
**FINAL REPORT
STRUCTURE FOUNDATION EXPLORATION
SUM-77/SR8-11.65/0.00
SUMMIT COUNTY, OHIO
PID: 114865**

Prepared For:

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NEAS PROJECT 22-0033

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EXECUTIVE SUMMARY

The Ohio Department of Transportation (ODOT) has proposed a roadway improvement project SUM-77/SR8-11.65/0.00 (PID 114865) along IR-77 just north of Lovers Lane to SR-8 and along SR-8 from IR-77 to just north of Perkins Street in Summit County, Ohio.

National Engineering & Architectural Services, Inc. (NEAS) 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/rock 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 53 test borings for the proposed construction; laboratory testing of soil samples in accordance with the SGE; performing geotechnical engineering analysis to assess subgrade stabilization requirements, pavement design parameters; and development of this summary report.

A subgrade exploration and subsequent analysis was completed for the referenced project. The exploration was conducted in general accordance with NEAS's proposal to Arcadis U.S. Inc., dated May 02, 2022, and with the provisions of ODOT's Specifications for Geotechnical Explorations (SGE)(ODOT, 2024). The subgrade analysis was performed in accordance with ODOT's Geotechnical Design Manual (ODOT [1], 2024) and Pavement Design Manual (PDM) (ODOT, 2024).

The subgrade conditions in the project area are relatively consistent and generally comprise existing pavement sections underlain by cohesive fine-grained soils and non-cohesive coarse-grained soils. The pavement sections are composed of asphalt ranging from five to twelve inches in thickness, or concrete ranging from eight to nine inches in thickness. Underneath the asphalt layer, a base layer encountered ranging from zero to thirteen inches in thickness. The cohesive fine-grained soils are classified as A-4a, A-4b, A-6a, A-6b, and A-7-6, while the non-cohesive coarse-grained soils are classified as A-1-b, A-2-4, A-3a, and non-plastic A-4a.

Based on our subgrade analysis, unstable materials, including weak soils and those with high moisture content, were encountered throughout the project limits. These soils exhibited an N₆₀ value below 12 bpf and/or a moisture content exceeding the optimum level by more than 3 percent within 3 feet of subgrade. Unsuitable soils, including rock and prohibited soils were encountered throughout the project limits as well. According to ODOT GDM Section 605, "For all Interstates and other divided highways with four or more lanes more than 1-mile in project length, chemically stabilize the subgrade for the entire project (global stabilization)". Also, according to ODOT GDM Section 609, "When global chemical stabilization includes areas where shallow bedrock is anticipated to be encountered during the stabilization process, excavate bedrock to a depth of 18 inches below the top of proposed subgrade. Replace the excavation with Item 204 Embankment and then proceed with chemical stabilization. The Item 204 Embankment will need to be tested for sulfate content using the area/volume guidance in Supplement 1120.03.". Therefore, NEAS recommends undercutting and replacing rock encountered within 6 inches of the final subgrade elevation with suitable material conforming to Item 204.02, Granular Material Type B, in accordance with the gradation requirements of 703.17. Global chemical stabilization of the project subgrade using cement to a depth of 12 inches is recommended, except along SR-8 where unsuitable A-4b material is encountered. In these areas, the stabilization depth should be increased to 14 inches from Station 329+96 to Station 331+97 and from Station 342+47 to Station 344+64.

NEAS's opinion that the subgrade soils will provide adequate pavement support assuming it is designed and constructed in accordance with the recommendations provided within this report, as well as all applicable ODOT standards and specifications.

The foundation system was checked for each of the proposed structures. At the abandoned Beacon Street Bridge site, the estimated undrained factored bearing resistance is between 4.7 ksf and 5.9 ksf, while the estimated drained factored bearing resistance is between 4.7 ksf and 5.2 ksf. Based on our analysis, the

ground surface is estimated to experience up to 3.60 inches of immediate settlement and up to 5.28 inches of long-term (consolidation) settlement due to the induced loads from the 26-foot-high new fill.

At the Exchange Street Bridge site, the estimated factored bearing resistance is between 50.6 ksf and 51.7 ksf.

At the Carroll Street Bridge site, the estimated factored bearing resistances are 123.6 ksf and 133 ksf at the rear abutment and rear abutment wingwall, respectively; 226.7 ksf to 228.5 ksf at the center pier; and 142.3 ksf and 151 ksf at the forward abutment and forward abutment wingwall, respectively.

At the Braid Bridge site, the estimated factored bearing resistances are between 96.7 ksf and 140 ksf at the rear abutment, and between 96.9 ksf and 109.3 ksf at the forward abutment.

At the RW 1 site, it is recommended to use W24x62 steel reinforcing beam with yield strength of 50 ksi (ASTM grade 50). Based on our lateral load analysis, the proposed 12-ft-long shaft in bedrock should be sufficient to constrain the deflection at the pile head with appropriate serviceability limits.

At RW 2 site, based on our external stability, the recommended soil reinforcement length is 70% of the design wall height.

At RW 3 site, it is recommended to use a W24x162 as the steel reinforcing beam with yield strength of 50 ksi (ASTM grade 50). The recommended soil reinforcement length for the MSE wall option is 80% of design height.

At the RW 4 site, the recommended soil reinforcement length is 70% of the design wall height.

At the RW 5 site, the recommended soil reinforcement length is 70% of the design wall height.

At the RW 6 site, the factored bearing resistance on soils is about 20.6 ksf, while the factored bearing resistance on bedrock is estimated to be 110.7 ksf using the RMR method.

At the RW 7 site, the recommended soil reinforcement length is 70% of the design wall height.

At the RW 8 site, the factored bearing resistance on bedrock is estimated to be 82.0 ksf using the RMR method.

At the RW 9 site, the factored bearing resistance on bedrock is estimated to range between 102.6 ksf and 142.8 ksf using the RMR method.

At the RW 10 site, the factored bearing resistance on bedrock is estimated to range between 97.9 ksf and 174.8 ksf using the RMR method.

A seismic site class was also determined at the overall project site, in which a Seismic Site Class D is recommended.

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1. INTRODUCTION

1.1. General

National Engineering and Architectural Services Inc. (NEAS) presents our Structure Foundation Exploration Report for a roadway improvement project SUM-77/SR8-11.65/0.00 project (PID: 114865) along IR-77 just north of Lovers Lane to SR-8 and along SR-8 from IR-77 to just north of Perkins Street in Summit County, Ohio. The proposed project consists of service road and ramps reconfigurations, widening and resurfacing SR-8 and its associated ramps, the abandonment of one bridge, the addition of a ramp braid, the construction of 10 retaining walls, and the replacement of two bridges. The overall project objective is to improve the roadway condition of SR-8, enhance safety, and reduce congestion.

The geotechnical scope of work includes:

- Roadway widening and resurfacing of SR-8 and its associated ramps
- Bridge widening for the bridge carrying SR-8 over Beacon Street
- Bridge widening for the bridge carrying SR-8 over E. Exchange Street
- Construction of southbound (SB) entrance ramp bridge over the SB exit ramp
- Replacement for the Carroll Street bridge over SR-8
- Construction of 10 Retaining walls along SR-8 northbound (NB) and SB

The report presents a summary of the encountered surficial and subsurface conditions and our recommendations for bridge 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, 10th Edition (BDS) (AASHTO, 2024), ODOT's 2025 Bridge Design Manual (BDM) (ODOT, 2025) and ODOT's 2025 Geotechnical Design Manual (GDM) (ODOT, 2025).

The exploration was conducted in general accordance with NEAS, Inc.'s original proposal to Arcadis U.S., Inc. dated May 2, 2022, and additional proposals to Arcadis U.S., Inc. dated May 5, 2023 and May 13, 2024 with the provisions of ODOT's *Specifications for Geotechnical Explorations* (SGE) (ODOT[2], 2025).

The scope of work performed included: 1) reviewing published geotechnical information; 2) performing 53 test borings as part of the referenced structure foundation exploration; 3) performing laboratory testing of soil and rock samples in accordance with the SGE; 4) conducting geotechnical engineering analysis to assess foundation design and construction considerations; and 5) developing of this summary report.

1.2. Proposed Construction

It is our understanding that the bridge carrying SR-8 over abandoned Beacon Street is proposed for removal. The existing structure consists of continuous steel beams with composite reinforced deck, supported by wall-type semi-integral abutments and a cap and column pier. The proposed replacement structure is a precast reinforced concrete box culvert, backfilled with soil to support the roadway. The culvert will have a span of 15 feet and a rise of 12 feet.

NEAS understands that the bridge (Structure File Number: 7700156) carrying SR-8 over Exchange Street is proposed to be widened, and a new bridge carrying Ramp CD over Exchange Street is proposed to be constructed. The existing structure is a single-span steel girder bridge with a non-composite reinforced concrete deck on wall-type semi-integral abutments. The proposed ramp bridge will be a single-span steel beam bridge with a composite reinforced concrete deck on wall-type semi-integral abutments. The

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widening bridge will have a span of 86'-6" (center to center of bearings), while the ramp bridge will have a span of 86'-7 1/4" (center to center of bearings).

A new ramp bridge (No. SUM-77/8-1165/000) is proposed to carry the SB entrance ramp to SR-8 over the SB exit ramp to IR-76, between Vine Street and Orchard Street. The proposed structure is a single-span prestressed concrete I-beams bridge with a composite reinforced concrete deck slab, supported by reinforced concrete full-height wall abutments and cap-and-column abutments on spread footings. The ramp bridge will have a span of 46'-10 1/2" (center to center of bearings) and a roadway width of approximately 26'-0".

Additionally, it is our understanding that the bridge (Structure File Number: 7700180) carrying Carroll Street over SR-8 is proposed to be replaced. The existing structure is a continuous steel beam bridge with a reinforced concrete deck and substructure. The proposed replacement bridge will be a two-span continuous rolled steel beam bridge with a composite reinforced concrete deck, supported by semi-integral abutments and a reinforced concrete cap-and-column pier. The new bridge will be supported by spread footings and will have spans of 71'-6" and 68'-6" (center to center of bearings).

Retaining Wall 1 (RW 1) is proposed along the west side of Ramp CD, extending from STA. 105+26.63 to STA. 111+77.99. RW 1 will be approximately 650.00 ft long and designed as a soldier pile wall. The maximum exposed wall height is approximately 17.41 ft at STA. 6+00, referenced to the alignment of RW 1.

Retaining Wall 2 (RW 2) is proposed along the west side of Ramp CD, extending from STA. 113+19.42 to STA. 127+43.17. RW 2 will be approximately 1,423.79 ft long and designed as an MES wall. The maximum design wall height (measured from the top of the coping to top of leveling pad) is 33.0 ft at STA. 3+00, referenced to RW 2.

Retaining Wall 3 (RW 3) is proposed along the east side of SR-8, extending from STA. 335+26.90 to STA. 337+72.83. RW 3 will be a combination of an MSE wall and a soldier pile lagging wall, with a total length of 251'-7". The maximum design wall height (measured from the top of coping to the top of the leveling pad) for the MSE wall section is 28.2 ft at the beginning (STA. 0+00) of RW 3. The exposed wall height for the soldier pile lagging wall section is approximately 12.6 ft.

Retaining Wall 4 (RW 4) is proposed along the west side of the SR-8 southbound exit CD (to IR-76), extending from STA. 128+47.86 to STA. 132+24.01. RW 4 will be approximately 376'-6 3/8" long and designed as an MES wall. The maximum design wall height (measured from the top of the coping to the top of the leveling pad) is 22.5 ft at STA. 400+00, referenced to RW 4.

Retaining Wall 5 (RW 5) is proposed along the west side of the SR-8 Southbound Entrance Ramp (to SR-8) from STA. 352+06.83 to STA. 357+22.52. RW 5 is about 515'-10 1/4" long and is proposed as a MES wall. The max design wall height (from top of coping to top of leveling pad) is 24.4 ft at STA. 504+28.33 in reference to RW 5.

Retaining Wall 6 (RW 6) is proposed along the west side of Goodkirk Street, extending from STA. 504+72.71 to STA. 508+48.78. RW 6 will be approximately 411.83 ft long and designed as a cast-in-place wall. The maximum design wall height (measured from the top of the wall to the bottom of the footing) is 15.69 ft at STA. 602+25, referenced to RW 6.

Retaining Wall 7 (RW 7) is proposed along the east side of the SR-8 southbound exit CD (to IR-76), extending from STA. 134+50.00 to STA. 135+65.74. RW 7 will be approximately 213.75 ft long and designed as an MSE wall. The maximum design wall height (measured from the top of the coping to the top of the leveling pad) is 21.80 ft at STA. 702+13.70, referenced to RW 7.

Retaining Wall 8 (RW 8) is proposed along the west side of the SR-8 southbound exit CD (to IR-76), extending from STA. 134+63.00 to STA. 136+87.50. RW 8 will be approximately 225.39 ft long and designed as a cast-in-place wall. The maximum design wall height (measured from the top of the wall to the bottom of the footing) is 9.56 ft at STA. 802+25.39, referenced to RW 8.

Retaining Wall 9 (RW 9) is proposed along the west side of the SR-8 southbound exit CD (to IR-76), extending from STA. 139+72.79 to STA. 142+90.02. RW 9 will be approximately 317.51 ft long and designed as a cast-in-place wall. The maximum design wall height (measured from the top of the wall to the bottom of the footing) is 25.74 ft at STA. 1000+00, referenced to RW 9.

Retaining Wall 10 (RW 10) is proposed along the west side of the SR-8 southbound exit CD (to IR-76), extending from STA. 362+60.09 to STA. 366+56.40 in reference to SR-8. RW 10 will be approximately 398.875 ft long and designed as a cast-in-place wall. The maximum design wall height (measured from the top of the wall to the bottom of the footing) is 26.46 ft at STA. 1100+00, referenced to RW 10.

2. GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1. Geology and Physiography

The project site is located within the Akron-Canton Interlobate Plateau, part of the Glaciated Allegheny Plateaus (ODGS, 1998). This is a moderate relief, hummocky area between two converging glacial lobes dominated by kames, kame terraces, eskers, kettles, kettle lakes, and bogs/fens. Soils in this region are characteristically sandy Wisconsinan-age clay and older drift over Devonian to Pennsylvanian-age sandstones, conglomerates and shales.

The geology at the project site is mapped as an average of 20 ft of Wisconsinan-age till, underlain by Pennsylvanian-age Shale, Sandstone, Limestone, and Coal bedrock (ODGS, 2005). The till is described as an unsorted mix of clay, silt, sand, gravel, and boulders and may contain silt, sand, and gravel lenses. Till in deep buried valleys is noted as potentially being older than Wisconsinan.

The exception to this is a small area at the southern end of the project area that is mapped as an average of 300 ft of Wisconsinan-age sand and gravel underlain by Mississippian-age sandstone and shale bedrock (ODGS, 2005) the sand and gravel is described as interbedded sand and gravel commonly containing thin, discontinuous layers of silt and clay. The grains are well to moderately sorted, moderately to well rounded, finely stratified to massive, may be cross-bedded, and may locally contain organics. Sand and gravel in deep buried valleys is noted as potentially being older than Wisconsinan.

Based on the Bedrock Geologic Units Map of Ohio (USGS & ODGS, 2006), bedrock within the project area consists of shale, sandstone, limestone, and coal, of the Allegheny and Pottsville groups, undivided. This undivided group is comprised of Pennsylvanian-age shale, sandstone, limestone, and coal. The shale in this formation is described as black, gray, and olive in color, clayey to silty, and calcareous in part. The sandstone in this formation is described as light to medium gray and weathers to shades of yellow-brown, mostly very fine to medium grained, locally quartzose and conglomeratic in the lower third of the unit, and thin to massive to cross bedded. The limestone in this formation is described as black to light gray in color, micritic to medium grained, locally grades to flint, thin to medium bedded. The coal in this formation is described as mostly banded bituminous, locally cannel, thin to locally as much as 12 ft thick. The bedrock appears to follow the natural topography of the site which slopes very gently from south to north. (ODGS, 2003). Based on the ODNR bedrock topography map of Ohio, bedrock elevations at the project site can be expected to be between 1000 to 1100 ft amsl, putting bedrock at a depth of about 5 to 40 ft below ground surface (bgs).

The soils at the project site have been mapped (Web Soil Survey) by the Natural Resources Conservation Service (USDA, 2015) as primarily Udorthents and Canfield-Urban land complex. Soils classified as Udorthents are soils that have been disturbed by cutting and filling and as such are not classified according to the AASHTO method of soil classification. Soils in the Canfield series are characterized as very deep, moderately well drained soils formed in Wisconsinan-age till on plains. The Canfield series is comprised of both coarse- and fine-grained soils and classifies as A-1, A-2, A-4, A-6, and A-7 type soils according to the AASHTO method of soil classification.

2.2. Hydrology/Hydrogeology

Groundwater at the project site can be expected at an elevation consistent with that of the nearby Little Cuyahoga River in the northern portion of the project and the Tuscarawas River in the southern portion of the project as they are the most dominant hydraulic influences in the vicinity of the project's boundaries. The water level of the two aforementioned rivers may be generally representative of the local groundwater table. However, it should be noted that perched groundwater systems may be existent in areas due to the presence of fine-grained soils making it difficult for groundwater to permeate to the phreatic surface.

The project site is not located within a regulatory flood hazard area based on available mapping by the Federal Emergency Management Agency's (FEMA) National Flood Hazard mapping program (FEMA, 2019).

2.3. Mining and Oil/Gas Production

An abandoned mine (ID# ST-OGS-019) located along IR-77 between E Wilbeth Rd. and E Catawba Ave. is noted on ODNR's Abandoned Underground Mine Locator in the vicinity of the project site. The exact extents of this abandoned mine are unknown (ODNR [1], 2020).

A directionally drilled oil and gas well (ID# 3415322866) is also noted on ODNR's Oil and Gas Well Locator near the project site (ODNR [1], 2020). The wellhead was located approximately 0.04 miles south and 0.04 miles west of where Beacon St. crosses under SR-8, while the bottom of the well is approximately 0.1 miles north and 0.02 miles east of the same point (ODNR [1], 2020).

2.4. Historical Records and Previous Phases of Project Exploration

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

- Soil Profile Sheets and Boring Logs for the Soil Profile Investigation Project SUM-8-0.63 (PID: 20356), Sub Batch: 19060, dated 2000.
- Subsurface Exploration Data Report SUM-76/77 Phase III, PID: 102329 dated April 25, 2020.

A summary of the historical boring information, including location, elevation, and depth, is provided in Table 1. The historical boring logs are provided in Appendix B.

Table 1: Historical Boring Summary

Boring Number	Latitude	Longitude	Elevation (ft)	Depth (ft)
Subgrade				
R-002-0-99	41.067381	-81.504350	1064.0	10.0
R-004-0-99	41.069453	-81.504272	1046.8	10.0
R-005-0-99	41.070650	-81.504033	1045.0	10.0
R-006-0-99	41.071744	-81.504392	1038.6	10.0
R-008-0-99	41.072842	-81.504206	1037.3	7.6
R-009-0-99	41.072997	-81.504798	1039.0	8.7
R-010-0-99	41.073831	-81.504311	1036.4	8.8
R-011-0-99	41.073995	-81.504810	1047.3	8.8
R-012-0-99	41.074906	-81.504122	1035.0	5.7
R-013-0-99	41.075106	-81.504771	1057.3	8.6
B-004-0-99	41.072478	-81.504790	1038.2	9.5
B-006-0-99	41.073677	-81.504805	1043.6	8.8
B-008-0-99	41.075198	-81.504708	1057.4	10.5
P-011-0-20	41.063858	-81.504556	1068.8	9.1
P-012-0-20	41.063861	-81.504878	1069.9	10.0
P-013-0-20	41.064340	-81.504591	1072.2	8.9
P-014-0-20	41.064663	-81.504279	1075.0	9.2
P-015-0-20	41.065760	-81.504185	1077.7	8.8
P-016-0-20	41.066053	-81.504384	1076.9	8.9
P-017-0-20	41.066858	-81.504167	1070.3	9.3
P-018-0-20	41.067149	-81.504219	1067.0	10.0
P-019-0-20	41.067750	-81.503970	1060.5	10.0
P-020-0-20	41.068249	-81.504397	1056.0	10.0
P-022-0-20	41.069214	-81.504357	1048.3	10.0
P-024-0-20	41.070312	-81.504432	1044.9	10.0
P-025-0-20	41.071338	-81.504014	1042.7	10.0
P-027-0-20	41.072435	-81.503947	1039.1	10.0
P-028-0-20	41.072509	-81.504521	1039.3	8.7
P-029-0-20	41.073065	-81.504556	1038.0	10.0
P-030-0-20	41.073381	-81.503868	1033.1	8.9
P-032-0-20	41.074705	-81.504453	1034.6	8.7
E3-001-0-20	41.064138	-81.504881	1039	9.5
E3-002-0-20	41.064471	-81.504774	1043.6	10.00
SUM-8-0086 Bridge over Exchange Street				
GB-001-0-99	41.071006	-81.504584	1023.4	28.00
GB-002-0-99	41.071283	-81.504597	1023.5	18.00
Retaining Wall 1				
E3-003-0-20	41.064805	-81.504724	1093.2	50.00
E3-004-0-20	41.065216	-81.504644	1098.3	50.00
Retaining Wall 2				
B2-001-0-20	41.067411	-81.504525	1064.1	40.00

2.5. Field Reconnaissance

A field reconnaissance visit for the overall project area was conducted on October 4, 2022, along SR-8 between Johnston Street and Perkins Street. Site conditions were observed and documented through photographs. Notable geotechnical and drainage features were photographed, and a summary of our observations by roadway segment is provided below.

2.5.1. Land Use and Cover

The land use of most of the project area consists of 1) Industrial properties; 2) Residential Properties; 3) Commercial Properties; and 4) Institutional Properties.

2.5.2. Proposed Retaining Wall Location North of Johnston Street

The existing slope in the area of the proposed retaining wall was observed to be approximately 2H:1V (2 ft horizontal to 1 ft vertical). The slope itself was lightly vegetated, while the area at the top of the slope was heavily vegetated (Photograph 1). A traffic barrier/retaining wall was observed at the toe of the slope and appeared to be in excellent condition, with no signs of wear. The slope appeared stable, with no indications of geotechnical issues.

Photograph 1: Existing Embankment in the Area of the Proposed Retaining Wall



2.5.3. Bridge Carrying SR-8 over Abandoned Beacon Street

The existing bridge carrying SR-8 over Beacon Street is a two-span, steel multi-beam bridge with four lanes of traffic in each direction on a concrete deck with a concrete wearing course (Photograph 2). The bridge sits atop full-height wall-type concrete abutments and a cap and column piers. The foundation type was unknown at the time of the site visit. The roadway embankment slopes at the site generally appeared to be stable, with no signs of instability observed during our site visit. The existing roadway embankments appeared to have a 2 Horizontal to 1 Vertical (2H:1V) slope and were heavily vegetated.

Overall, the bridge appeared to be in good condition, with minor wear and degradation observed on the bridge substructures. Both the concrete bridge deck and the main steel superstructure beams were observed to be in excellent condition. Some water infiltration through the expansion joints was observed (Photograph 3). A steel retaining wall at the southeast corner of the bridge was observed to be in fair condition, with rust and through-holing concentrated on the top of the wall (Photograph 4). No apparent signs of structural distress due to geotechnical concerns were observed during our field reconnaissance visit.

In general, the existing bridge structure appeared to be adequately drained, with no signs of significant erosion at the bridge site. The concrete wearing course was observed to be in good condition, with some

signs of surface wear. Some concrete pop-outs were observed. Water appeared to run down the shoulders of the bridge to the road going northwards.

Photograph 2: Beacon Street Bridge



Photograph 3: Signs of Infiltration at Expansion Joints



Photograph 4: Steel Retaining Wall at Southeastern Corner of Bridge



2.5.4. SR-8 between E Exchange Street and Perkins Street

In general, the pavement condition of the SR-8 in this section was observed to be fair, with signs of weathering and surface wear. Moderate severity longitudinal and transverse cracking was common along this section, along with potholing, raveling, and crack sealing deficiencies (Photograph 5). The on and off ramps were observed to be in worse condition than the mainline SR-8, showing poor condition overall. Moderate severity longitudinal and transverse cracking was common, as well as potholing, patching, raveling, map cracking, and crack sealing deficiencies (Photograph 6). The project area in this section is situated in a cut below the surrounding land, with slopes up to the surrounding land of roughly 3H:1V (3 ft horizontal to 1 ft vertical). The roadway in this section of the project drains to drainage ditches off both shoulders, which eventually drains to drainage basins, as well as to drainage basins between the ramps and SR-8 (Photograph 7). The area is moderately vegetated, and no signs of standing water were observed. No issues due to geotechnical conditions were noted.

Photograph 5: Overall Pavement Condition of SR-8



Photograph 6: Overall Pavement Condition of Ramps



Photograph 7: Example of Drainage Ditches and Drainage Basin



2.5.5. *Fountain Street*

In general, the pavement condition of the project portion of Fountain Street was observed to be good to fair, with signs of weathering and surface wear. Moderate potholing, D-cracking, and patching with asphalt were observed but were confined to a few locations (Photograph 8). The project area in this section was observed to be level with the surrounding land. The roadway in this section of the project drains to drainage basins in both curbs of the roadway. The area is lightly vegetated, and no signs of standing water were observed. No issues due to geotechnical conditions were observed.

Photograph 8: Pavement Condition of Fountain Street



2.5.6. Bridge carrying Carroll Street over SR-8

The existing bridge carrying Carroll Street over SR-8 is a two-span, continuous steel beam bridge with a reinforced concrete deck and substructure. The bridge carries two lanes of traffic in each direction. The roadway embankment slopes at the site generally appeared to be stable, with no signs of instability observed during the visit. The existing roadway embankments are approximately at a 2 Horizontal to 1 Vertical (2H:1V) slope and are heavily vegetated. Overall, the bridge is in good condition, with only minor wear and degradation observed on the substructures. Both the concrete bridge deck and the main steel superstructure beams are in good condition. No apparent signs of structural distress due to geotechnical concerns were observed during the field reconnaissance visit.

In general, the existing bridge structure appeared to be adequately drained, with no signs of significant erosion at the bridge site. The concrete wearing course is in good condition, with some signs of surface wear.

3. GEOTECHNICAL EXPLORATION

3.1. Field Exploration Program

The project subsurface exploration was conducted by NEAS between March 7, 2023, and October 28, 2024, and included 53 borings drilled to depths between 6.1 ft and 70.9 ft below ground surface. The boring locations were selected by NEAS in general accordance with the guidelines contained in the SGE with the intent to evaluate subsurface soil and groundwater conditions. Borings were typically located within the planned project construction areas that were not restricted by underground utilities or dictated by terrain (e.g. steep embankment slopes). Project boring locations were located in the field after drilling by the project surveyor. Each individual project boring log (included within Appendix B) includes the recorded boring latitude and longitude location (based on the surveyed Ohio State Plane North, NAD83, location) and the corresponding ground surface elevation. The boring locations are depicted on the Boring Location Plan provided in Appendix A. Latitude/Longitude, elevations and stationing and offsets of the borings are shown on Table 2 below.

Structure Foundation Exploration – FINAL
SUM-77/SR8-11.65/0.00
Summit County, Ohio
PID: 114865

Table 2: Project Boring Summary

Boring Number	Latitude	Longitude	Elevation (NAVD 88) (ft)	Depth (ft)
Bridge Over Beacon Street				
B-003-0-22	41.067542	-81.504370	1063.0	55.5
B-004-0-22	41.067795	-81.504522	1033.3	35.5
B-005-0-22	41.068235	-81.504410	1054.8	70.9
Braid Bridge				
B-023-0-24	41.073715	-81.504701	1043.3	57.6
B-024-0-24	41.073746	-81.504528	1036.2	45.2
B-025-0-24	41.074196	-81.504711	1051.4	50.3
B-026-0-24	41.073969	-81.504443	1036.1	45.3
Carroll Bridge Over SR-8				
B-027-0-24	41.075365	-81.504645	1057.1	25.1
B-028-0-24	41.075208	-81.504277	1033.8	27.7
B-029-0-24	41.075271	-81.503786	1050.9	35.0
Retaining Wall 1				
B-001-0-22	41.065799	-81.504614	1093.5	45.7
B-002-0-22	41.066166	-81.504561	1090.6	40.8
B-030-0-24	41.066627	-81.504407	1071.3	40.5
Retaining Wall 2				
B-031-0-24	41.067120	-81.504408	1066.7	26.2
B-021-0-24	41.067670	-81.504477	1034.3	25.5
B-032-0-24	41.068013	-81.504656	1027.0	35.5
B-033-0-24	41.068414	-81.504402	1053.3	65.4
B-034-0-24	41.068830	-81.504409	1049.4	55.9
B-035-0-24	41.069225	-81.504765	1007.1	20.0
B-036-0-24	41.069613	-81.504760	1007.9	13.9
B-037-0-24	41.070062	-81.504710	1009.9	14.3
B-038-0-24	41.070457	-81.504738	1014.6	14.3
B-039-0-24	41.070856	-81.504724	1022.2	16.0
Retaining Wall 3				
B-022-0-24	41.068026	-81.503908	1034.3	35.5
Retaining Wall 4				
B-040-0-24	41.071567	-81.504695	1027.1	15.5
B-041-0-24	41.071900	-81.504772	1032.6	15.5
B-042-0-24	41.072273	-81.504709	1037.0	20.4
Retaining Wall 5				
B-043-0-24	41.072750	-81.504473	1038.8	17.9
B-044-0-24	41.073034	-81.504437	1038.5	15.1
B-024-0-24	41.073747	-81.504527	1036.2	45.2
Retaining Wall 6				
B-045-0-24	41.072442	-81.504931	1044.4	26.1
B-046-0-24	41.072861	-81.504915	1049.9	27.6
B-047-0-24	41.073232	-81.504892	1048.4	25.1
Retaining Wall 7				
B-048-0-24	41.073139	-81.504590	1037.8	16.2
B-049-0-24	41.073513	-81.504566	1040.4	15.1
Retaining Wall 8				
B-050-0-24	41.073021	-81.504721	1038.1	18.2
B-051-0-24	41.073349	-81.504716	1038.7	18.0
Retaining Wall 9				
B-052-0-24	41.074433	-81.504727	1053.7	25.1
B-053-0-24	41.074434	-81.504509	1034.5	15.2
B-054-0-24	41.074829	-81.504710	1056.2	30.1
Retaining Wall 10				
B-055-0-24	41.075531	-81.504438	1034.4	20.1
B-056-0-24	41.075930	-81.504659	1060.6	35.1
B-057-0-24	41.076345	-81.504404	1039.3	20.1
CCTV Pole				
B-058-0-24	41.073072	-81.503844	1036.3	20.5

Boring Number	Latitude	Longitude	Elevation (NAVD 88) (ft)	Depth (ft)
Subgrade			(Continue)	
B-001-0-22	41.065799	-81.504614	1093.5	45.7
B-002-0-22	41.066166	-81.504561	1090.6	40.8
B-006-0-22	41.073914	-81.504048	1036.1	6.5
B-007-0-22	41.075986	-81.504043	1036.0	6.1
B-008-0-22	41.076990	-81.504379	1041.8	6.8
B-009-0-22	41.078164	-81.503859	1040.3	7.5
B-010-0-22	41.078854	-81.503677	1034.1	6.3
B-011-0-22	41.079310	-81.504147	1030.7	6.8
B-012-0-22	41.081340	-81.503812	1026.4	7.5
B-013-0-22	41.083172	-81.503430	1031.4	6.1
B-018-0-22	41.078552	-81.504364	1038.0	6.2
B-019-0-22	41.082223	-81.503757	1031.7	6.2
B-020-0-22	41.082184	-81.503095	1030.5	6.2

Project borings were drilled using a CME 45B, CME 55X, CME 55T, or CME 55TB truck-mounted drilling rig with a 3.25-inch (inner diameter) hollow stem auger. In general, soil samples were recovered continuously or at 2.5-ft interval to the top of bedrock using an 18-inch split spoon sampler (AASHTO T-206 “Standard Method for Penetration Test and Split Barrel Sampling of Soils.”). The soil samples obtained from the exploration program were visually observed in the field by the NEAS field representative and preserved for review by a Geologist for possible laboratory testing. Standard penetration tests (SPT) were conducted using a CME auto hammer calibrated to be 72.6%, 79% and 63.4% efficient for CME 45B, CME 55X, CME 55T rigs, respectively, on January 24, 2022, and 89% efficient for the CME 55TB on July 30, 2024, as indicated on the boring logs. The CME 55T rig has been recalibrated on May 15, 2024 to be 71%. The CME 55X rig has been recalibrated on March 25, 2026 to be 83%.

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

3.2. Laboratory Testing Program

The laboratory testing program consisted of classification testing and moisture content determinations. Data from the laboratory testing program was incorporated onto the boring logs (Appendix B). Soil samples are retained at the laboratory through completion and ODOT approval of Stage 2 plans, 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 33% of the samples. At each boring location, samples were selected for testing with the intent of identification and classification of all significant soil units. Soils not selected for testing were compared to laboratory tested samples/strata and classified visually. Moisture content testing was conducted on all samples. The laboratory testing was performed in general accordance with applicable AASHTO specifications.

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

3.2.3. Sulfate Testing

Sulfate testing was generally performed on one sample from each subgrade/roadway boring performed for pavement/subgrade design purposes. The selected samples were tested in accordance with ODOT Supplement 1122, “Determining Sulfate Content in Soils” dated July 17, 2015. In general, the upper most sample (within 3 ft of the proposed subgrade elevation) from each boring was tested when feasible. Testing results are summarized in Table 3 below, and presented on the boring logs within Appendix B.

Table 3: Sulfate Test Summary by Boring

Boring ID	Sample	Depth (ft)	Dilution Ratio	Average Sulfate Content (ppm)
B-006-0-22	SS-1	1.5 - 2.17	20	0
B-007-0-22	SS-1	1.5 - 3.0	20	120
B-008-0-22	SS-1	1.5 - 3.0	20	327
B-009-0-22	SS-1	1.5 - 3.0	20	13
B-011-0-22	SS-1	1.5 - 3.0	20	53
B-012-0-22	SS-1	1.5 - 3.0	20	133
B-019-0-22	SS-1	1.5 - 3.0	20	0

3.2.4. Unconfined Compressive Strength of Rock Core Results

Unconfined Compressive Strength of Rock Core Testing was conducted in accordance with ASTM D7012, "Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying Sates of Stress and Temperatures", on twenty-three rock core samples obtained during the exploration program. The elastic modulus of sandstone was also determined using NQ2-1 from B-048-0-24, which yielded a value of 7.361 GPa. The Unconfined Compressive Strength test results are shown in Table 4 below and provided in Appendix B.

Table 4: Unconfined Compressive Strength Test Results

Boring ID	Rock Core	Depth (ft)	Elevation (ft)	Bedrock Type	Unconfined Compressive Strength (psi)	Stain at Failure (%)
B-022-0-24	NQ2-1	32.7 - 33.1	1001.2 - 1001.6	Very weak, slightly weathered Shale	390	2.0
B-023-0-24	NQ2-1	14.9 - 15.3	1028.0 - 1028.4	Moderately strong, slightly weathered Sandstone	5,315	1.0
B-024-0-24	NQ2-1	7.4 - 7.8	1028.4 - 1028.8	Slightly strong, slightly weathered Sandstone	1,752	0.9
B-025-0-24	NQ2-1	15.6 - 16.0	1035.4 - 1035.8	Moderately strong, unweathered Sandstone	4,242	1.2
B-026-0-24	NQ2-1	7.3 - 7.6	1028.5 - 1028.8	Slightly strong, moderately weathered Sandstone	2,936	2.8
B-026-0-24	NQ2-2	15.8 - 16.1	1020.0 - 1020.3	Moderately strong, slightly weathered Sandstone	6,808	1.0
B-027-0-24	NQ2-1	14.0 - 14.4	1042.7 - 1043.1	Slightly strong, moderately weathered Sandstone	2,600	1.2
B-028-0-24	NQ2-1	6.3 - 6.7	1027.1 - 1027.5	Moderately strong, slightly weathered Sandstone	6,587	0.7
B-029-0-24	NQ2-1	19.4 - 19.8	1031.1 - 1031.5	Moderately strong, slightly weathered Sandstone	5,519	1.0
B-030-0-24	NQ2-1	16.3 - 16.8	1054.5 - 1055.0	Moderately strong, slightly weathered Sandstone	4,843	1.1
B-031-0-24	NQ2-1	18.2 - 18.6	1048.1 - 1048.5	Moderately strong, slightly weathered Sandstone	6,083	1.3
B-042-0-24	NQ2-1	13.7 - 14.1	1022.9 - 1023.3	Moderately strong, slightly weathered Sandstone	5,900	1.4
B-043-0-24	NQ2-1	8.9 - 9.3	1029.5 - 1029.9	Moderately strong, slightly weathered Sandstone	6,775	1.0
B-044-0-24	NQ2-1	7.8 - 8.2	1030.3 - 1030.7	Moderately strong, slightly weathered Sandstone	6,400	1.4
B-046-0-24	NQ2-1	19.4 - 19.8	1030.1 - 1030.5	Moderately strong, slightly weathered Sandstone	5,536	1.3
B-048-0-24	NQ2-1	11.6 - 12.0	1025.8 - 1026.2	Moderately strong, slightly weathered Sandstone	6,916	1.1
B-049-0-24	NQ2-1	8.6 - 9.0	1031.4 - 1031.8	Moderately strong, moderately weathered Sandstone	6,540	1.0
B-051-0-24	NQ2-1	11.3 - 11.7	1027.0 - 1027.4	Moderately strong, slightly weathered Sandstone	4,620	1.1
B-052-0-24	NQ2-1	17.9 - 18.3	1035.4 - 1035.8	Slightly strong, slightly weathered Sandstone	2,331	1.2
B-053-0-24	NQ2-1	6.2 - 6.6	1027.9 - 1028.3	Moderately strong, slightly weathered Sandstone	4,942	1.0
B-054-0-24	NQ2-1	16.4 - 16.8	1039.4 - 1039.8	Moderately strong, slightly weathered Sandstone	4,874	1.4
B-055-0-24	NQ2-1	6.5 - 6.9	1027.5 - 1027.9	Moderately strong, slightly weathered Sandstone	5,520	0.9
B-057-0-24	NQ2-1	6.7 - 7.1	1032.2 - 1032.6	Moderately strong, unweathered Sandstone	5,541	1.1

4. GEOTECHNICAL FINDINGS

4.1. Existing Pavement

The pavement section thicknesses in terms of asphalt, brick, sand, concrete and granular base were measured at representative project subgrade borings during the subsurface exploration for the project and are recorded on the test boring logs provided in Appendix B. A summary of these measurements is provided in Table 5 below.

Table 5: Measured Pavement Thickness at Boring Locations

Boring ID	Drilled Depth (ft)	Asphalt Thickness (in)	Concrete Thickness (in)	Base Thickness (in)	Total Thickness (in)	Boring ID	Drilled Depth (ft)	Asphalt Thickness (in)	Concrete Thickness (in)	Base Thickness (in)	Total Thickness (in)
B-001-0-22	45.7	11.0	-	8.0	19.0	B-030-0-24	40.5	8.0	-	12.5	20.5
B-002-0-22	40.8	11.0	-	8.0	19.0	B-031-0-24	26.2	7.0	-	11.0	18.0
B-003-0-22	55.5	12.0	-	6.0	18.0	B-033-0-24	65.4	9.0	-	13.0	22.0
B-005-0-22	70.9	11.0	-	8.0	19.0	B-034-0-24	55.9	8.0	-	10.0	18.0
B-006-0-22	6.5	10.0	-	7.0	17.0	B-036-0-24	13.9	-	8.0	-	8.0
B-007-0-22	6.1	10.0	-	8.0	18.0	B-037-0-24	14.3	-	9.0	-	9.0
B-008-0-22	6.8	10.0	-	8.0	18.0	B-038-0-24	14.3	-	9.0	-	9.0
B-009-0-22	7.5	9.0	-	8.0	17.0	B-039-0-24	16.0	-	9.0	-	9.0
B-010-0-22	6.3	10.0	-	6.0	16.0	B-040-0-24	15.5	-	9.0	-	9.0
B-011-0-22	6.8	8.0	-	10.0	18.0	B-041-0-24	15.5	-	8.0	-	8.0
B-012-0-22	7.5	10.0	-	8.0	18.0	B-043-0-24	17.9	8.0	-	10.0	18.0
B-013-0-22	6.1	11.0	-	7.0	18.0	B-044-0-24	15.1	8.0	-	10.0	18.0
B-018-0-22	6.2	9.0	-	8.0	17.0	B-048-0-24	16.2	5.0	-	-	5.0
B-019-0-22	6.2	10.0	-	8.0	18.0	B-049-0-24	15.1	-	8.0	12.0	20.0
B-020-0-22	6.2	8.0	-	7.0	15.0	B-050-0-24	18.2	-	8.0	11.0	19.0
B-022-0-24	35.5	7.0	-	-	7.0	B-051-0-24	18.0	-	8.0	-	8.0
B-023-0-24	57.6	-	8.0	10.0	18.0	B-052-0-24	25.1	-	8.0	-	8.0
B-025-0-24	50.3	-	8.0	12.0	20.0	B-054-0-24	30.1	-	8.0	-	8.0
B-027-0-24	25.1	-	8.0	11.0	19.0	B-055-0-24	20.1	8.0	-	10.0	18.0
B-028-0-24	27.7	8.0	-	11.0	19.0	B-056-0-24	35.1	-	8.0	10.0	18.0
B-029-0-24	35.0	-	8.0	12.0	20.0	B-057-0-24	20.1	7.0	-	11.0	18.0

4.2. Subgrade Conditions

The subgrade conditions in the project area are relatively consistent and generally comprise existing pavement sections underlain by cohesive fine-grained soils and non-cohesive coarse-grained soils. The pavement sections are composed of asphalt ranging from eight to eleven inches in thickness. Underneath the asphalt layer, a base layer encountered ranging from six to ten inches in thickness. The cohesive fine-grained soils are classified as A-4a, A-4b, A-6a, A-6b, and A-7-6, while the non-cohesive coarse-grained soils are classified as A-1-b, A-2-4, A-3a, and non-plastic A-4a. A brief summary of the subgrade conditions encountered along the project site is provided below.

4.2.1. SR-8

This section consists of 41 borings along SR-8, designated as P-011-0-20 to P-020-0-20, P-022-0-20, P-024-0-20, P-025-0-20, P-027-0-20, P-028-0-20, P-030-0-20, P-032-0-20, R-002-0-99, R-004-0-99 to R-006-0-99, R-008-0-99, R-010-0-99, R-012-0-99, B-003-0-22, B-005-0-22 to B-013-0-22, B-018-0-22 to B-020-0-22, B-028-0-24, B-044-0-24, B-055-0-24 and B-057-0-24.

Along SR-8, twenty-five percent (25%) of the samples taken were classified as cohesive fine-grained soils, which were comprised of: 1) Silt (A-4b, 2% of samples); 2) Silt and Clay (A-6a, 5% of samples); 3) Silty Clay (A-6b, 2% of sample); 4) Clay (A-7-6, 1% of samples), and 5) Sandy Silt (A-4a, 15% of samples).. In terms of consistency of the cohesive soils, the description varies from stiff to hard correlating to converted SPT-N values (N_{60}) ranging from 10 blows per foot (bpf) to refusal. Natural moisture contents ranged from 6 to 16 percent. Based on Atterberg limits test performed on representative samples of the cohesive soils, the liquid and plastic limit were estimated to range from 14 to 30 percent and from 13 to 22 percent, respectively.

Thirty-five percent (35 %) of the samples taken along SR-8 were classified as non-cohesive coarse-grained soils, which were comprised of: 1) Gravel and Stone Fragments with Sand (A-1-b, 7% of samples); 2) Gravel and Stone Fragments with Sand and Silt (A-2-4, 9% of samples); 3) Coarse and Fine Sand (A-3a, 15% of samples); and 4) non-cohesive Sandy Silt (A-4a, 4% of samples). With respect to the relative density of the non-cohesive granular soils, the material description varies from loose to very dense correlating to N_{60} values ranging from 6 bpf to refusal. The natural moisture content of granular soil ranged from 6 to 14 percent.

Forty percent (40%) of the remaining samples are classified as a combination of Claystone, Shale, and Sandstone.

4.2.2. Exist Ramp to I-76

This section consists of 19 borings, designated as B-004-0-99, R-009-0-99, P-012-0-20, P-032-0-20, E3-001-0-20 to E3-004-0-20., B-001-0-22, B-002-0-22, B-023-0-24, B-028-0-24, B-030-0-24, B-031-0-24, B-042-0-24, B-050-0-24, B-051-0-24, B-053-0-24, B-055-0-24.

Along the exit ramp to I-76, nineteen percent (19%) of the samples taken were classified as cohesive fine-grained soils, which were comprised of: 1) Silt and Clay (A-6a, 3% of samples); 2) Clay (A-7-6, 3% of samples), and 3) Sandy Silt (A-4a, 14% of soil samples). In terms of consistency of the cohesive soils, the description varies from stiff to hard correlating to converted SPT-N values (N_{60}) ranging from 17 blows per foot (bpf) to refusal. Natural moisture contents ranged from 8 to 16 percent. Based on Atterberg limits test performed on representative samples of the cohesive soils, the liquid and plastic limit were estimated to range from 17 to 30 percent and from 15 to 22 percent, respectively.

Thirty-five percent (35 %) of the samples taken along the exit ramp were classified as non-cohesive coarse-grained soils, which were comprised of: 1) Gravel and Stone Fragments with Sand (A-1-b, 8% of samples); 2) Coarse and Fine Sand (A-3a, 11% of samples); 4) non-cohesive Sandy Silt (A-4a, 5% of samples), and 5) Gravel and Stone Fragments with Sand and Silt (A-2-4, 11% of samples). With respect to the relative density of the non-cohesive soils, the material description varies from medium dense to very dense correlating to N_{60} values ranging from 30 bpf to refusal. The natural moisture content of granular soil ranged from 7 to 12 percent.

Forty-six percent (46%) of the remaining samples are classified as a combination of Sandstone and Siltstone.

4.2.3. Goodkirk Street

This section consists of 6 borings, designated as B-004-0-99, B-006-0-99, B-008-0-99, R-009-0-99, R-011-0-99, and R-013-0-99.

Along Goodkirk Street, thirty-eight percent (38%) of the samples taken were classified as cohesive fine-grained soils, which were comprised of: 1) Silt and Clay (A-6a, 4% of samples); and 2) Sandy Silt (A-4a, 34% of samples). In terms of consistency of the cohesive soils, the description varies from medium stiff to very stiff correlating to converted SPT-N values (N_{60}) ranging from 7 blows per foot (bpf) to 19 bpf. Natural moisture contents ranged from 14 to 17 percent. Based on Atterberg limits test performed on representative samples of the cohesive soils, the liquid and plastic limit were estimated to range from 18 to 27 percent and from 12 to 15 percent, respectively.

Eight percent (8%) of the samples taken along Goodkirk Street were classified as non-cohesive coarse-grained soils, which were comprised of 1) Gravel and Stone Fragments with Sand and Silt (A-2-4, 8% of

samples). With respect to the relative density of the non-cohesive granular soils, the material is described as medium dense correlating to N_{60} values ranging from 22 bpf to 23 bpf. The natural moisture content of granular soil was 11 percent.

Fifty-four percent (54%) of the remaining samples are classified as Sandstone.

4.3. Subsurface Conditions

The subsurface conditions encountered during NEAS's explorations are described in the following subsections and/or on each boring log presented in Appendix B. The boring logs represent NEAS's interpretation of the subsurface conditions encountered at each boring location based on our site observations, field logs, 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 logs 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, and consideration of the geological history of the site.

4.3.1. SR-8 Bridge Over Abandoned Beacon Street

At the bridge site of SR-8 over Beacon Street, fill soils were encountered at the rear abutment location in boring B-003-0-22, immediately beneath the pavement section, and extended 15.5 ft below ground surface (bgs). At the rear abutment, the fills were encountered between elevations 1063 ft and 1047.5 ft. However, at the forward abutment and center pier locations, no fills were encountered during our subsurface exploration. The fills consisted of Stone Fragments with Sand (A-1-b), Stone Fragments with Sand, Silt and Clay (A-2-6), and Sandy Silt (A-4a). The cohesive fills can be described as having a stiff to hard consistency based on N_{60} values between 5 and 15 bpf and unconfined compressive strengths (estimated by means of hand penetrometer) between approximately 4.25 and 4.50 tons per square foot (tsf). Natural moisture contents of the cohesive fills ranged from 10 to 16 percent. Based on Atterberg Limit tests performed on representative samples of the cohesive fills, the liquid and plastic limits ranged from 21 to 23 percent and 14 to 15 percent, respectively. These non-cohesive fills are described as having a relative compactness of medium dense correlating to N_{60} values between 12 and 29. The natural moisture content of the non-cohesive soils ranged from 4 to 16 percent.

Below the fills, the subsurface soils encountered primarily consisted of cohesive fine-grained soils, with non-cohesive coarse-grained soils present secondarily. At the rear abutment, cohesive soils were encountered between elevations 1047.5 ft and 1035.5 ft, and between 1024.7 ft and 1019.7 ft. At the center pier location, cohesive soils were encountered between elevations 1026.3 ft and 1019.3 ft, and between 1016.3 ft and 1013.3 ft. At the forward abutment location, cohesive soils were encountered between elevations 1041.8 ft and 993.8 ft. The remaining soils consisted of granular materials. The cohesive glacial till soils are classified on the boring logs as Sandy Silt (A-4a), Silt (A-4b), and Silt and Clay (A-6a). The cohesive soils can be described as being a stiff to hard consistency based on N_{60} values between 7 bpf and refusal and unconfined compressive strengths (estimated by means of hand penetrometer) between approximately 1.50 and 4.50 tons per square foot (tsf). Natural moisture contents of the cohesive soils ranged from 13 to 23 percent. Based on Atterberg Limit tests performed on representative samples of the cohesive fills, the liquid and plastic limits ranged from 23 to 34 percent and 16 to 20 percent, respectively. The non-cohesive glacial till soils encountered are classified on the boring logs as Stone Fragments with Sand (A-1-b), Stone Fragments with Sand, Silt and Clay (A-2-6), Coarse and Fine Sand (A-3a), non-plastic Sandy Silt (A-4a), and non-plastic Silt (A-4b). These non-cohesive soils are described as having a relative

compactness of loose to very dense correlating to N_{60} values between 8 bpf and refusal. The natural moisture content of the non-cohesive soils ranged from 4 to 34 percent.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. Groundwater was observed in only one of the project borings (B-004-0-22) during drilling, at a depth of 15 ft bgs, corresponding to an elevation of 1018.3 ft amsl. It should be noted that groundwater levels can be influenced by various hydrologic factors in the area and may fluctuate from those measured at the time of the exploration.

Bedrock was encountered in all three structure borings at the site, with depths ranging from 993.8 ft to 1017.5 ft. The bedrock is described as slightly weathered, weak to slightly strong Shale. The recovery of shale ranges from 83% to 91%, and the RQD of the shale ranged from 0% to 62%.

4.3.2. SR-8 Bridge Over Exchange Street

At the bridge site of SR-8 over Exchange Street, granular soils were encountered at the rear abutment location in boring GB-001-0-99 immediately beneath the topsoil, extending between elevations 1021.9 ft and 1015.4 ft. The non-cohesive glacial till soils encountered were classified on the boring logs as Coarse and Fine Sand (A-3a) and Sandy Silt (A-4a). These soils are described as having a relative compactness ranging from loose to medium dense, based on N_{60} values between 8 bpf and 10 bpf. The natural moisture content of the non-cohesive soils was approximately 12 percent. Underlying the granular soils, cohesive soils were encountered between elevations 1015.4 ft and 1012.9 ft. The cohesive glacial till soils were classified on the boring logs as Silt and Clay (A-6a). Based on N_{60} values of 34 bpf to refusal and unconfined compressive strengths (estimated using a hand penetrometer) of approximately 2.25 tons per square foot (tsf), the cohesive soils are described as having a very stiff consistency.

At the bridge site of SR-8 over Exchange Street, fill soil was encountered at the forward abutment location in boring GB-002-0-99 immediately beneath the topsoil, extending between elevations 1022.2 ft and 1015.5 ft. The upper fill consisted of Sandy Silt (A-4a) between elevations 1022.5 ft and 1020.5 ft. Beneath this layer, the fill consisted of shale, sandstone, and concrete fragments between elevations 1020.5 ft and 1015.5 ft. Based on N_{60} values ranging from 12 bpf to refusal, the upper granular fill can be described as medium dense. The natural moisture content of the upper granular fill was approximately 12 percent.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. Groundwater was observed in one of the historical borings (GB-001-0-99) at a depth of 8.5 ft bgs, corresponding to an elevation of 1014.9 ft amsl. It should be noted that groundwater levels can be influenced by various hydrologic characteristics in the area and may vary from those measured during exploration.

Bedrock was encountered in both historical borings at the site, ranging from elevations 1012.9 ft to 1015.5 ft. The bedrock is described as moderately weathered, soft to moderately hard Shale. The RQD of shale ranges from 13% to 88%.

4.3.3. Carroll Street Bridge Over SR-8

At the Carroll Street Bridge over SR-8, the subsurface soils encountered consisted primarily of non-cohesive, coarse-grained soils. Cohesive soils were only encountered at the rear abutment between the elevations 1057.1 ft and 1050.1 ft, and at the forward abutment between elevations 1033.9 ft and 1032.4 ft. The remaining soils were all granular materials. The cohesive glacial till soils are classified on the boring logs as Sandy Silt (A-4a). The cohesive soils can be described as having a stiff to very stiff consistency based on N_{60} values of 21 bpf and refusal and unconfined compressive strengths (estimated by means of

hand penetrometer) between 1.5 and 3.0 tons per square foot (tsf). The natural moisture content of the non-cohesive soils is between 12 and 14 percent. The non-cohesive glacial till soils encountered are classified on the boring logs as Stone Fragments with Sand and Silt (A-2-4) and Coarse and Fine Sand (A-3a). These non-cohesive soils are described as having a relative compactness of medium dense to very dense correlating to N_{60} values between 13 bpf and refusal. The natural moisture content of the non-cohesive soils is between 10 and 23 percent.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. No groundwater was observed in any of the project borings. However, it should be noted that groundwater levels are influenced by various hydrologic factors in the area and may fluctuate over time.

Bedrock was encountered in all three borings at the site, at depths ranging from elevations 1031.3 ft to 1048.7 ft. The bedrock is described as slightly to moderately weathered, slightly to moderately strong Sandstone. The unconfined compressive strength of the sandstone ranges from 2,600 psi to 6,587 psi. The RQD of the shale ranges from 72% to 90%.

4.3.4. Braid Bridge

At the proposed Braid Bridge, the subsurface soils encountered consisted primarily of non-cohesive, coarse-grained materials. Cohesive soils were only encountered in boring B-025-0-24, between elevations 1049.7 ft and 1046.9 ft. The remaining soils consisted entirely of granular materials. The cohesive glacial till soils are classified on the boring log as Sandy Silt (A-4a). The cohesive soils can be described as having a hard consistency based on N_{60} value of 13 bpf and unconfined compressive strengths (estimated by means of hand penetrometer) of approximately 4.50 tons per square foot (tsf). The non-cohesive glacial till soils encountered are classified on the boring logs as Stone Fragments with Sand (A-1-b), Stone Fragments with Sand and Silt (A-2-4), Coarse and Fine Sand (A-3a) and Sandy Silt (A-4a). These non-cohesive soils are described as having a relative compactness of medium dense to very dense correlating to N_{60} values between 19 bpf and refusal. The natural moisture content of the non-cohesive soils is between 8 and 12 percent.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. No groundwater was observed in any of the project borings. However, it should be noted that groundwater levels are influenced by various hydrologic factors in the area and may fluctuate over time.

Bedrock was encountered in all four project borings at depths ranging from elevations 1033.7 ft to 1041.4 ft. The bedrock is described as slightly to moderately weathered, slightly to moderately strong Sandstone. The unconfined compressive strength of the sandstone ranges from 1,752 psi to 6,808 psi. The RQD of the shale ranges from 62.8% to 82%. In boring log B-024-0-24, a 2.5-ft-thick shale layer was interbedded with sandstone between elevations 1026.7 ft and 1024.2 ft. The shale is described as gray, moderately weathered, and weak.

4.3.5. Retaining Wall 1

At the proposed RW 1 site, 1-ft-thick layer of uncontrolled fill was encountered at the beginning of RW1 in both historical borings E3-003-0-20 and E3-004-0-20. Below the fill and pavement section, the subsurface soils consisted of both cohesive fine-grained soils and non-cohesive coarse-grained soils. At the beginning (south) of RW 1, cohesive soils were encountered between elevations between 1097.3 ft and 1083 ft. At the end (north) of RW 1, non-cohesive soils were encountered between elevations 1089 ft and 1086.5 ft and between 1090.6 ft and 1085.6 ft. The cohesive glacial till soils are classified in the boring logs as Sandy Silt (A-4a) and Silt (A-4b). The cohesive soils can be described as having a stiff to hard

consistency based on N_{60} values between 29 bpf and refusal and unconfined compressive strengths (estimated by means of hand penetrometer) between approximately 3.50 and 4.50 tons per square foot (tsf). Natural moisture contents of the cohesive soils ranged from 8 to 14 percent. Based on Atterberg Limit tests performed on representative samples of the cohesive soils, the liquid and plastic limits ranged from 24 to 29 percent and 15 to 21 percent, respectively. The non-cohesive glacial till soils encountered are classified on the boring logs as Stone Fragments with Sand (A-1-b), and non-cohesive Sandy Silt (A-4a). These non-cohesive soils are described as having a relative compactness of dense to very dense correlating to N_{60} values between 35 bpf and 52 bpf. The natural moisture content of the non-cohesive soils ranged from 7 to 8 percent.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. Groundwater was observed in only one historical borings (E3-004-0-20) during drilling, at a depth of 48 ft bgs, corresponding to an elevation of 1050.3 ft amsl. However, groundwater levels are affected by various hydrologic factors and may fluctuate over time.

Bedrock was encountered in all four structure borings at depths ranging from elevations 1083 ft to 1087.3 ft. The bedrock consists of Sandstone, Shale and Siltstone. Bedrock recovery at the RW1 site ranges from 39% to 100%, with RQD values between 0% and 42%.

4.3.1. Retaining Wall 2

At the proposed RW 2 site, a 22-ft-thick layer of fill was encountered in boring B-033-0-24; no fill soils were encountered in other areas of RW 2. Below the fill and pavement section, the subsurface soils consisted of both cohesive fine-grained soils and non-cohesive coarse-grained soils. The cohesive glacial till soils are classified on the boring logs as Sandy Silt (A-4a), Silt (A-4b), Silt and Clay (A-6a), Silty Clay (A-6b) and Clay (A-7-6). These soils exhibit a stiff to hard consistency, with N_{60} values ranging from 5 bpf to 103 bpf and estimated unconfined compressive strengths (estimated using a hand penetrometer) between approximately 3.50 and 4.50 tons per square foot. The natural moisture contents of the cohesive soils ranged from 9% to 29%. Atterberg Limit tests performed on representative samples indicate liquid limits ranging from 20% to 52% and plastic limits ranging 15% to 28%. The non-cohesive glacial till soils encountered are classified in the boring logs as Coarse and Fine Sand (A-3a), Fine Sand (A-3), Stone Fragments with Sand (A-1-b), Gravel and Stone Fragments with Sand and Silt (A-2-4), Gravel with Sand, Silt and Clay (A-2-6), non-cohesive Sandy Silt (A-4a), and non-cohesive Silt (A-4b). These non-cohesive soils are described as having a relative compactness of loose to very dense correlating to N_{60} values between 6 bpf and 60 bpf. The natural moisture content of the non-cohesive soils ranged from 5 to 18 percent.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. Groundwater was observed in two structure borings at the RW2 site (B-035-0-24 and B2-001-0-20) during drilling. Groundwater was encountered at a depth of 14 ft bgs in boring B-035-0-24 (at an elevation of 993.1 ft amsl) and at a depth of 33 ft bgs in boring B2-001-0-20 (at an elevation of 1031.1 ft amsl). However, groundwater levels are influenced by various hydrologic factors and may fluctuate over time.

Bedrock was not encountered in only two structure borings (B-035-0-24 and B2-001-0-20). In all other structure borings, bedrock was encountered at elevations ranging from 999.9 ft to 1054.2 ft. The bedrock consists of slightly weathered Shale and Siltstone. Bedrock recovery at the RW2 site ranges from 82% to 100%, with RQD values between 18% and 86%.

4.3.2. Retaining Wall 3

At the proposed RW 3 site, the subsurface soils encountered below the pavement section consisted of both cohesive fine-grained soils and non-cohesive coarse-grained soils. The cohesive glacial till soils are classified in the boring logs as Sandy Silt (A-4a) and were encountered at depths of 0.5 ft to 4.5 ft and 12.0 to 14.5 ft below ground surface. These soils exhibit a stiff to very stiff consistency, with N_{60} values ranging from 16 bpf to 18 bpf and estimated unconfined compressive strengths (determined using a hand penetrometer) between approximately 0 and 1.50 tons per square foot. The natural moisture content of the cohesive soils ranged from 13% to 17%. Atterberg Limit tests performed on representative samples indicate liquid limits ranging from 24% to 26% and plastic limits 16% to 20%. The non-cohesive glacial till soils encountered are classified in the boring logs as Stone Fragments with Sand and Silt (A-2-4), which was encountered at 4.5 ft to 12.0 ft and 14.5 to 20.0 ft below ground surface (bgs). These soils exhibit a relative compactness ranging from medium dense to very dense, with N_{60} values between 16 bpf and 52 bpf. The natural moisture content of the non-cohesive soils ranged from 11% to 15%.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. Groundwater was not observed in the structure borings at the RW3 site during drilling. However, groundwater levels are influenced by various hydrologic factors and may fluctuate over time.

Bedrock was encountered in the structure boring at a depth of 20.0 ft below ground surface, at an approximate elevation of 1014.3 ft. The bedrock consists of gray to dark gray shale. Bedrock recovery at the RW3 site is 100%, with a RQD of 78%.

4.3.3. Retaining Wall 4

At the proposed RW 4 site, an 8 ft deep fill layer was encountered at the beginning of RW4 in historical boring GB-002-0-99. The upper 3 ft of this fill consisted of Sandy Silt (A-4a), while the lower 5 ft contained shale, sandstone, concrete, and gravel. No fill was encountered in structure borings B-040-0-24 to B-041-0-24. Beneath the fill and pavement section, the subsurface soils encountered consisted of both cohesive fine-grained soils and non-cohesive coarse-grained soils. The cohesive soils were found between elevations 1032.7 ft and 1027.2 ft, while the non-cohesive soils were presented between elevations 1016.1 ft and 1027.2 ft. The cohesive glacial till soils, classified as Sandy Silt (A-4a), exhibited a very stiff consistency, with N_{60} values ranging from 7 bpf to 21 bpf. Unconfined compressive strengths, estimated using hand penetrometer, ranged from approximately 2.75 and 3.00 tons per square foot (tsf). The natural moisture contents of the cohesive soils ranged from 11% to 15%. Atterberg Limit tests performed on representative samples indicated liquid limits of 24% to 25% and plastic limits of 16%. The non-cohesive glacial till soils encountered are classified in the boring logs as Stone Fragments with Sand (A-1-b), and Gravel with Sand and Silt (A-2-4). These non-cohesive soils are described as having a relative compactness of medium dense to very dense correlating to N_{60} values between 19 bpf and refusal. The natural moisture content of the non-cohesive soils ranged from 6% to 8%.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. No groundwater was observed in any of the nearby borings. However, groundwater levels are influenced by various hydrologic factors and may fluctuate over time.

Bedrock was encountered in all four structure borings at the site, at elevations ranging from 1016.1 ft to 1025.8 ft. The bedrock consisted of highly to moderately weathered Sandstone and Shale. Bedrock recovery at the RW4 site ranged from 30% to 90%, with RQD values between 13% and 70%.

4.3.4. Retaining Wall 5

At the proposed RW 5 site, the subsurface soils encountered consisted of both cohesive fine-grained soils and non-cohesive coarse-grained soils. Cohesive soils were encountered only in boring B-043-0-24, between elevations 1034.3 ft and 1032.3 ft, while the rest of soils at the site consisted of granular materials. The cohesive glacial till soils, classified as Sandy Silt (A-4a), exhibited a hard consistency based on N_{60} value of 77 bpf. The natural moisture content of the cohesive soils was 10 percent. Based on Atterberg Limit tests performed on representative samples of the cohesive soils, the liquid and plastic limits were 25 percent and 19 percent, respectively. The non-cohesive glacial till soils encountered are classified on the boring logs as Coarse and Fine Sand (A-3a), and Gravel with Sand and Silt (A-2-4). These non-cohesive soils are described as having a relative compactness of dense to very dense correlating to N_{60} values between 49 bpf and refusal. The natural moisture content of the non-cohesive soils ranged from 5 to 10 percent.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. No groundwater was observed in any of the nearby borings. However, groundwater levels are influenced by various hydrologic factors and may fluctuate over time.

Bedrock was encountered in all three structure borings at the site at elevations ranging from 1032.3 ft to 1035.5 ft. The bedrock consists of Sandstone and interbedded Sandstone and Shale. Bedrock recovery at the RW5 site ranges from 93% to 99%, with RQD values between 68% and 78%. The unconfined compressive strength of the bedrock ranges from 1,752 psi to 6,775 psi.

4.3.5. Retaining Wall 6

At the proposed RW 6 site, the subsurface soils encountered beneath the pavement section consisted of both cohesive fine-grained soils and non-cohesive coarse-grained soils. Cohesive soils were encountered only in boring B-046-0-24, between elevations 1049.9 ft and 1045.4 ft, while the remaining soils consisted of granular materials. The cohesive glacial till soils are classified on the boring logs as Sandy Silt (A-4a). The cohesive soils can be described as having a stiff consistency based on N_{60} value of 3 bpf and unconfined compressive strengths (estimated by means of hand penetrometer) of 3.00 tons per square foot (tsf). Natural moisture contents of the cohesive soils was 16 percent. Based on Atterberg Limit tests performed on representative samples of the cohesive soils, the liquid and plastic limits was 23 percent and 17 percent, respectively. The non-cohesive glacial till soils encountered are classified on the boring logs as Stone Fragments with Sand (A-1-b), Gravel with Sand and Silt (A-2-4), Coarse and Fine Sand (A-3a), Fine Sand (A-3) and non-cohesive Sandy Silt (A-4a). These non-cohesive soils are described as having a relative compactness of dense to very dense correlating to N_{60} values between 10 bpf and refusal. The natural moisture content of the non-cohesive soils ranged from 5 to 15 percent.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. No groundwater was observed in any of the nearby borings. However, groundwater levels are influenced by various hydrologic factors and may fluctuate over time.

Bedrock was encountered in all three structure borings at elevations ranging from 1028.4 ft to 1035.4 ft. The bedrock consists of slightly to moderately weathered Sandstone and Shale. Bedrock recovery at the RW6 site ranges from 90% to 100%, with RQD values between 0% and 78%. The unconfined compressive strength of the bedrock was measured 5,536 psi.

4.3.6. Retaining Wall 7

At the proposed RW 7 site, the subsurface soils encountered below the pavement section consisted of only non-cohesive coarse-grained soils, which were encountered between elevations 1040.4 ft and 1031.6 ft. The

non-cohesive glacial till soils encountered are classified in the boring logs as Gravel with Sand and Silt (A-2-4), Coarse and Fine Sand (A-3a) and non-cohesive Sandy Silt (A-4a). These non-cohesive soils are described as having a relative compactness of very dense correlating to N_{60} values between 73 bpf and refusal. The natural moisture content of the non-cohesive soils ranged from 7% to 9%.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. No groundwater was observed in any of the nearby borings. However, groundwater levels are influenced by various hydrologic factors and may fluctuate over time.

Bedrock was encountered in both structure borings at elevations ranging from 1031.6 ft to 1037.4 ft. The bedrock consists of slightly to highly weathered Sandstone. The recovery of bedrock at the RW7 site ranges from 96% to 97%, with RQD values between 48% and 72%. The unconfined compressive strength of the bedrock ranged from 6,540 psi to 6,916 psi.

4.3.7. Retaining Wall 8

At the proposed RW 8 site, the subsurface soils encountered below the pavement section consisted of both cohesive fine-grained soils and non-cohesive coarse-grained soils. Cohesive soils were only encountered in boring B-050-0-24, between elevations 1038.1 ft and 1033.6 ft. The remaining soils were all granular materials. The cohesive glacial till soils are classified in the boring logs as Sandy Silt (A-4a). The cohesive soils can be described as having a hard consistency based on N_{60} value of 71 bpf. Natural moisture content of the cohesive soils was 8 percent. Based on Atterberg Limit tests performed on representative samples of the cohesive soils, the liquid and plastic limits was 25 percent and 17 percent, respectively. The non-cohesive glacial till soils encountered are classified on the boring logs as Stone Fragments with Sand (A-1-b) and Gravel with Sand and Silt (A-2-4). These non-cohesive soils are described as having a relative compactness of dense to very dense correlating to N_{60} values between 62 bpf and refusal. The natural moisture content of the non-cohesive soils ranged from 6% to 9%.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. No groundwater was observed in any of the nearby borings. However, groundwater levels are influenced by various hydrologic factors and may fluctuate over time in the area and may vary from those measured at the time of the exploration.

Bedrock was encountered in both structure borings at the site, ranging from elevations 1030.1 ft to 1033.7 ft. The bedrock consists of slightly to moderately weathered Sandstone. The recovery of bedrock at the RW8 site ranges from 96% to 97%, with an RQD of bedrock between 46% and 67%. The unconfined compressive strength of the bedrock was 4,620 psi.

4.3.8. Retaining Wall 9

At the proposed RW 9 site, below pavement section, subsurface soils encountered consisted of only non-cohesive coarse-grained soils between the elevation of 1031.5 ft and 1056.2 ft. The non-cohesive glacial till soils encountered are classified on the boring logs as Coarse and Fine Sand (A-3a) and non-cohesive Sandy Silt (A-4a). These non-cohesive soils are described as having a relative compactness of medium dense to very dense correlating to N_{60} values between 19 bpf and refusal. The natural moisture content of the non-cohesive soils ranged from 9 to 14 percent.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. No groundwater was observed in any of the nearby borings. However, groundwater levels are influenced by various hydrologic factors and may fluctuate over time.

Bedrock was encountered in both structure borings at the site, at elevations ranging from 1031.5 ft to 1046.2 ft. The bedrock encountered includes slightly to moderately weathered Sandstone. Bedrock recovery at the RW9 site ranges from 97% to 98%, and the RQD ranges from 59% to 83%. The unconfined compressive strength of the bedrock ranged from 4,942 psi to 4,874 psi.

4.3.9. Retaining Wall 10

At the proposed RW 10 site, below the pavement section, subsurface soils encountered consisted of both cohesive fine-grained soils and non-cohesive coarse-grained soils. The cohesive soils were only encountered in boring B-056-0-24 between elevations 1051.1 ft and 1048.6 ft, while the remaining soils consisted of granular materials. The cohesive glacial till soils are classified on the boring logs as Sandy Silt (A-4a). The cohesive soils can be described as having a very stiff consistency based on N_{60} value of 27 bpf. Natural moisture content of the cohesive soils was 17 percent. Based on Atterberg Limit tests performed on representative samples of the cohesive soils, the liquid and plastic limits were 27 percent and 18 percent, respectively. The non-cohesive glacial till soils encountered are classified on the boring logs as Gravel with Sand and Silt (A-2-4) and non-cohesive Sandy Silt (A-4a). These non-cohesive soils are described as having a relative compactness of medium dense to very dense correlating to N_{60} values between 13 bpf and refusal. The natural moisture content of the non-cohesive soils ranged from 7 to 17 percent.

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. No groundwater was observed in any of the nearby borings. However, groundwater levels are influenced by various hydrologic factors and may fluctuate over time.

Bedrock was encountered in both structure borings at the site, ranging from elevations 1031.9 ft to 1056.2 ft. The bedrock encountered includes unweathered to slightly weathered Sandstone. Bedrock recovery at the RW10 site ranges from 97% to 98%, and the RQD of the bedrock ranges from 72% to 92%. The unconfined compressive strength of the bedrock ranged from 5,520 psi to 5,541 psi.

5. ANALYSES AND RECOMMENDATIONS

5.1. Subgrade Analysis

NEAS understands that Arcadis U.S., Inc. is working with the ODOT to develop construction plans for a roadway improvement project (SUM-77/SR8-11.65/0.00 project, PID: 114865) along IR-77 just north of Lovers Lane to SR-8 and along SR-8 from IR-77 to just north of Perkins Street in Summit County, Ohio. The subgrade exploration and subsequent analyses were completed for the referenced project. The subgrade analysis was performed in accordance with ODOT's GDM criteria utilizing the ODOT provided subgrade analysis spreadsheet (RoadwayAnalysis.xls, version 14.81 dated July 21, 2025). Input information for the spreadsheet was based on the soil characteristics gathered during NEAS's exploration (i.e., SPT results, laboratory test results, etc.).

Subgrade analyses were performed to identify the method, location, and dimensions (including depth) of required subgrade stabilization for the project. In addition to identifying stabilization recommendations, pavement design parameters are also determined to aid in pavement section design. The subsections below present the results of our subgrade analyses including pavement design parameters and unsuitable subgrade conditions identified within the project limits. Subgrade analysis spreadsheets are provided in Appendix C.

It should be noted that for the purposes of this report and our analysis, the term 'proposed subgrade' has been assumed to represent soils and/or soil conditions extending to a depth of 7.5 ft below the bottom of proposed pavement section (i.e., top of subgrade).

5.1.1. Pavement Design Recommendations

It is our understanding that pavement analysis and design is to be performed to determine the proposed pavement section within the project limits to undergo the proposed improvements. A subgrade analysis was performed using the subgrade soil data obtained during our field exploration program to evaluate the soil characteristics to develop pavement parameters for use in pavement design. The subgrade analysis parameters recommended for use in pavement design are presented in Table 6 below. Provided in the table are ranges of maximum, minimum and average N_{60L} values for the indicated segments as well as the design CBR value recommended for use in pavement design.

Table 6: Pavement Design Parameters

SUM-077/008-11.65/00.00 Subgrade Analysis Summary					
Segment	Maximum N_{60L}	Minimum N_{60L}	Average N_{60L}	Average PI Values	Design CBR
SR-8	30	6	22	6	11
Exit ramp to I-76	30	9	28	8	12
GoodKirk Street	22	7	14	7	9
Entire Project	30	6	23	6	11

5.1.2. Unsuitable Soils

Per ODOT's GDM, the presence of select subgrade conditions are prohibited within the subgrade zone for new pavement construction. These prohibited subgrade conditions generally include the presence of rock, specific soil types, and soils with a liquid limit greater than 65 percent. With respect to the proposed improvement project these subgrade conditions are further discussed in the following subsections.

5.1.2.1. Rock

Rock was encountered at or close to subgrade elevation at various boring locations performed within the project limits. Per ODOT's GDM, if rock is encountered within 6 inches of the final subgrade elevation, it is to be removed in accordance with 204.05 of the ODOT CMS and replaced with suitable material conforming to 204.02 Granular Material Type B, with the gradation of 703.17.

5.1.2.2. Prohibited Soils

Prohibited soil types per the GDM include A-4b, A-2-5, A-5, A-7-5, A-8a, A-8b, and soils with liquid limits greater than 65. Prohibited soil type A-4b was encountered in one or more project boring, along the project limits. PER ODOT's GDM, when A-4b encountered in natural ground or an existing embankment within 3 feet of the subgrade, A-4b should be removed and excavated to 36 inches below the subgrade and replaced with item 204 embankment.

All unsuitable material encountered along the project limits are summarized in Table 7 below.

Table 7: Unsuitable Materials Summary

Boring ID	Prohibited Soil Type	Depth Below Subgrade (ft)
SR-8		
P-015-0-20	Rock	0.0 - 7.2
P-017-0-20	A-4b	0.0 - 1.9
P-024-0-20	A-4b	1.9 - 3.4
R-012-0-99	Rock	0.0 - 4.2
B-018-0-22	Rock	0.8 - 5.5
B-010-0-22	Rock	1.9 - 6.7
B-020-0-22	Rock	0.0 - 4.6
B-013-0-22	Rock	1.2 - 5.8
B-028-0-24	Rock	1.2 - 3.9
B-055-0-24	Rock	1.0 - 3.6
Goodkirk Street		
R-009-0-99	Rock	0.0 - 1.5
Exit Ramp to I-76		
E3-001-0-20	Rock	0.0 - 7.5
E3-002-0-20	Rock	0.0 - 7.5
E3-003-0-20	Rock	0.0 - 7.5
E3-004-0-20	Rock	0.0 - 7.5
B-001-0-22	Rock	0.0 - 7.5
B-002-0-22	Rock	0.0 - 7.5
B-004-0-99	Rock	0.0 - 0.7
R-009-0-99	Rock	0.0 - 6.0
B-028-0-24	Rock	1.0 - 3.7
B-055-0-24	Rock	0.9 - 3.5

5.1.3. Unstable Soils

The GDM recommends subgrade stabilization for soils in which the N_{60} value of a particular soil sample (SS) at a referenced boring location is less than 12 bpf and in some cases less than 15 bpf (i.e., where moisture content is greater than optimum plus 3 percent). Based on the specific N_{60} value at the subject boring, *Figure B - Subgrade Stabilization* within the subgrade analysis recommends a depth of subgrade stabilization for ODOT standard stabilization methods. For the purposes of this report, the term 'weak soils' has been assumed to represent subgrade soils of these conditions. It should be noted that although a soil sample's N_{60} value may meet the criteria to be considered a weak soil, the depth in which the weak soil is encountered in relation to the proposed subgrade is considered when each individual subgrade boring is analyzed. For example, if the GDM recommends an excavate and replace of 12 inches within a weak soil underlying 18 inches of stable material, it would be unreasonable to recommend the removal of both the stable and unstable material for a total of 30 inches of excavate and replace.

Information on the boring location where weak soils were encountered and determined to have a potential impact on subgrade performance is shown in Table 8 below.

Table 8: Unstable (Weak) Soils Location Summary

Boring ID	N _{60L}	Depth Below Subgrade (ft)
SR-8		
R-006-0-99	10	0.0 - 2.5
P-019-0-20	6	1.9 - 3.4
P-028-0-20	10	2.9 - 4.4
Goodkirk street		
R-013-0-99	7	0.0 - 1.3
B-008-0-99	9	0.0 - 1.5
Exit Ramp to I-76		
B-031-0-24	9	0.9 - 2.4

5.1.3.1. High Moisture Content Soils

High moisture content soils are defined by the GDM as soils that exceed the estimated optimum moisture content (per *Figure A - Optimum Moisture Content* within the GDM) for a given classification by 3 percent or more. Per the GDM, soils determined to be above the identified moisture content levels are a likely indication of the presence of an unstable subgrade and may require some form of subgrade stabilization. Similar to our analysis of weak soils, although a soil sample's moisture content may meet the criteria to be considered high, the depth in which the high moisture soil is encountered in relation to the proposed subgrade is considered when each individual subgrade boring is analyzed for stabilization recommendations. Based on the subsurface exploration performed, high moisture content soils within the proposed subgrade of the project were encountered in borings as shown in Table 9 below.

Table 9: High Moisture Content Soils Summary

Boring ID	Sample ID	Soil Type	Moisture Content (%)	Optimum Moisture Content (%)	Moisture Above Optimum (%)	Depth Below Subgrade (ft)
SR-8						
R-005-0-99	SS-1	A-4a	14	10	4	0.0 - 2.0
P-012-0-20	SS-1	A-4a	16	10	6	0.0 - 1.1
P-013-0-20	SS-1	A-4a	14	11	3	0.0 - 0.9
P-022-0-20	SS-1	A-4a	13	10	3	0.0 - 1.1
P-028-0-20	SS-2	A-4a	14	11	3	1.9 - 3.4
Goodkirk Street						
B-004-0-99	SS-1	A-4a	14	10	4	0.0 - 1.0
R-011-0-99	SS-1	A-4a	14	10	4	0.0 - 1.7
R-013-0-99	SS-1	A-4a	15	10	5	0.0 - 1.3

5.1.4. Stabilization Recommendations

Based on our subgrade analysis, unstable materials, including weak soils and those with high moisture content, were encountered throughout the project limits. These soils exhibited an N₆₀ value below 12 bpf and/or a moisture content exceeding the optimum level by more than 3 percent within 3 feet of subgrade. Unsuitable soils, including rock and prohibited soils were encountered throughout the project limits as well. According to ODOT GDM Section 605, "For all Interstates and other divided highways with four or more lanes more than 1-mile in project length, chemically stabilize the subgrade for the entire project (global stabilization)". Also, according to ODOT GDM Section 609, "When global chemical stabilization includes areas where shallow bedrock is anticipated to be encountered during the stabilization process, excavate bedrock to a depth of 18 inches below the top of proposed subgrade. Replace the excavation with Item 204 Embankment and then proceed with chemical stabilization. The Item 204 Embankment will need to be

tested for sulfate content using the area/volume guidance in Supplement 1120.03.”. Therefore, NEAS recommends undercutting and replacing rock encountered within 6 inches of the final subgrade elevation with suitable material conforming to Item 204.02, Granular Material Type B, in accordance with the gradation requirements of 703.17. Limits of undercut and replacement for rock are provided in Table 10.

Table 10: The undercut and replacement area for rock

Start Station	End Station	Problem		Excavate and Replace w/ Item 204		Borings Considered
		Unsuitable	Unstable	Unsuitable	Unstable	
SR-8						
326+00	327+96	Rock	-	6	-	P-015-0-20
360+41	363+88	Rock	-	6	-	R-012-0-99, B-028-0-24, B-055-0-24
373+35	376+02	Rock	-	6	-	B-018-0-22, B-010-0-22
387+48	END	Rock	-	6	-	B-020-0-22, B-013-0-22
Exit Ramp to I-76						
101+83	110+66	Rock	-	6	-	E3-001-0-20, E3-002-0-20, E3-003-0-20, E3-004-0-20, B-001-0-22 & B-002-0-22
132+46	133+78	Rock	-	6	-	B-004-0-99
141+93	END	Rock	-	6	-	B-028-0-24, B-055-0-24

Global chemical stabilization of the project subgrade using cement to a depth of 12 inches is recommended, except along SR-8 where unsuitable A-4b material is encountered. In these areas, the stabilization depth should be increased to 14 inches from Station 329+96 to Station 331+97 and from Station 342+47 to Station 344+64.

Remove rock, shale or coal to 12 inches beyond the edge of the surface of the pavement, paved shoulders, or paved medians, including under new curbs and gutters. Stabilization limits should extend 18-inches beyond the edge of the proposed paved roadway, shoulder or median and it is recommended removing any topsoil, existing pavement materials or abandoned structure foundation materials.

The mix design should be conducted in accordance with ODOT's CMS Supplement 1120 (Mixture Design for Chemically Stabilized Soils). For design purposes it may be assumed that the cement addition will be 5% using the following formula.

$$\text{Cement: } C = 0.75 \times T \times 115 \times 0.05$$

Where:

C = amount of chemical in pounds / square yard and

T = thickness of the treatment zone in inches

A dry density of 115-pounds per cubic foot (pcf) is assumed.

It should be noted that per ODOT's GDM, *typical chemical stabilization equipment cannot stabilize areas less than 8 ft in width*. If it is anticipated that the project will require multiple maintenance of traffic phases, it is recommended that the roadway work is coordinated with the maintenance of traffic schemes in such a way that an 8-ft minimum width for chemical stabilization exists. If areas of less than 8 ft in width are

anticipated, subgrade soils may be excavated out, mixed with stabilization chemical, and compacted in place, though this method is not practical for large areas.

NEAS's opinion that the subgrade soils will provide adequate pavement support assuming it is designed and constructed in accordance with the recommendations provided within this report, as well as all applicable ODOT standards and specifications.

5.2. SR-8 Bridge Over Abandoned Beacon Street

It is our understanding that the bridge carrying SR-8 over Beacon Street is planned for removal and will be backfilled with soil to match the proposed roadway. The existing structure is a continuous steel beams bridge with composite reinforced deck, supported by wall-type semi-integral abutments and a cap-and-column pier. A precast reinforced concrete box culvert will be constructed beneath the existing span 1 to accommodate a bike trail. The proposed culvert will have a 15-foot span and a 12-foot rise.

5.2.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 11 and 13 below.

Table 11: B-003-0-22 soil profile for analysis

Bridge Over Beacon Street : Soil Profile B-003-0-22							
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)	Setup Factor (f_{su})
Gravel with Sand Depth (1063 ft - 1060 ft)	118	118	128	-	-	40	1.00
Sandy Silt Depth (1060 ft - 1057 ft)	110	110	120	1600	150	24	1.50
Gravel with Sand Depth (1057 ft - 1055 ft)	112	112	122	-	-	35	1.00
Sandy Silt Depth (1055 ft - 1050 ft)	110	110	120	1400	150	24	1.50
Gravel with Sand, Silt and Clay Depth (1050 ft - 1047.5 ft)	118	118	128	-	-	34	1.20
Silt and Clay Depth (1047.5 ft - 1035.5 ft)	112	112	122	1900	200	24	1.50
Gravel with Sand Depth (1035.5 ft - 1032 ft)	122	122	132	-	-	40	1.00
Gravel with Sand Depth (1032 ft - 1024.7 ft)	112	112	122	-	-	33	1.00
Silt and Clay Elevation (1024.7 ft - 1019.7 ft)	108	108	118	850	100	22	1.50
Gravel with Sand, Silt and Clay Elevation (1019.7 ft - 1017.5 ft)	122	122	132	-	-	38	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N_{160-52} , else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 12: B-004-0-22 soil profile for analysis

Bridge Over Beacon Street: Soil Profile B-004-0-22							
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)	Setup Factor (f_{su})
Silt Depth (1033.3 ft - 1026.3 ft)	112	112	122	2350	200	26	1.50
Sandy Silt Depth (1026.3 ft - 1019.3 ft)	112	112	122	1800	200	25	1.50
Gravel with Sand Depth (1019.3 ft - 1016.3 ft)	120	110	120	-	-	33	1.00
Sandy Silt Depth (1016.3 ft - 1013.3 ft)	122	112	122	-	-	30	1.20

Notes:
1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405.
2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used.
3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.

Table 13: B-005-0-22 soil profile for analysis

Bridge Over Beacon Street: Soil Profile B-005-0-22							
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)	Setup Factor (f_{su})
Gravel with Sand Depth (1054.8 ft - 1050.3 ft)	120	120	130	-	-	40	1.00
Sandy Silt Depth (1050.3 ft - 1048.8 ft)	115	115	125	-	-	32	1.20
Gravel with Sand Depth (1048.8 ft - 1046.8 ft)	115	115	125	-	-	39	1.00
Coarse and Fine Sand Depth (1046.8 ft - 1041.8 ft)	115	115	125	-	-	34	1.00
Sandy Silt Depth (1041.8 ft - 1021.5 ft)	112	112	122	2250	200	26	1.50
Silt Depth (1021.5 ft - 1016.5 ft)	108	108	118	1000	100	23	1.50
Silt and Clay Depth (1016.5 ft - 1006.5 ft)	112	112	122	2300	200	25	1.50
Sandy Silt Depth (1006.5 ft - 1001.5 ft)	115	115	125	2850	250	26	1.50
Silt and Clay Elevation (1001.5 ft - 993.8 ft)	120	120	130	4600	350	27	1.50

Notes:
1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405.
2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used.
3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.

5.2.2. Bearing Resistance

Based on the site plan prepared by Gannett Fleming, the proposed box culvert (15' span x 12' rise) will be placed on the existing ground at an approximately elevation of 1033.5 ft. The culvert's bottom width is approximately 17.0 ft, with a total length of 231.0 ft. Bearing resistance analyses were performed at the abandoned Beacon Street culvert site. The estimated undrained factored bearing resistance is between 4.7 ksf and 5.9 ksf, while the estimated drained factored bearing resistance is between 4.7 ksf and 5.2 ksf. The bearing resistance analysis results can be found in Appendix D.

5.2.3. Settlement

The planned backfill of the abandoned Beacon Street will require approximately 26 feet of new fill. To estimate the maximum total and differential settlement that may occur within the subsurface soils, NEAS reviewed: 1) the proposed culvert site plan prepared by Gannett Fleming; 2) roadway cross sections; and 3) test borings and laboratory data developed as part of this report. Utilizing this information and the software entitled FoSSA 2.0 by ADAMA Engineering, Inc., a settlement model was developed and analyzed for both elastic (immediate) and consolidation (long term) settlement.

Based on our analysis, the ground surface is estimated to experience up to 3.60 inches of immediate settlement and up to 5.28 inches of long-term (consolidation) settlement due to the induced loads from the 26-foot-high new fill. Due to the similar fill heights along the proposed culvert and the relatively uniform

subsurface soil conditions on both sides of the culvert, differential settlement is expected to be minimal. The settlement analysis results can be found in Appendix D. Immediate settlement is expected to take place during construction, prior to bridge loading, and is not anticipated to be a concern. The estimated time for 90% consolidation at the project site is approximately 14 days following embankment construction. If the project schedule allows, delaying pavement construction until settlements have largely occurred should be considered to mitigate potential costs associated with reconstruction, repairs and maintenance.

To determine the appropriate waiting period before structure and pavement construction, we recommend implementing a settlement monitoring program. This program should confirm that settlements have dissipated to an acceptable level, as determined by the Geotechnical Engineer. Specifically, settlement platforms should be installed at the backfill site in accordance Section 1106.2, Site Monitoring and Confirmation Testing, of ODOT's Geotechnical Design Manual (dated July 15, 2024).

5.3. SR-8 Bridge Over Exchange Street

NEAS understands that the bridge (Structure File Number: 7700156) carrying SR-8 over Exchange Street is proposed to be widened, and a new bridge carrying Ramp CD over Exchange Street is proposed for construction. The proposed Ramp CD bridge will be a single-span steel beam bridge with a composite reinforced concrete deck supported by wall-type semi-integral abutments. The proposed bridge will be founded on spread footings.

5.3.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 14 and 15 below.

Table 14: GB-001-0-99 soil profile for analysis

Bridge Over Exchange Street: Soil Profile GB-001-0-99							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1023.4 ft - 1017.9 ft)	112	112	122	-	-	31	1.20
Coarse and Fine Sand (A-3a) Depth (1017.9 ft - 1015.4 ft)	112	112	122	-	-	32	1.00
Silt and Clay (A-6a) Depth (1015.4 ft - 1012.9 ft)	118	118	128	4250	300	26	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 15: GB-002-0-99 soil profile for analysis

Bridge Over Exchange Street: Soil Profile GB-002-0-99							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1023.5 ft - 1020.5 ft)	112	112	122	-	-	31	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

5.3.2. Bearing Resistance

Based on the site plan prepared by Gannett Fleming, the proposed bottom of footing elevations are 1011.0 ft at the rear abutment and 1013.4 ft at the forward abutment. Historical borings indicate that bedrock was encountered at 1012.9 ft and 1015.5 at the rear and forward abutment, respectively. Therefore, both abutments will be supported on bedrock. The encountered bedrock is described as moderately weathered, soft to moderately hard Shale. The RQD of the shale ranges from 13% to 88%.

Bearing resistance analyses were performed for the proposed bridge site, assuming an unconfined compressive strength of shale of 200 psi based on ODOT GDM Table 400-6. The estimated factored bearing resistance is between 50.6 ksf and 51.7 ksf. The bearing resistance analysis results can be found in Appendix D.

5.3.1. Settlement

Due to the presence of very shallow bedrock, settlement is not expected to be a concern.

5.4. Carroll Street Bridge Over SR-8

We understand that the bridge (Structure File Number: 7700180) carrying Carroll Street over SR-8 is proposed for replacement. The proposed bridge is a two-span continuous rolled steel beam bridge with composite reinforced concrete deck, supported by semi-integral abutments and a reinforced concrete cap-and-column pier. The proposed bridge will be founded on spread footings.

5.4.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 16 and 18 below.

Table 16: B-027-0-24 soil profile for analysis

Carroll Street Bridge: Soil Profile B-027-0-24							
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)	Setup Factor (f_{su})
Sandy Silt Depth (1057.1 ft - 1050.1 ft)	115	115	125	2650	250	26	1.50
Gravel with Sand and Silt Depth (1050.1 ft - 1048.7 ft)	122	122	132	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 17: B-028-0-24 soil profile for analysis

Carroll Street Bridge: Soil Profile B-028-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand and Silt Depth (1033.8 ft - 1031.3 ft)	122	122	132	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 18: B-029-0-24 soil profile for analysis

Carroll Street Bridge: Soil Profile B-029-0-24							
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)	Setup Factor (f_{su})
Coarse and Fine Sand Depth (1050.9 ft - 1038.9 ft)	115	115	125	-	-	34	1.00
Gravel with Sand and Silt Depth (1038.9 ft - 1033.9 ft)	120	120	130	-	-	39	1.20
Sandy Silt Depth (1033.9 ft - 1032.4 ft)	125	125	135	6250	400	29	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N ₁₆₀ <52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

5.4.2. Bearing Resistance

Based on the site plan prepared by Arcadis, the proposed bottom of footing elevations is 1028.85 ft at the rear abutment, 1027.5 ft at the interior center pier, 1028.2 ft at the exterior center pier, and 1028.90 ft at the forward abutment. The bedrock encountered is described as slightly to moderately weathered, slightly to moderately strong Sandstone. The unconfined compressive strength of sandstone ranges from 2,600 psi to 6,587 psi. The RQD of the sandstone ranges from 72% to 90%.

Bearing resistance analyses were performed for the proposed bridge site. The estimated factored bearing resistances are 123.6 ksf and 133 ksf at the rear abutment and rear abutment wingwall, respectively; 226.7 ksf to 228.5 ksf at the center pier; and 142.3 ksf and 151 ksf at the forward abutment and forward abutment wingwall, respectively. The bearing resistance analysis results are included in Appendix D.

5.4.3. Settlement

Due to the presence of very shallow bedrock, settlement is not expected to be a concern.

5.5. Braid Bridge

A new ramp bridge (No. SUM-77/8-1165/000) is proposed carrying the SB entrance ramp to SR-8 over the SB exit ramp to IR-7) between Vine Street and Orchard Street. The proposed structure will be a single span prestressed concrete I-beams bridge with composite reinforced concrete deck slab, supported by reinforced concrete full-height wall and cap-and-column abutments on spread footings.

5.5.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 19 and 22 below.

Table 19: B-023-0-24 soil profile for analysis

Braid Bridge: Soil Profile B-023-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand and Silt Depth (1043.3 ft - 1038.8 ft)	115	115	125	-	-	38	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 20: B-024-0-24 soil profile for analysis

Braid Bridge: Soil Profile B-024-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand and Silt Depth (1036.2 ft - 1033.7 ft)	118	118	128	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 21: B-025-0-24 soil profile for analysis

Braid Bridge: Soil Profile B-025-0-24							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1051.4 ft - 1046.9 ft)	110	110	120	1600	150	24	1.50
Sandy Silt (A-4a) Depth (1046.9 ft - 1044.4 ft)	115	115	125	-	-	35	1.20
Gravel with Sand (A-1-b) Depth (1044.4 ft - 1041.4 ft)	130	130	140	-	-	40	1.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 22: B-026-0-24 soil profile for analysis

Braid Bridge: Soil Profile B-026-0-24							
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)	Setup Factor (f_{su})
Coarse and Fine Sand Depth (1036.1 ft - 1030.8 ft)	122	122	132	-	-	40	1.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

5.5.2. Bearing Resistance

Based on the site plan prepared by Arcadis, the proposed bottom of footing elevations is 1035.65 ft at the rear abutment and 1034.5 ft at the forward abutment. Bedrock was encountered in the project borings between elevations 1033.7 ft and 1041.4 ft. The bedrock encountered is described as slightly to moderately weathered, slightly to moderately strong Sandstone. The unconfined compressive strength of sandstone ranges from 1,752 psi to 6,808 psi. The RQD of the sandstone ranges from 62.8% to 82%.

Bearing resistance analyses were performed at the proposed bridge site. The estimated factored bearing resistances are between 96.7 ksf and 140 ksf at the rear abutment, and between 96.9 ksf and 109.3 ksf at the forward abutment. The bearing resistance analysis results are included in Appendix D.

Due to stronger rock (unconfined compressive strength larger than 1500 psi) was encountered at the bridge site, sliding check can be non-performed according to ODOT GDM Section 1303.3.5.

5.5.3. Settlement

Due to the presence of very shallow bedrock, settlement is not expected to be a concern.

5.6. Retaining Wall 1

A foundation review was completed for the proposed Soldier Pile Lagging Wall – RW 1, located along the proposed Southbound Exit Ramp CD between STA. 105+26.63 and STA. 111+78.10. The following information was used in our design: 1) the soil/rock characteristics gathered during the subsurface exploration (i.e., SPT results, laboratory test results, etc.); and 2) the referenced Site Plan prepared by Gannett Fleming.

5.6.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The boring B-030-0-24 was not used in our design because B-30-0-24 was offset 35 ft east on SR-8 due to thick brush, steep embankment and homeless camp. Also, no survey information was collected for B-030-0-24. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 23 and 27 below.

Table 23: EB-003-0-20 soil profile for analysis

Retaining Wall 1: Soil Profile EB-003-0-20							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1093.2 ft - 1087.2 ft)	122	122	132	4900	375	28	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 24: EB-004-0-20 soil profile for analysis

Retaining Wall 1: Soil Profile E3-004-0-20							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1098.3 ft - 1089.8 ft)	125	125	135	5800	400	29	1.50
Silt (A-4b) Depth (1089.8 ft - 1087.3 ft)	125	125	135	6250	400	29	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 25: B-001-0-22 soil profile for analysis

Retaining Wall 1: Soil Profile B-001-0-22							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1093.5 ft - 1089 ft)	125	125	135	6250	400	29	1.50
Gravel with Sand (A-1-b) Depth (1089 ft - 1086.5 ft)	120	120	130	-	-	40	1.00
Silt (A-4b) Depth (1086.5 ft - 1083 ft)	118	118	128	3600	300	27	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

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Table 26: B-002-0-22 soil profile for analysis

Retaining Wall 1: Soil Profile B-002-0-22							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1090.6 ft - 1085.6 ft)	122	122	132	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 27: B-030-0-24 soil profile for analysis

Retaining Wall 1: Soil Profile B-030-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand (A-1-b) Depth (1071.3 ft - 1065.9 ft)	125	125	135	-	-	40	1.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Deep foundation elements subjected to horizontal loads and/or moments should be analyzed for maximum bending moment, maximum shear force and lateral deflection and should be checked whether the foundation element is structurally capable of resisting the lateral load. The required lateral load capacity can be obtained by increasing the diameter or the embedment depth of the foundation element. For the purpose of evaluating the shaft resistance in the lateral direction, drilled shaft has been analyzed by using the software entitled Lpile by Ensoft, Inc. The generalized soil/rock parameters, to be used to analyze the laterally loaded shaft by the p-y curve method for soil and rock strata are shown in Table 28 and Table 32, respectively.

Table 28: E3-003-0-20 Lpile Parameters

Retaining Wall 1 E3-003-0-20						
Soil Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Soil Class	LPILE p-y Model	Soil Strain Parameter ϵ_{50}	Soil Modulus Parameter p-y k (pci)
1	1093.2	1087.2	A-4a	Stiff Clay w/o Water	0.0042	1646
Rock Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Rock Type	LPILE p-y Model	krm	Rock Initial Modulus Ei (psi)
1	1087.2	1077.2	Sandstone	Weak Rock	0.00016	100000
2	1077.2	1043.2	Shale	Weak Rock	0.00025	68000

Table 29: E3-004-0-20 Lpile Parameters

Retaining Wall 1 E3-004-0-20						
Soil Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Soil Class	LPILE p-y Model	Soil Strain Parameter ϵ_{50}	Soil Modulus Parameter p-y k (pci)
1	1098.3	1089.8	A-4a	Stiff Clay w/o Water	0.0039	1944
2	1089.8	1087.3	A-4b	Stiff Clay w/o Water	0.0038	2083
Rock Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Rock Type	LPILE p-y Model	krm	Rock Initial Modulus Ei (psi)
1	1087.3	1083.3	Shale	Weak Rock	0.00025	68000
2	1083.3	1082.3	Shale	Weak Rock	0.00050	18000
3	1082.3	1075.3	Shale	Weak Rock	0.00050	18000
4	1075.3	1072.3	Shale	Weak Rock	0.00016	100000
5	1072.3	1062.3	Shale	Weak Rock	0.00016	100000
6	1062.3	1048.3	Shale	Weak Rock	0.00025	68000

Table 30: B-001-0-22 Lpile Parameters

Retaining Wall 1 B-001-0-22						
Soil Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Soil Class	LPILE p-y Model	Soil Strain Parameter ϵ_{50}	Soil Modulus Parameter p-y k (pci)
1	1093.5	1089	A-4a	Stiff Clay w/o Water	0.0038	2083
2	1089	1086.5	A-1-b	Sand (Reese)	-	656
3	1086.5	1083	A-4b	Stiff Clay w/o Water	0.0047	1208
Rock Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Rock Type	LPILE p-y Model	krm	Rock Initial Modulus E_i (psi)
1	1083	1047.8	Siltstone	Weak Rock	0.00050	32000

Table 31: B-002-0-22 Lpile Parameters

Retaining Wall 1 B-002-0-22						
Soil Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Soil Class	LPILE p-y Model	Soil Strain Parameter ϵ_{50}	Soil Modulus Parameter p-y k (pci)
1	1090.6	1085.6	A-4a	Sand (Reese)	-	437
Rock Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Rock Type	LPILE p-y Model	krm	Rock Initial Modulus E_i (psi)
1	1085.6	1084.8	Siltstone	Weak Rock	0.00025	68000
2	1084.8	1055.0	Siltstone	Weak Rock	0.00016	100000
3	1055.0	1049.8	Shale	Weak Rock	0.00025	68000

Table 32: B-030-0-24 Lpile Parameters

Retaining Wall 1 B-030-0-24						
Soil Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Soil Class	LPILE p-y Model	Soil Strain Parameter ϵ_{50}	Soil Modulus Parameter p-y k (pci)
1	1071.3	1065.9	A-1-b	Sand (Reese)	-	965
Rock Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Rock Type	LPILE p-y Model	krm	Rock Initial Modulus E_i (psi)
1	1065.9	1055.8	Shale	Weak Rock	0.00050	32000
2	1055.8	1050.0	Siltstone	Weak Rock	0.00006	200000
3	1050	1030.8	Shale	Weak Rock	0.00005	440000

5.6.2. Laterally Loaded Pile Analysis

NEAS reviewed the cross-section and project boring logs at the proposed RW 1 site. Based on our review, NEAS developed representative cross-sectional models at STA. 2+00, STA. 2+50 and STA. 5+80 in reference to RW1 to use as the basis for deep foundation design. The model was developed from NEAS's interpretation of the available information which included: 1) the roadway cross sections; 2) test borings and laboratory data (i.e., soil/rock classifications, SPT data, and strata elevation/thickness information) developed as part of this report; and 3) the estimate soil/rock properties as described in the Section 5.2.1 of this report. The Service I and Strength I lateral loads were calculated per AASHTO's LRFD BDS, Article 3.11.5.3 (AASHTO LRFD, 2020). The lateral load calculation process is presented in Appendix E – Retaining Wall 1.

The laterally loaded pile analyses were run twice in LPILE for two loading conditions, unfactored loading for Service (I) Limit State and factored loading for Strength (I) Limit State. The former loading condition is to determine pile head deflection; and the latter loading condition is to determine the structural shear and moment on the pile. For the factored Strength Limit State condition, a load factor of $\gamma_{EH}=1.50$ was applied

to the horizontal earth pressure (EH), per AASHTO's LRFD BDS, Article 3.4.1 (AASHTO LRFD, 2020). The horizontal earth pressure for a nongravity cantilever wall that directly supports a cut rock face refers to ODOT GDM Section 1501.1.4. The results of laterally loaded shaft analyses by using LPILE (maximum bending moments, maximum shearing force and lateral deflection) are summarized in Table 33. The Lpile analyses output can be found in Appendix E.

Table 33: Summary of Laterally Loaded Shaft Analyses

Retaining Wall 1 LPILE Analysis Summary								
Station	Boring	Maximum Moment in Pile - Strength Limit State (in-lbs)	Depth of Maximum Bending Moment below Pile Head (ft)	Average Shear in Shaft (lbs) ⁽¹⁾	Depth of Maximum Shear Force below Pile Head (ft)	Pile Head Deflection - Service Limit State (in)	First Fixity Point below Pile Head (ft)	Second Fixity Point below Pile Head (ft)
STA. 2+00	E3-004-0-20	2,901,536	17.0	-51,760	19.5	0.566	19.5	26.5
STA. 2+50	B-001-0-22	5,485,279	18.0	-82,069	21.0	0.781	21.0	28.0
STA. 5+80	B-002-0-22	4,303,403	20.0	-76,735	23.0	1.233	23.0	30.0
Note: 1. Average shear in shaft was calculated in according to ODOT GDM Section 1501.7.9								

With respect to the reinforcement design in drilled shaft, NEAS followed AASHTO's LRFD BDS (AASHTO LRFD, 2024) and ODOT's GDM Section 904 (ODOT, 2024). Please note the minimum concrete cover between soil and steel reinforcement for a drilled shaft of 4 feet or less in diameter is 3 inches. A resistance factor $\phi_f=1.00$ for flexural resistance and a resistance factor $\phi_v=1.00$ for shear resistance per AASHTO Article 6.5.4.2. were used. The factored maximum moment and the factored maximum shear for all the cross sections from LPILE analyses are listed in Table 33 above. It is recommended to use W24x62 steel reinforcing beam, with structural steel having a yield strength of 50 ksi (ASTM grade 50). Per AASHTO's LRFD BDS, Article 6.10.8.1.3 and 6.10.9.2 (AASHTO LRFD, 2017), the factored flexure resistance for W24x62 is $\phi_f * M_n = \phi_f * R_h * f_y * S_x = 6,550,000$ lb-in ($R_h=1.0$, $S_x=131$ in³) and the factored shear resistance is $\phi_v * V_n = \phi_v * C * 0.58 * F_{yw} * D * t_w = 280,824$ lb ($C=1.0$, $D=d-2*t_f=22.52$ in, $t_w=0.43$ in), which are larger than the required factored maximum moment M_u and the factored maximum shear V_u , respectively. Therefore, with the steel beam W24x62, the 3.5-ft diameter structural shafts are capable of meeting the structural strength requirements for both flexure and shear.

In terms of shaft length, it is important to ensure that the drilled shaft is embedded in a solid stratum to constrain deflection at the pile head within appropriate serviceability limits. According to ODOT's GDM (ODOT, 2024), the shaft should ideally be embedded into bedrock to provide sufficient resistance to deflection. Based on the subsurface exploration, bedrock is shallow at the proposed RW 1 site, with bedrock encountered between 5 ft and 11 ft below ground surface and at elevations ranging from 1083.0 ft to 1087.3 ft. The site plan indicates that the top of shaft is proposed between 1067.91 and 1073.91 ft, which confirms that the shafts will be socketed in the bedrock. For the cross-sectional model at STA 2+00, the second fixity point is at 26.5 ft below the pile head (from the top of the steel pile); for the cross-sectional model at STA. 2+50, the second fixity point is at 28.0 ft below the pile head; for the cross-sectional model at STA. 5+80, the second fixity point is at 30.0 ft below the pile head. Based on our lateral load analysis, the proposed 12-ft long shaft in bedrock should be sufficient to constrain the deflection at the pile head to appropriate serviceability limits. As per ODOT GDM Section 1501.6, the horizontal deflection limit for the retaining wall is one percent (1%) of the design retained height (the height of the wall above design grade).

5.7. Retaining Wall 2

A foundation review was completed for the proposed MSE wall – RW 2, located along the proposed Ramp CD between STA. 113+19.42 and STA. 127+43.17. The following information was used in our design: 1) the soil/rock characteristics gathered during the subsurface exploration (i.e., SPT results, laboratory test

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results, etc.); 2) the referenced Site Plan prepared by Gannet Fleming; and, 3) other design assumptions presented in subsequent sections of this report. Geotechnical analyses consisting of external stability (i.e., bearing resistance, eccentricity, and sliding resistance) were performed for the proposed MSE wall in accordance with ODOT's BDM (ODOT, 2024) and AASHTO's LRFD BDS (AASHTO, 2020).

5.7.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 34 and 44 below.

Table 34: B-031-0-24 soil profile for analysis

Retaining Wall 2: Soil Profile B-031-0-24							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1066.7 ft - 1057.2 ft)	112	112	122	2000	200	25	1.50
Silt and Clay (A-6a) Depth (1057.2 ft - 1054.2 ft)	105	105	115	600	75	21	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 35: B-001-0-20 soil profile for analysis

Retaining Wall 2: Soil Profile B-001-0-20							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1064.1 ft - 1060.6 ft)	118	118	128	3750	300	27	1.50
Fine Sand (A-3) Depth (1060.6 ft - 1055.6 ft)	115	115	125	-	-	33	1.20
Sandy Silt (A-4a) Depth (1055.6 ft - 1048.1 ft)	112	112	122	2100	200	25	1.50
Silt and Clay (A-6a) Depth (1048.1 ft - 1045.6 ft)	118	118	128	3750	300	26	1.50
Silt and Clay (A-6a) Depth (1045.6 ft - 1035.6 ft)	112	112	122	2000	200	24	1.50
Coarse and Fine Sand (A-3a) Depth (1035.6 ft - 1024.1 ft)	122	112	122	-	-	32	1.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 36: B-021-0-24 soil profile for analysis

Retaining Wall 2: Soil Profile B-021-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand (A-1-b) Depth (1034.3 ft - 1028.8 ft)	115	115	125	-	-	36	1.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

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Table 37: B-032-0-24 soil profile for analysis

Retaining Wall 2: Soil Profile B-032-0-24							
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)	Setup Factor (f_{su})
Silty Clay (A-6b) Depth (1027 ft - 1020 ft)	118	118	128	3850	300	26	1.75
Coarse and Fine Sand (A-3a) Depth (1020 ft - 1017.5 ft)	118	118	128	-	-	37	1.00
Sandy Silt (A-4a) Depth (1017.5 ft - 1015 ft)	110	110	120	1600	150	24	1.50
Silt (A-4b) Depth (1015 ft - 1007.5 ft)	110	110	120	1450	150	24	1.50
Sandy Silt (A-4a) Depth (1007.5 ft - 1005 ft)	110	110	120	1600	150	24	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 38: B-033-0-24 soil profile for analysis

Retaining Wall 2: Soil Profile B-033-0-24							
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)	Setup Factor (f_{su})
Silt (A-4b) Depth (1053.3 ft - 1041.3 ft)	115	115	125	-	-	35	1.50
Sandy Silt (A-4a) Depth (1041.3 ft - 1036.3 ft)	110	110	120	1250	150	24	1.50
Silt and Clay (A-6a) Depth (1036.3 ft - 1031.3 ft)	108	108	118	1000	100	22	1.50
Sandy Silt (A-4a) Depth (1031.3 ft - 1026.3 ft)	115	115	125	-	-	31	1.20
Silt (A-4b) Depth (1026.3 ft - 1020 ft)	110	110	120	1250	150	24	1.50
Coarse and Fine Sand (A-3a) Depth (1020 ft - 1015 ft)	112	112	122	-	-	31	1.00
Silt (A-4b) Depth (1015 ft - 1010 ft)	110	110	120	1500	150	24	1.50
Gravel with Sand, Silt and Clay (A-6) Depth (1010 ft - 1003.3 ft)	112	112	122	-	-	30	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 39: B-034-0-24 soil profile for analysis

Retaining Wall 2: Soil Profile B-034-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand and Silt (A-2-4) Depth (1049.4 ft - 1044.9 ft)	118	118	128	-	-	40	1.20
Silt and Clay (A-6a) Depth (1044.9 ft - 1042.4 ft)	112	112	122	2100	200	24	1.50
Sandy Silt (A-4a) Depth (1042.4 ft - 1039.9 ft)	115	115	125	-	-	31	1.20
Silt and Clay (A-6a) Depth (1039.9 ft - 1029.9 ft)	112	112	122	1900	200	24	1.50
Sandy Silt (A-4a) Depth (1029.9 ft - 1027.4 ft)	115	115	125	-	-	31	1.20
Coarse and Fine Sand (A-3a) Depth (1027.4 ft - 1024.9 ft)	120	120	130	-	-	38	1.00
Gravel with Sand and Silt (A-2-4) Depth (1024.9 ft - 1016.1 ft)	120	120	130	-	-	38	1.20
Sandy Silt (A-4a) Depth (1016.1 ft - 1011.1 ft)	112	112	122	2100	200	25	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

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Table 40: B-035-0-24 soil profile for analysis

Retaining Wall 2: Soil Profile B-035-0-24							
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)	Setup Factor (f_{su})
Silt and Clay (A-6a) Depth (1007.1 ft - 1004.1 ft)	110	110	120	1500	150	23	1.50
Gravel with Sand (A-1-b) Depth (1004.1 ft - 1001.6 ft)	110	110	120	-	-	33	1.00
Clay (A-7-6) Depth (1001.6 ft - 999.1 ft)	105	105	115	750	75	20	2.00
Clay (A-7-6) Depth (999.1 ft - 994.1 ft)	115	115	125	3050	250	24	2.00
Silt and Clay (A-6a) Depth (994.1 ft - 987.1 ft)	140	130	140	9100	450	31	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 41: B-036-0-24 soil profile for analysis

Retaining Wall 2: Soil Profile B-036-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand (A-1-b) Depth (1007.9 ft - 1002.4 ft)	115	115	125	-	-	36	1.00
Clay (A-7-6) Depth (1002.4 ft - 999.9 ft)	110	110	120	1600	150	22	2.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 42: B-037-0-24 soil profile for analysis

Retaining Wall 2: Soil Profile B-037-0-24							
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)	Setup Factor (f_{su})
Clay (A-7-6) Depth (1009.9 ft - 1000.9 ft)	108	108	118	1200	100	21	2.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 43: B-038-0-24 soil profile for analysis

Retaining Wall 2: Soil Profile B-038-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand (A-1-b) Depth (1014.6 ft - 1009.1 ft)	115	115	125	-	-	40	1.00
Silt and Clay (A-6a) Depth (1009.1 ft - 1006.1 ft)	112	112	122	2250	200	25	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 44: B-039-0-24 soil profile for analysis

Retaining Wall 2: Soil Profile B-039-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand (A-1-b) Depth (1022.2 ft - 1016.2 ft)	115	115	125	-	-	36	1.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

5.7.2. MSE Wall Design Assumptions

ODOT's BDM and AASHTO's LRFD BDS dictate analysis parameters and design minimums/constraints that were significant to our analyses consist of the following:

- Minimum reinforcement strap lengths of proposed MSE wall is to be 70% of the total wall height (as measured from proposed profile grade at the face of the wall to the top of the leveling pad) or 8 ft, whichever is greater, at the particular section of wall being analyzed, per BDM section 307.4-A;
- Minimum MSE wall embedment depths (as measured from top of the leveling pad to the lowest point on the ground surface within 4-ft of the face of the wall) are to conform to Figure 201-5 presented in ODOT's BDM and be the larger of 3 ft or the local frost depth;
- Soils below the bottom of leveling pad will be undercut a minimum of 1 ft and replaced Granular Material Type C according to SS840 and the requirements of ODOT Construction & Materials Specifications Section 204.07 (CMS 204.07);
- Reinforced Zone and Retained Fill soils will meet the minimum design soil parameters per Table 307-1 of the ODOT BDM and SS 840 Table 840.04-1 as shown in Table 45 below.

Table 45: Design Soil Parameters for Fill Materials

Fill Zone	Type of Soil	Soil Unit Weight (pcf)	Friction Angle (°)	Cohesion (psf)
Reinforced Zone	Select Granular Backfill	120	34	0
Retained Soil	On-site soil varying from sandy lean clay to silty sand	120	30	0

With respect to design constraints and assumptions specific to the proposed RW 2, the geometry of the proposed walls (i.e., exposed wall heights, existing ground elevations, proposed bottom of wall elevation, etc.) is assumed to be consistent with that shown in the proposed Site Plan prepared by Gannet Fleming.

5.7.3. External Stability

Based on our estimated geotechnical soil properties and the retaining wall design assumptions provided in Sections 5.3.1. of this report, an external stability analysis of the proposed MSE wall was performed. Five cross-sections were evaluated for resistance to bearing pressure, sliding forces and overturning at the Strength Limit State for both drained and undrained conditions in accordance with Section 11.10.5 of the AASHTO's LRFD BDS. The capacity to demand ratios (CDRs) calculated for the referenced cross-sections with respect to bearing, sliding and overturning, as well as the calculated factored bearing resistances are presented in Table 46 below. (External Stability and Bearing Resistance Calculation Results can be found in Appendix E)

Table 46: External Stability Analysis Summary

MSE Wall Analysis Summary - Retaining Wall 2											
Station		STA. 1+00		STA. 3+00		STA. 5+50		STA. 8+00		STA. 11+00	
Top of Coping (ft)		1063.8		1058.0		1050.7		1046.9		1045.0	
Existing Ground in front of wall (ft)		1045.8		1029.1		1030.9		1025.0		1019.8	
Bottom of Leveling Pad (ft)		1040.4		1024.4		1027.4		1019.4		1014.9	
Exposed Wall Height (ft)		18.0		28.9		19.8		21.9		25.2	
Design Wall Height (ft)		23.4		33.6		23.3		27.4		30.1	
Length of Reinforcement (ft)		16.4		23.5		16.3		19.1		21.1	
Length of Reinf. To Height Ratio		0.7		0.7		0.7		0.7		0.7	
Adjacent Boring Locations		B2-001-0-20		B-032-0-24		B-033-0-24		B-035-0-24		B-037-0-24	
Capacity Demand Ratio (CDR)											
Condition		Undrained	Drained	Undrained	Drained	Undrained	Drained	Undrained	Drained	Undrained	Drained
Top of Foundation Preparation Layer	Sliding	2.0	2.0	2.1	2.1	2.0	2.0	2.0	2.0	2.0	2.0
	Bottom of Foundation Preparation Layer	1.5	1.3	1.5	1.4	1.5	1.4	1.5	1.4	1.5	1.4
Overtopping / Eccentricity		1.9	1.9	2.0	2.0	1.9	1.9	2.0	2.0	2.0	2.0
Top of Foundation Preparation Layer	Bearing Resistance	1.2	1.2	1.1	1.2	1.1	1.2	1.3	1.3	1.3	1.1
	Factored Bearing Resistance (ksf) ⁽¹⁾	6.9	6.9	8.9	9.1	6.0	6.7	8.8	8.8	9.3	8.0
Bottom of Foundation Preparation Layer	Bearing Resistance	1.2	1.4	1.1	1.3	1.1	1.4	1.3	1.6	1.3	1.3
	Factored Bearing Resistance (ksf) ⁽¹⁾	6.9	8.1	8.9	10.4	6.0	8.0	8.8	10.1	9.3	9.1
Notes:											
1. Bearing Resistance calculated in accordance to Section 11.10.5.4 of 2024 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of LRFD BDS.											

5.7.1. Settlement

There will be varying depths of new embankment fills along the proposed RW 2. The maximum new fill is going to be located around Beacon Street at STA. 3+00. A settlement model was developed and analyzed for both elastic (immediate) and consolidation (long term) settlement at STA. 3+00 and STA. 4+50 by using the software entitled FoSSA 2.0 by ADAMA Engineering, Inc.

Based on our analysis, at STA. 3+00 the ground surface is estimated to experience up to 6.0 inches of immediate settlement and up to 5.52 inches of long-term (consolidation) settlement due to the induced loads from the 26-ft high new fills. At STA. 4+50, the ground surface is estimated to experience up to 1.44 inches of immediate settlement and up to 3.24 inches of long-term (consolidation) settlement due to the induced loads from the 15-ft high new fills. The differential settlement is about 0.0013, which is less than 1/100. The settlement analysis results can be found in Appendix E. Immediate settlement is expected to take place during construction, prior to roadway loading, and is not anticipated to be a concern. The estimated time required for 90% consolidation at the project site is approximately 46 days following embankment construction. If the project schedule can tolerate a delay between the substantial completion of the site earthwork in this area and commencement of the structure/pavement construction, postponing construction and allowing the potentially damaging settlements to take place should be considered to prevent additional costs associated with reconstruction, repairs and maintenance.

In terms of the actual waiting period until the beginning of structure/pavement construction, it is recommended that a settlement monitoring program be designed and implemented to verify that the settlements have dissipated to a level acceptable by the Geotechnical Engineer and determine the time period at which permanent structure/pavement construction may begin. With respect to the settlement monitoring, it is recommended that settlement platforms per 1106.2 Site Monitoring and Confirmation Testing of ODOT's Geotechnical Design Manual, dated July 15, 2024, be installed at the backfill site.

5.7.2. Global Stability Analysis

For the purpose of evaluating the stability of the proposed RW 2, NEAS reviewed two critical cross-sections that were 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, weak or thick soil layer, etc.).

Based on our review of the available information at the referenced locations and the associated soil properties, two (2) separate cross-sections were initially estimated to be most "critical" and were analyzed for global stability for RW 2. The two cross-sections analyzed for global stability include: the maximum total wall height and maximum exposed wall height section, STA 3+25 in reference to RW 2; and the other one section at the weak soil, STA 6+00 in reference to RW 2.

For these cross-sections, NEAS developed a representative cross-sectional model to use as the basis for global stability analyses. The models were developed from NEAS's interpretation of the available information which included: 1) the proposed Retaining Wall Site Plan prepared by Gannett Fleming; 2) a live load surcharge of 250 psf, accounting for traffic induced loads; and, 3) test borings and laboratory data developed as part of this report. With respect to the soil's engineering properties, the provided Soil Profile Estimated Engineering Properties presented in Section 5.6.1 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 Rocscience, Inc. Specifically, the Bishop, GLE and Spencer analysis methods were used to calculate a factor of safety (FOS) for circular type slope failures. The FOS is the ratio of the resisting forces and the driving forces, with the desired safety factor being more than about 1.3 which approximately 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 contains or supports a structural element.

Based on our global stability analyses for the referenced retaining wall sections, the minimum slope stability safety factors walls for short-term (Total Stress) and long term (Effective Stress) conditions exceeded the desired value of 1.3. The results of the analyses are summarized in Table 47. Based on the results of the analyses, it is our opinion that the subsurface conditions encountered at the retaining wall locations are generally satisfactory and the site can be considered to be stable at short-term and long-term conditions. The graphical output of the slope stability program (cross-sectional model, calculated safety factor, and critical failure plane) is presented in Appendix E.

Table 47: Global Stability Analysis Summary

Global Stability Analysis at Retaining Wall 2					
Location	Boring No.	Description	Minimum Factor of Safety	Equivalent Resistance Factor	Status (OK/NG)
STA. 3+25	B-032-0-24	Short Term	2.95	0.34	OK
		Long Term	1.59	0.63	OK
STA. 6+00	B-033-0-24	Short Term	2.59	0.39	OK
		Long Term	1.82	0.55	OK

5.7.3. MSE Wall Recommendation

MSE wall soil reinforcement: the recommended soil reinforcement length is presented in Table 4 in Section 5.3.4 of this report.

MSE wall reinforced backfill: we recommend the use of granular material meeting the requirements of ODOT's Supplemental Specification 840 (SS-840) Section 840.03.E "Select Granular Backfill" (SGB). Furthermore, it is recommended that, at a minimum, SGB be placed as backfill material within the limits shown in Figures 303.5.1-3 and 303.5.1-5 of ODOT's BDM (ODOT, 2024). With respect to placement, it

is recommended that SGB be placed in accordance with SS-840 Section 840.06.I "Select Granular Backfill Placement".

Drainage: 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 reinforced or retained backfill soil, loss of shear strength of foundation soils due to saturation, and other drainage related issues.

It is recommended that internal drainage of the retaining wall (reinforced fill) be designed as indicated in Section 307.4 and as shown in Figures 201-5 through 201-7 of the ODOT BDM and SS840.04.A.6. We recommend the wall drainage material conform to the requirements of SS-840, Section 840.03.F "Backfill Drainage Material" and wall drainage be constructed in accordance with SS-840 Section 840.06.F "Wall Drainage". Furthermore, it is recommended that the barrier or curb at the roadway extend at least 25 ft beyond the MSE wall limits, and outlet to a piped collection system (i.e., collection basin/inlet) located beyond the extents of the wall. Where a barrier or curb is not present, it is recommended that a paved channel (swale) be placed directly behind the top of the wall. The paved channel should be designed to intercept surface water and direct it to an outlet as well as reduce the potential for surface water from overtopping 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.

5.8. Retaining Wall 3

Retaining Wall 3 consists of two types of walls: MSE wall and Soldier Pile Lagging Wall. The foundation conditions for each wall type was evaluated separately to ensure appropriate design and performance.

5.8.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 48-49 below.

Table 48: B-022-0-24 soil profile for analysis

Retaining Wall 3: Soil Profile B-022-0-24							
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)	Setup Factor (f_w)
Sandy Silt (A-4a) Depth (1034.3 ft - 1029.8 ft)	112	112	122	2250	200	26	1.50
Gravel with Sand and Silt (A-2-4) Depth (1029.8 ft - 1022.3 ft)	115	115	125	-	-	35	1.20
Sandy Silt (A-4a) Depth (1022.3 ft - 1019.8 ft)	112	112	122	2000	200	25	1.50
Gravel with Sand and Silt (A-2-4) Depth (1019.8 ft - 1014.3 ft)	120	120	130	-	-	40	1.20

Notes:
1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405.
2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used.
3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.

Table 49: R-003-0-99 soil profile for analysis

Retaining Wall 3: Soil Profile R-003-0-99							
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)	Setup Factor (f_{su})
Gravel with Sand and Silt (A-2-4) Depth (1054.5 ft - 1052 ft)	115	115	125	-	-	39	1.20
Coarse and Fine Sand (A-3a) Depth (1052 ft - 1046 ft)	110	110	120	-	-	32	1.00
Sandy Silt (A-4a) Depth (1046 ft - 1030 ft)	112	112	122	2350	200	26	1.50
Silty Clay (A-6b) Depth (1030 ft - 1016 ft)	112	112	122	1750	200	24	1.75
Silt and Clay (A-6a) Depth (1016 ft - 1000 ft)	122	112	122	2350	200	25	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160-52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Deep foundation elements subjected to horizontal loads and/or moments should be analyzed for maximum bending moment, maximum shear force and lateral deflection and should be checked whether the foundation element is structurally capable of resisting the lateral load. For the purpose of evaluating the shaft resistance in the lateral direction, drilled shaft has been analyzed by using the software entitled Lpile by Ensoft, Inc. The generalized soil and rock parameters for analyzing the laterally loaded shaft using the p-y curve method for soil and rock strata are shown in Tables 50 and 51 below.

Table 50: B-022-0-24 Lpile Parameters

Retaining Wall 3 B-022-0-24						
Soil Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Soil Class	LPILE p-y Model	Soil Strain Parameter ϵ_{50}	Soil Modulus Parameter p-y k (pci)
1	1034.3	1029.8	A-4a	Stiff Clay w/o Water	0.0059	750
2	1029.8	1022.3	A-2-4	Sand (Reese)	-	226
3	1022.3	1019.8	A-4a	Stiff Clay w/o Water	0.0062	667
4	1019.8	1014.3	A-2-4	Sand (Reese)	-	537
Rock Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Rock Type	LPILE p-y Model	krm	Rock Initial Modulus E_i (psi)
1	1014.3	998.8	Shale	Weak Rock	0.00050	35000

Table 51: R-003-0-99 Lpile Parameters

Retaining Wall 3 R-003-0-99						
Soil Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Soil Class	LPILE p-y Model	Soil Strain Parameter ϵ_{50}	Soil Modulus Parameter p-y k (pci)
1	1054.5	1052	A-2-4	Sand (Reese)	-	437
2	1052	1046	A-3a	Sand (Reese)	-	83
3	1046	1030	A-4a	Stiff Clay w/o Water	0.0057	799
4	1030	1016	A-6b	Stiff Clay w/o Water	0.0067	583
5	1016	1000	A-6a	Stiff Clay w/o Water	0.0057	792

5.8.2. External Stability Analysis for MSE Wall Portion

Based on our estimated geotechnical soil properties and the retaining wall design assumptions provided in Sections 5.6.2. of this report, an external stability analysis of the proposed MSE wall portion was performed. One cross-section at STA. 0+00 was evaluated for resistance to bearing pressure, sliding forces and overturning at the Strength Limit State for both drained and undrained conditions in accordance with Section 11.10.5 of the AASHTO's LRFD BDS. The capacity to demand ratios (CDRs) calculated for the referenced cross-sections with respect to bearing, sliding and overturning, as well as the calculated factored

bearing resistances are presented in Table 52 below. (External Stability and Bearing Resistance Calculation Results can be found in Appendix E)

Based on the external stability analysis summary, the recommended soil reinforcement length is 80% of the design wall height.

Table 52: External Stability Analysis Summary

MSE Wall - Retaining Wall 3			
Station		STA. 0+00	
Top of Coping (ft)		1058.6	
Existing Ground in front of wall (ft)		1033.4	
Bottom of Leveling Pad (ft)		1029.9	
Exposed Wall Height (ft)		25.2	
Design Wall Height (ft)		28.7	
Length of Reinforcement (ft)		22.9	
Length of Reinf. To Height Ratio		0.8	
Adjacent Boring Locations		R-003-0-99	
Capacity Demand Ratio (CDR)			
Condition		Undrained	Drained
Top of Foundation Preparation Layer	Sliding	2.3	2.3
Bottom of Foundation Preparation Layer	Sliding	1.7	1.5
Overturning / Eccentricity		2.6	2.6
Top of Foundation Preparation Layer	Bearing Resistance	1.0	1.1
	Factored Bearing Resistance (ksf) ⁽¹⁾	6.5	7.1
Bottom of Foundation Preparation Layer	Bearing Resistance	1.0	1.3
	Factored Bearing Resistance (ksf) ⁽¹⁾	6.5	8.2
Notes:			
1. Bearing Resistance calculated in accordance to Section 11.10.5.4 of 2024 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of LRFD BDS.			

5.8.3. Laterally Loaded Pile Analysis for Soldier Pile Wall Section

NEAS reviewed the cross-section and project boring logs at the proposed RW 3 site. Based on our review, NEAS developed a representative cross-sectional model at STA. 1+25 in reference to RW3 to use as the basis for deep foundation design. The model was developed from NEAS's interpretation of the available information which included: 1) the roadway cross sections; 2) test borings and laboratory data (i.e., soil/rock classifications, SPT data, and strata elevation/thickness information) developed as part of this report; 3) the estimate soil/rock properties as described in the Section 5.4.1 of this report; and 4) assumed embankment fill properties (granular materials with internal friction angle of 32°) listed in ODOT GDM Table 500-2. The Service I and Strength I lateral loads were calculated per AASHTO's LRFD BDS, Article 3.11.5.3 (AASHTO LRFD, 2020). The lateral load calculation process is presented in Appendix E – Retaining Wall 3.

The laterally loaded pile analyses were run twice in LPILE for two loading conditions, unfactored loading for Service (I) Limit State and factored loading for Strength (I) Limit State. The former loading condition is to determine pile head deflection; and the latter loading condition is to determine the structural shear and moment on the pile. For the factored Strength Limit State condition, a load factor of $\gamma_{EH}=1.50$ was applied to the horizontal earth pressure (EH), per AASHTO's LRFD BDS, Article 3.4.1 (AASHTO LRFD, 2020). The results of laterally loaded shaft analyses by using LPILE (maximum bending moments, maximum shearing force and lateral deflection) are summarized in Table 53. The Lpile analyses output can be found in Appendix E.

Table 53: Summary of Laterally Loaded Pile Analyses

Retaining Wall 3 LPILE Analysis Summary								
Station	Boring	Maximum Moment in Pile - Strength Limit State (in-lbs)	Depth of Maximum Bending Moment below Pile Head (ft)	Average Shear in Shaft (lbs) ⁽¹⁾	Depth of Maximum Shear Force below Pile Head (ft)	Pile Head Deflection - Service Limit State (in)	First Fixity Point below Pile Head (ft)	Second Fixity Point below Pile Head (ft)
STA. 1+25	R-003-0-99	9,300,799	21.5	-88,136	14.1	1.7	28.5	37.0

With respect to the reinforcement design in drilled shaft, NEAS followed AASHTO's LRFD BDS (AASHTO LRFD, 2020) and ODOT's GDM Section 904(ODOT, 2024). Please note the minimum concrete cover between soil and steel reinforcement for a drilled shaft of 4 feet or less in diameter is 3 inches. A resistance factor $\phi_r=1.00$ for flexural resistance and a resistance factor $\phi_v=1.00$ for shear resistance per AASHTO Article 6.5.4.2. were used. The factored maximum moment and the factored maximum shear for both cross sections from LPILE analyses are listed in Table 53 above. It is recommended to use W24x162 steel reinforcing beam, with structural steel having a yield strength of 50 ksi (ASTM grade 50). Per AASHTO's LRFD BDS, Article 6.10.8.1.3 and 6.10.9.2 (AASHTO LRFD, 2017), the factored flexure resistance for W24x162 is $\phi_r * M_n = \phi_r * R_h * f_y * S_x = 20,700,000 \text{ lb-in}$ ($R_h=1.0$, $S_x=414 \text{ in}^3$) and the factored shear resistance is $\phi_v * V_n = \phi_v * C * 0.58 * F_{yw} * D * t_w = 461,239 \text{ lb}$ ($C=1.0$, $D=d-2*t_f=22.56 \text{ in}$, $t_w=0.705 \text{ in}$), which are larger than the required factored maximum moment M_u and the factored maximum shear V_u , respectively. Therefore, with the steel beam W24x162, the 3.5-ft diameter structural shafts are capable of meeting the structural strength requirements for both flexure and shear.

In terms of the pile length, it is important to note that the pile should extend to the second fixity point. For the cross-sectional model at STA 1+25, the second fixity point is at 36.3 ft below the pile head (top of wall 1046.41 ft), which is about 22.3 ft below the top of shaft (1032.41 ft). The 22.3-ft long shaft should be long enough to constrain the deflection at the pile head to appropriate serviceability limits. In reference to ODOT GDM Section 1501.6, the horizontal deflection limit for all retaining walls is one percent (1%) of the design retained height (height of wall above design grade) of wall. Therefore, it is recommended that shafts extend to the elevation 1010.11 ft

5.8.4. Global Stability Analysis

To evaluate the stability of the proposed RW 3, NEAS reviewed cross-sections that were 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, 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 was initially estimated as the most "critical" and was analyzed for global stability for RW 3. The analyzed cross-section represent the maximum total wall height and maximum exposed wall height at STA 0+00, in reference to RW 3.

For this 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 Site Plan prepared by Gannet Fleming; 2) a live load surcharge of 250 psf, accounting for traffic induced loads; and, 3) test borings and laboratory data developed as part of this report. With respect to the soil's engineering properties, the provided Soil Profile Estimated Engineering Properties presented in Section 5.7.1 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 Rocscience, Inc. Specifically, the Bishop, GLE and Spencer analysis methods were used to calculate a factor of safety (FOS) for circular type slope failures. The FOS is the ratio of the resisting forces and the driving forces, with the desired safety factor being more than about 1.3 which approximately 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 contains or supports a structural element.

Based on our global stability analyses for the referenced retaining wall section, the minimum slope stability safety factors for short-term (Total Stress) and long term (Effective Stress) conditions exceeded the desired value of 1.3. The results of the analyses are summarized in Table 54. Based on the results of the analyses, it is our opinion that the subsurface conditions encountered at the retaining wall location are generally satisfactory and the site can be considered to be stable at short-term and long-term conditions. The graphical output of the slope stability program (cross-sectional model, calculated safety factor, and critical failure plane) is presented in Appendix E.

Table 54: Global Stability Analysis Summary

Global Stability Analysis at Retaining Wall 3					
Location	Boring No.	Description	Minimum Factor of Safety	Equivalent Resistance Factor	Status (OK/NG)
STA. 0+00	B-022-0-24	Short Term	3.01	0.33	OK
		Long Term	1.80	0.56	OK

5.9. Retaining Wall 4

A foundation review was conducted for the proposed MSE wall – RW 4, located along the proposed Southbound Exit Ramp CD between STA. 128+47.86 and STA. 132+24. The following information was used in our design: 1) the soil/rock characteristics gathered during the subsurface exploration (i.e., SPT results, laboratory test results, etc.); 2) the referenced Site Plan prepared by Arcadis; and, 3) other design assumptions presented in Section 5.3.2 of this report. Geotechnical analyses consisting of external stability (i.e., bearing resistance, eccentricity, and sliding resistance), global stability, and settlement were performed for the proposed MSE wall in accordance with ODOT's BDM (ODOT, 2024) and AASHTO's LRFD BDS (AASHTO, 2020).

5.9.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 55 and 58 below.

Table 55: GB-002-0-99 soil profile for analysis

Bridge Over Exchange Street: Soil Profile GB-002-0-99							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1023.5 ft - 1020.5 ft)	112	112	122	-	-	31	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 56: B-040-0-24 soil profile for analysis

Retaining Wall 4: Soil Profile B-040-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand (A-1-b) Depth (1027.1 ft - 1019.1 ft)	115	115	125	-	-	39	1.00
Gravel with Sand and Silt (A-2-4) Depth (1019.1 ft - 1016.1 ft)	115	115	125	-	-	35	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 57: B-041-0-24 soil profile for analysis

Retaining Wall 4: Soil Profile B-041-0-24							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1032.6 ft - 1027.1 ft)	112	112	122	1750	200	25	1.50
Gravel with Sand (A-1-b) Depth (1027.1 ft - 1025.7 ft)	122	122	132	-	-	40	1.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 58: B-042-0-24 soil profile for analysis

Retaining Wall 4: Soil Profile B-042-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand and Silt (A-2-4) Depth (1037 ft - 1030 ft)	120	120	130	-	-	40	1.20
Sandy Silt (A-4a) Depth (1030 ft - 1027.5 ft)	130	130	140	10500	450	33	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

5.9.2. External Stability

Based on our estimated geotechnical soil properties and the retaining wall design assumptions provided in Sections 5.6.2. of this report, an external stability analysis of the proposed MSE wall was performed. Two cross-sections were evaluated for resistance to bearing pressure, sliding forces and overturning at the Strength Limit State for both drained and undrained conditions in accordance with Section 11.10.5 of the AASHTO's LRFD BDS. The capacity to demand ratios (CDRs) calculated for the referenced cross-sections with respect to bearing, sliding and overturning, as well as the calculated factored bearing resistances are presented in Table 59 below. (External Stability and Bearing Resistance Calculation Results can be found in Appendix E)

Based on the external stability analysis summary, the recommended soil reinforcement length is 70% of the design wall height.

Table 59: External Stability Analysis Summary

MSE Wall Analysis Summary - Retaining Wall 4					
Station		STA. 400+00		STA. 402+25	
Top of Coping (ft)		1043.0		1041.3	
Proposed Ground in front of wall (ft)		1026.5		1033.0	
Bottom of Leveling Pad (ft)		1020.0		1026.0	
Exposed Wall Height (ft)		16.5		8.3	
Design Wall Height (ft)		23.0		15.3	
Length of Reinforcement (ft)		16.1		10.7	
Length of Reinf. To Height Ratio		0.7		0.7	
Adjacent Boring Locations		GB-002-0-99		B-041-0-24	
Capacity Demand Ratio (CDR)					
Condition		Undrained	Drained	Undrained	Drained
Top of Foundation Preparation Layer	Sliding	2.0	2.0	2.0	2.0
Bottom of Foundation Preparation Layer	Sliding	1.8	1.8	1.6	1.6
Overturning / Eccentricity		1.9	1.9	1.8	1.8
Top of Foundation Preparation Layer	Bearing Resistance	2.1	2.1	-	17.8
	Factored Bearing Resistance (ksf) ⁽¹⁾	11.7	11.7	-	70.1
Bottom of Foundation Preparation Layer	Bearing Resistance	2.1	2.6	-	17.2
	Factored Bearing Resistance (ksf) ⁽¹⁾	11.7	14.7	-	72.3
Notes:					
1. Bearing Resistance calculated in accordance to Section 11.10.5.4 of 2024 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of LRFD BDS.					

5.9.3. Settlement

Due to the presence of very shallow bedrock, settlement is not expected to be a concern.

5.9.4. MSE Wall Recommendation

MSE wall soil reinforcement: the recommended soil reinforcement length is 70% of the design wall height or 8 feet, whichever is greater in according with ODOT GDM Section 1504 and BDM Section 307.4.

MSE wall reinforced backfill: we recommend the use of granular material meeting the requirements of ODOT's Supplemental Specification 840 (SS-840) Section 840.03.E "Select Granular Backfill" (SGB). Furthermore, it is recommended that, at a minimum, SGB be placed as backfill material within the limits shown in Figures 303.5.1-3 and 303.5.1-5 of ODOT's BDM (ODOT, 2024). With respect to placement, it is recommended that SGB be placed in accordance with SS-840 Section 840.06.I "Select Granular Backfill Placement".

Drainage: 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 reinforced or retained backfill soil, loss of shear strength of foundation soils due to saturation, and other drainage related issues.

It is recommended that internal drainage of the retaining wall (reinforced fill) be designed as indicated in Section 307.4 and as shown in Figures 201-5 through 201-7 of the ODOT BDM. We recommend the wall drainage material conform to the requirements of SS-840, Section 840.03.F "Backfill Drainage Material" and wall drainage be constructed in accordance with SS-840 Section 840.06.F "Wall Drainage". Furthermore, it is recommended that the barrier or curb at the roadway extend at least 25 ft beyond the MSE wall limits, and outlet to a piped collection system (i.e., collection basin/inlet) located beyond the extents of the wall. Where a barrier or curb is not present, it is recommended that a paved channel (swale) be placed directly behind the top of the wall. The paved channel should be designed to intercept surface water and

direct it to an outlet as well as reduce the potential for surface water from overtopping 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.

5.10. Retaining Wall 5

A foundation review was completed for the proposed MSE wall – RW 5, located along the proposed Southbound Entrance Ramp between STA. 1+80 and Southbound Exit Ramp CD STA. 7+25.44. The following information was used in our design: 1) the soil/rock characteristics gathered during the subsurface exploration (i.e., SPT results, laboratory test results, etc.); 2) the referenced Site Plan prepared by Arcadis; and, 3) other design assumptions presented in Section 5.3.2 of this report. Geotechnical analyses consisting of external stability (i.e., bearing resistance, eccentricity, and sliding resistance), global stability, and settlement were performed for the proposed MSE wall in accordance with ODOT's BDM (ODOT, 2024) and AASHTO's LRFD BDS (AASHTO, 2020).

5.10.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 60 and 63 below.

Table 60: B-043-0-24 soil profile for analysis

Retaining Wall 5: Soil Profile B-043-0-24							
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)	Setup Factor (f_{su})
Coarse and Fine Sand (A-3a) Depth (1038.8 ft - 1034.3 ft)	122	122	132	-	-	40	1.00
Sandy Silt (A-4a) Depth (1034.3 ft - 1032.3 ft)	130	130	140	8000	450	32	1.50
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 61: B-044-0-24 soil profile for analysis

Retaining Wall 5: Soil Profile B-044-0-24							
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)	Setup Factor (f_{su})
Coarse and Fine Sand (A-3a) Depth (1038.5 ft - 1035.5 ft)	122	122	132	-	-	40	1.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 62: B-024-0-24 soil profile for analysis

Retaining Wall 5: Soil Profile B-024-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand and Silt (A-2-4) Depth (1036.2 ft - 1033.7 ft)	122	122	132	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

5.10.2. External Stability

Based on our estimated geotechnical soil properties and the retaining wall design assumptions provided in Sections 5.3.2. of this report, an external stability analysis of the proposed MSE wall was performed. One cross-section was evaluated for resistance to bearing pressure, sliding forces and overturning at the Strength Limit State for both drained and undrained conditions in accordance with Section 11.10.5 of the AASHTO's LRFD BDS. The capacity to demand ratios (CDRs) calculated for the referenced cross-sections with respect to bearing, sliding and overturning, as well as the calculated factored bearing resistances are presented in Table 63 below. (External Stability and Bearing Resistance Calculation Results can be found in Appendix E).

Based on the external stability analysis summary, the recommended soil reinforcement length is 70% of the design wall height or 8 feet, whichever is greater in according with ODOT GDM Section 1504 and BDM Section 307.4.

Table 63: External Stability Analysis Summary

MSE Wall Analysis Summary - Retaining Wall 5		
Station	STA. 504+28.33	
Top of Coping (ft)	1057.3	
Proposed Ground in front of wall (ft)	1037.2	
Top of Leveling Pad (ft)	1032.8	
Exposed Wall Height (ft)	20.0	
Design Wall Height (ft)	24.4	
Length of Reinforcement (ft)	17.2	
Length of Reinf. To Height Ratio	0.7	
Adjacent Boring Locations	B-024-0-24	
Capacity Demand Ratio (CDR)		
Condition	Undrained	Drained
Sliding	-	1.9
Overturning / Eccentricity	-	2.0
Bearing Resistance	-	26.3
Factored Bearing Resistance (ksf) ⁽¹⁾	-	157.4
Notes:		
1. Bearing Resistance calculated in accordance to Section 11.10.5.4 of 2024 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of LRFD BDS.		

5.10.3. Settlement

Settlement should not be a concern due to the presence of very shallow bedrock.

5.11. Retaining Wall 6

A foundation review was completed for RW 6 (Cast-in-place Wall), along the proposed Goodkirk Street between STA. 504+72.71 and STA. 508+48.78, based on the following design information: 1) the Site Plan for RW 6 conducted by Arcadis; and 2) subsurface exploration and lab testing results. The bottom of the RW 6 footing is at the elevation of 1035.00 ft. At the RW 6 site, bedrock was encountered at elevations of 1028.4 ft in the boring B-045-0-24, 1032.9 ft in the boring B-046-0-24 and 1035.4 ft in the boring B-047-0-24. Therefore, the end portion of RW 6 could be sitting on bedrock.

5.11.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were estimated based on their field (i.e., SPT N₆₀ Values, hand penetrometer values, etc.) and laboratory (i.e., Atterberg

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Limits, grain size, etc.) test results using correlations provided in published engineering manuals, research reports and guidance documents. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 64 and 66 below.

Table 64: B-045-0-24 soil profile for analysis

Retaining Wall 6: Soil Profile B-045-0-24							
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)	Setup Factor (f_{su})
Coarse and Fine Sand (A-3a) Depth (1044.4 ft - 1040.4 ft)	112	112	122	-	-	34	1.00
Coarse and Fine Sand (A-3a) Depth (1040.4 ft - 1029.9 ft)	120	120	130	-	-	39	1.00
Gravel with Sand and Silt (A-2-4) Depth (1029.9 ft - 1028.4 ft)	122	122	132	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 65: B-046-0-24 soil profile for analysis

Retaining Wall 6: Soil Profile B-046-0-24							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1049.9 ft - 1045.4 ft)	100	100	110	350	35	20	1.50
Fine Sand (A-3) Depth (1045.4 ft - 1042.9 ft)	112	112	122	-	-	32	1.20
Sandy Silt (A-4a) Depth (1042.9 ft - 1037.9 ft)	112	112	122	-	-	30	1.20
Gravel with Sand (A-1-b) Depth (1037.9 ft - 1035.4 ft)	125	125	135	-	-	40	1.00
Gravel with Sand and Silt (A-2-4) Depth (1035.4 ft - 1032.9 ft)	122	122	132	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 66: B-047-0-24 soil profile for analysis

Retaining Wall 6: Soil Profile B-047-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand and Silt (A-2-4) Depth (1048.4 ft - 1041.4 ft)	112	112	122	-	-	34	1.20
Gravel with Sand and Silt (A-2-4) Depth (1041.4 ft - 1038.9 ft)	118	118	128	-	-	39	1.20
Gravel with Sand (A-1-b) Depth (1038.9 ft - 1035.4 ft)	130	130	140	-	-	40	1.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

5.11.1. External Stability

Based on our estimated geotechnical soil properties, an external stability analysis was performed for the proposed CIP wall. Two cross-sections were evaluated for resistance to bearing pressure, sliding forces and overturning at the Strength Limit State for both drained and undrained conditions in accordance with Section 11.10.5 of the AASHTO's LRFD BDS. The capacity to demand ratios (CDRs) calculated for the referenced cross-sections with respect to bearing, sliding and overturning, as well as the calculated factored bearing resistances are presented in Table 67 below. External Stability and Bearing Resistance Calculation Results can be found in Appendix E.

Table 67: External Stability Analysis Summary

CIP Wall Analysis Summary - Retaining Wall 6				
Station	STA. 601+50		STA. 602+25	
Top of Wall (ft)	1048.2		1050.7	
Proposed Ground in front of wall (ft)	1040.0		1038.6	
Bottom of Wall (ft)	1035.0		1032.7	
Exposed Wall Height (ft)	8.2		12.1	
Design Wall Height (ft)	13.2		18.1	
Adjacent Boring Locations	B-045-0-24		B-046-0-24	
Capacity Demand Ratio (CDR)				
Condition	Undrained	Drained	Undrained	Drained
Sliding	2.2	2.2	-	-
Overturning / Eccentricity	7.4	7.4	-	1.7
Bearing Resistance	10.5	10.5	-	26.1
Factored Bearing Resistance (ksf) ⁽¹⁾	20.6	20.6	-	110.7
Notes:				
1. Bearing Resistance calculated in accordance to Section 11.6 of 2020 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of 2020 LRFD BDS.				

5.11.1. Global Stability

Since the bottom of footing will be keyed into bedrock, global stability should not be a concern, as the bedrock provides a solid foundation for stability.

5.12. Retaining Wall 7

A foundation review was completed for the proposed MSE wall – RW 7, located along the proposed Southbound Exit Ramp CD between STA. 134+50 and STA. 136+65.74. The following information was used in our design: 1) the soil/rock characteristics gathered during the subsurface exploration (i.e., SPT results, laboratory test results, etc.); 2) the referenced Site Plan prepared by Arcadis; and, 3) other design assumptions presented in Section 5.3.2 of this report. Geotechnical analyses consisting of external stability (i.e., bearing resistance, eccentricity, and sliding resistance), global stability, and settlement were performed for the proposed MSE wall in accordance with ODOT's BDM (ODOT, 2024) and AASHTO's LRFD BDS (AASHTO, 2020).

5.12.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 68 and 69 below.

Table 68: B-048-0-24 soil profile for analysis

Retaining Wall 7: Soil Profile B-048-0-24							
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)	Setup Factor (f_{su})
Coarse and Fine Sand (A-3a) Depth (1037.8 ft - 1033.3 ft)	130	130	140	-	-	40	1.00
Sandy Silt (A-4a) Depth (1033.3 ft - 1031.6 ft)	130	130	140	-	-	40	1.20
Notes:							
1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405.							
2. Values calculated from Terzaghi and Peck (1967) if N_{160-52} , else Stroud and Butler (1975) was used.							
3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 69: B-049-0-24 soil profile for analysis

Retaining Wall 7: Soil Profile B-049-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand and Silt (A-2-4) Depth (1040.4 ft - 1037.4 ft)	125	125	135	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

5.12.2. External Stability

Based on our estimated geotechnical soil properties and the retaining wall design assumptions provided in Sections 5.3.2. of this report, an external stability analysis of the proposed MSE wall was performed. The most critical cross-section with tallest design height was evaluated for resistance to bearing pressure, sliding forces and overturning at the Strength Limit State for both drained and undrained conditions in accordance with Section 11.10.5 of the AASHTO's LRFD BDS. The capacity to demand ratios (CDRs) calculated for the referenced cross-sections with respect to bearing, sliding and overturning, as well as the calculated factored bearing resistances are presented in Table 70 below. (External Stability and Bearing Resistance Calculation Results can be found in Appendix E)

Based on the external stability analysis summary, the recommended soil reinforcement length is 70% of the design wall height.

Table 70: External Stability Analysis Summary

MSE Wall Analysis Summary- RW 7		
Station	STA. 702+13.70	
Top of Coping (ft)	1055.9	
Proposed Ground in front of wall (ft)	1036.8	
Bottom of Leveling Pad (ft)	1032.6	
Exposed Wall Height (ft)	19.1	
Design Wall Height (ft)	23.3	
Length of Reinforcement (ft)	16.3	
Length of Reinf. To Height Ratio	0.7	
Adjacent Boring Locations	B-048-0-24	
Capacity Demand Ratio (CDR)		
Condition	Undrained	Drained
Sliding	-	1.8
Overturning / Eccentricity	-	1.9
Bearing Resistance	-	21.8
Factored Bearing Resistance (ksf) ⁽¹⁾	-	124.7
Notes:		
1. Bearing Resistance calculated in accordance to Section 11.10.5.4 of 2024 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of LRFD BDS.		

5.12.3. Settlement

Settlement should not be a concern due to the presence of very shallow bedrock.

5.13. Retaining Wall 8

A foundation review was completed for the proposed Cast-in-place wall – RW 8, located along the proposed Southbound Exit Ramp CD between STA. 134+63 and STA. 136+87.50. The following information was used in our design: 1) the soil/rock characteristics gathered during the subsurface exploration (i.e., SPT results, laboratory test results, etc.); and 2) the referenced Site Plan prepared by Arcadis. Geotechnical analyses consisting of external stability (i.e., bearing resistance, eccentricity, and sliding resistance), global

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stability, and settlement were performed for the proposed CIP wall in accordance with ODOT's BDM (ODOT, 2024) and AASHTO's LRFD BDS (AASHTO, 2020).

5.13.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 71 and 72 below.

Table 71: B-050-0-24 soil profile for analysis

Retaining Wall 8: Soil Profile B-050-0-24							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1038.1 ft - 1033.6 ft)	130	130	140	8400	450	31	1.50
Gravel with Sand (A-1-b) Depth (1033.6 ft - 1031.1 ft)	125	125	135	-	-	40	1.00
Gravel with Sand and Silt (A-2-4) Depth (1031.1 ft - 1030.1 ft)	130	130	140	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 72: B-051-0-24 soil profile for analysis

Retaining Wall 8: Soil Profile B-051-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand and Silt (A-2-4) Depth (1038.7 ft - 1033.7 ft)	130	130	140	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

5.13.1. External Stability

A foundation review was completed for RW 8 based on the following design information: 1) the Site Plan for RW 8 conducted by Arcadis; and 2) subsurface exploration and lab testing results. The bottom of the RW 8 footing is at an elevation of 1032.35 ft. At the RW 8 site, bedrock was encountered at elevations of 1030.1 ft in boring B-050-0-23 and 1033.7 ft in boring B-051-0-23, indicating that a portion of RW 8 may be founded on bedrock.

Based on our estimated geotechnical soil properties, an external stability analysis of the proposed CIP wall was performed. The most critical cross-section with tallest design height was evaluated for resistance to bearing pressure, sliding forces and overturning at the Strength Limit State for both drained and undrained conditions in accordance with Section 11.10.5 of the AASHTO's LRFD BDS. The capacity to demand ratios (CDRs) calculated for the referenced cross-sections with respect to bearing, sliding and overturning, as well as the calculated factored bearing resistances are presented in Table 73 below. (External Stability and Bearing Resistance Calculation Results can be found in Appendix E)

Table 73: External Stability Analysis Summary

CIP Wall Analysis Summary- RW 8		
Station	STA. 802+25.39	
Top of Wall (ft)	1042.1	
Proposed Ground in front of wall (ft)	1035.9	
Bottom of Wall (ft)	1031.6	
Exposed Wall Height (ft)	6.3	
Design Wall Height (ft)	10.5	
Adjacent Boring Locations	B-050-0-24	
Capacity Demand Ratio (CDR)		
Condition	Undrained	Drained
Sliding	-	-
Overturning / Eccentricity	-	4.6
Bearing Resistance	-	37.0
Factored Bearing Resistance (ksf) ⁽¹⁾	-	82.0
Notes:		
1. Bearing Resistance calculated in accordance to Section 11.10.5.4 of 2024 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of LRFD BDS.		

5.13.2. Global Stability

Since the bottom of footing will be keyed into bedrock, global stability should not be a concern.

5.14. Retaining Wall 9

A foundation review was completed for the proposed Cast-in-place wall – RW 9, located along the proposed Southbound Exit Ramp CD between STA. 139+72.29 and STA. 142+90.02. The following information was used in our design: 1) the soil/rock characteristics gathered during the subsurface exploration (i.e., SPT results, laboratory test results, etc.); and 2) the referenced Site Plan prepared by Arcadis. Geotechnical analyses consisting of external stability (i.e., bearing resistance, eccentricity, and sliding resistance), and global stability were performed for the proposed CIP wall in accordance with ODOT's BDM (ODOT, 2024) and AASHTO's LRFD BDS (AASHTO, 2020).

5.14.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 74 and 76 below.

Table 74: B-052-0-24 soil profile for analysis

Retaining Wall 9: Soil Profile B-052-0-24							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1053.7 ft - 1049.2 ft)	110	110	120	1500	150	24	1.50
Gravel with Sand (A-1-b) Depth (1049.2 ft - 1041.7 ft)	120	120	130	-	-	40	1.00
Gravel with Sand and Silt (A-2-4) Depth (1041.7 ft - 1040.7 ft)	122	122	132	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 75: B-053-0-24 soil profile for analysis

Retaining Wall 9: Soil Profile B-053-0-24							
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)	Setup Factor (f_{su})
Coarse and Fine Sand (A-3a) Depth (1034.5 ft - 1031.5 ft)	122	122	132	-	-	40	1.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 76: B-054-0-24 soil profile for analysis

Retaining Wall 9: Soil Profile B-054-0-24							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1056.2 ft - 1049.2 ft)	115	115	125	-	-	35	1.20
Sandy Silt (A-4a) Depth (1049.2 ft - 1046.2 ft)	125	125	135	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

5.14.1. External Stability

A foundation review was completed for RW 9 based on the following design information: 1) the Site Plan for RW 9 conducted by Arcadis; and 2) subsurface exploration and lab testing results. The bottom of the RW 9 footing ranges between 1029.25 ft and 1030.0 ft. At the RW 9 site, bedrock was encountered at elevations of 1040.7 ft in boring B-052-0-24, 1031.5 ft in boring B-053-0-24, and 1046.2 ft in boring B-054-0-24, indicating that RW 9 will be founded on bedrock.

Based on our estimated geotechnical soil properties, an external stability analysis of the proposed CIP wall was performed. The most critical cross-section with tallest design height was evaluated for resistance to bearing pressure, sliding forces and overturning at the Strength Limit State for both drained and undrained conditions in accordance with Section 11.10.5 of the AASHTO's LRFD BDS. The capacity to demand ratios (CDRs) calculated for the referenced cross-sections with respect to bearing, sliding and overturning, as well as the calculated factored bearing resistances are presented in Table 77 below. (External Stability and Bearing Resistance Calculation Results can be found in Appendix E)

Table 77: External Stability Analysis Summary

CIP Wall Analysis Summary - Retaining Wall 9				
Station	STA. 997+08		STA. 999+50	
Top of Wall (ft)	1055.6		1042.8	
Proposed Ground in front of wall (ft)	1035.3		1033.5	
Bottom of Wall (ft)	1030.0		1029.3	
Exposed Wall Height (ft)	20.3		9.3	
Design Wall Height (ft)	25.6		13.6	
Adjacent Boring Locations	B-052-0-24		B-054-0-24	
Capacity Demand Ratio (CDR)				
Condition	Undrained	Drained	Undrained	Drained
Sliding	-	-	-	-
Overturning / Eccentricity	-	5.4	-	3.8
Bearing Resistance	-	24.0	-	50.5
Factored Bearing Resistance (ksf) ⁽¹⁾	-	102.6	-	142.8
Notes:				
1. Bearing Resistance calculated in accordance to Section 11.6 of 2024 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of LRFD BDS.				

5.14.2. Global Stability

Since the bottom of footing will be keyed into bedrock, global stability should not be a concern.

5.15. Retaining Wall 10

A foundation review was completed for the proposed Cast-in-place wall – RW 10, located along the proposed Southbound Exit Ramp CD between STA. 143+52.85 and STA. 147+40.26. The following information was used in our design: 1) the soil/rock characteristics gathered during the subsurface exploration (i.e., SPT results, laboratory test results, etc.); and 2) the referenced Site Plan prepared by Arcadis. Geotechnical analyses consisting of external stability (i.e., bearing resistance, eccentricity, and sliding resistance), and global stability were performed for the proposed CIP wall in accordance with ODOT's BDM (ODOT, 2024) and AASHTO's LRFD BDS (AASHTO, 2020).

5.15.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were 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. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 78 and 80 below.

Table 78: B-055-0-24 soil profile for analysis

Retaining Wall 10: Soil Profile B-055-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand and Silt (A-2-4) Depth (1034.4 ft - 1031.9 ft)	122	122	132	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 79: B-056-0-24 soil profile for analysis

Retaining Wall 10: Soil Profile B-056-0-24							
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)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (1060.6 ft - 1051.1 ft)	115	115	125	-	-	32	1.20
Sandy Silt (A-4a) Depth (1051.1 ft - 1048.6 ft)	115	115	125	3350	250	27	1.50
Gravel with Sand and Silt (A-2-4) Depth (1048.6 ft - 1041.1 ft)	118	118	128	-	-	35	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 80: B-057-0-24 soil profile for analysis

Retaining Wall 10: Soil Profile B-057-0-24							
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)	Setup Factor (f_{su})
Gravel with Sand and Silt (A-2-4) Depth (1039.3 ft - 1034.8 ft)	130	130	140	-	-	40	1.20
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

5.15.1. External Stability

A foundation review was completed for RW 10 based on the following design information: 1) the Site Plan for RW 10 conducted by Arcadis; and 2) subsurface exploration and lab testing results. The bottom of the RW 10 footing ranges from elevations of 1029.60 ft to 1035.39 ft. At the RW 10 site, bedrock was

encountered at elevations of 1048.7 ft in boring B-027-0-24, 1031.9 ft in boring B-055-0-24, 1041.1 ft in boring B-056-0-24, and 1034.8 ft in boring B-057-0-24, indicating that RW 10 will be founded on bedrock.

Based on our estimated geotechnical soil properties, an external stability analysis of the proposed CIP wall was performed. The most critical cross-section with tallest design height was evaluated for resistance to bearing pressure, sliding forces and overturning at the Strength Limit State for both drained and undrained conditions in accordance with Section 11.10.5 of the AASHTO's LRFD BDS. The capacity to demand ratios (CDRs) calculated for the referenced cross-sections with respect to bearing, sliding and overturning, as well as the calculated factored bearing resistances are presented in Table 81 below. (External Stability and Bearing Resistance Calculation Results can be found in Appendix E).

Table 81: External Stability Analysis Summary

CIP Wall Analysis Summary - Retaining Wall 10				
Station	STA. 1100+02.87		STA. 1102+00	
Top of Wall (ft)	1056.1		1043.0	
Proposed Ground in front of wall (ft)	1033.6		1036.1	
Bottom of Wall (ft)	1029.6		1031.8	
Exposed Wall Height (ft)	22.5		6.9	
Design Wall Height (ft)	26.5		11.2	
Adjacent Boring Locations	B-055-0-24		B-056-0-24	
Capacity Demand Ratio (CDR)				
Condition	Undrained	Drained	Undrained	Drained
Sliding	-	-	-	-
Overturning / Eccentricity	-	2.2	-	5.6
Bearing Resistance	-	28.5	-	46.1
Factored Bearing Resistance (ksf) ⁽¹⁾	-	174.8	-	97.9
Notes:				
1. Bearing Resistance calculated in accordance to Section 11.6 of 2024 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of LRFD BDS.				

5.15.2. Global Stability

Since the bottom of footing will be keyed into bedrock, global stability should not be a concern.

5.16. CCTV Camera Pole

5.16.1. Foundation Analysis

We understand that one new CCTV Camera pole is planned at the approximate STA. 354+05 in reference to SR-8 NB. The proposed structure is a standard 70 feet CCTV concrete pole.

To aid in the design of the proposed CCTV Camera Pole foundation at STA. 354+05, the boring log performed nearby was reviewed, and a generalized material profile was developed. Utilizing the generalized soil profile, engineering properties for each soil strata were estimated based on their field (i.e., SPT N60 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. The developed soil profile and estimated engineering soil properties for evaluation of the proposed pole structure foundation (with sited correlation/reference material) is summarized within Table 82 and Table 83 below.

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Summit County, Ohio
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Table 82: Soil Profile and Estimated Properties - Boring B-058-0-26

Light Tower: Soil Profile B-058-0-26							
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)	Setup Factor (f_{su})
Gravel with Sand (A-1-b) Depth (1036.3 ft - 1031.8 ft)	120	120	130	-	-	40	1.00
Gravel (A-1-a) Depth (1031.8 ft - 1029.3 ft)	115	115	125	-	-	37	1.00
Coarse and Fine Sand (A-3a) Depth (1029.3 ft - 1027.9 ft)	122	122	132	-	-	40	1.00
Notes: 1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405. 2. Values calculated from Terzaghi and Peck (1967) if N160-52, else Stroud and Butler (1975) was used. 3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.							

Table 83: Soil Properties For p-y curve Method - Boring B-058-0-26

Layer Number (No.)	Top Elev. (ft)	Bottom Elev. (ft)	Layer Depth (ft)	Soil/Rock Class	LPILE p-y Model	Soil Strain Parameter ϵ_{50}	Soil Modulus Parameter p-y k (pci)	Rock RQD (%)	Rock Uniaxial Compressive Strength q_u (psi)	Rock Strain Factor k_m ¹	Rock Initial Modulus of Rock Mass (psi) ²
B-058-0-26											
1	1036.3	1031.8	4.5	A-1-b	Sand (Reese)	-	965	-	-	-	-
2	1031.8	1029.3	7	A-1-a	Sand (Reese)	-	353	-	-	-	-
3	1029.3	1027.9	8.4	A-3a	Sand (Reese)	-	656	-	-	-	-
4	1027.9	1025.8	10.5	Sandstone	Weak Rock	-	-	0	1,125	0.00016	3,000
5	1025.8	1015.8	20.5	Interbedded Sandstone and Shale	Weak Rock	-	-	26	1,067	0.00017	5,760
Note: 1. Values interpreted from LPILE Technical Manual. 2. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 406.											

A lateral load analysis (p-y analysis) was performed utilizing the software entitled LPILE v2022 by Ensoft, Inc for checking excessive lateral deflection in Service I limit state and for checking overturning in Extreme I Limit State. Our p-y analyses were performed based on: 1) our estimated engineering soil properties and p-y method soil parameters provided in Table 82 and Table 83 of this report; 2) ODOT standard drawing ITS-12.10 (Concrete CCTV Camera Pole); 3) ODOT's ITS-10.10 (CCTV Assembly Details, Ground Mounted Cabinet) standard details; 4) embedded pole into ground; and 5) Service and Extreme limit state load, pole properties, calculation steps and failure criterion listed in the ODOT'S GDM Section 1200. It should be noted that the top 3 ft starting from the proposed ground surface is assumed to be soft clay with undrained shear strength $S_u = 250$ psf to account for soil weakened by freeze-thaw or other disturbance, according to ODOT'S GDM Section 1200.

Based on our p-y analysis, by using the pole foundation length $L = 10$ ft, the Fixed tip condition can be achieved for the Service I Limit State. The projected deflection at the top of the CCTV Camera Pole is 0.093 in. For the Extreme I Limit State design, no overturning issue was detected. The p-y analysis results are summarized in Table 84 below. The LPILE outputs are included within Appendix F.

Table 84: STA. CCTV Pole p-y Analysis Results

CCTV Camera Pole Location	CCTV Camera Pole Height Above Ground (ft)	CCTV Camera Pole Foundation Length (ft)	Extreme I Limit State		Service I Limit State						
			Maximum Moment in Pole Foundation (in-lbs)	Maximum Shear in Pole Foundation (lbs)	Foundation Head Deflection (in)	Foundation Butt Deflection (in)	Differential Deflection over Foundation Length (in)	Differential Deflection / Foundation Head Deflection	Is It Fixed-Tip (Differential Deflection / Foundation Head Deflection < 1.005)	Lateral Ration at Foundation Head (radians)	Lateral Deflection at Pole Head (in)
STA. 354+05	70	10	533,108	-13,256	0.00402	0.000005	0.004025	1.001366	Yes	0.0001	0.093

5.17. Seismic Site Class

Based on the results of the subsurface exploration, laboratory test data, and the AASHTO Site Class Definitions indicated in Table 3.10.3.1-1 of the *LRFD Bridge Design Specifications, 9th Edition* (AASHTO LRFD, 2020), the average Standard Penetration Test blow count $\bar{N}$ is between 15 blows/ft and 50 blows/ft. A Seismic Site Class D is recommended for the overall project site.

6. QUALIFICATIONS

This investigation was performed in accordance with accepted geotechnical engineering practice for the purpose of characterizing the subsurface conditions at the proposed roadway improvement project (SUM-77/SR8-11.65/0.00 project, PID: 114865) in Summit County, Ohio. This report has been prepared for Arcadis U.S., Inc., ODOT and their design consultants to be used solely in evaluating the soils underlying the indicated structures 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 test results from representative soil/rock 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 occur in the nature, design or location of the proposed structural work, 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 Arcadis U.S., Inc. in performing this geotechnical exploration for the SUM-77/SR8-11.65/0.00 project. Please call if there are any questions, or if we can be of further service.

Respectfully Submitted,

Melinda He, Ph.D., P.E.
Project Manager

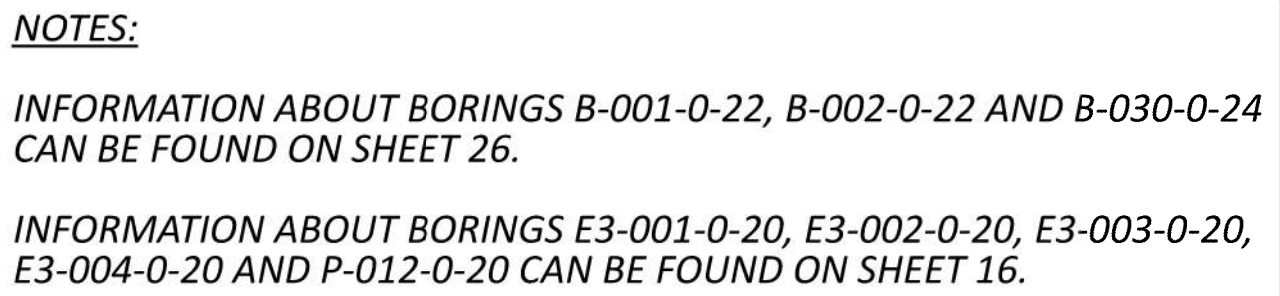
Zhao Mankoci, Ph.D., P.E.
Geotechnical Engineer

REFERENCES

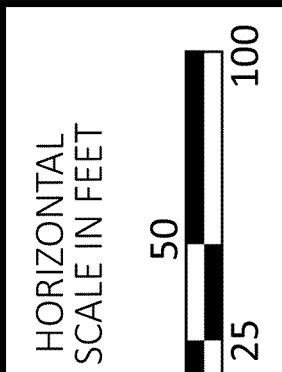
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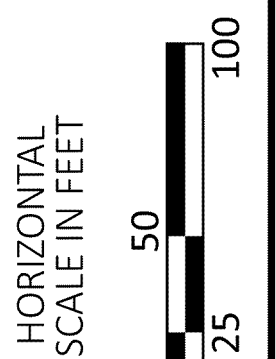
APPENDIX A

BORING LOCATION PLAN



NOTE: BORING LOCATIONS PROVIDED IN
PROFILE VIEW REFERENCE C CONST. SR-8.





DESIGN AGENCY
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National Engineering & Architectural Services Inc.
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DESIGNER	
RS	
REVIEWER	
CH 08/14/26	
PROJECT ID	
114865	
SUBSET	TOTAL
9	116
SHEET	TOTAL
P.518	625

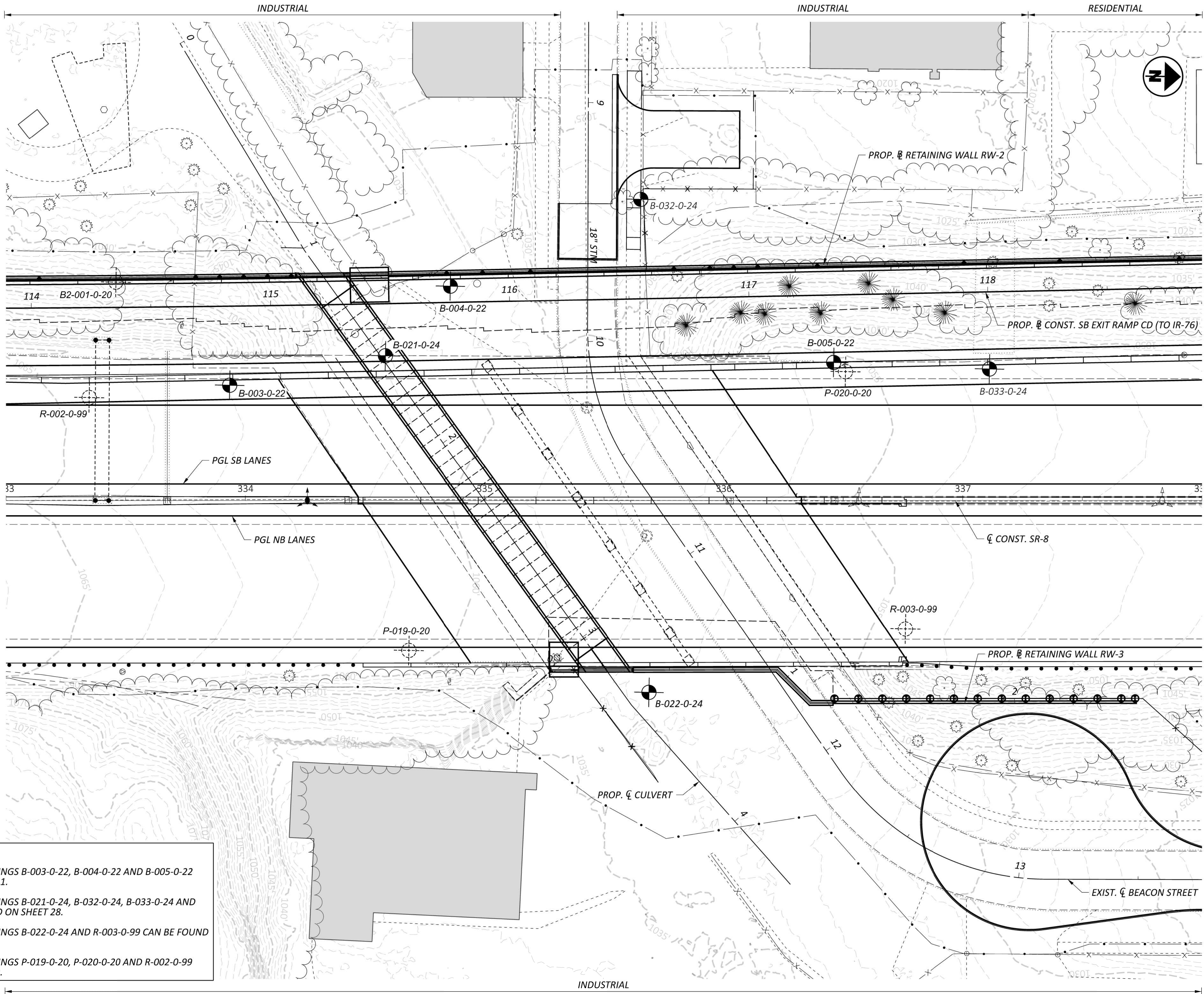
NOTES:

INFORMATION ABOUT BORINGS B-003-0-22, B-004-0-22 AND B-005-0-22
CAN BE FOUND ON SHEET 11.

INFORMATION ABOUT BORINGS B-021-0-24, B-032-0-24, B-033-0-24 AND
B2-001-0-20 CAN BE FOUND ON SHEET 28.

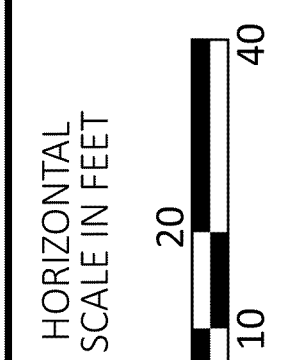
INFORMATION ABOUT BORINGS B-022-0-24 AND R-003-0-99 CAN BE FOUND
ON SHEET 33.

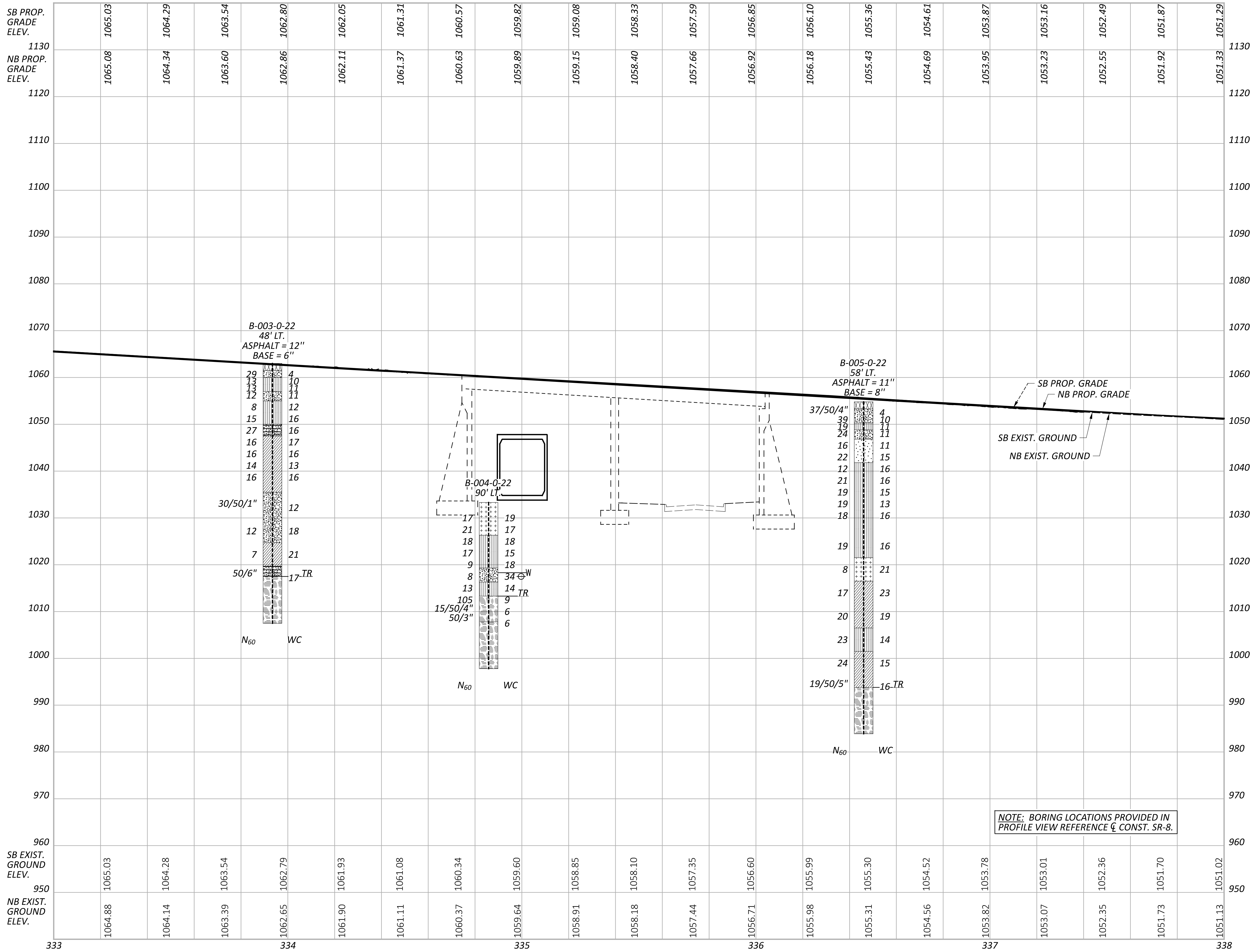
INFORMATION ABOUT BORINGS P-019-0-20, P-020-0-20 AND R-002-0-99
CAN BE FOUND ON SHEET 9.



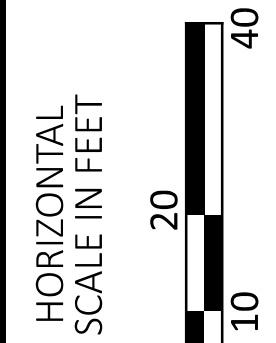
GEOTECHNICAL PROFILE - ROADWAY
BRIDGE NO. SUM-8-0.770
OVER BIKE TRAIL AND ABANDONED BEACON STREET

SFN 7700091	
DESIGN AGENCY NEAS NEAS Engineering & Consulting, Inc.	
2800 CORPORATE EXCHANGE DR. SUITE 240 COLUMBUS, OH, 43231 TEL: 614.714.0299 WWW.NEASINC.COM	
DESIGNER RS	CHECKER DT
REVIEWER CH 08/14/26	
PROJECT ID 114865	
SUBSET 10	TOTAL 116
SHEET P.519	TOTAL 625

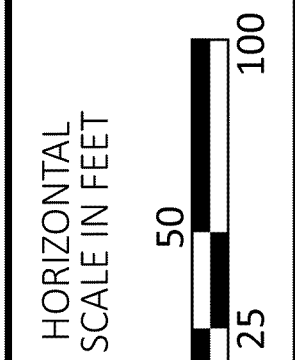




GEOTECHNICAL PROFILE - ROADWAY
BRIDGE NO. SUM-8-0.770
OVER BIKE TRAIL AND ABANDONED BEACON STREET



SFN 7700091	
DESIGN AGENCY NEAS 2800 CORPORATE EXCHANGE DR, SUITE 240 COLUMBUS, OH, 43231 TEL: 614.714.0299 WWW.NEASINC.COM	
DESIGNER RS	CHECKER DT
REVIEWER CH 08/14/26	
PROJECT ID 114865	
SUBSET 11	TOTAL 116
SHEET P.520	TOTAL 625

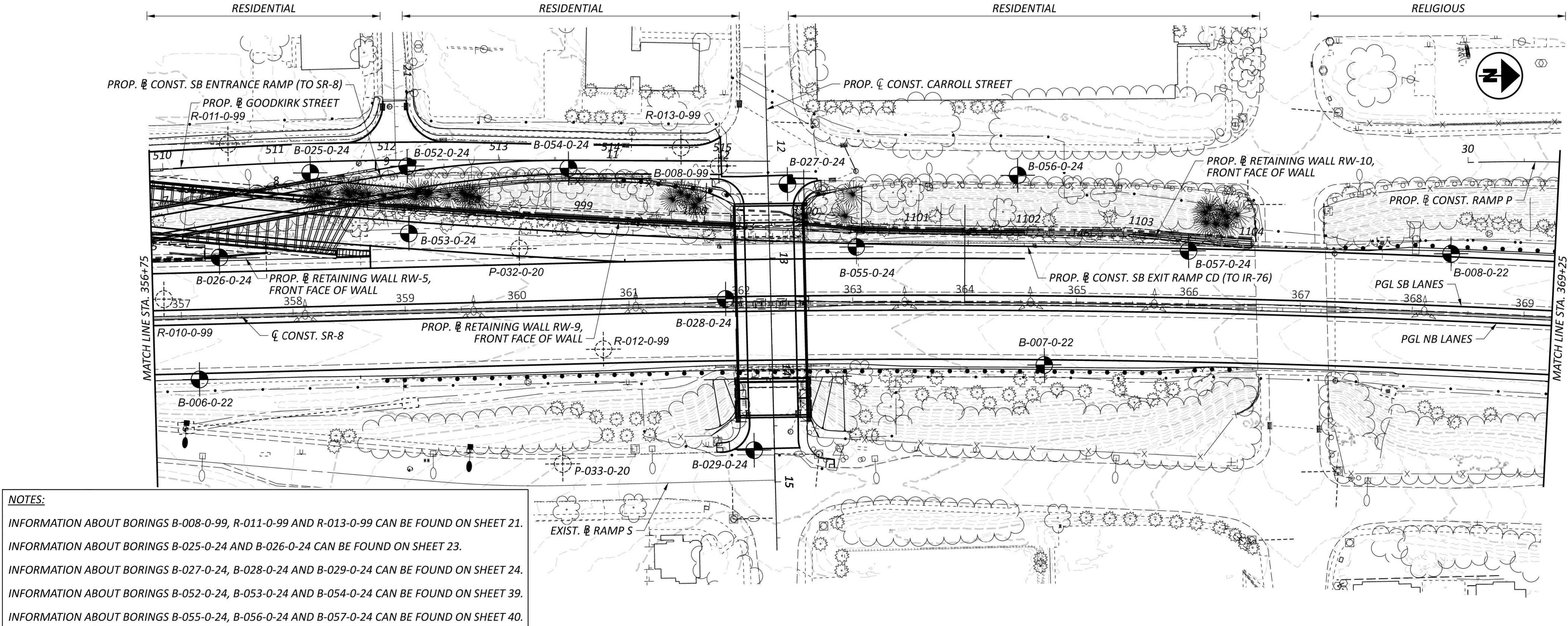


INFORMATION ABOUT BORINGS B-043-0-24 AND B-044-0-24
CAN BE FOUND ON SHEET 35.

INFORMATION ABOUT BORINGS B-045-0-24, B-046-0-24 AND B-047-0-24 CAN BE FOUND ON SHEET 36.

INFORMATION ABOUT BORINGS GB-001-0-99 AND GB-002-0-
CAN BE FOUND ON SHEET 19.





NOTES:

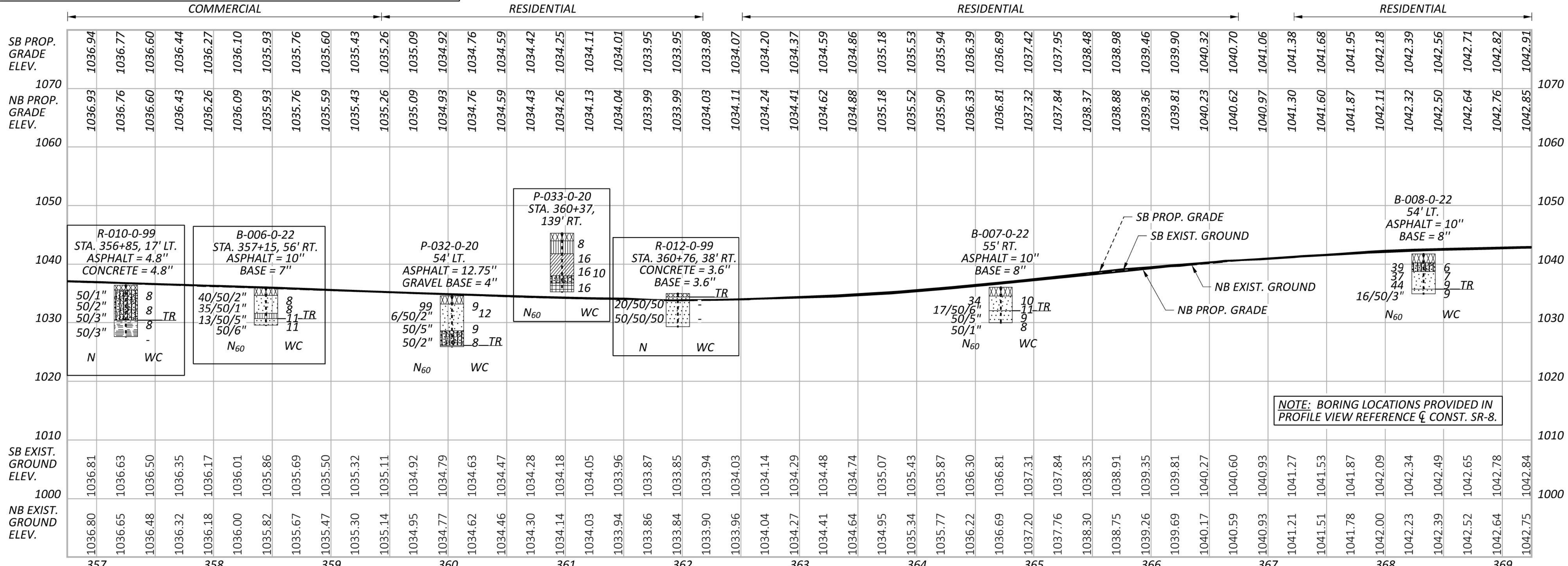
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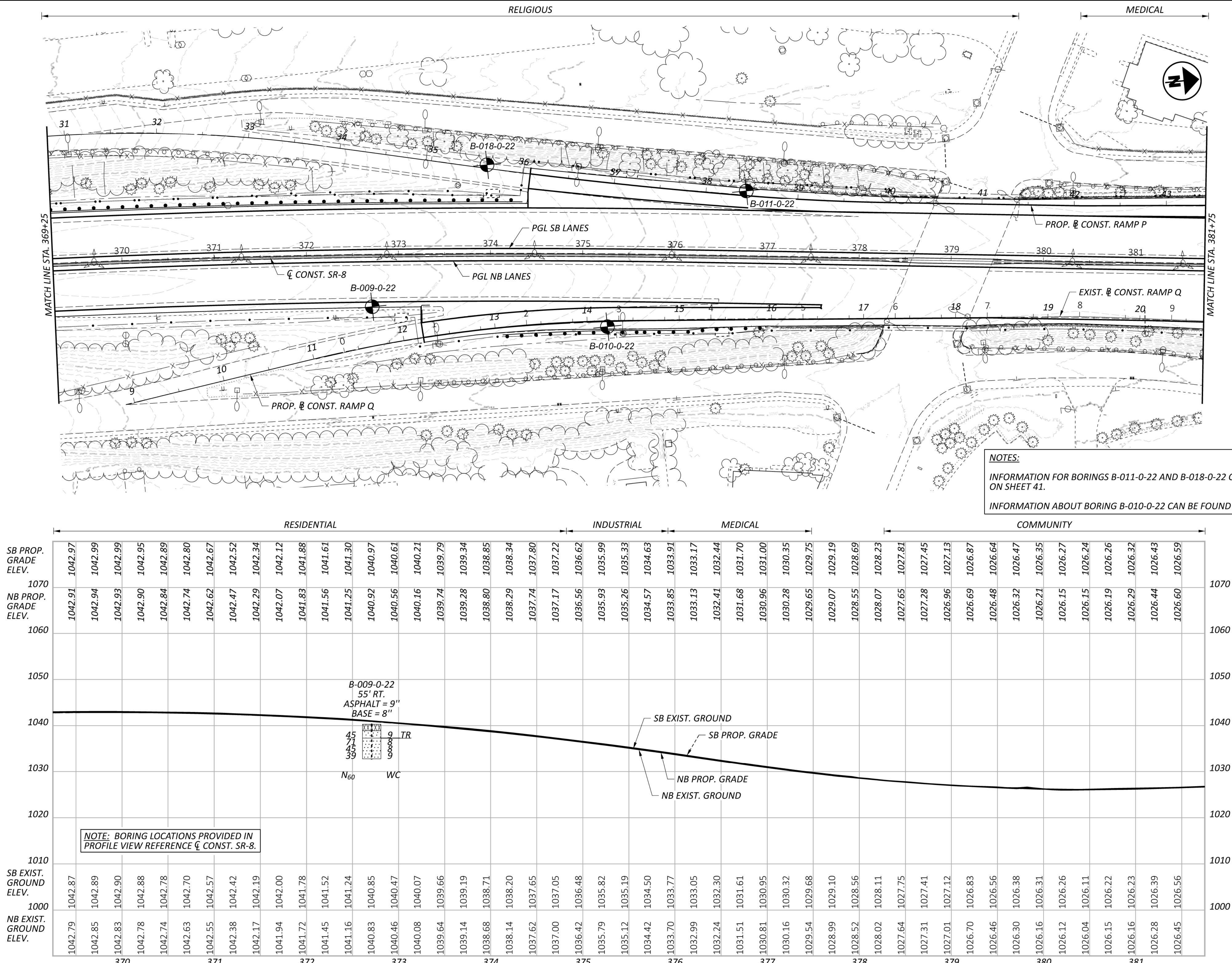
INFORMATION ABOUT BORINGS B-025-0-24 AND B-026-0-24 CAN BE FOUND ON SHEET 23.

INFORMATION ABOUT BORINGS B-027-0-24, B-028-0-24 AND B-029-0-24 CAN BE FOUND ON SHEET 24.

INFORMATION ABOUT BORINGS B-052-0-24, B-053-0-24 AND B-054-0-24 CAN BE FOUND ON SHEET 39.

INFORMATION ABOUT BORINGS B-055-0-24, B-056-0-24 AND B-057-0-24 CAN BE FOUND ON SHEET 40.



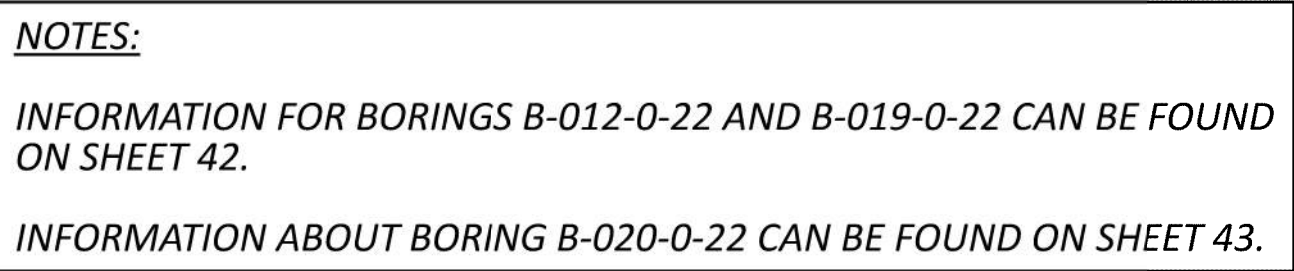


GEOTECHNICAL PROFILE - ROADWAY
SR-8 - STA. 369+25 TO STA. 381+75

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DESIGNER	RS
REVIEWER	CH
PROJECT ID	114865
SUBSET	14
TOTAL	116
SHEET	P.523
TOTAL	625

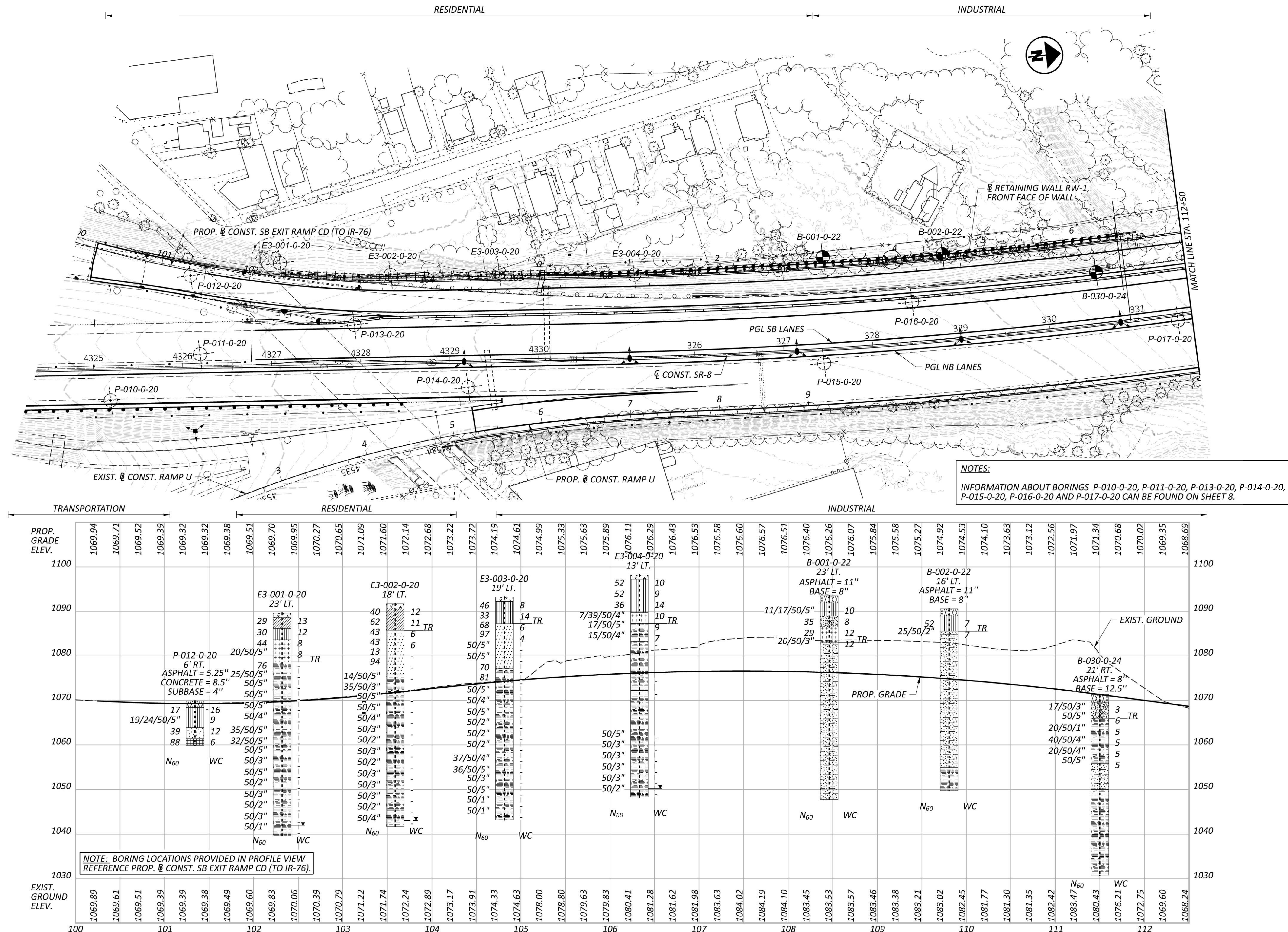
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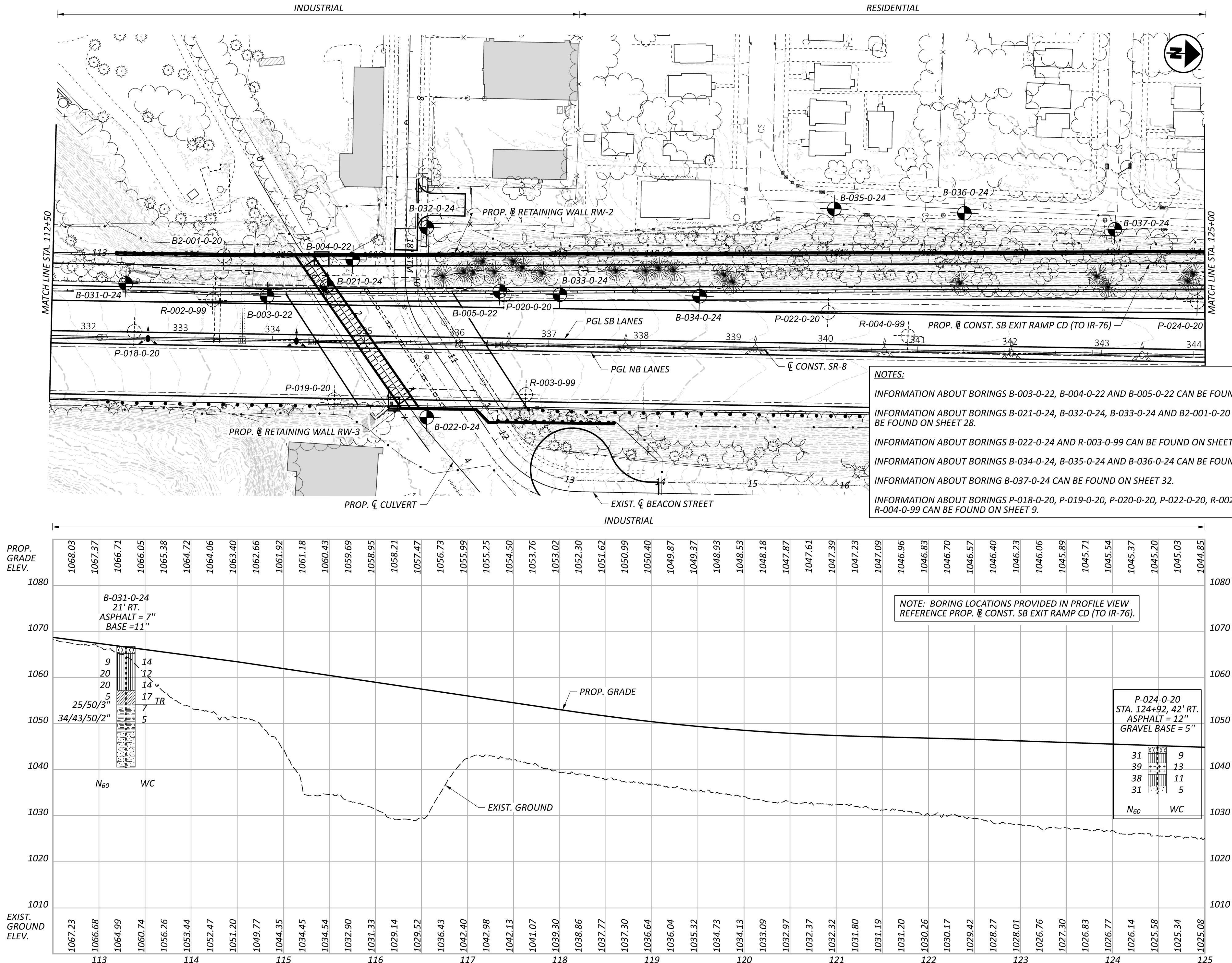


DESIGN AGENCY

National Engineering & Architectural Services Inc.
 2800 CORPORATE
 EXCHANGE DR,
 SUITE 240
 COLUMBUS, OH,
 43231
 TEL: 614.714.0299
 WWW.NEASINC.COM

DESIGNER	
RS	
REVIEWER	
CH	08/14/20
PROJECT ID	
114865	
SUBSET	TOTAL
15	116
SHEET	TOTAL
P.524	625

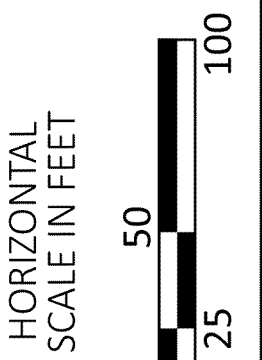
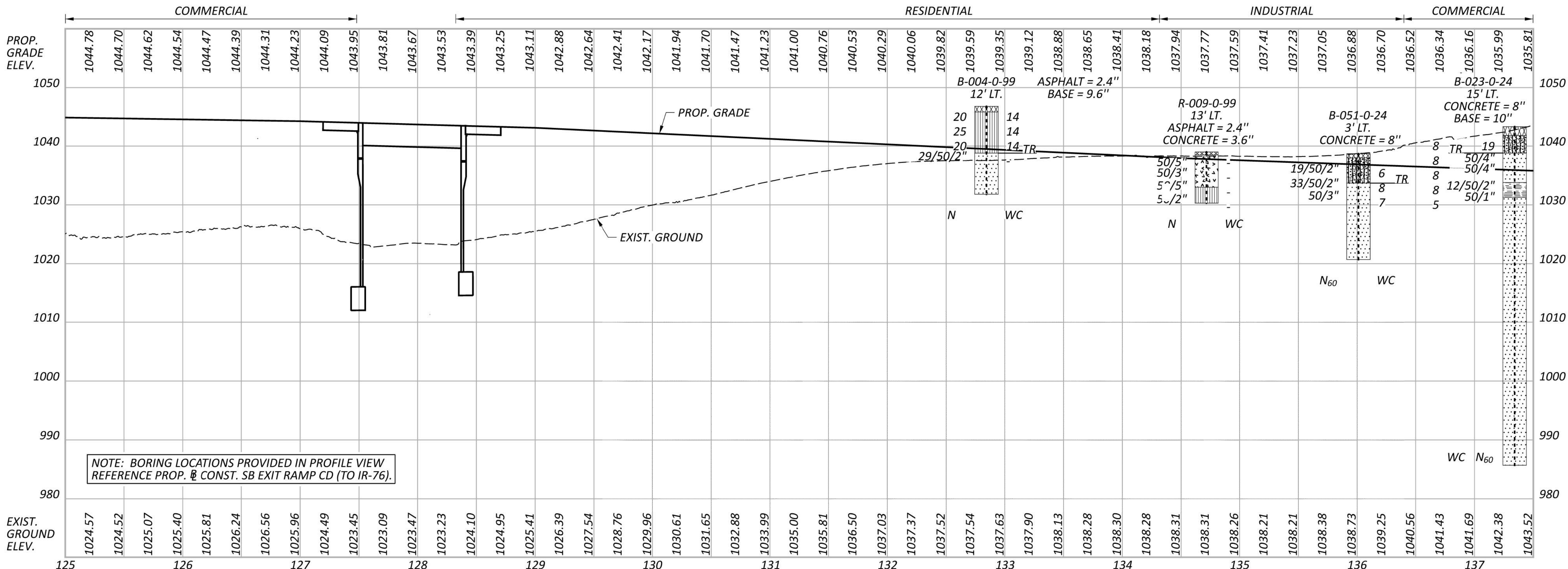
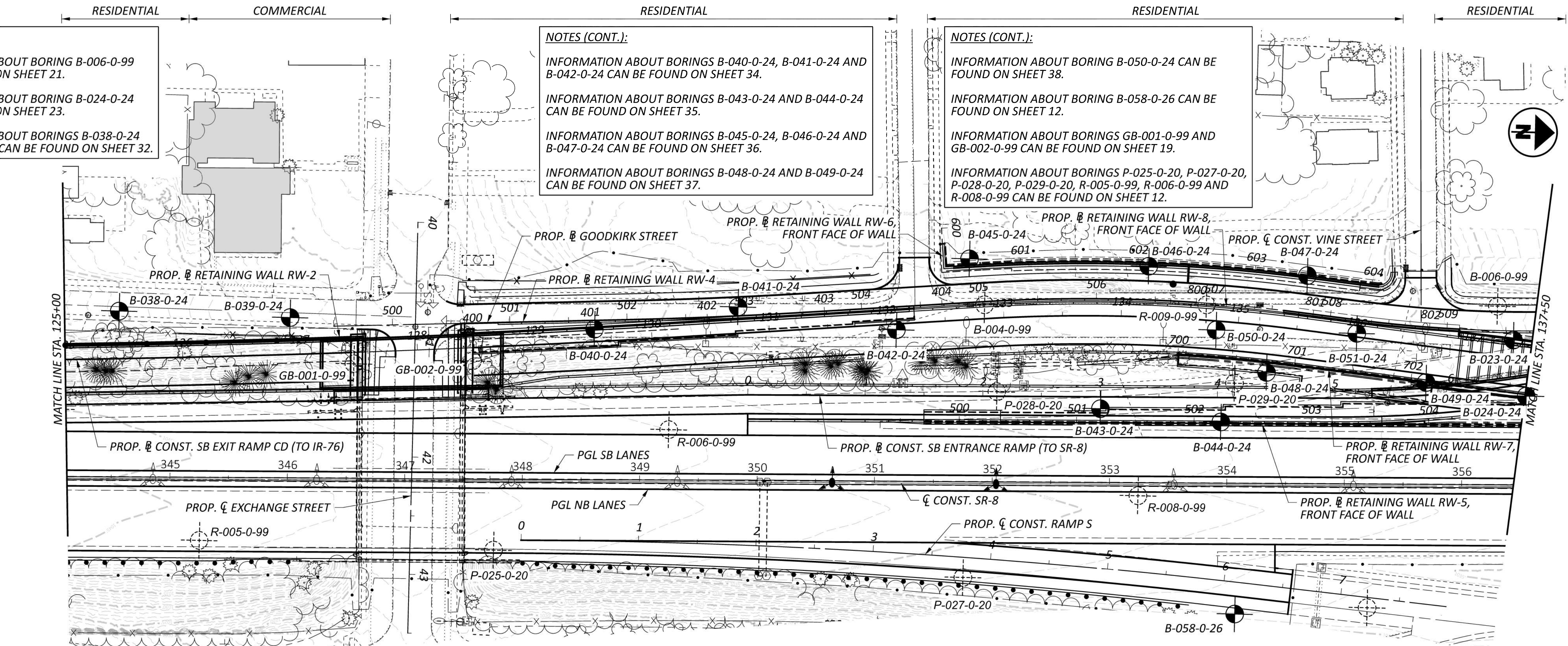




NOTES:
INFORMATION ABOUT BORING B-006-0-99
CAN BE FOUND ON SHEET 21.
INFORMATION ABOUT BORING B-024-0-24
CAN BE FOUND ON SHEET 23.
INFORMATION ABOUT BORINGS B-038-0-24
AND B-039-0-24 CAN BE FOUND ON SHEET 32.

NOTES (CONT.):
INFORMATION ABOUT BORINGS B-040-0-24, B-041-0-24 AND
B-042-0-24 CAN BE FOUND ON SHEET 34.
INFORMATION ABOUT BORINGS B-043-0-24 AND B-044-0-24
CAN BE FOUND ON SHEET 35.
INFORMATION ABOUT BORINGS B-045-0-24, B-046-0-24 AND
B-047-0-24 CAN BE FOUND ON SHEET 36.
INFORMATION ABOUT BORINGS B-048-0-24 AND B-049-0-24
CAN BE FOUND ON SHEET 37.

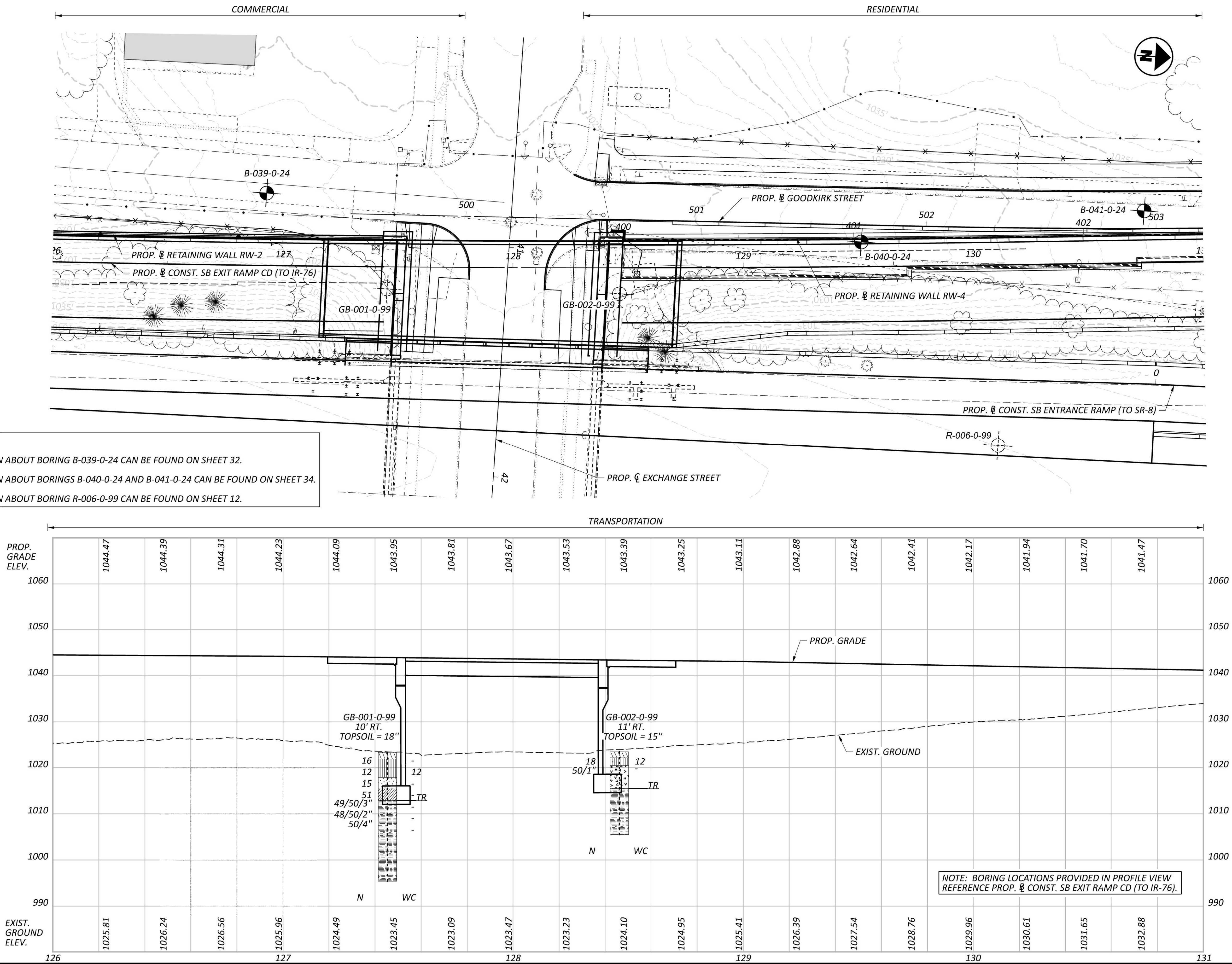
NOTES (CONT.):
INFORMATION ABOUT BORING B-050-0-24 CAN BE
FOUND ON SHEET 38.
INFORMATION ABOUT BORING B-058-0-26 CAN BE
FOUND ON SHEET 12.
INFORMATION ABOUT BORINGS GB-001-0-99 AND
GB-002-0-99 CAN BE FOUND ON SHEET 19.
INFORMATION ABOUT BORINGS P-025-0-20, P-027-0-20,
P-028-0-20, P-029-0-20, R-005-0-99, R-006-0-99 AND
R-008-0-99 CAN BE FOUND ON SHEET 12.



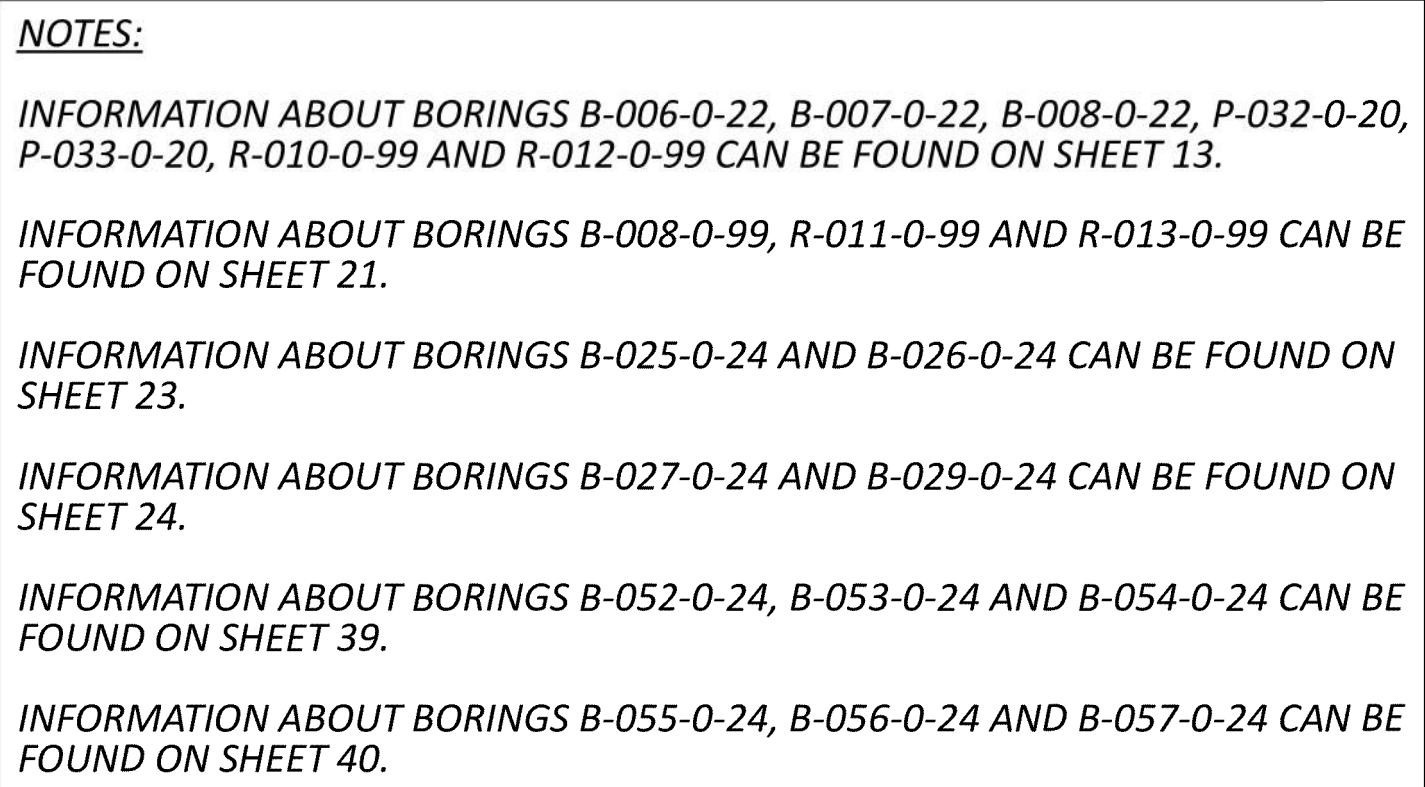
GEOTECHNICAL PROFILE - ROADWAY
SB EXIT RAMP CD (TO IR-76) - STA. 125+00 TO STA. 137+50

DESIGN AGENCY	
NEAS Engineering & Construction Services, Inc.	
2800 CORPORATE EXCHANGE DR, SUITE 240 COLUMBUS, OH, 43231 TEL: 614.714.0299 WWW.NEASINC.COM	
DESIGNER	
RS	
REVIEWER	
CH 08/14/26	
PROJECT ID	
114865	
SUBSET	TOTAL
18	116
SHEET	TOTAL
P.527	625

NOTES:
INFORMATION ABOUT BORING B-039-0-24 CAN BE FOUND ON SHEET 32.
INFORMATION ABOUT BORINGS B-040-0-24 AND B-041-0-24 CAN BE FOUND ON SHEET 34.
INFORMATION ABOUT BORING R-006-0-99 CAN BE FOUND ON SHEET 12.

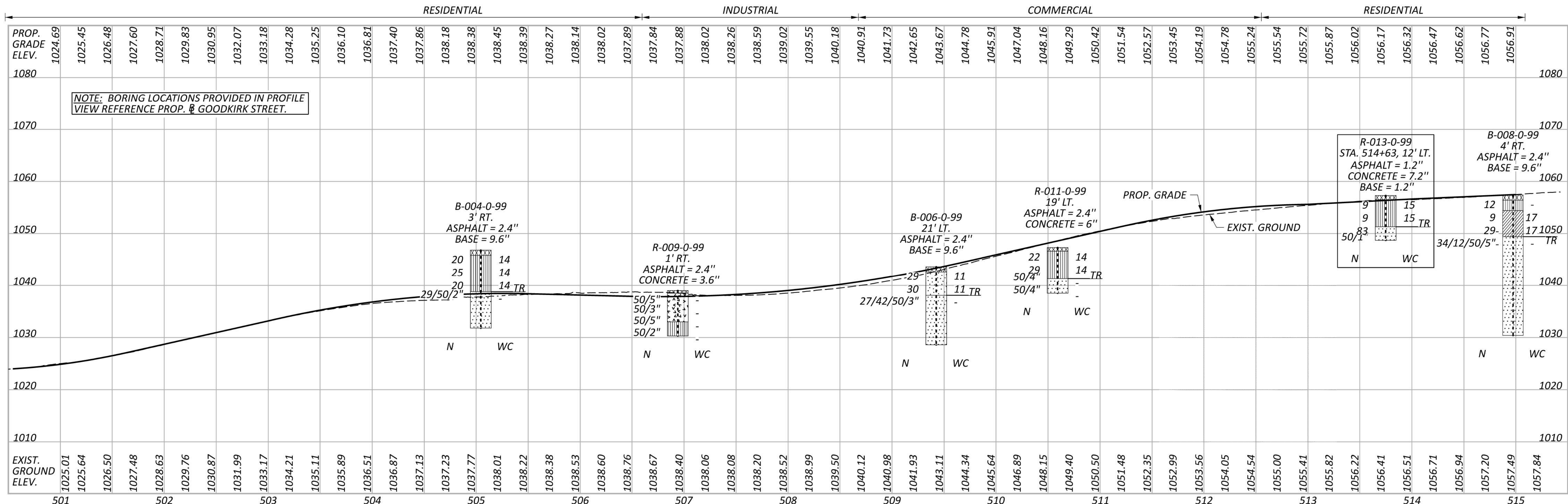
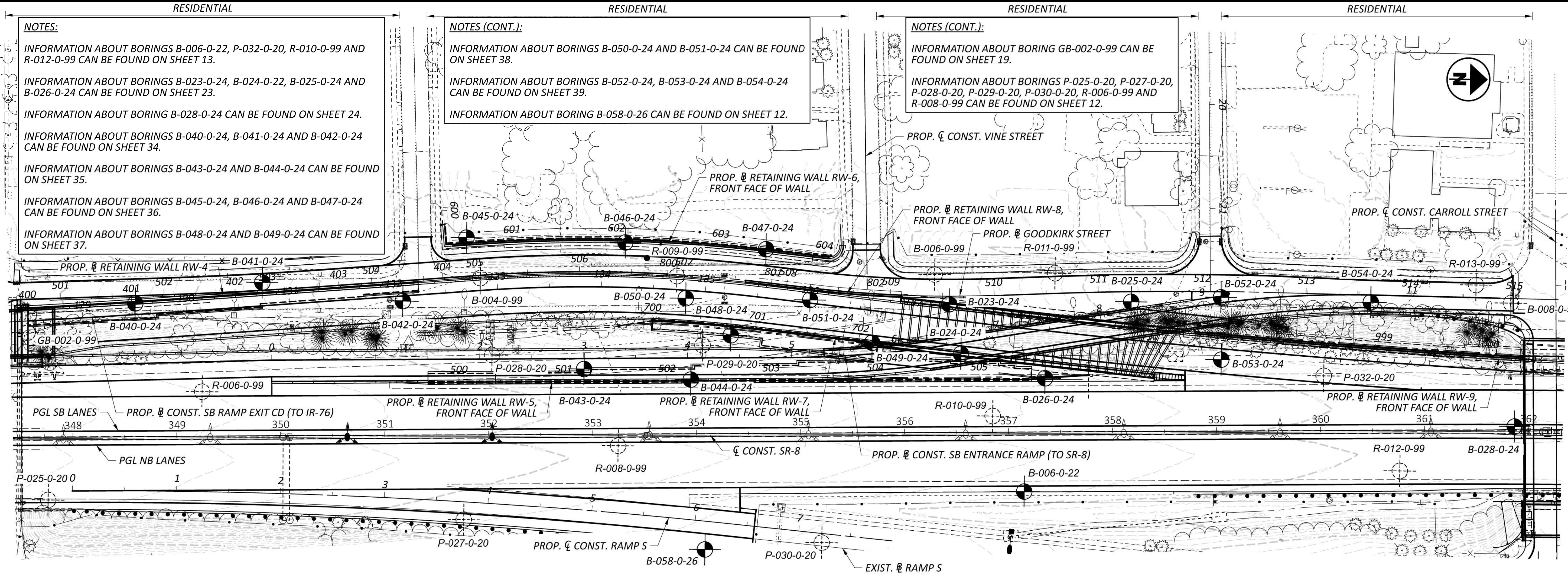


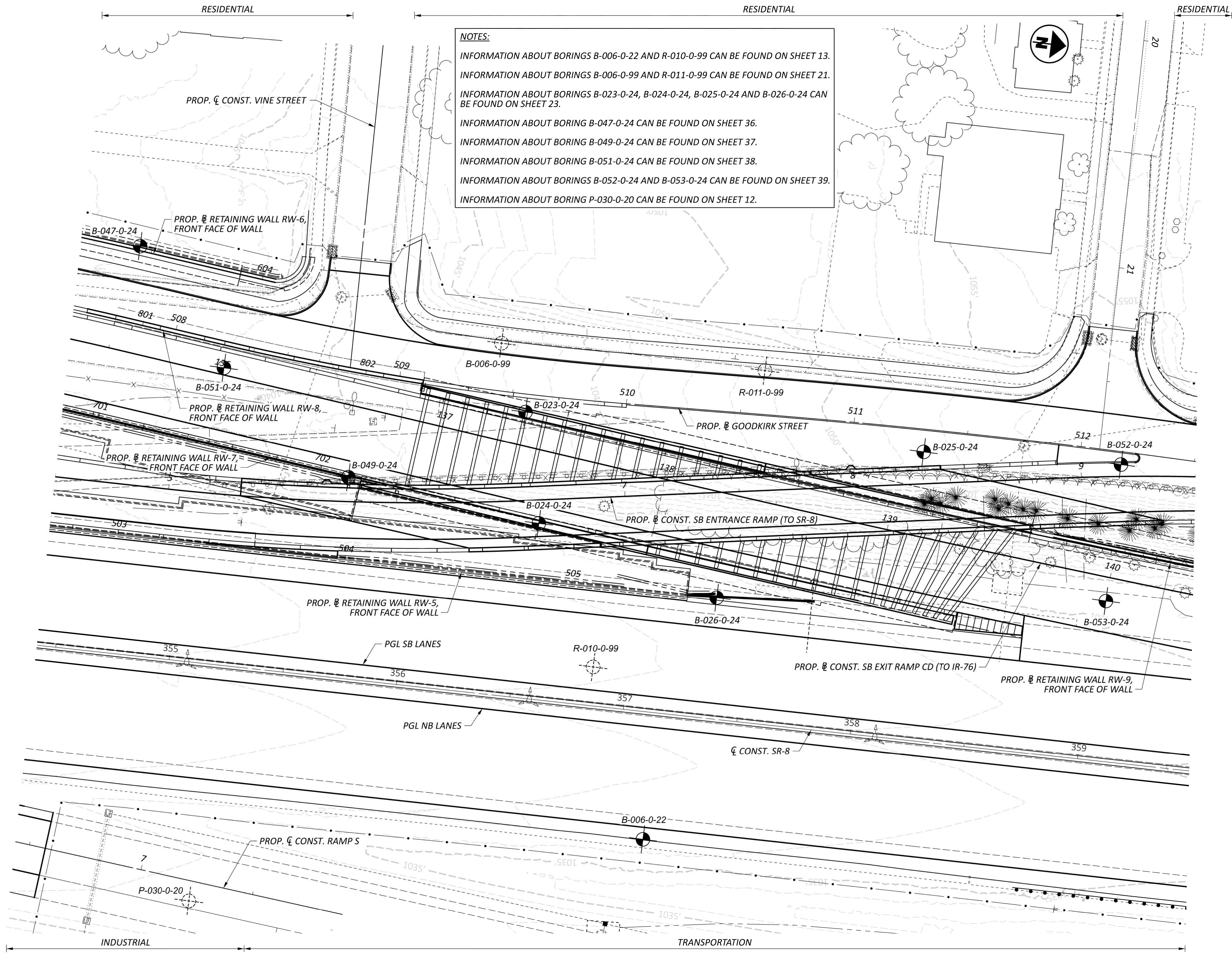
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pw:\arcadis-us-pw.bentley.com\arcadis-us-01\Documents\01 Active Projects\30149976\400 CAD\403-Engineering NEAS\Geotechnical\Sheets\114865 IP011.dgn



GEOTECHNICAL PROFILE - ROADWAY
 USB EXIT RAMP CD (TO IR-76) - STA. 137+50 TO END

DESIGN AGENCY
NEAS
 National Engineering & Architectural Services Inc.
 2800 CORPORATE
 EXCHANGE DR.,
 SUITE 240
 COLUMBUS, OH,
 43231
 TEL. 614.714.0299
 WWW.NEASINC.COM





NOTES:

INFORMATION ABOUT BORINGS B-006-0-22 AND R-010-0-99 CAN BE FOUND ON SHEET 13.

INFORMATION ABOUT BORINGS B-006-0-99 AND R-011-0-99 CAN BE FOUND ON SHEET 21.

INFORMATION ABOUT BORINGS B-023-0-24, B-024-0-24, B-025-0-24 AND B-026-0-24 CAN BE FOUND ON SHEET 23.

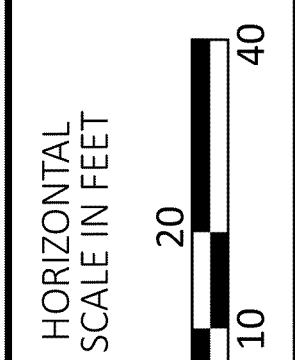
INFORMATION ABOUT BORING B-047-0-24 CAN BE FOUND ON SHEET 36.

INFORMATION ABOUT BORING B-049-0-24 CAN BE FOUND ON SHEET 37.

INFORMATION ABOUT BORING B-051-0-24 CAN BE FOUND ON SHEET 38.

INFORMATION ABOUT BORINGS B-052-0-24 AND B-053-0-24 CAN BE FOUND ON SHEET 39.

INFORMATION ABOUT BORING P-030-0-20 CAN BE FOUND ON SHEET 12.



GEOTECHNICAL PROFILE - ROADWAY

BRIDGE NO. SUM-8-1.040

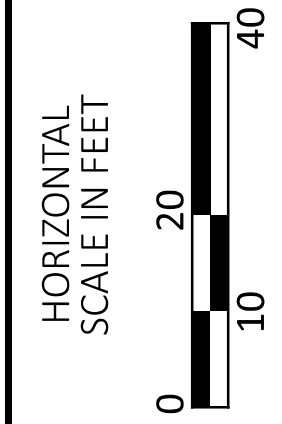
SB ENTRANCE RAMP (TO SR-8) OVER SB EXIT RAMP CD (TO IR-76)

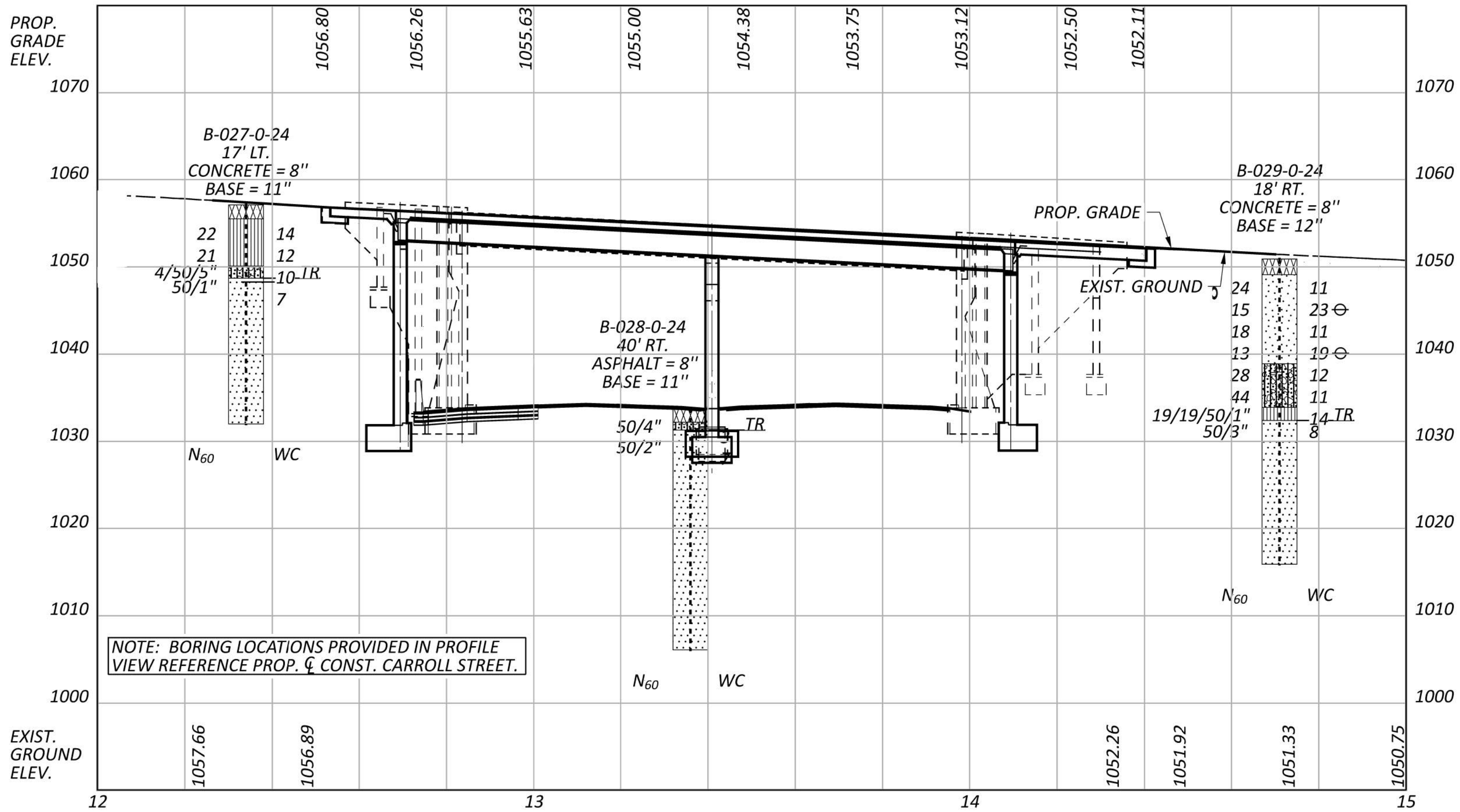
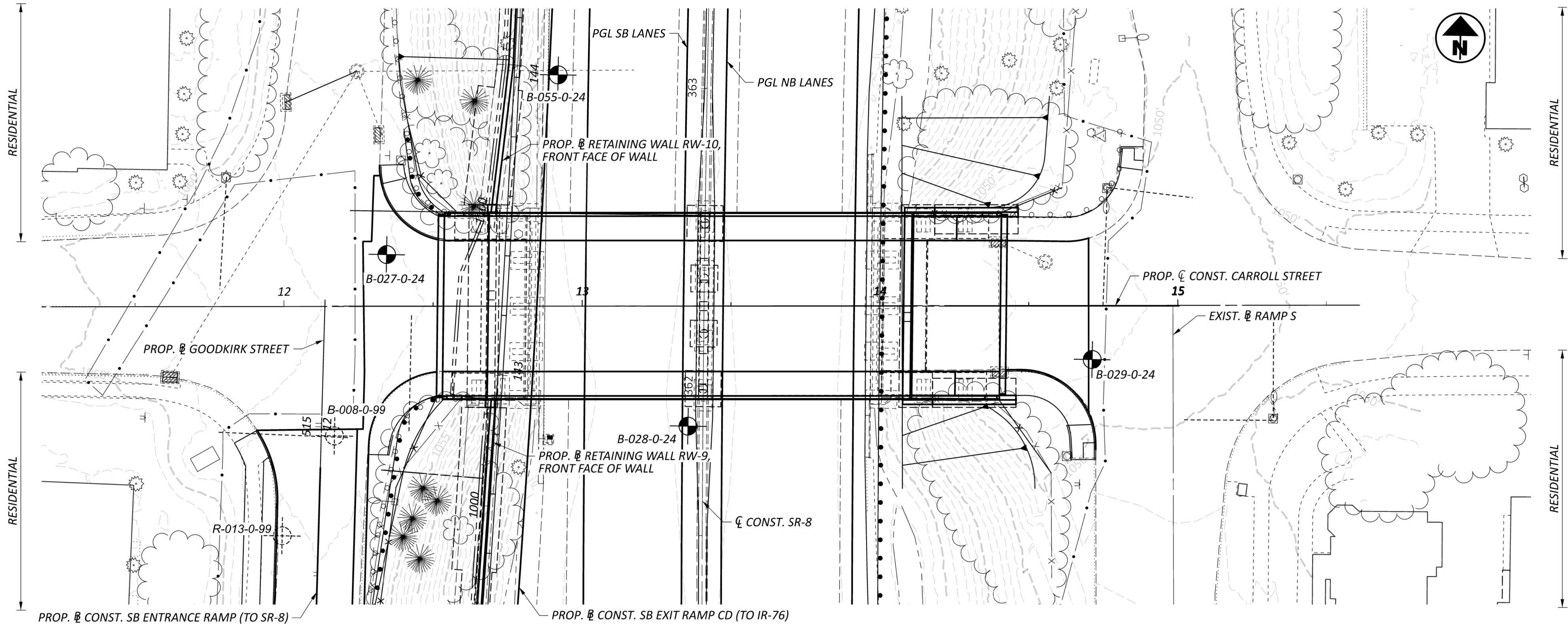
SFN	7700372
DESIGN AGENCY	NEAS
2800 CORPORATE EXCHANGE DR, SUITE 240 COLUMBUS, OH, 43231 TEL: 614.714.0299 WWW.NEASINC.COM	
DESIGNER	CHECKER
RS	DT
REVIEWER	
CH	08/14/26
PROJECT ID	114865
SUBSET	TOTAL
22	116
SHEET	TOTAL
P.531	625

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pw:\\arcadis-us-pw.bentley.com:arcadis-us-01\\Documents\\01 Active Projects\\30149976\\400_CAD\\403-Engineering_NEAS\\Geotechnical\\Sheets\\114865_ZF005.dgn



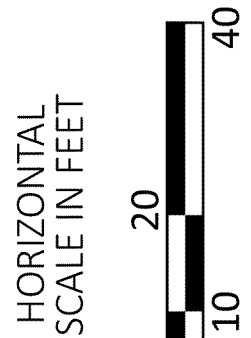
SFN	
7700372	
DESIGN AGENCY	
	
NEAS Engineering & Architectural Services Inc.	
2800 CORPORATE	
ECHANGE DR,	
SUITE 240	
COLUMBUS, OH,	
43231	
TEL:614.714.0299	
WWW.NEASINC.COM	
DESIGNER	CHECKER
RS	DT
REVIEWER	
CH 08/14/26	
PROJECT ID	
114865	
SUBSET	TOTAL
23	116
SHEET	TOTAL
P.532	625



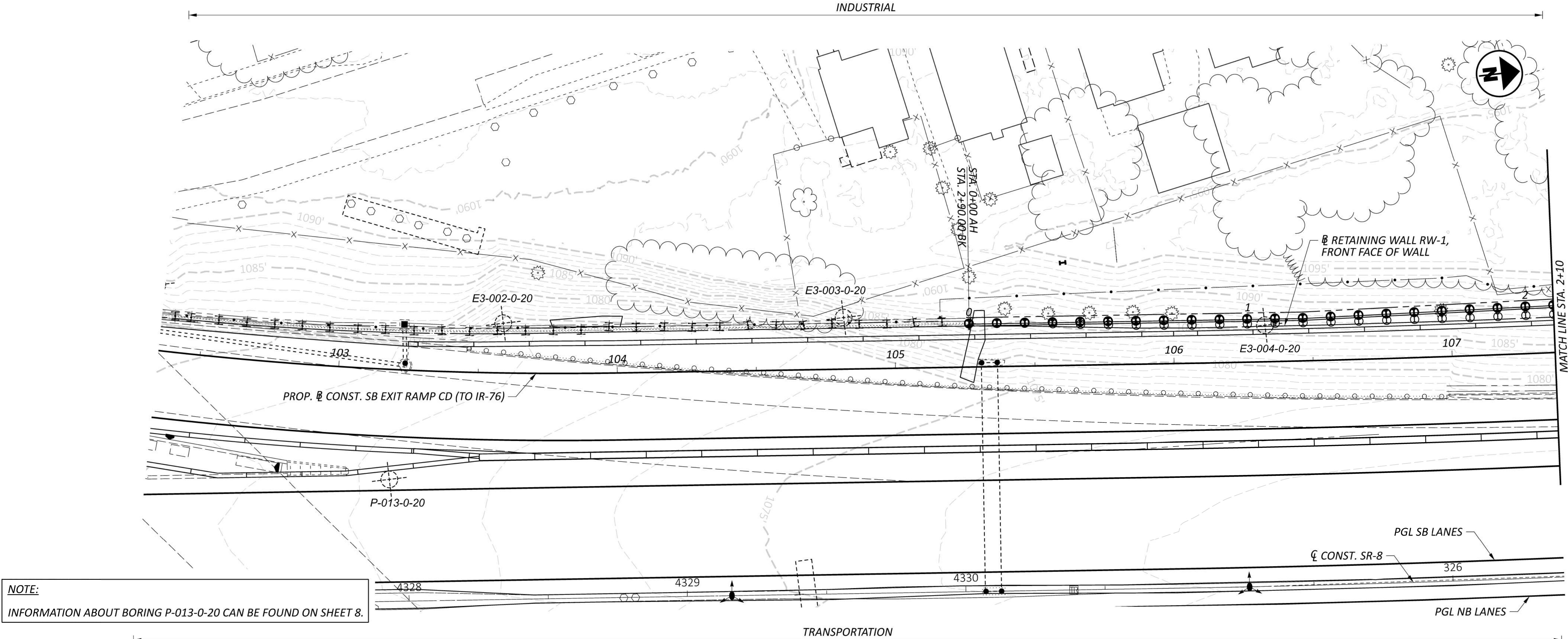


NOTES:
INFORMATION ABOUT BORINGS B-008-0-99 AND R-013-0-99 CAN BE FOUND ON SHEET 21.
INFORMATION ABOUT BORING B-055-0-24 CAN BE FOUND ON SHEET 40.

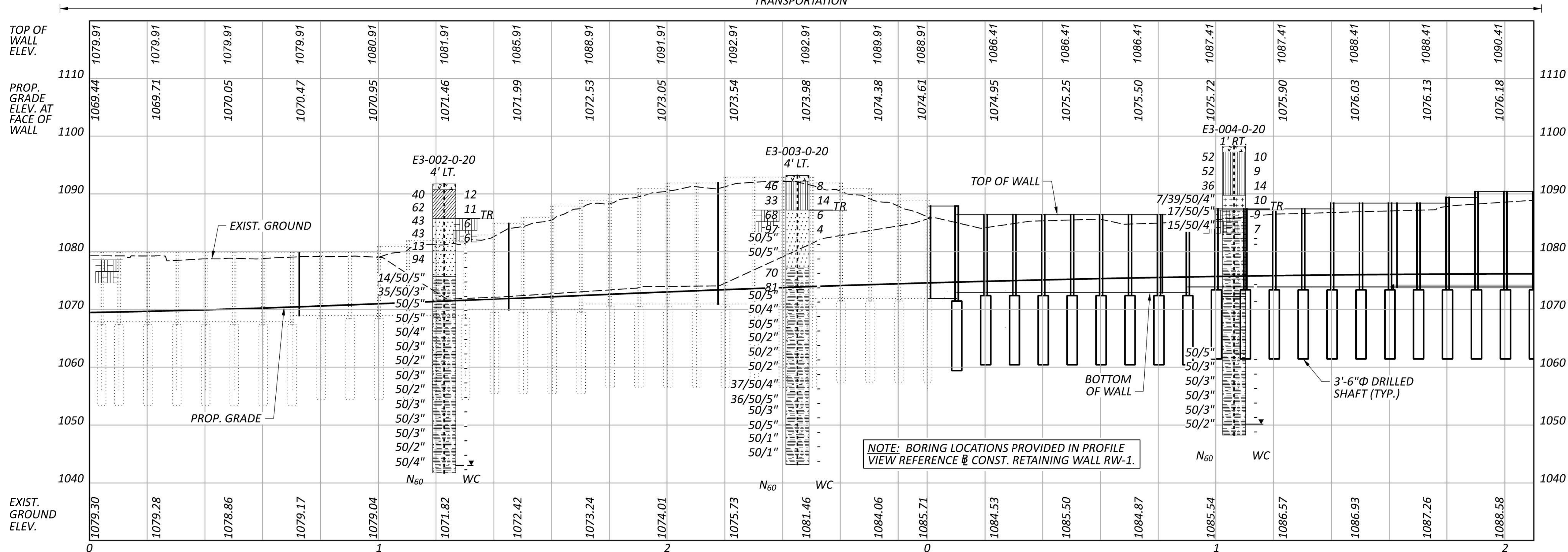
GEOTECHNICAL PROFILE - ROADWAY
BRIDGE NO. SUM-8-1.162
CARROLL STREET OVER SR-8



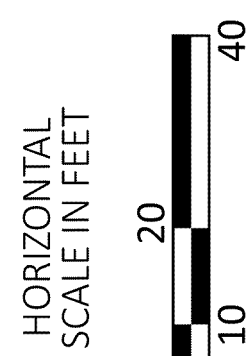
SFN	
7700373	
DESIGN AGENCY	
NEAS NEAS Engineering & Construction Services, Inc.	
2800 CORPORATE EXCHANGE DR, SUITE 240 COLUMBUS, OH, 43231 TEL: 614.714.0299 WWW.NEASINC.COM	
DESIGNER	CHECKER
RS	DT
REVIEWER	
CH 08/14/26	
PROJECT ID	
114865	
SUBSET	TOTAL
24	116
SHEET	TOTAL
P.533	625



NOTE:
INFORMATION ABOUT BORING P-013-0-20 CAN BE FOUND ON SHEET 8.



GEOTECHNICAL PROFILE - ROADWAY
RETAINING WALL RW-1



DESIGN AGENCY
NEAS
2800 CORPORATE
EXCHANGE DR,
SUITE 240
COLUMBUS, OH,
43231
TEL: 614.714.0299
WWW.NEASINC.COM

DESIGNER
RS

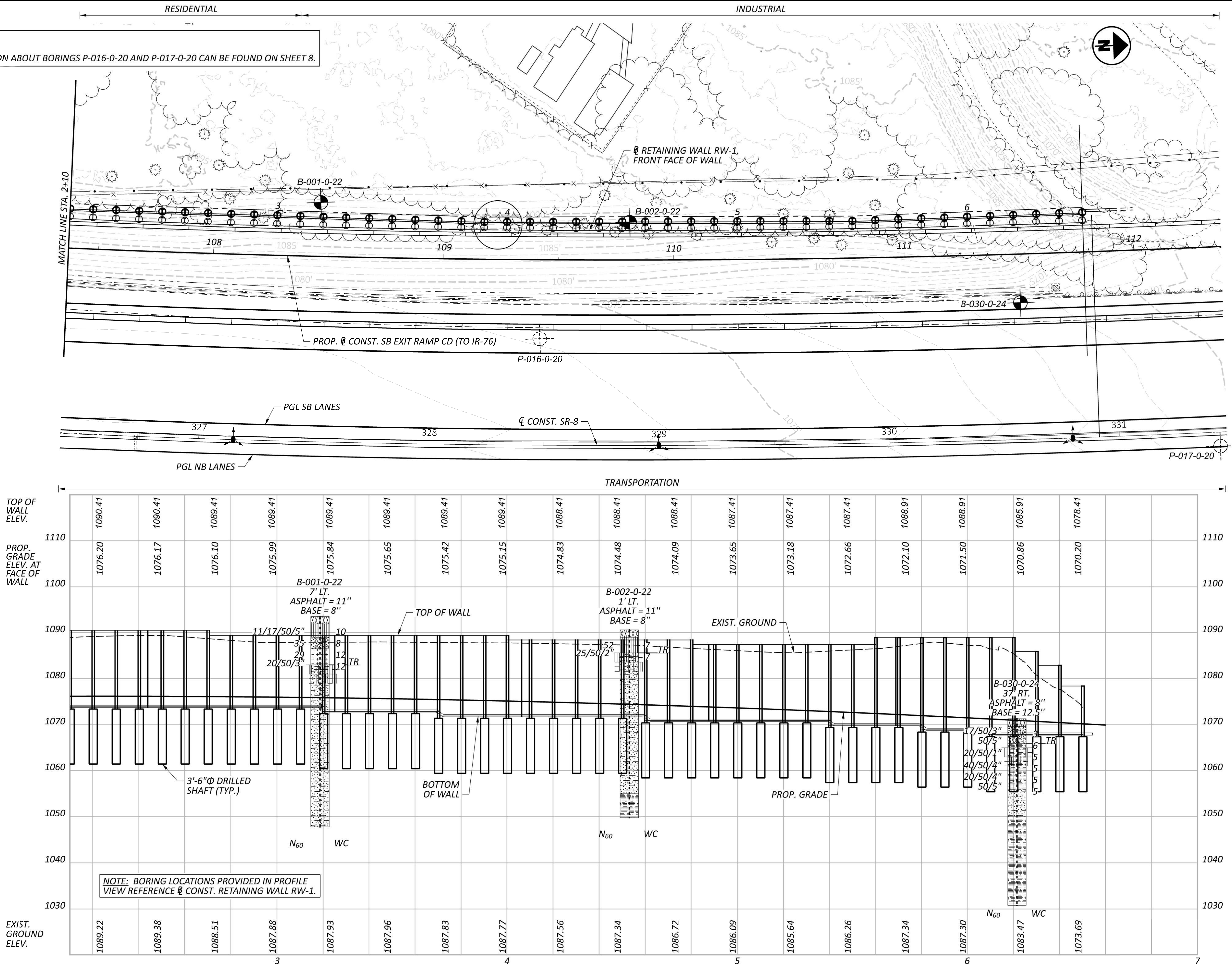
REVIEWER
CH 08/14/26

PROJECT ID
114865

SUBSET TOTAL
25 116

SHEET TOTAL
P.534 625

NOTE:
INFORMATION ABOUT BORINGS P-016-0-20 AND P-017-0-20 CAN BE FOUND ON SHEET 8.



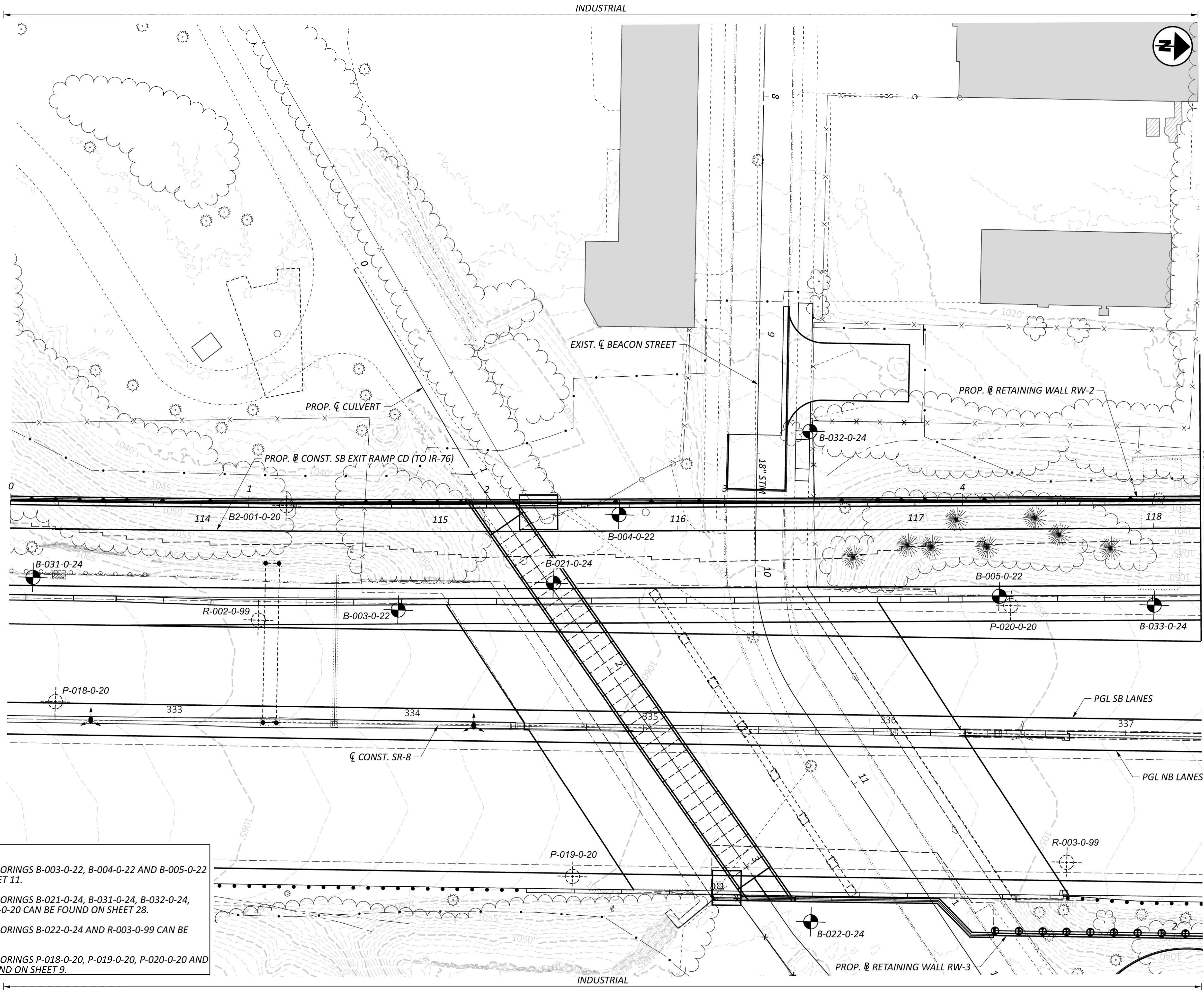
NOTES:

INFORMATION ABOUT BORINGS B-003-0-22, B-004-0-22 AND B-005-0-22 CAN BE FOUND ON SHEET 11.

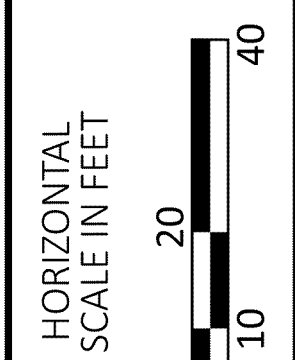
INFORMATION ABOUT BORINGS B-021-0-24, B-031-0-24, B-032-0-24, B-033-0-24 AND B2-001-0-20 CAN BE FOUND ON SHEET 28.

INFORMATION ABOUT BORINGS B-022-0-24 AND R-003-0-99 CAN BE FOUND ON SHEET 33.

INFORMATION ABOUT BORINGS P-018-0-20, P-019-0-20, P-020-0-20 AND R-002-0-99 CAN BE FOUND ON SHEET 9.



GEOTECHNICAL PROFILE - ROADWAY
RETAINING WALL RW-2



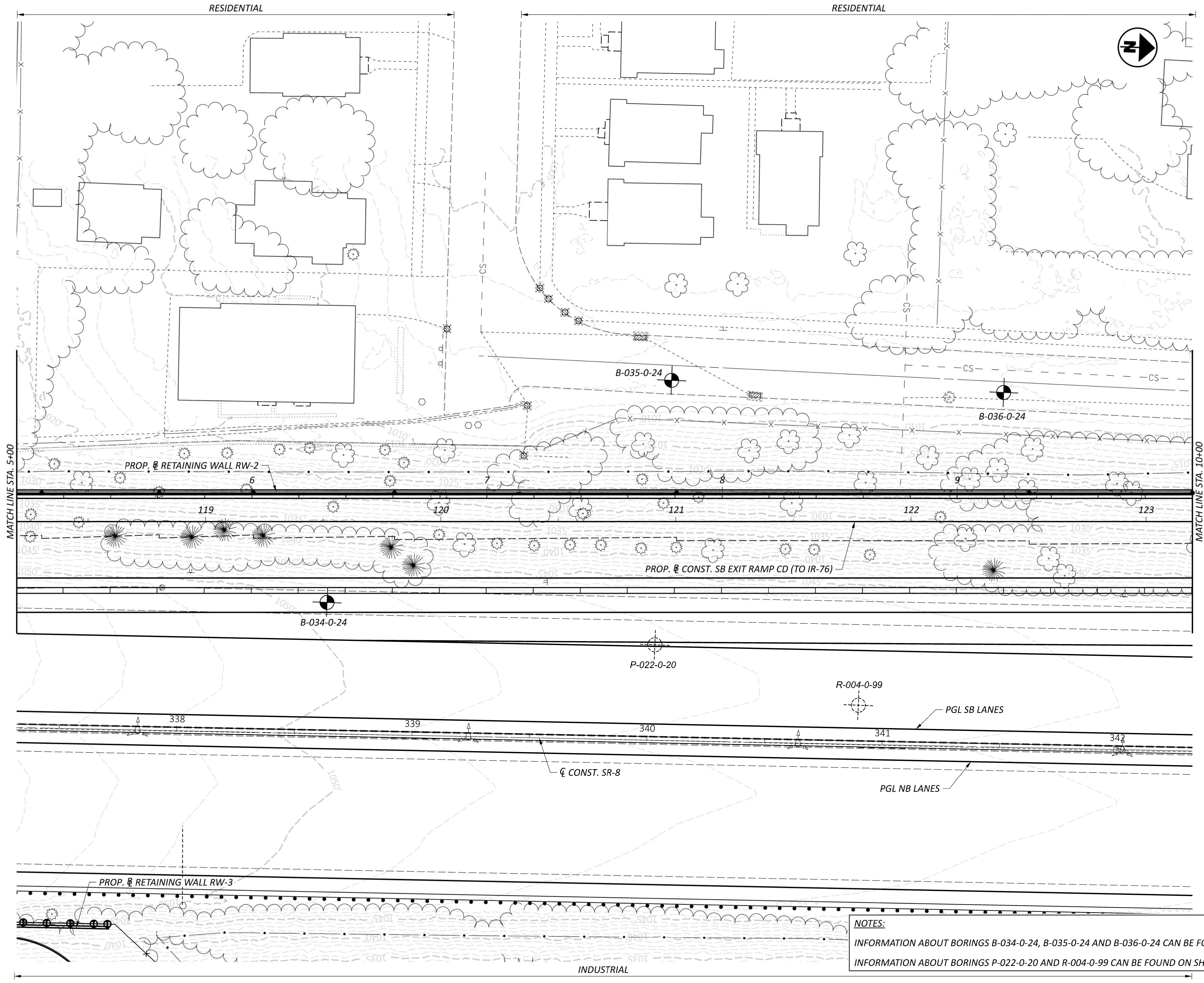
DESIGN AGENCY	
NEAS Neas Engineering & Technology, Inc.	
2800 CORPORATE EXCHANGE DR. SUITE 240 COLUMBUS, OH, 43231 TEL: 614.714.0299 WWW.NEASINC.COM	
DESIGNER	
RS	
REVIEWER	
CH 08/14/26	
PROJECT ID	
114865	
SUBSET	TOTAL
27	116
SHEET	TOTAL
P.536	625

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DESIGN AGENCY
NEAS
National Engineering & Architectural Services Inc.
2800 CORPORATE
EXCHANGE DR,
SUITE 240
COLUMBUS, OH,
43231
TEL:614.714.0299
WWW.NEASINC.COM

DESIGNER	
RS	
REVIEWER	
CH 08/14/26	
PROJECT ID	
114865	
SUBSET	TOTAL
28	116
SHEET	TOTAL
P.537	625



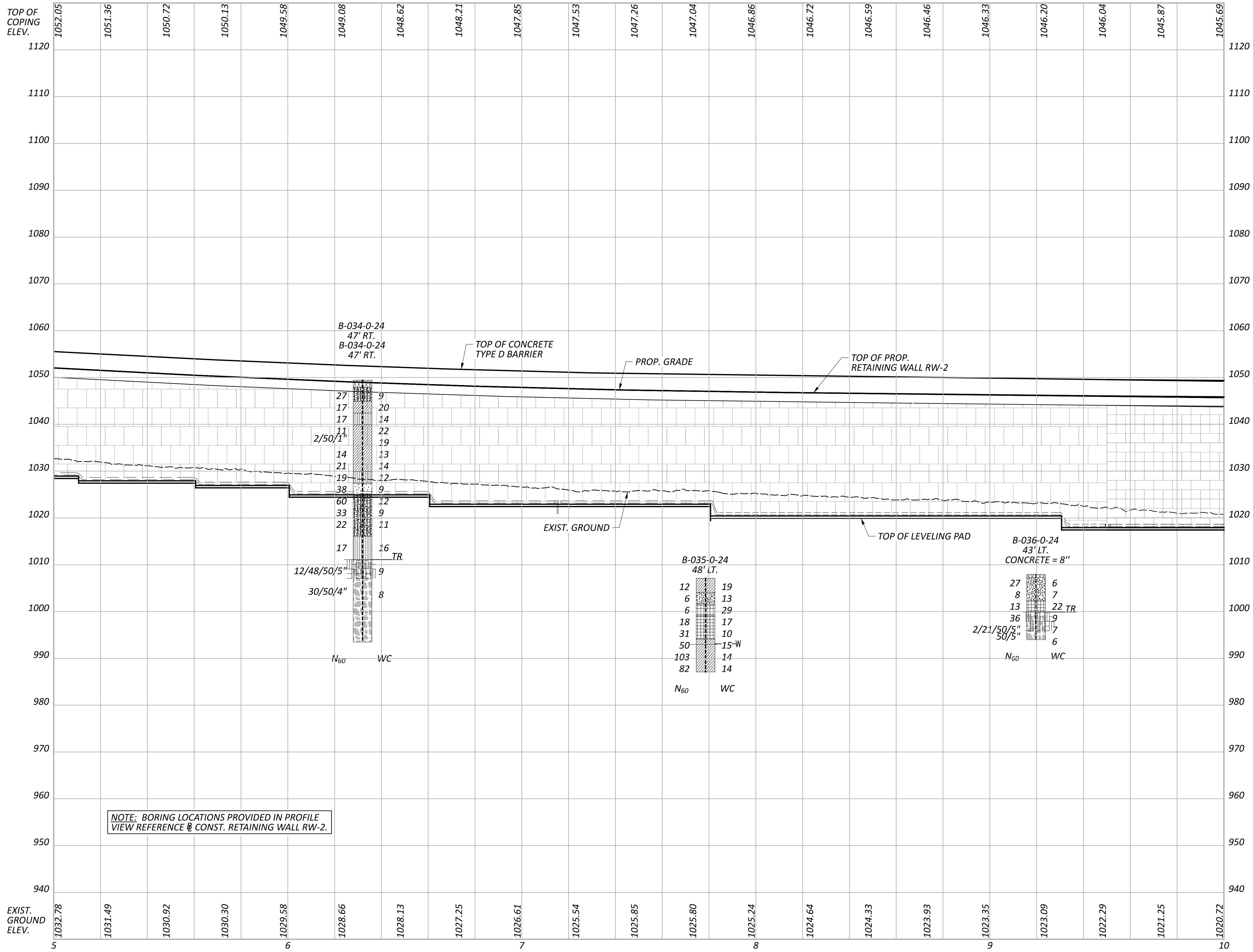
NOTES:
INFORMATION ABOUT BORINGS B-034-0-24, B-035-0-24 AND B-036-0-24 CAN BE FOUND ON SHEET 30.
INFORMATION ABOUT BORINGS P-022-0-20 AND R-004-0-99 CAN BE FOUND ON SHEET 9.

GEOTECHNICAL PROFILE - ROADWAY
RETAINING WALL RW-2 (CONT.)

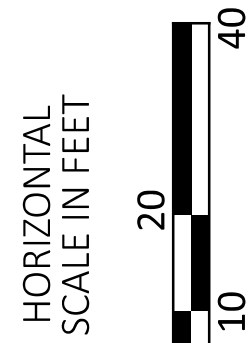


DESIGN AGENCY
NEAS
2800 CORPORATE
EXCHANGE DR,
SUITE 240
COLUMBUS, OH,
43231
TEL: 614.714.0299
WWW.NEASINC.COM

DESIGNER	RS
REVIEWER	CH
PROJECT ID	114865
SUBSET	TOTAL
29	116
SHEET	TOTAL
P.538	625



GEOTECHNICAL PROFILE - ROADWAY
RETAINING WALL RW-2 (CONT.)



DESIGN AGENCY
NEAS
2800 CORPORATE
EXCHANGE DR,
SUITE 240
COLUMBUS, OH,
43231
TEL: 614.714.0299
WWW.NEASINC.COM

DESIGNER	
RS	
REVIEWER	
CH 08/14/26	
PROJECT ID	
114865	
SUBSET	TOTAL
30	116
SHEET	TOTAL
P.539	625

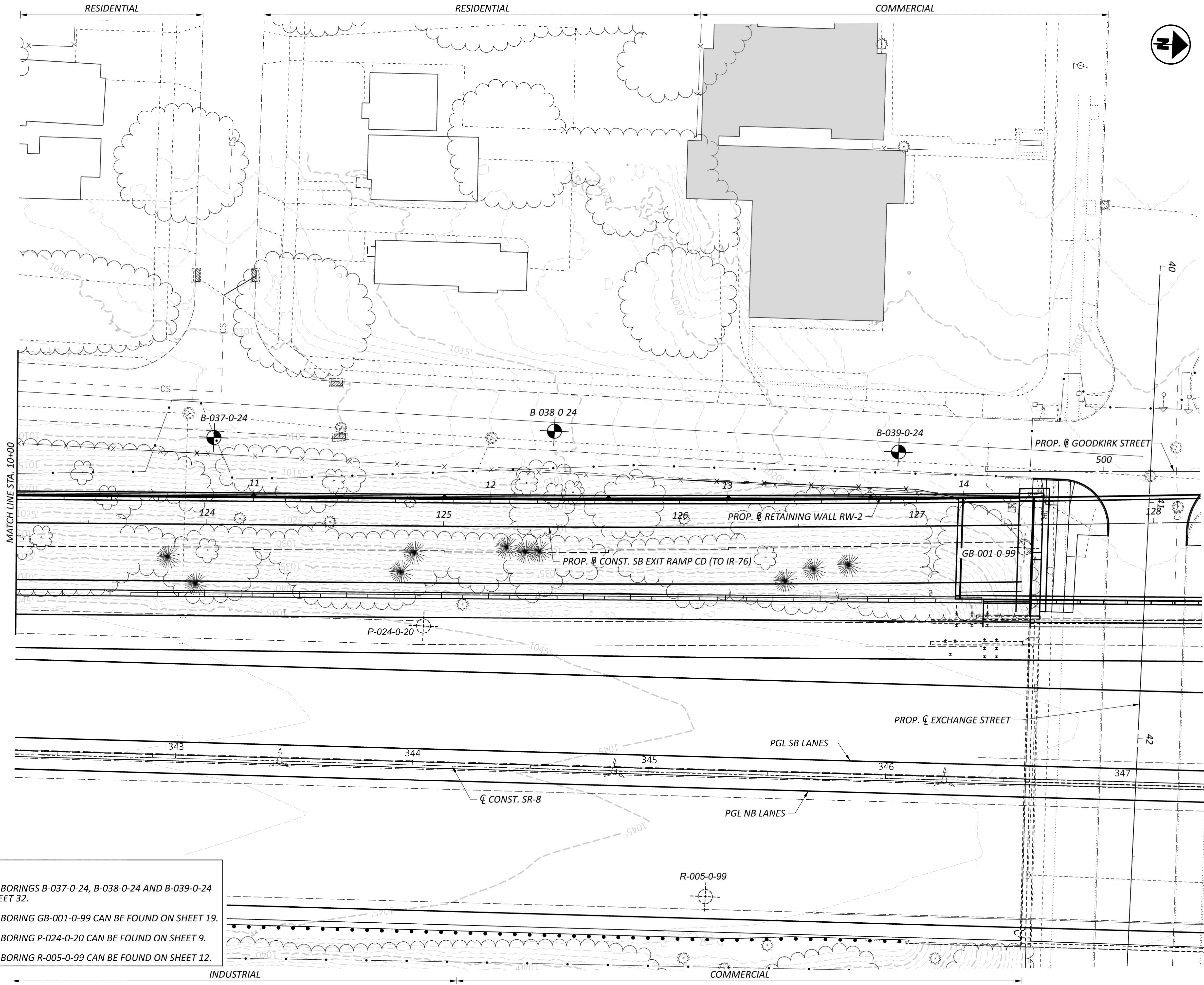
NOTES:

INFORMATION ABOUT BORINGS B-037-0-24, B-038-0-24 AND B-039-0-24
CAN BE FOUND ON SHEET 32.

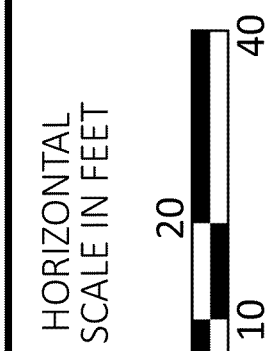
INFORMATION ABOUT BORING GB-001-0-99 CAN BE FOUND ON SHEET 19.

INFORMATION ABOUT BORING P-024-0-20 CAN BE FOUND ON SHEET 9.

INFORMATION ABOUT BORING R-005-0-99 CAN BE FOUND ON SHEET 12.

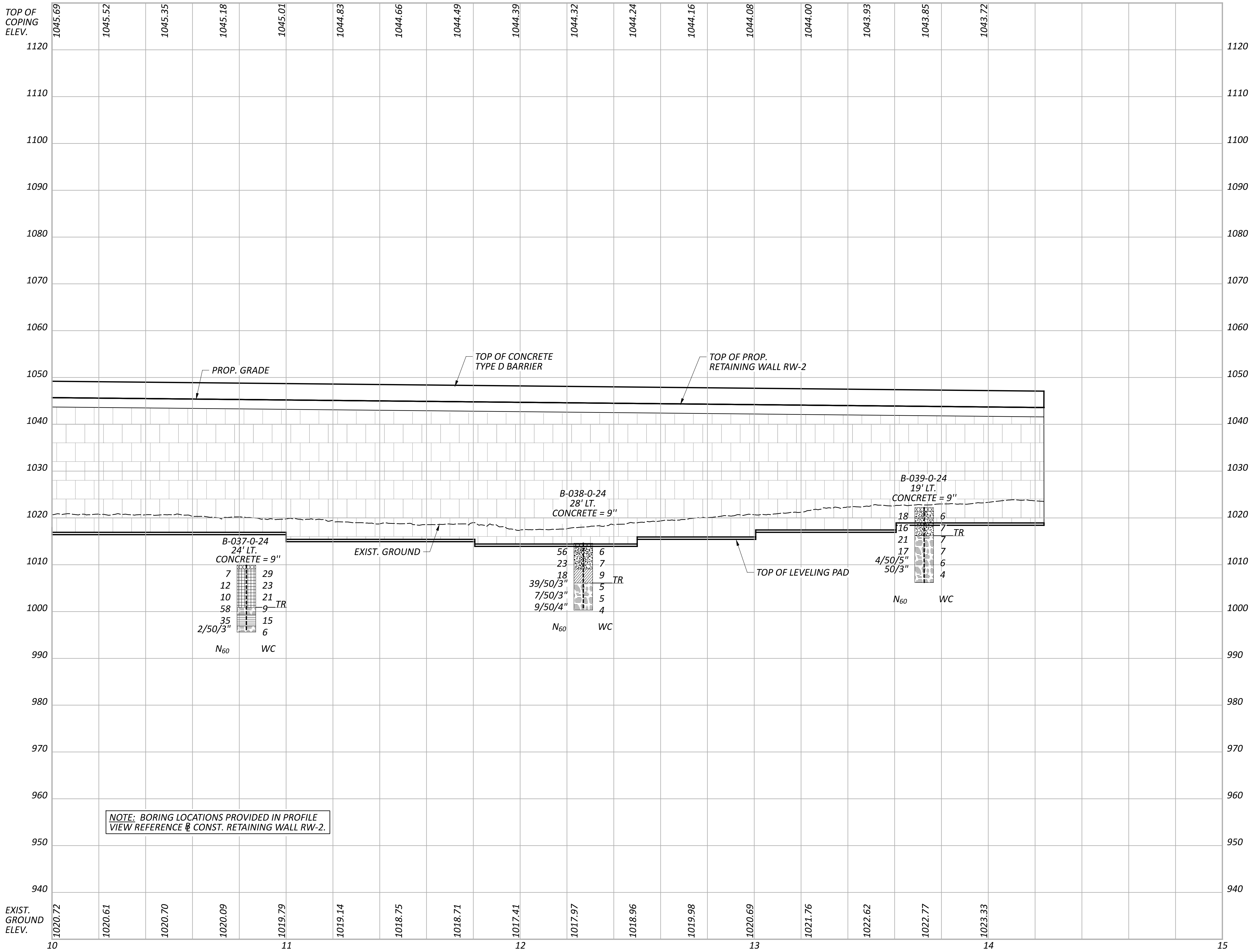


GEOTECHNICAL PROFILE - ROADWAY
RETAINING WALL RW-2 (CONT.)

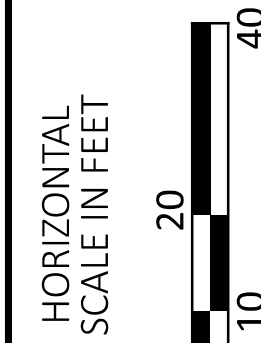


DESIGN AGENCY
NEAS
2800 CORPORATE
EXCHANGE DR.
SUITE 240
COLUMBUS, OH,
43231
TEL: 614.714.0299
WWW.NEASINC.COM

DESIGNER	RS
REVIEWER	CH 08/14/26
PROJECT ID	114865
SUBSET	TOTAL
31	116
SHEET	TOTAL
P.540	625

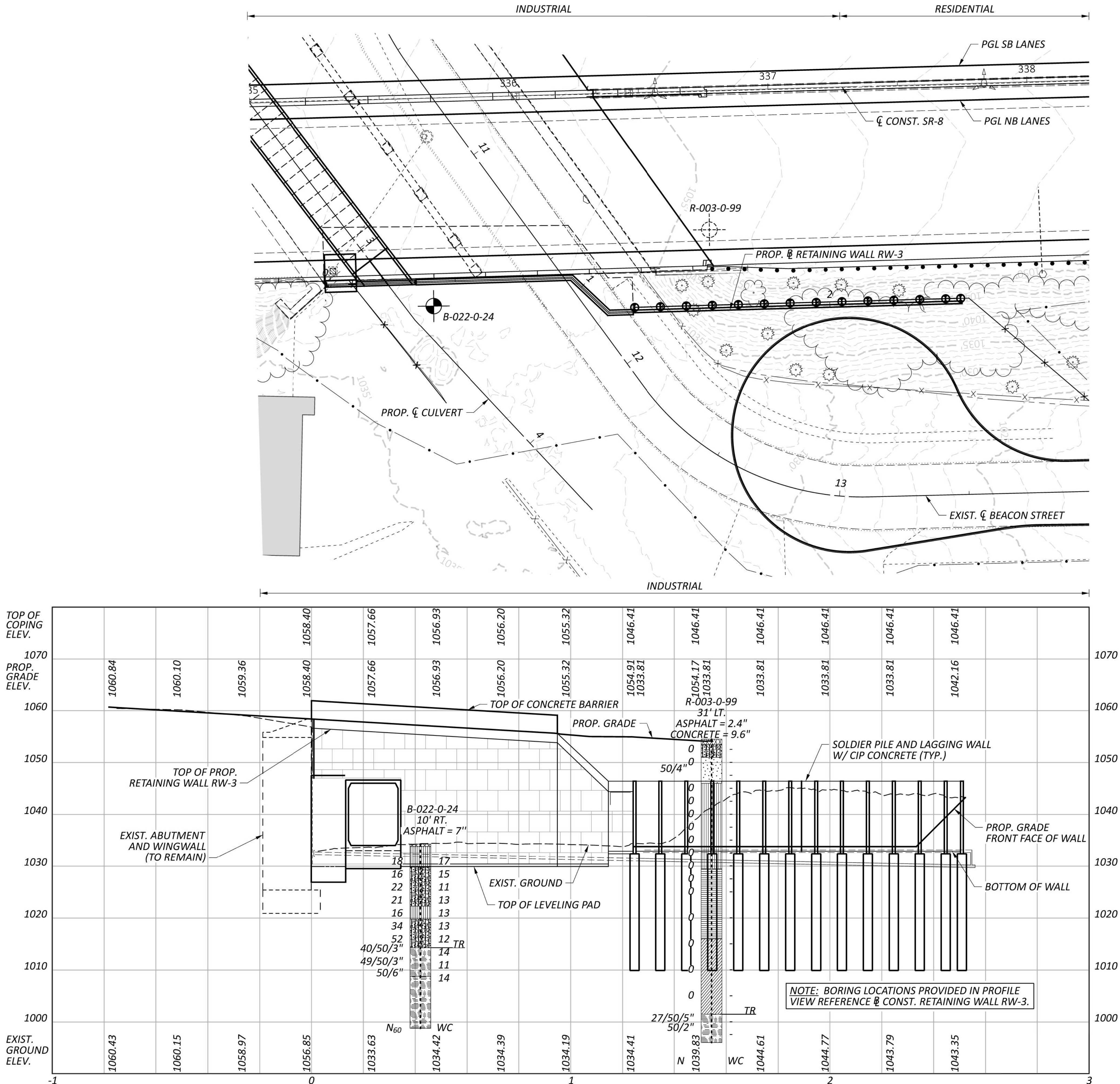


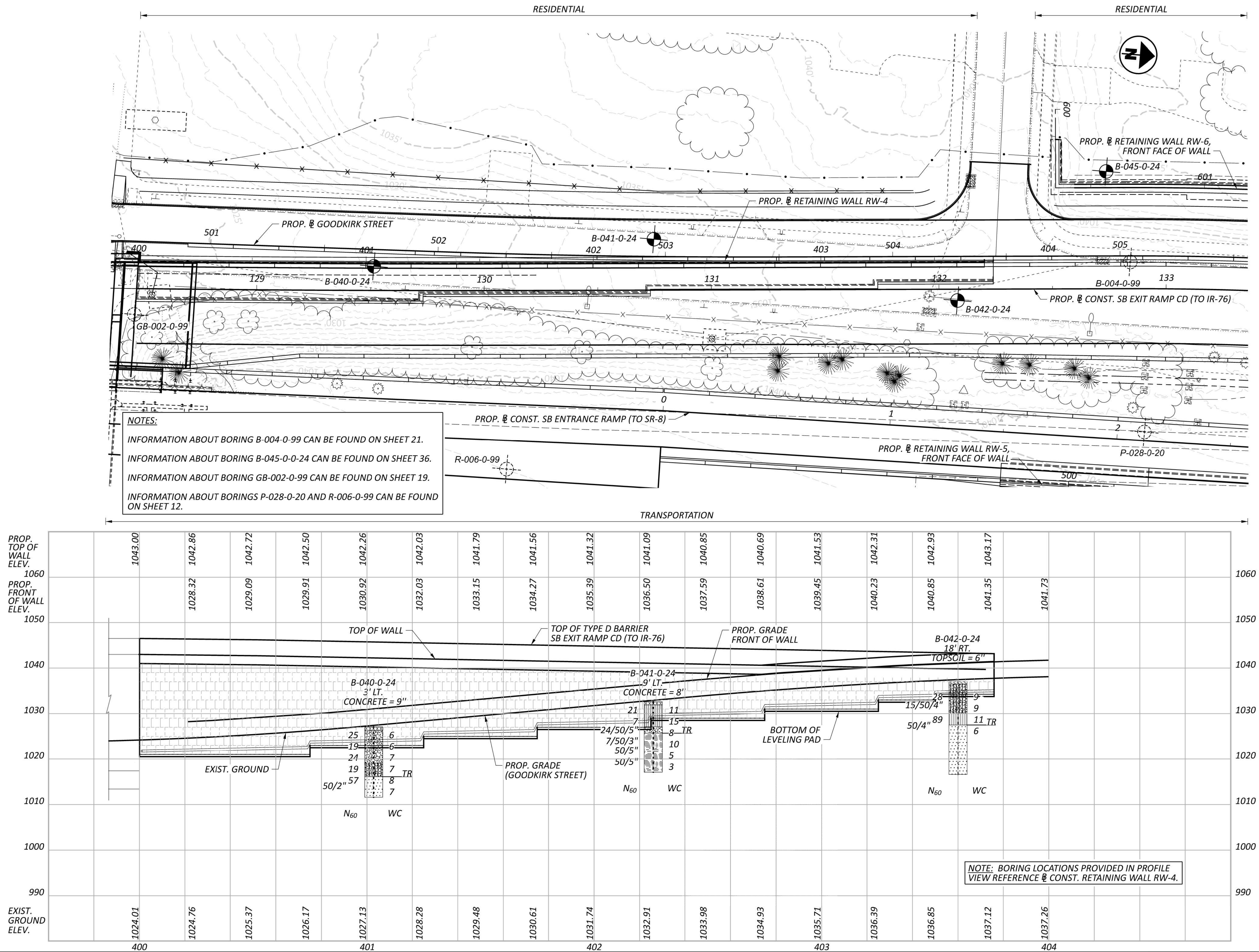
GEOTECHNICAL PROFILE - ROADWAY
RETAINING WALL RW-2 (CONT.)

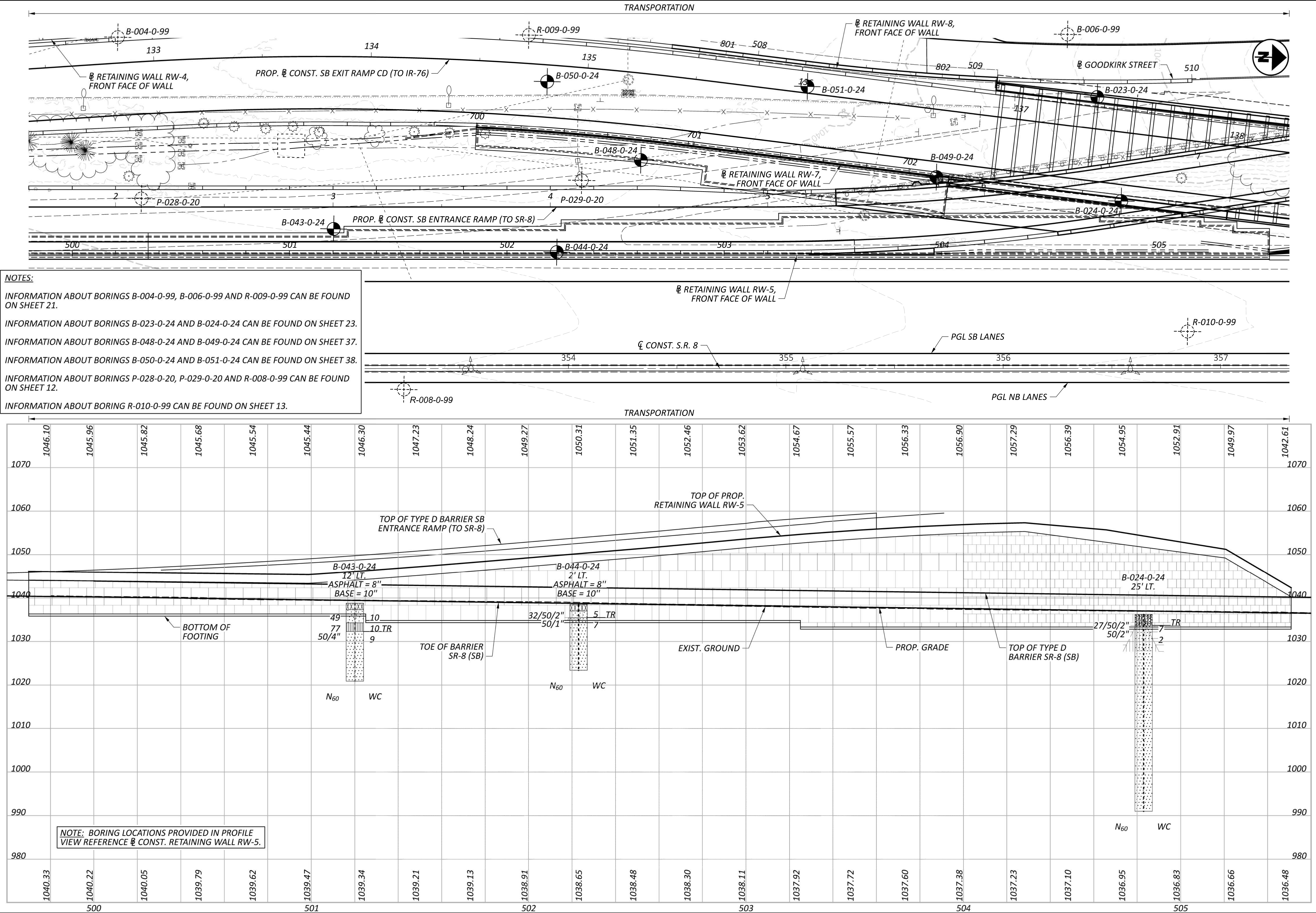


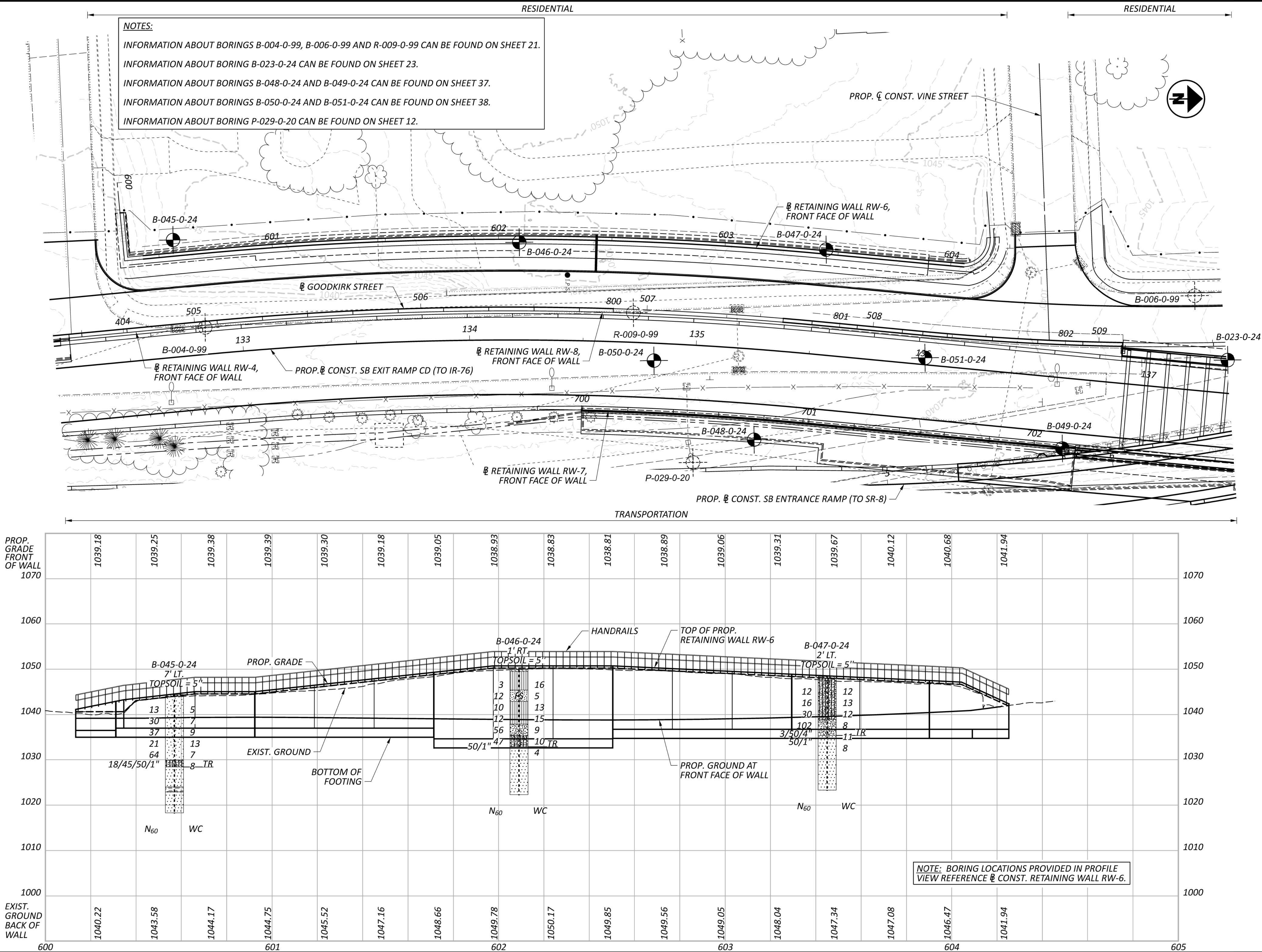
DESIGN AGENCY
NEAS
2800 CORPORATE
EXCHANGE DR,
SUITE 240
COLUMBUS, OH,
43231
TEL: 614.714.0299
WWW.NEASINC.COM

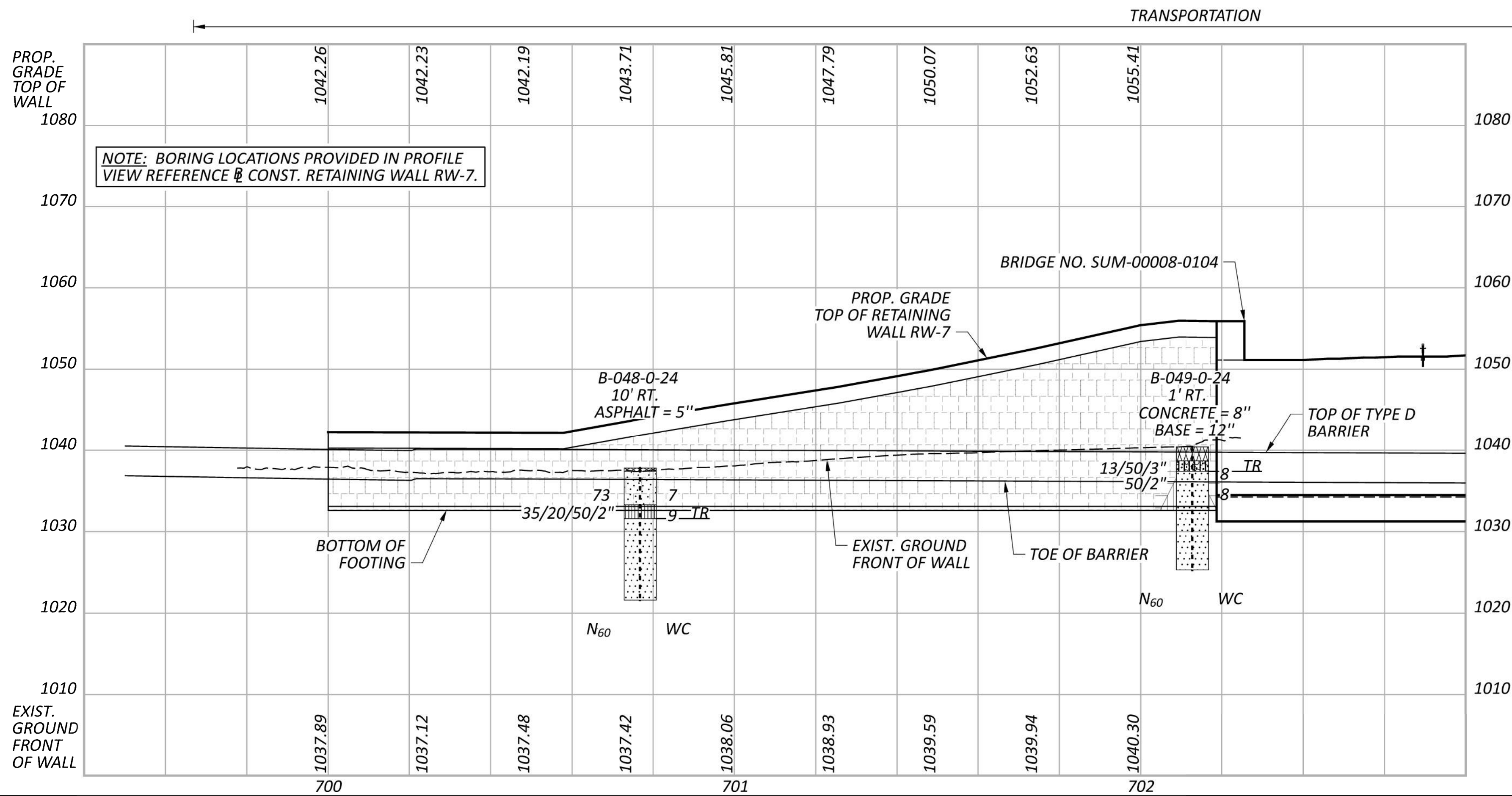
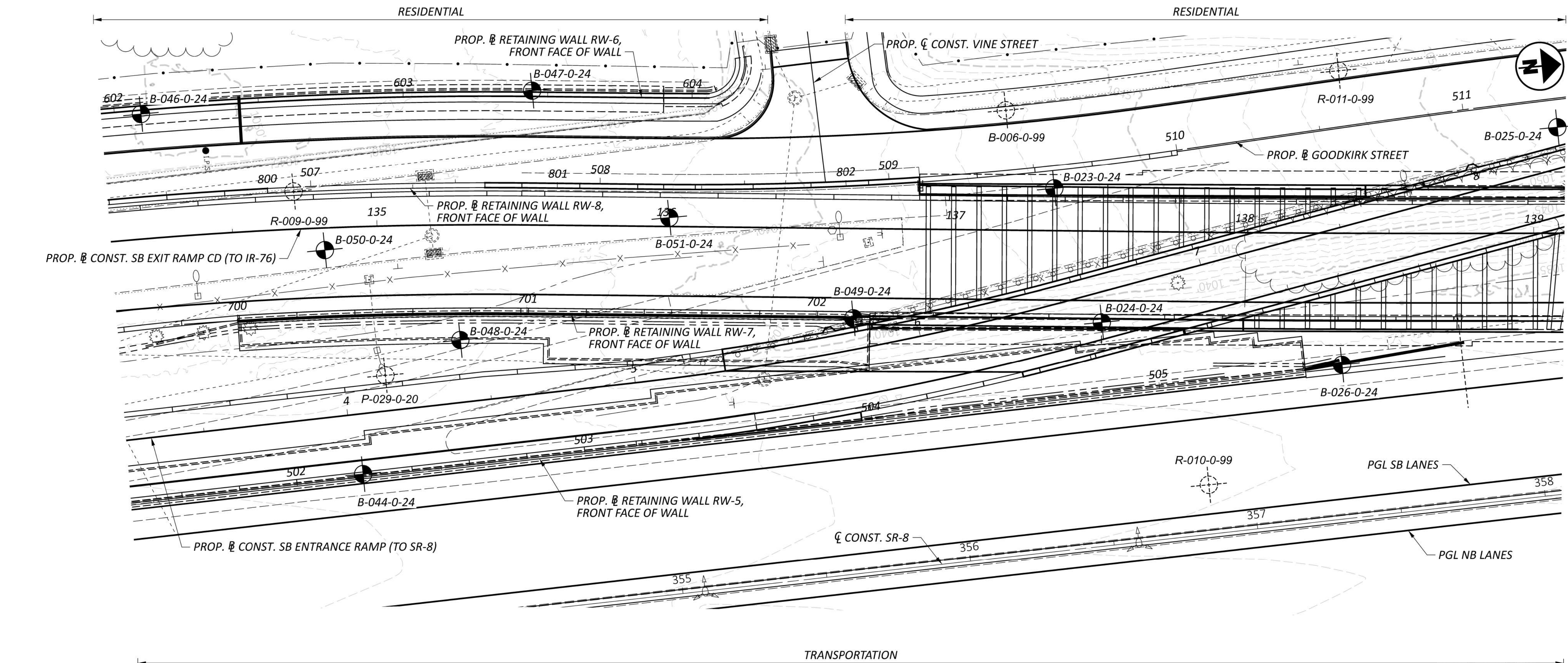
DESIGNER	RS
REVIEWER	CH
PROJECT ID	114865
SUBSET	TOTAL
32	116
SHEET	TOTAL
P.541	625





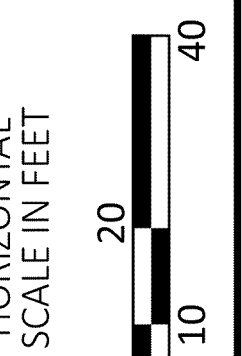






NOTES:

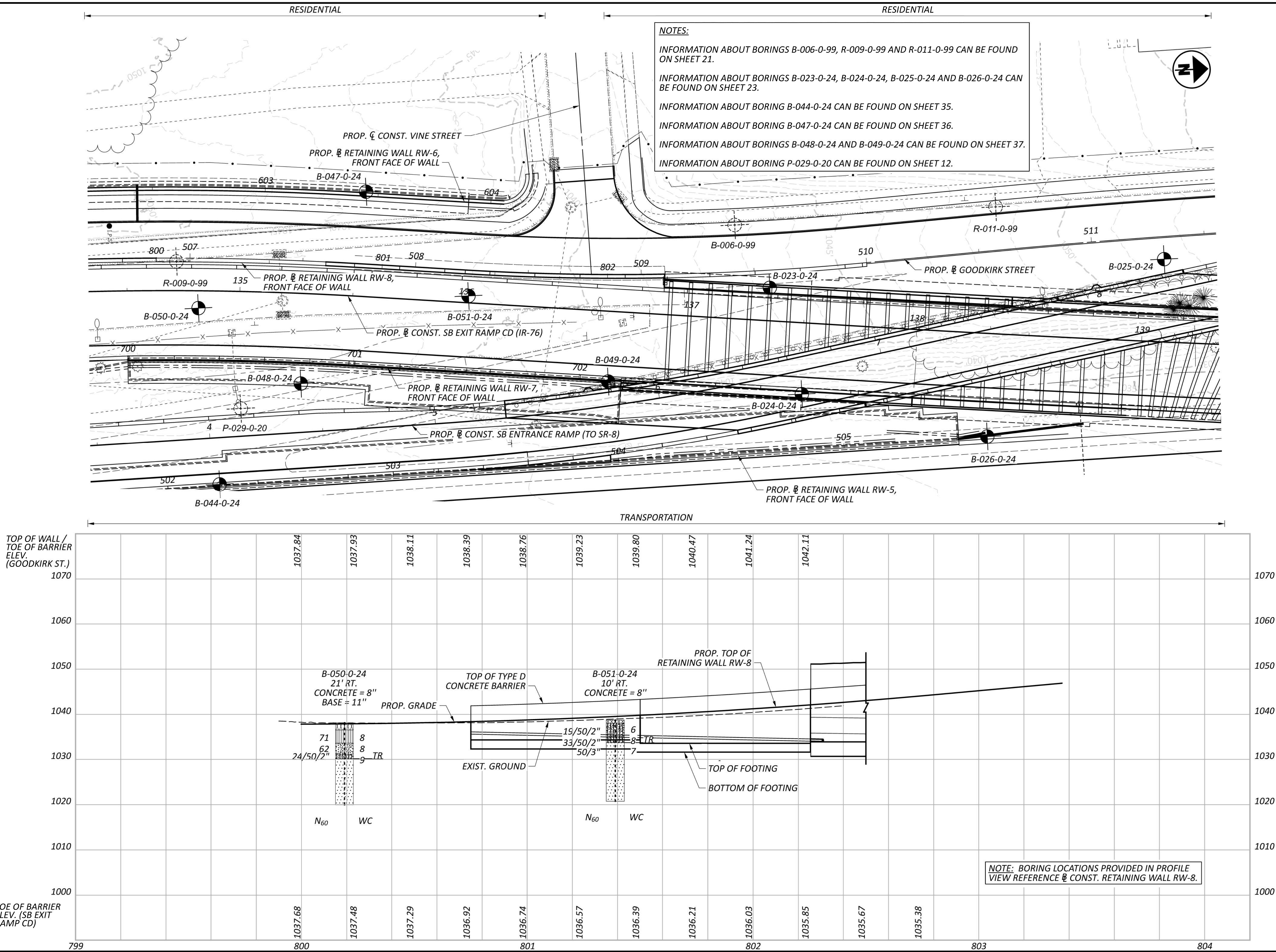
- INFORMATION ABOUT BORINGS B-006-0-99, R-009-0-99 AND R-011-0-99 CAN BE FOUND ON SHEET 21.
- INFORMATION ABOUT BORINGS B-023-0-24, B-024-0-24, B-025-0-24 AND B-026-0-24 CAN BE FOUND ON SHEET 23.
- INFORMATION ABOUT BORING B-044-0-24 CAN BE FOUND ON SHEET 35.
- INFORMATION ABOUT BORINGS B-046-0-24 AND B-047-0-24 CAN BE FOUND ON SHEET 36.
- INFORMATION ABOUT BORINGS B-050-0-24 AND B-051-0-24 CAN BE FOUND ON SHEET 38.
- INFORMATION ABOUT BORING P-029-0-20 CAN BE FOUND ON SHEET 12.
- INFORMATION ABOUT BORING R-010-0-99 CAN BE FOUND ON SHEET 13.



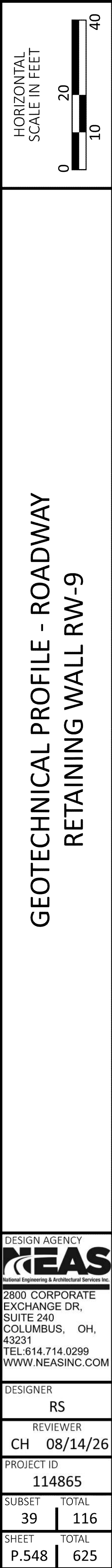
**GEOTECHNICAL PROFILE - ROADWAY
RETAINING WALL RW-7**

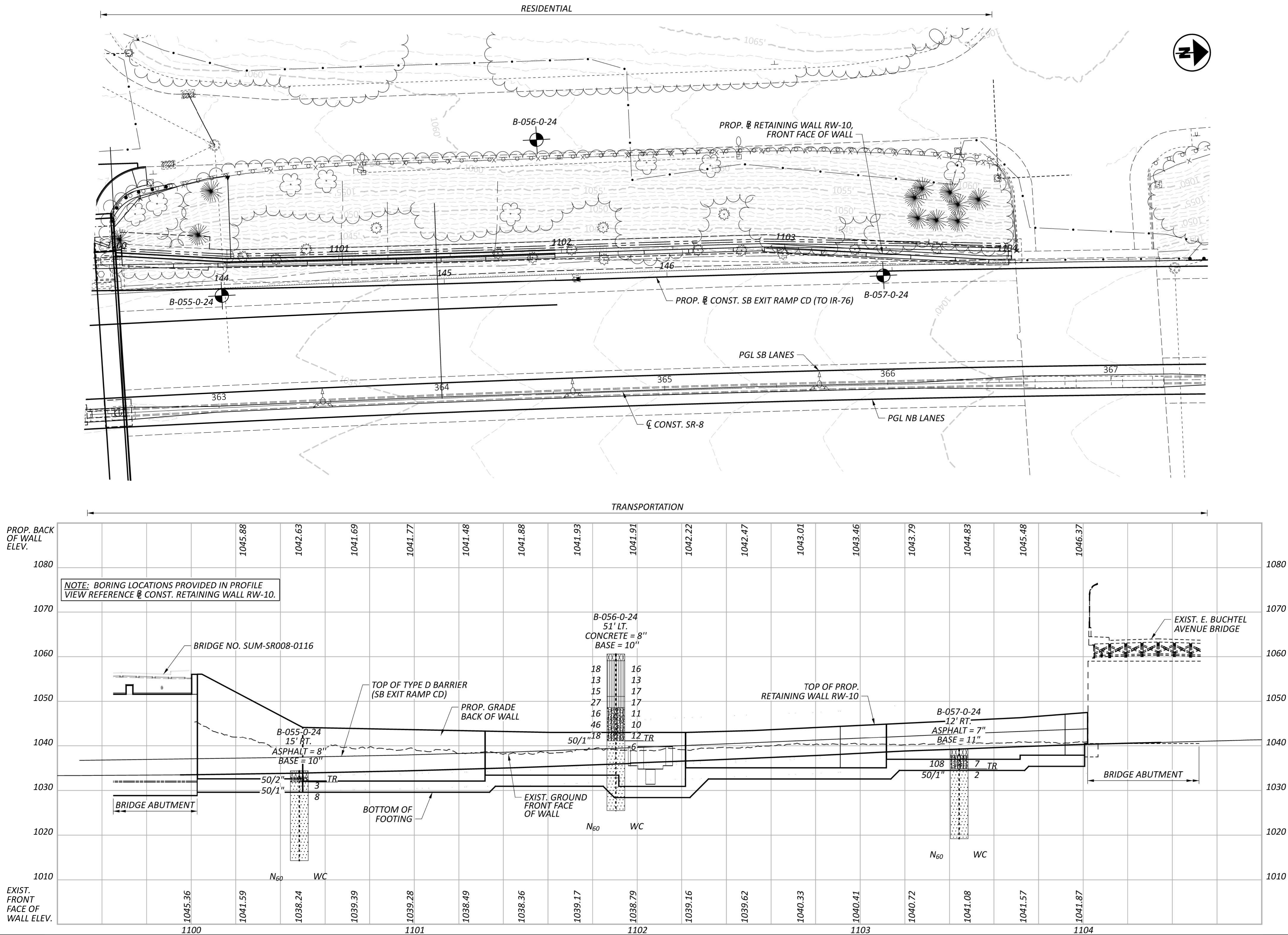
DESIGN AGENCY
NEAS
2800 CORPORATE
EXCHANGE DR,
SUITE 240
COLUMBUS, OH,
43231
TEL: 614.714.0299
WWW.NEASINC.COM

DESIGNER	RS
REVIEWER	CH
PROJECT ID	114865
SUBSET	TOTAL
37	116
SHEET	TOTAL
P.546	625

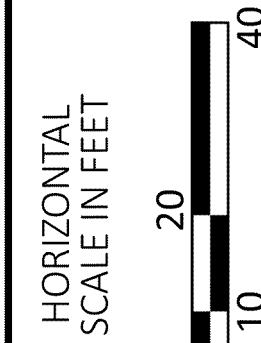


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www.arcadis-us-pw.bentley.com:arcadis-us-01/Documents/01 Active Projects/30149976/400_CAD/403-Engineering_NEAS/Geotechnical/Sheets/114865_IP020.dgn

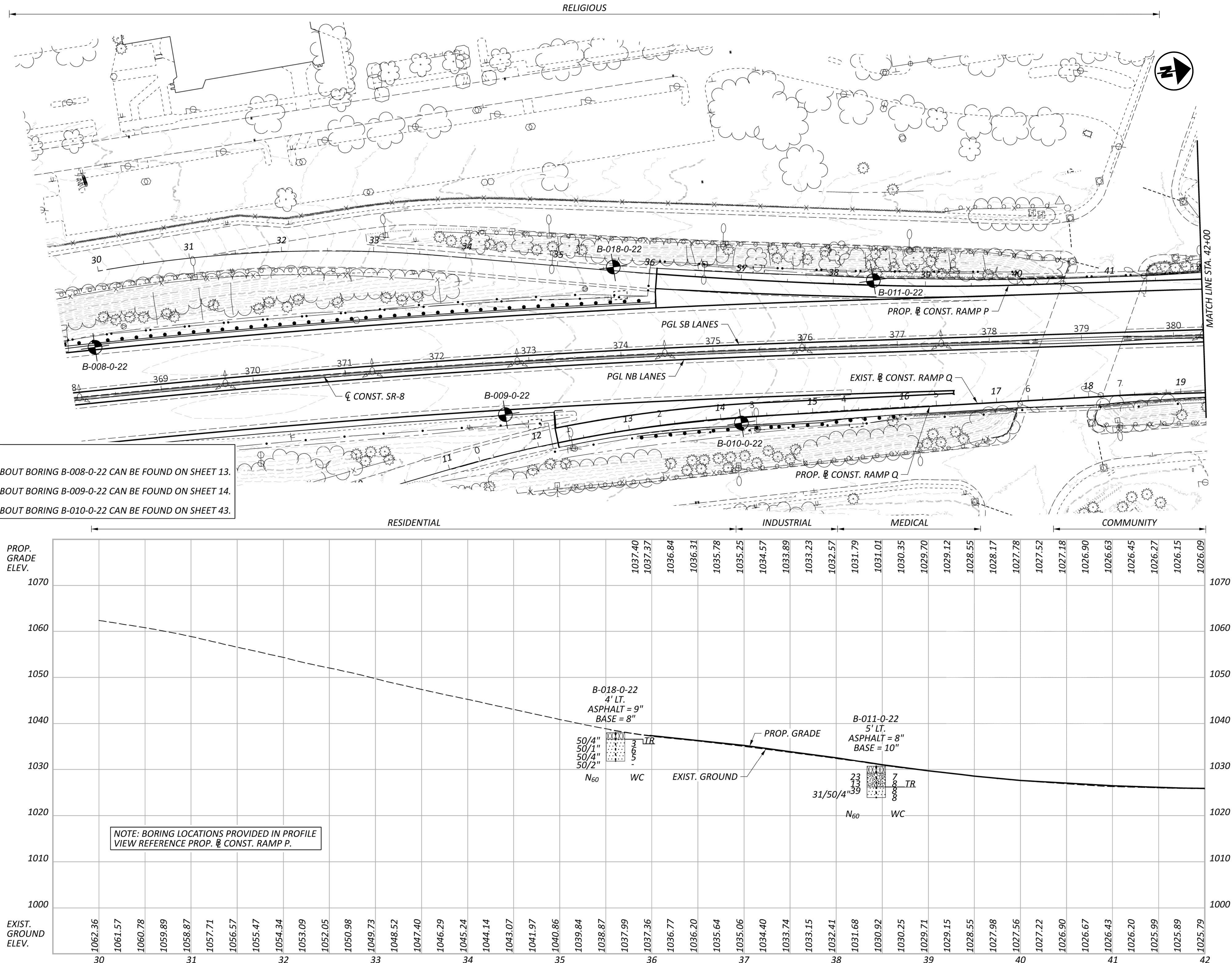


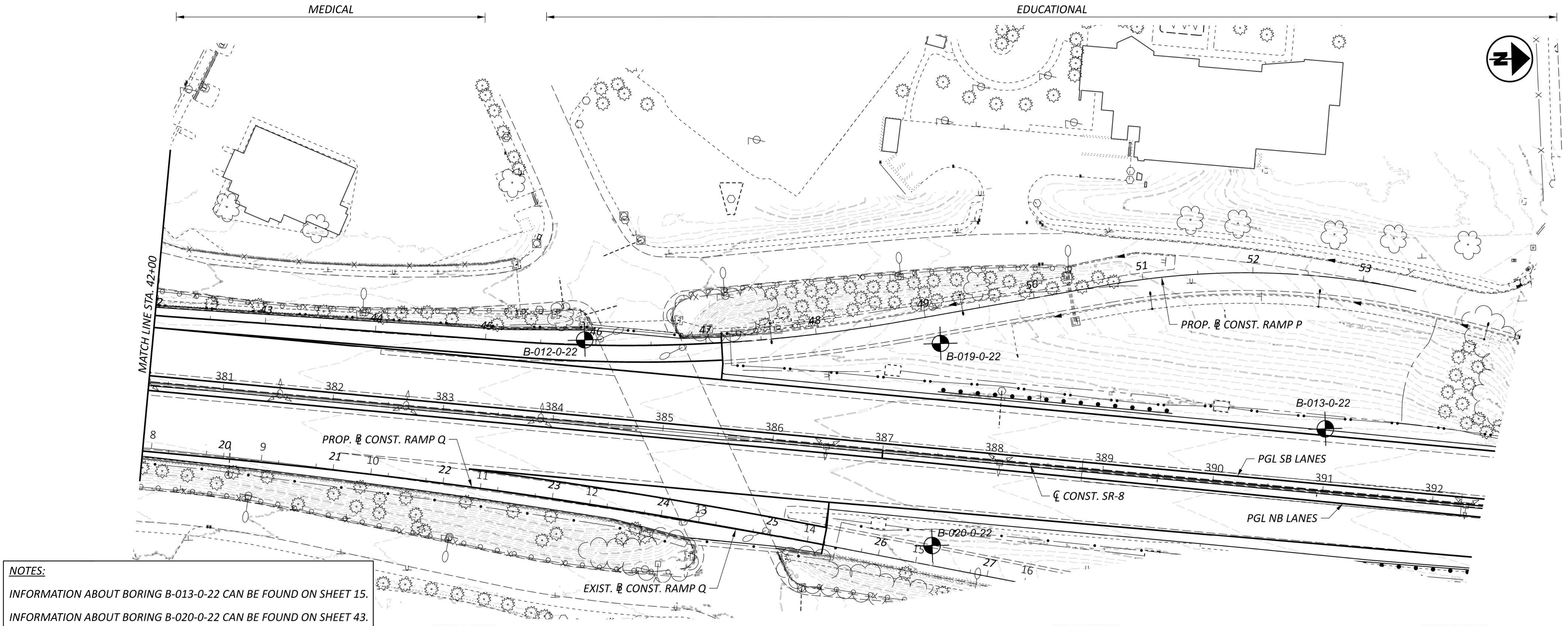


GEOTECHNICAL PROFILE - ROADWAY
RETAINING WALL RW-10

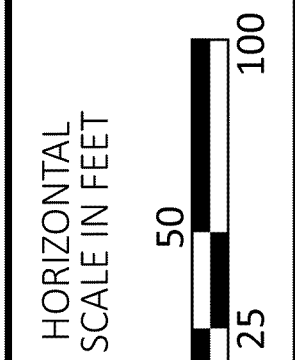
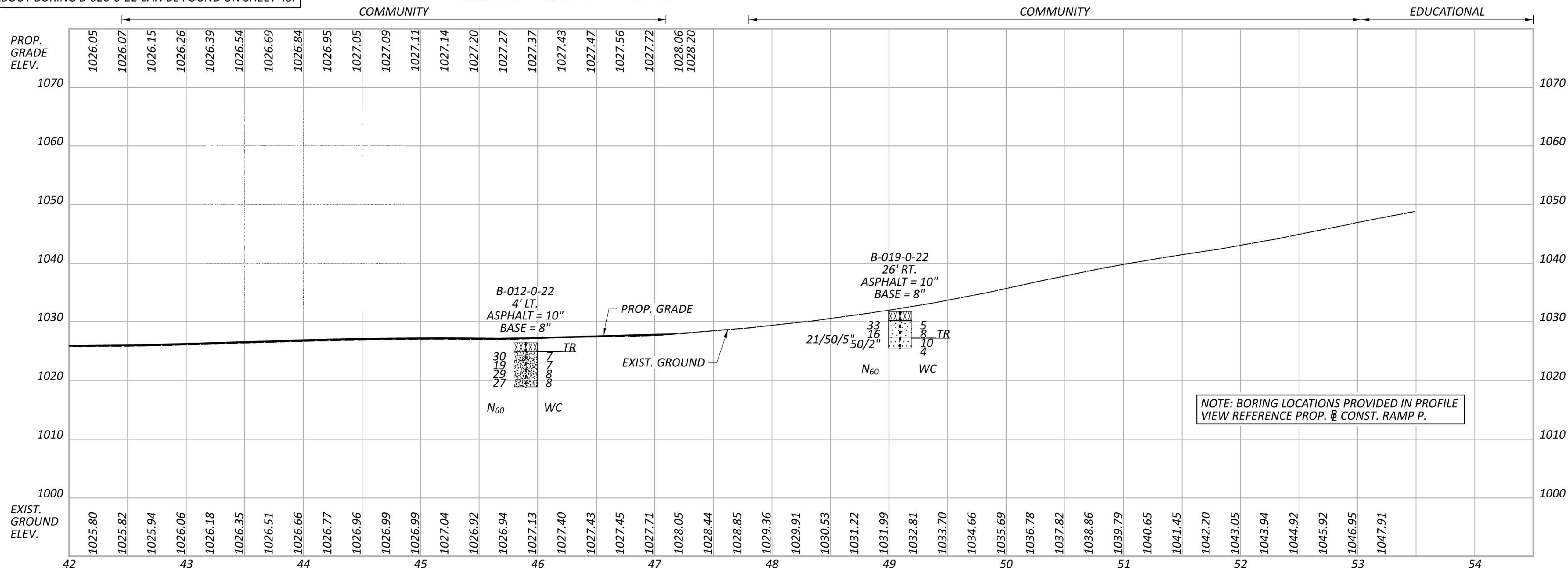


NOTES:
INFORMATION ABOUT BORING B-008-0-22 CAN BE FOUND ON SHEET 13.
INFORMATION ABOUT BORING B-009-0-22 CAN BE FOUND ON SHEET 14.
INFORMATION ABOUT BORING B-010-0-22 CAN BE FOUND ON SHEET 43.



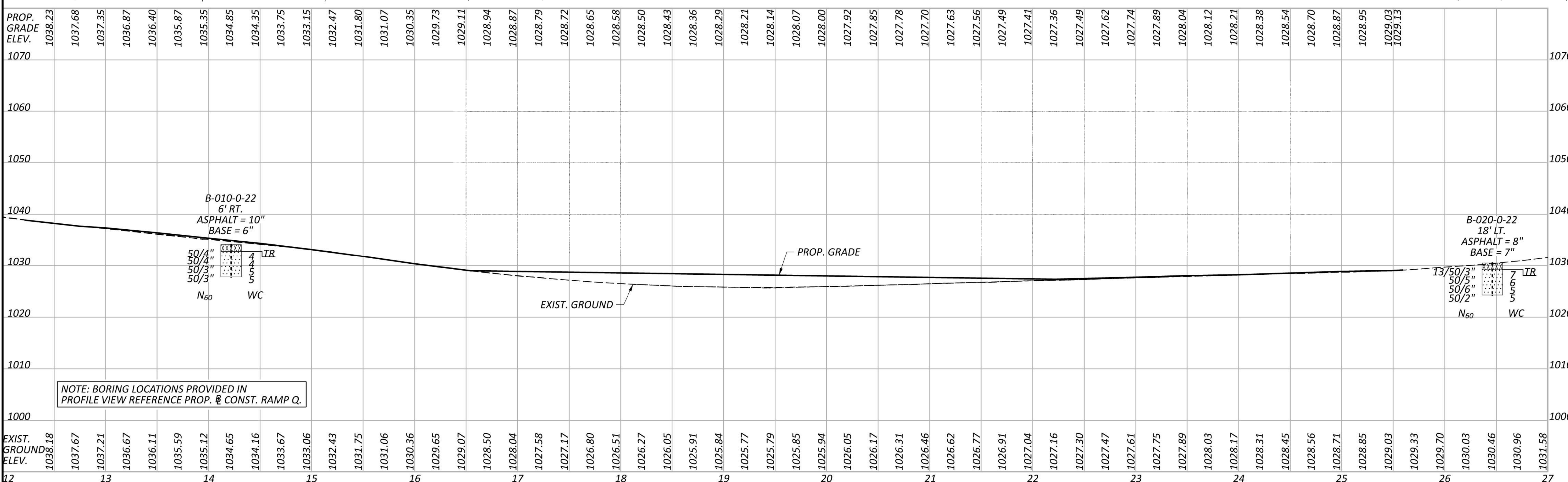
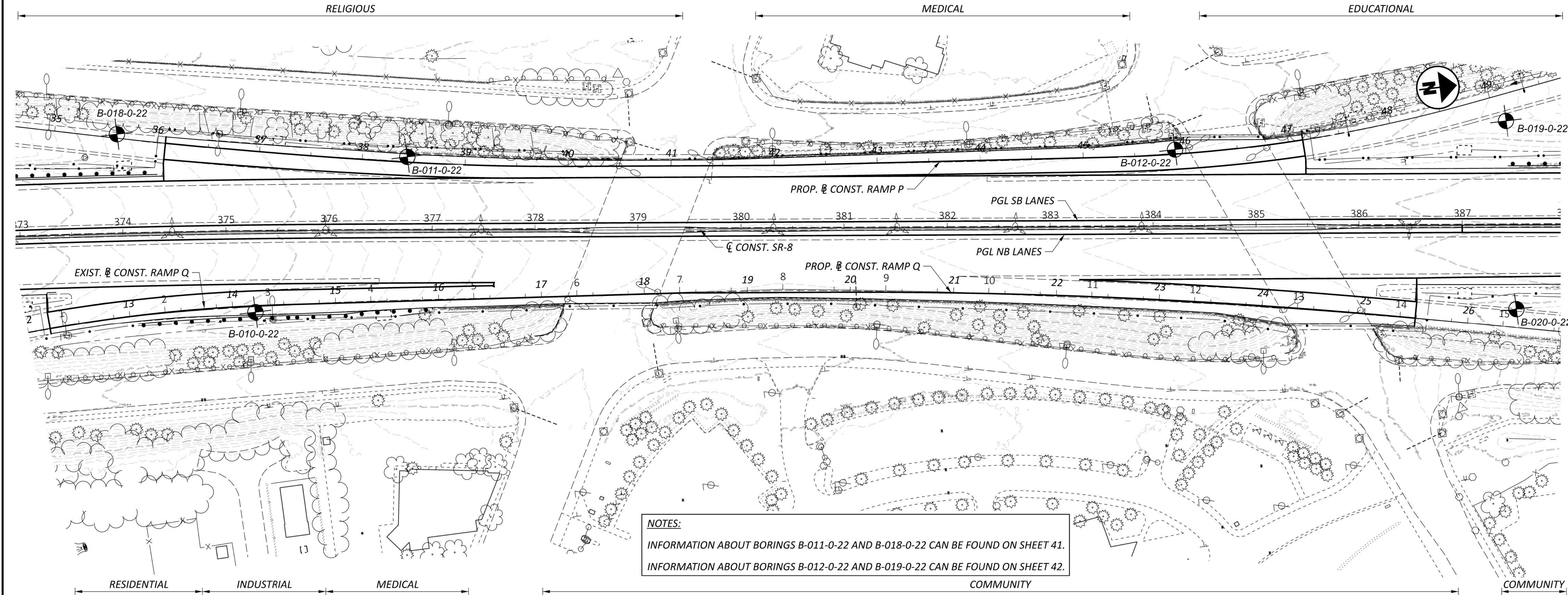


NOTES:
INFORMATION ABOUT BORING B-013-0-22 CAN BE FOUND ON SHEET 15.
INFORMATION ABOUT BORING B-020-0-22 CAN BE FOUND ON SHEET 43.



GEOTECHNICAL PROFILE - ROADWAY
SR-8 RAMP P - STA. 42+00 TO END

DESIGN AGENCY	
NEAS	
2800 CORPORATE EXCHANGE DR, SUITE 240 COLUMBUS, OH, 43231 TEL: 614.714.0299 WWW.NEASINC.COM	
DESIGNER	
RS	
REVIEWER	
CH 08/14/26	
PROJECT ID	
114865	
SUBSET	TOTAL
42	116
SHEET	TOTAL
P.551	625



HORIZONTAL SCALE IN FEET

0 25 50 100

GEOTECHNICAL PROFILE - ROADWAY

SR-8 RAMP Q - STA. 12+00 TO STA. 27+00

DESIGN AGENCY

NEAS

2800 CORPORATE EXCHANGE DR, SUITE 240 COLUMBUS, OH, 43231 TEL: 614.714.0299 WWW.NEASINC.COM

DESIGNER

RS

REVIEWER

CH 08/14/26

PROJECT ID

114865

SUBSET TOTAL

43 116

SHEET TOTAL

P.552 625

APPENDIX B

BORING LOGS AND TEST RESULTS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:20 - X11ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11 65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PID: 114865	SFN:	PROJECT: SUM-IR-77/SR8-11.65/0.00	STATION / OFFSET: 4+53, 1' LT.		START: 3/15/23	END: 3/16/23	PG 2 OF 2		B-002-0-22														
MATERIAL DESCRIPTION AND NOTES			ELEV. 1060.6	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL			
				31							GR	CS	FS	SI	CL	LL	PL	PI	WC				
				32																			
				33																			
				34																			
			1055.0	35																			
SHALE, DARK GRAY, UNWEATHERED, WEAK, LAMINATED, FISSILE, BEDDING DISCONTINUITIES: LOW ANGLE, JOINT DISCONTINUITIES: HIGH ANGLE FROM 32.0'-33.8', HIGHLY FRACTURED TO MODERATELY FRACTURED, NARROW TO TIGHT, SLIGHTLY ROUGH, BLOCKY/DISTURBED/SEAMY, GOOD TO VERY GOOD SURFACE CONDITION; RQD 93%, REC 96%.				36	96		98	NQ2-4													CORE		
				37																			
				38																			
				39																			
				40																			
			1049.8	EOB																			
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																							
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; SHOVELED SOIL CUTTINGS																							

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:20 - X11ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11 65-0-00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PID: 114865	SFN:	PROJECT: SUM-IR-77/SR8-11.65/0.00	STATION / OFFSET: 334+86, 90' LT.		START: 3/9/23		END: 3/9/23		PG 2 OF 2		B-004-0-22										
MATERIAL DESCRIPTION AND NOTES			ELEV. 1003.3	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED
SHALE , GRAY, SEVERELY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, LAMINATED, FISSILE, INTERBEDDED 1/8" CLAY SEAMS, BEDDING DISCONTINUITIES: LOW ANGLE, HIGHLY FRACTURED TO FRACTURED, OPEN TO NARROW, SLIGHTLY ROUGH, LAMINATED, POOR TO FAIR SURFACE CONDITION; RQD 0%, REC 83%. <i>(continued)</i>						0		83	NQ2-1											CORE	
					31																
					32																
					33																
					34																
	35																				
			997.8	EOB																	
NOTES: GROUNDWATER ENCOUNTERED AT 15.0' DURING DRILLING. HOLE DID NOT CAVE. ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 50 GAL. BENTONITE GROUT																					

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:20 - X11ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55T		STATION / OFFSET: 357+15, 56' RT.		EXPLORATION ID														
TYPE: SUBGRADE		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: CLX S008		B-006-0-22														
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 1/24/22		ELEVATION: 1036.1 (MSL) EOB: 6.5 ft.		PAGE														
START: 3/7/23 END: 3/7/23		SAMPLING METHOD: SPT		ENERGY RATIO (%): 63.4		LAT / LONG: 41.073915, -81.504048		1 OF 1														
MATERIAL DESCRIPTION AND NOTES			ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL	
			1036.1								GR	CS	FS	SI	CL	LL	PL	PI	WC			
10.0" ASPHALT AND 7.0" BASE (DRILLERS DESCRIPTION)			1034.7	1																		
VERY DENSE, BROWN, COARSE AND FINE SAND, LITTLE SILT, TRACE CLAY, TRACE STONE FRAGMENTS, RELIC ROCK STRUCTURE, STONE FRAGMENTS ARE FRIABLE SANDSTONE, DAMP				2		40 50/2"	-	75	SS-1	-	0	3	71	20	6	NP	NP	NP	8	A-3a (0)		
				3		35 50/1"	-	86	SS-2	-	-	-	-	-	-	-	-	-	8	A-3a (V)		
			1031.6	4																		
VERY DENSE, DARK GRAY, SANDY SILT, LITTLE CLAY, TRACE STONE FRAGMENTS, RELIC ROCK STRUCTURE, DAMP			1030.7	5		13 50/5"	-	91	SS-3	-	4	15	41	27	13	NP	NP	NP	11	A-4a (1)		
			1029.6	6		50	-	83	SS-4	-	-	-	-	-	-	-	-	-	11	Rock (V)		
SANDSTONE, DARK GRAY AND BROWN, HIGHLY WEATHERED, WEAK, FRIABLE.				EOB																		
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																						
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; SHOVELED SOIL CUTTINGS																						

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:20 - X11ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11 65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / ASHBAUGH		DRILL RIG: CME 55T		STATION / OFFSET: 368+33, 54' LT.		EXPLORATION ID											
TYPE: SUBGRADE		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: CLX S008		B-008-0-22											
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 1/24/22		ELEVATION: 1041.8 (MSL) EOB: 6.8 ft.		PAGE											
START: 3/9/23 END: 3/9/23		SAMPLING METHOD: SPT		ENERGY RATIO (%): 63.4		LAT / LONG: 41.076990, -81.504379		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		1041.8							GR	CS	FS	SI	CL	LL	PL	PI	WC		
10.0" ASPHALT AND 8.0" BASE (DRILLERS DESCRIPTION)		1040.3	1																
DENSE, GRAY, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, DAMP		1038.8	2	14	39	33	SS-1	-	34	19	16	23	8	NP	NP	NP	6	A-2-4 (0)	
DENSE, BROWN, COARSE AND FINE SAND, LITTLE SILT, TRACE STONE FRAGMENTS, TRACE CLAY, RELIC ROCK STRUCTURE, STONE FRAGMENTS ARE FRIABLE SANDSTONE, DAMP			3	16															
			4	20	37	28	SS-2	-	6	7	68	14	5	NP	NP	NP	7	A-3a (0)	
			5	21															
		1035.8	6	17	44	56	SS-3	-	-	-	-	-	-	-	-	-	9	A-3a (V)	
SANDSTONE, BROWN, MODERATELY WEATHERED, SLIGHTLY STRONG, FRIABLE.		1035.0	TR	20															
			EOB	22															
				16	-	78	SS-4	-	-	-	-	-	-	-	-	-	9	Rock (V)	
				50/3"															
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; SHOVELED SOIL CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:20 - X11ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55T		STATION / OFFSET: 372+71, 55' RT.		EXPLORATION ID														
TYPE: SUBGRADE		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: CLX S008		B-009-0-22														
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 1/24/22		ELEVATION: 1040.3 (MSL) EOB: 7.5 ft.		PAGE														
START: 3/7/23 END: 3/7/23		SAMPLING METHOD: SPT		ENERGY RATIO (%): 63.4		LAT / LONG: 41.078164, -81.503859		1 OF 1														
MATERIAL DESCRIPTION AND NOTES			ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL	
			1040.3								GR	CS	FS	SI	CL	LL	PL	PI	WC			
9.0" ASPHALT AND 8.0" BASE (DRILLERS DESCRIPTION)			1038.9	1																		
DENSE, LIGHT BROWN AND BROWN, COARSE AND FINE SAND , LITTLE SILT, TRACE CLAY, TRACE STONE FRAGMENTS, RELIC ROCK STRUCTURE, STONE FRAGMENTS ARE FRIABLE SANDSTONE, DAMP SANDSTONE , LIGHT BROWN AND BROWN, HIGHLY WEATHERED, WEAK TO SLIGHTLY STRONG, FRIABLE.			1037.3	2		9	45	78	SS-1	-	3	6	67	18	6	NP	NP	NP	9	A-3a (0)		
				3		14	29															
				4		12	27	71	67	SS-2	-	-	-	-	-	-	-	-	-	8	Rock (V)	
				5		10	18	45	28	SS-3	-	-	-	-	-	-	-	-	-	8	Rock (V)	
				6		13	20															
			1032.8	7		20	17	39	44	SS-4	-	-	-	-	-	-	-	-	9	Rock (V)		
				EOB																		
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																						
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; SHOVELED SOIL CUTTINGS																						

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:20 - X11ACTIVE PROJECTSACTIVE SOIL PROJECTS SUM-77-SR8-11 65-0.00 (114865) GINT FILES SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55T		STATION / OFFSET: 375+28, 78' RT.		EXPLORATION ID															
TYPE: SUBGRADE		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: CLX S008		B-010-0-22															
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 1/24/22		ELEVATION: 1034.1 (MSL) EOB: 6.3 ft.		PAGE															
START: 3/8/23 END: 3/8/23		SAMPLING METHOD: SPT		ENERGY RATIO (%): 63.4		LAT / LONG: 41.078854, -81.503677		1 OF 1															
MATERIAL DESCRIPTION AND NOTES			ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL		
			1034.1								GR	CS	FS	SI	CL	LL	PL	PI	WC				
10.0" ASPHALT AND 6.0" BASE (DRILLERS DESCRIPTION)			1032.8	TR		1																	
SANDSTONE, LIGHT BROWN AND PINKISH BROWN, MODERATELY WEATHERED, SLIGHTLY STRONG, FRIABLE.						2	50/4"	-	75	SS-1	-	-	-	-	-	-	-	-	4	Rock (V)			
						3	50/4"	-	50	SS-2	-	-	-	-	-	-	-	-	-	4	Rock (V)		
						4																	
						5	50/3"	-	67	SS-3	-	-	-	-	-	-	-	-	-	-	5	Rock (V)	
						6	50/3"	-	33	SS-4	-	-	-	-	-	-	-	-	-	-	5	Rock (V)	
			1027.8	EOB																			
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																							
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; SHOVELED SOIL CUTTINGS																							

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:20 - X11ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / ASHBAUGH		DRILL RIG: CME 55T		STATION / OFFSET: 376+76, 72' LT.		EXPLORATION ID											
TYPE: SUBGRADE		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: CLX S008		B-011-0-22											
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 1/24/22		ELEVATION: 1030.7 (MSL) EOB: 6.8 ft.		PAGE											
START: 3/9/23 END: 3/9/23		SAMPLING METHOD: SPT		ENERGY RATIO (%): 63.4		LAT / LONG: 41.079310, -81.504147		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		1030.7							GR	CS	FS	SI	CL	LL	PL	PI	WC		
8.0" ASPHALT AND 10.0" BASE (DRILLERS DESCRIPTION)		1029.2	1																
MEDIUM DENSE, GRAY, GRAVEL AND STONE FRAGMENTS WITH SAND, LITTLE SILT, TRACE CLAY, RELIC ROCK STRUCTURE, DAMP			2	6	11	23	22	SS-1	-	33	21	23	18	5	NP	NP	NP	7	A-1-b (0)
			3	5	11														
		1026.2	4	5	7	13	33	SS-2	-	-	-	-	-	-	-	-	8	A-1-b (V)	
SANDSTONE, LIGHT BROWN, SLIGHTLY TO MODERATELY WEATHERED, WEAK TO SLIGHTLY STRONG, FRIABLE.			5	8	12	39	67	SS-3	-	-	-	-	-	-	-	-	8	Rock (V)	
		1023.9	6	31	25														
				50/4"		-	30	SS-4	-	-	-	-	-	-	-	-	8	Rock (V)	
			EOB																
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; SHOVELED SOIL CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:20 - X11ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11 65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / ASHBAUGH		DRILL RIG: CME 55T		STATION / OFFSET: 384+22, 75' LT.		EXPLORATION ID													
TYPE: SUBGRADE		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: CLX S008		B-012-0-22													
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 1/24/22		ELEVATION: 1026.4 (MSL) EOB: 7.5 ft.		PAGE													
START: 3/8/23 END: 3/8/23		SAMPLING METHOD: SPT		ENERGY RATIO (%): 63.4		LAT / LONG: 41.081340, -81.503812		1 OF 1													
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL		
		1026.4							GR	CS	FS	SI	CL	LL	PL	PI	WC				
10.0" ASPHALT AND 8.0" BASE (DRILLERS DESCRIPTION)		1024.9	TR	1																	
MEDIUM DENSE, GRAY AND BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND, LITTLE SILT, TRACE CLAY, RELIC ROCK STRUCTURE, DAMP			TR	2	11																
				3	13	30	56	SS-1	-	46	17	15	17	5	NP	NP	NP	7	A-1-b (0)		
				4	15																
				5	9	19	44	SS-2	-	41	16	18	20	5	NP	NP	NP	7	A-1-b (0)		
				6	8																
				7	10																
				8	14	29	50	SS-3	-	-	-	-	-	-	-	-	8	A-1-b (V)			
				9	13																
				10	11																
				11	8	27	44	SS-4	-	-	-	-	-	-	-	-	8	A-1-b (V)			
		1018.9	EOB	12	15																
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																					
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; SHOVELED SOIL CUTTINGS																					

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

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:20 - X11ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55T		STATION / OFFSET: 373+97, 98' LT.		EXPLORATION ID											
TYPE: SUBGRADE		SAMPLING FIRM / LOGGER: NEAS / ASHBAUGH		HAMMER: CME AUTOMATIC		ALIGNMENT: CLX S008		B-018-0-22											
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 1/24/22		ELEVATION: 1038.0 (MSL) EOB: 6.2 ft.		PAGE											
START: 3/9/23 END: 3/9/23		SAMPLING METHOD: SPT		ENERGY RATIO (%): 63.4		LAT / LONG: 41.078552, -81.504364		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		1038.0							GR	CS	FS	SI	CL	LL	PL	PI	WC		
9.0" ASPHALT AND 8.0" BASE (DRILLERS DESCRIPTION)																			
		1036.6	TR	1															
SANDSTONE, BROWN, SLIGHTLY WEATHERED, SLIGHTLY STRONG, FRIABLE.				2	50/4"	-	75	SS-1	-	-	-	-	-	-	-	-	3	Rock (V)	< \>
				3	50/1"	-	100	SS-2	-	-	-	-	-	-	-	-	6	Rock (V)	< \>
				4															< \>
				5	50/4"	-	100	SS-3	-	-	-	-	-	-	-	-	5	Rock (V)	< \>
		1031.8	EOB	6	50/2"	-	0	SS-4	-	-	-	-	-	-	-	-	-		< \>
SS-4 CONTAINS NO RECOVERY																			
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; SHOVELED SOIL CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:20 - X11ACTIVE PROJECTSACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55T		STATION / OFFSET: 387+43, 102' LT.		EXPLORATION ID											
TYPE: SUBGRADE		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: CLX S008		B-019-0-22											
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 1/24/22		ELEVATION: 1031.7 (MSL) EOB: 6.2 ft.		PAGE											
START: 3/8/23 END: 3/8/23		SAMPLING METHOD: SPT		ENERGY RATIO (%): 63.4		LAT / LONG: 41.082223, -81.503757		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		1031.7							GR	CS	FS	SI	CL	LL	PL	PI	WC		
10.0" ASPHALT AND 8.0" BASE (DRILLERS DESCRIPTION)		1030.2	1																
MEDIUM DENSE TO DENSE, LIGHT BROWN, COARSE AND FINE SAND, TRACE SILT, TRACE STONE FRAGMENTS, TRACE CLAY, RELIC ROCK STRUCTURE, STONE FRAGMENTS ARE FRIABLE SANDSTONE, DAMP			2	31															
			3	24	33	44	SS-1	-	8	12	68	10	2	NP	NP	NP	5	A-3a (0)	
			4	7															
			5	8	16	33	SS-2	-	-	-	-	-	-	-	-	-	8	A-3a (V)	
		1027.2	6	7															
SANDSTONE, LIGHT BROWN, SLIGHTLY WEATHERED, SLIGHTLY STRONG, FRIABLE.			1	21															
			2	50/5"	-	64	SS-3	-	-	-	-	-	-	-	-	-	10	Rock (V)	
		1025.5	3																
			4																
			5																
			6	60/2"	-	50	SS-4	-	-	-	-	-	-	-	-	-	4	Rock (V)	
			EOB																
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; SHOVELED SOIL CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X11ACTIVE PROJECTSACTIVE SOIL PROJECTS SUM-77-SR8-11 65-0.00 (114865) GINT FILES SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55T		STATION / OFFSET: 387+52, 81' RT.		EXPLORATION ID														
TYPE: SUBGRADE		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: CLX S008		B-020-0-22														
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 1/24/22		ELEVATION: 1030.5 (MSL) EOB: 6.2 ft.		PAGE														
START: 3/8/23 END: 3/8/23		SAMPLING METHOD: SPT		ENERGY RATIO (%): 63.4		LAT / LONG: 41.082184, -81.503095		1 OF 1														
MATERIAL DESCRIPTION AND NOTES			ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL	
			1030.5								GR	CS	FS	SI	CL	LL	PL	PI	WC			
8.0" ASPHALT AND 7.0" BASE (DRILLERS DESCRIPTION)			1029.2	TR		1																
SANDSTONE, LIGHT BROWN AND BROWN, SLIGHTLY TO MODERATELY WEATHERED, WEAK TO SLIGHTLY STRONG, FRIABLE.						2	13 50/3"	-	56	SS-1	-	-	-	-	-	-	-	-	7	Rock (V)		
						3	50/5"	-	60	SS-2	-	-	-	-	-	-	-	-	6	Rock (V)		
						4																
						5	50	-	50	SS-3	-	-	-	-	-	-	-	-	5	Rock (V)		
						6	50/2"	-	50	SS-4	-	-	-	-	-	-	-	-	5	Rock (V)		
			1024.3	EOB																		
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																						
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; SHOVELED SOIL CUTTINGS																						

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.
ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 2 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PID: 114865	SFN:	PROJECT: SUM-IR-77/SR8-11.65/0.00	STATION / OFFSET: 0+42, 10' RT.		START: 10/7/24		END: 10/7/24		PG 2 OF 2		B-022-0-24										
MATERIAL DESCRIPTION AND NOTES			ELEV. 1004.3	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
SHALE , DARK GRAY, SLIGHTLY WEATHERED, VERY WEAK TO WEAK, LAMINATED, FISSILE, BEDDING DISCONTINUITIES: LOW ANGLE, HIGHLY FRACTURED TO SLIGHTLY FRACTURED, NARROW TO TIGHT, SLIGHTLY ROUGH, BLOCKY/DISTURBED/SEAMY, GOOD TO FAIR SURFACE CONDITION; RQD 78%, REC 100%. <i>(continued)</i> @32.7'-33.1'; Qu = 390 PSI @ 2.0% STRAIN (DRY UNIT WEIGHT= 146.6 PCF)					31	78		100	NQ2-1											CORE	
					32																
					33																
					34																
			998.8		35																
				EOB																	
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.																					
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																					

[illegible]

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X11ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55TB		STATION / OFFSET: 12+34, 17' LT.		EXPLORATION ID														
TYPE: BRIDGE		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: PR. CAROL STREET		B-027-0-24														
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA / NQ2		CALIBRATION DATE: 7/30/24		ELEVATION: 1057.1 (MSL) EOB: 25.1 ft.		PAGE														
START: 10/2/24 END: 10/2/24		SAMPLING METHOD: SPT / NQ2		ENERGY RATIO (%): 89		LAT / LONG: 41.075365, -81.504645		1 OF 1														
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL		
		1057.1								GR	CS	FS	SI	CL	LL	PL	PI	WC				
8.0" CONCRETE AND 11.0" BASE (DRILLERS DESCRIPTION)		1055.5	1																			
STIFF TO VERY STIFF, BROWN, SANDY SILT, LITTLE CLAY, TRACE TO LITTLE GRAVEL, SS-1 CONTAINS IRON STAINING, DAMP			2																			
			3		8	7	22	78	SS-1	1.50	10	11	36	30	13	23	17	6	14	A-4a (2)		
			4																			
			5		4																	
			6		4	10	21	83	SS-2	3.00	14	10	24	36	16	22	16	6	12	A-4a (3)		
		1050.1	7																			
VERY DENSE, BROWN, SAND AND SILT, LITTLE CLAY, DAMP		1048.7	8		4																	
SANDSTONE, BROWN AND ORANGISH BROWN AND GRAY, SLIGHTLY TO MODERATELY WEATHERED, SLIGHTLY TO MODERATELY STRONG, FINE TO MEDIUM GRAINED, VERY THIN TO MEDIUM BEDDED, FRIABLE, VUGGY, CONTAINS SHALE LAMINATIONS AND INCLUSIONS, CONTAINS CONCRETIONS, CONTAINS 1/8" CLAY SEAM AT 19.6', BEDDING DISCONTINUITIES: LOW ANGLE, JOINT DISCONTINUITIES: HIGH ANGLE FROM 11.2-11.9' AND 15.8-16.9', HIGHLY FRACTURED TO SLIGHTLY FRACTURED, OPEN TO NARROW, SLIGHTLY ROUGH, VERY BLOCKY, GOOD TO POOR SURFACE CONDITION; RQD 79%, REC 97%. @14.0'-14.4'; Qu = 2600 PSI @ 1.2% STRAIN (DRY UNIT WEIGHT= 139.7 PCF)			TR 8		50/5"	-	82	SS-3	-	-	-	-	-	-	-	-	-	10	A-2-4 (V)			
			9																			
			10		50/1"	-	100	SS-4	-	-	-	-	-	-	-	-	-	-	7	Rock (V)		
			11																			
			12																			
			13																			
			14																			
			15		75		98	NQ2-1													CORE	
			16																			
			17																			
			18																			
			19																			
			20																			
			21																			
			22																			
23		88		95	NQ2-2														CORE			
24																						
		1032.0	EOB 25																			
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.																						
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																						

[illegible]

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2.0 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X11ACTIVE PROJECTS\SUM-77-SR8-11 65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PID: 114865	SFN:	PROJECT: SUM-IR-77/SR8-11.65/0.00	STATION / OFFSET: 14+71, 18' RT.			START: 10/3/24	END: 10/3/24	PG 2 OF 2	B-029-0-24											
MATERIAL DESCRIPTION AND NOTES		ELEV. 1020.9	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
		GR								CS	FS	SI	CL	LL	PL	PI				
		1015.9	EOB	31	57	92	NQ2-2											CORE		
				32																
				33																
				34																
				35																
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.																				
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 3 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																				

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-0.00

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55T 2024		STATION / OFFSET: 0+10, 33' RT.		EXPLORATION ID													
TYPE: RETAINING WALL		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: RETAINING WALL 2		B-031-0-24													
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA / NQ2		CALIBRATION DATE: 5/15/24		ELEVATION: 1066.7 (MSL) EOB: 26.2 ft.		PAGE													
START: 10/22/24 END: 10/23/24		SAMPLING METHOD: SPT / NQ2		ENERGY RATIO (%): 71		LAT / LONG: 41.067120, -81.504408		1 OF 1													
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL		
		1066.7							GR	CS	FS	SI	CL	LL	PL	PI	WC				
7.0" ASPHALT AND 11.0" BASE (DRILLERS DESCRIPTION)		1065.2	1																		
HARD, BROWN MOTTLED WITH GRAY, SANDY SILT, SOME CLAY, TRACE GRAVEL, TRACE IRON STAINING, DAMP			2																		
			3	5	9	44	SS-1	4.50	10	12	16	41	21	27	18	9	14	A-4a (5)			
			4																		
			5	5																	
			6	8	9	20	56	SS-2	4.50	5	11	25	37	22	26	17	9	12	A-4a (5)		
STIFF TO VERY STIFF, BROWN MOTTLED WITH GRAY, SILT AND CLAY, LITTLE SAND, TRACE GRAVEL, DAMP			7																		
			8	3	8	20	67	SS-3	4.50	-	-	-	-	-	-	-	-	14	A-4a (V)		
		1057.2	9																		
		1054.2	10	2	2	5	50	SS-4	2.00	-	-	-	-	-	-	-	17	A-6a (V)			
		1054.2	11																		
SHALE, GRAY AND BROWN, SEVERELY WEATHERED, VERY WEAK, FISSILE, IRON STAINING.			12																		
			13	25	50/3"	-	33	SS-5	-	-	-	-	-	-	-	-	-	7	Rock (V)		
		1050.5	14																		
		1050.5	15	34	43	-	21	SS-6	-	-	-	-	-	-	-	-	5	Rock (V)			
		1050.5	16	50/2"																	
SHALE, DARK GRAY, SLIGHTLY TO MODERATELY WEATHERED, WEAK TO MODERATELY STRONG, LAMINATED, FISSILE, SILTY, BEDDING DISCONTINUITIES: LOW ANGLE, HIGHLY FRACTURED TO MODERATELY FRACTURED, NARROW, SLIGHTLY ROUGH, LAMINATED/SHEARED, FAIR SURFACE CONDITION; RQD 18%, REC 82%. @18.2'-18.6'; Qu = 6083 PSI @ 1.3% STRAIN (DRY UNIT WEIGHT= 156.2 PCF)			17																		
			18																		
			19																		
			20																		
			21	70	96	NQ2-1														CORE	
			22																		
			23																		
			24																		
			25																		
			26																		
SILTSTONE, GRAY AND DARK GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, THIN TO VERY THICK BEDDED, ARGILLACEOUS, BEDDING DISCONTINUITIES: LOW ANGLE, FRACTURED TO INTACT, NARROW, SLIGHTLY ROUGH TO VERY ROUGH, INTACT, GOOD SURFACE CONDITION; RQD 86%, REC 100%.		1048.2																			
		1040.5																			
EOB																					
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.																					
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 3 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																					

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PID: 114865	SFN:	PROJECT: SUM-IR-77/SR8-11.65/0.00	STATION / OFFSET: 3+36, 29' LT.			START: 10/8/24	END: 10/8/24	PG 2 OF 2		B-032-0-24											
MATERIAL DESCRIPTION AND NOTES			ELEV. 997.0	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
SHALE , DARK GRAY, SLIGHTLY WEATHERED, WEAK, LAMINATED, FISSILE, BEDDING DISCONTINUITIES: LOW ANGLE, FRACTURED TO SLIGHTLY FRACTURED, NARROW TO TIGHT, SLIGHTLY ROUGH, BLOCKY/DISTURBED/SEAMY, GOOD TO FAIR SURFACE CONDITION; RQD 70%, REC 100%. <i>(continued)</i>					70		100	NQ2-1											CORE		
				31																	
				32																	
				33																	
				34																	
			991.7	EOB		35															
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED. ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 3 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																					

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X11ACTIVE PROJECTS\SUM-77-SR8-11 65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PID: 114865	SFN:	PROJECT: SUM-IR-77/SR8-11.65/0.00	STATION / OFFSET: 4+80, 45' RT.	START: 10/21/24	END: 10/22/24	PG 3 OF 3	B-033-0-24												
MATERIAL DESCRIPTION AND NOTES		ELEV. 991.2	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
		GR							CS	FS	SI	CL	LL	PL	PI				
		987.9	63																
			64																
			65																
		EOB																	
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 3 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																			

MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
										GR	CS	FS	SI	CL	LL	PL	PI			
9.0" CONCRETE (DRILLERS DESCRIPTION)		1009.9																		
STIFF TO HARD, BROWN MOTTLED WITH GRAY AND ORANGISH BROWN, CLAY , SOME SILT, LITTLE SAND, TRACE GRAVEL, IRON STAINING, MOIST TO DAMP		1009.1	TR	1	3	7	33	SS-1	2.00	-	-	-	-	-	-	-	29	A-7-6 (V)		
		2		3																
		3																		
		4		2	4	12	72	SS-2	3.00	6	6	6	28	54	47	25	22	23	A-7-6 (14)	
		5		6																
		6		1																
		7		2	6	10	50	SS-3	4.25	-	-	-	-	-	-	-	-	21	A-7-6 (V)	
		8																		
SHALE, GRAY AND ORANGISH BROWN, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK.		1000.9	TR	9	2	17	58	44	SS-4	-	-	-	-	-	-	-	9	Rock (V)		
		999.4		10	31															
VERY STIFF, GRAY AND ORANGISH BROWN, SILTY CLAY , SOME SAND, TRACE GRAVEL, RESIDUAL ROCK, DAMP		996.9	TR	11	2															
		996.9		12	6	23	35	44	SS-5	3.00	10	14	13	37	26	35	19	16	15	A-6b (8)
SHALE, GRAY AND ORANGISH BROWN, MODERATELY TO HIGHLY WEATHERED, VERY WEAK TO WEAK.		995.6	TR	13																
		995.6		14	2	50/3"	-	22	SS-6	-	-	-	-	-	-	-	-	6	Rock (V)	

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 1 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
										GR	CS	FS	SI	CL	LL	PL	PI			
9.0" CONCRETE (DRILLERS DESCRIPTION)		1014.6																		
MEDIUM DENSE TO VERY DENSE, LIGHT BROWN, STONE FRAGMENTS WITH SAND, LITTLE SILT, TRACE CLAY, RESEMBLES GRANULAR BASE, DAMP		1013.8	TR	1	17	56	44	SS-1	-	51	20	10	15	4	NP	NP	NP	6	A-1-b (0)	
				2	27															
				3																
				4	4															
				5	10	23	33	SS-2	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
				6																
				7	6															
				8	7	18	39	SS-3	-	34	11	9	31	15	33	20	13	9	A-6a (3)	
				9																
				10																
				11																
				12																
				13																
				14																
VERY STIFF, GRAY, SILT AND CLAY, SOME STONE FRAGMENTS, LITTLE SAND, RESIDUAL ROCK, CONTAINS NO INTACT SOIL FOR HP READINGS, DAMP		1009.1																		
SHALE, GRAY, MODERATELY WEATHERED, VERY WEAK TO WEAK.		1006.1	TR	9	39															
				10	50/3"	-	56	SS-4	-	-	-	-	-	-	-	-	-	5	Rock (V)	
				11																
				12	7															
				13	50/3"	-	44	SS-5	-	-	-	-	-	-	-	-	-	5	Rock (V)	
				14																
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				128																
				129																

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE CAVED AT 12' 3". DRILLED AS STAKED.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 1 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL		
									GR	CS	FS	SI	CL	LL	PL	PI					
9.0" CONCRETE (DRILLERS DESCRIPTION)		1022.2																			
MEDIUM DENSE, GRAY, STONE FRAGMENTS WITH SAND , LITTLE SILT, TRACE CLAY, RESEMBLES GRANULAR BASE, DAMP		1021.4		1	8	18	17	SS-1	-	-	-	-	-	-	-	-	6	A-1-b (V)			
				2	9	6				-	-	-	-	-	-	-					
				3																	
				4	4	7	16	22	SS-2	-	53	20	9	14	4	NP	NP	NP	7	A-1-b (0)	
				5																	
SHALE , GRAY, HIGHLY TO MODERATELY WEATHERED, VERY WEAK TO WEAK.		1016.2	TR	6	2																
				7	4	13	21	22	SS-3	-	-	-	-	-	-	-	-	7	Rock (V)		
				8																	
				9	4	6	8	17	28	SS-4	-	-	-	-	-	-	-	-	7	Rock (V)	
				10																	
				11	4																
				12	4	50/5"	-	36	SS-5	-	-	-	-	-	-	-	-	-	6	Rock (V)	
				13																	
				14	60/3"	-	0	SS-6	-	-	-	-	-	-	-	-	-	-	4	Rock (V)	
				15																	
ALSOED REFUSAL AT 16'		1006.2	EOB	16																	

AUGER REFUSAL AT 16.0'

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE CAVED AT 13' 2". DRILLED AS STAKED.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 1 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 45B		STATION / OFFSET: 401+03, 3' LT.		EXPLORATION ID											
TYPE: RETAINING WALL		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: RETAINING WALL 4		B-040-0-24											
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 1/24/22		ELEVATION: 1027.1 (MSL) EOB: 15.5 ft.		PAGE											
START: 8/28/24 END: 8/28/24		SAMPLING METHOD: SPT		ENERGY RATIO (%): 72.6		LAT / LONG: 41.071567, -81.504695		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		1027.1							GR	CS	FS	SI	CL	LL	PL	PI	WC		
9.0" CONCRETE (DRILLERS DESCRIPTION)		1026.3																	
MEDIUM DENSE, LIGHT BROWN, STONE FRAGMENTS WITH SAND, LITTLE SILT, TRACE CLAY, RESEMBLES GRANULAR BASE, DAMP			1	5															
			2	11 10	25	22	SS-1	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
			3																
			4	4 7 9	19	22	SS-2	-	48	22	10	16	4	NP	NP	NP	6	A-1-b (0)	
			5																
MEDIUM DENSE, GRAY, STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, DAMP			6	10															
			7	10 10	24	28	SS-3	-	-	-	-	-	-	-	-	7	A-1-b (V)		
			8																
SANDSTONE, GRAY AND ORANGISH BROWN, MODERATELY WEATHERED, WEAK TO MODERATELY STRONG, IRON STAINING.		1019.1	9	4 11 5	19	28	SS-4	-	38	18	13	21	10	NP	NP	NP	7	A-2-4 (0)	
		1016.1	10																
			11	3 9 38	57	50	SS-5	-	-	-	-	-	-	-	-	8	Rock (V)		
AUGER REFUSAL AT 15.5'			12																
			13																
			14	50/2"	-	100	SS-6	-	-	-	-	-	-	-	-	7	Rock (V)		
		1011.6	15																
			EOB																
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE CAVED AT 8'. DRILLED AS STAKED.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 1 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11 65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 45B		STATION / OFFSET: 402+26, 9' LT.		EXPLORATION ID											
TYPE: RETAINING WALL		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: RETAINING WALL 4		B-041-0-24											
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 1/24/22		ELEVATION: 1032.6 (MSL) EOB: 15.5 ft.		PAGE											
START: 8/28/24 END: 8/28/24		SAMPLING METHOD: SPT		ENERGY RATIO (%): 72.6		LAT / LONG: 41.071900, -81.504772		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		1032.6							GR	CS	FS	SI	CL	LL	PL	PI	WC		
8.0" CONCRETE (DRILLERS DESCRIPTION)		1031.9																	
VERY STIFF, BROWN AND ORANGISH BROWN, SANDY SILT, LITTLE CLAY, TRACE TO LITTLE GRAVEL, IRON STAINING, DAMP			1	7	21	56	SS-1	3.00	16	11	27	30	16	24	16	8	11	A-4a (2)	
			2	10	7														
			3																
			4	2	7	50	SS-2	2.75	7	16	31	30	16	25	16	9	15	A-4a (2)	
		1027.1	5	4															
VERY DENSE, BROWN, STONE FRAGMENTS WITH SAND, LITTLE SILT, TRACE CLAY, STONE FRAGMENTS ARE FRIABLE SANDSTONE, DAMP		1025.7	6	24	-	55	SS-3	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
SHALE, GRAY, HIGHLY TO MODERATELY WEATHERED, VERY WEAK TO WEAK.			7	50/5"															
			8																
			9	7	-	33	SS-4	-	-	-	-	-	-	-	-	-	10	Rock (V)	
			10	50/3"															
			11	50/5"	-	80	SS-5	-	-	-	-	-	-	-	-	-	5	Rock (V)	
			12																
			13																
			14	50/5"	-	0	SS-6	-	-	-	-	-	-	-	-	-	3	Rock (V)	
		1017.1	15																
AUGER REFUSAL AT 15.5'			EOB																
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE CAVED AT 13'. DRILLED AS STAKED.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 1 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11 65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55TB		STATION / OFFSET: 403+60, 18' RT.		EXPLORATION ID											
TYPE: RETAINING WALL		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: RETAINING WALL 4		B-042-0-24											
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA / NQ2		CALIBRATION DATE: 7/30/24		ELEVATION: 1037.0 (MSL) EOB: 20.4 ft.		PAGE											
START: 9/10/24 END: 9/10/24		SAMPLING METHOD: SPT / NQ2		ENERGY RATIO (%): 89		LAT / LONG: 41.072273, -81.504709		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		1037.0							GR	CS	FS	SI	CL	LL	PL	PI	WC		
6.0" TOPSOIL (DRILLERS DESCRIPTION)		1036.5																	
MEDIUM DENSE TO VERY DENSE, BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, RESIDUAL SANDSTONE ROCK STRUCTURE, DAMP			1																
			2																
			3	4	6	28	78	SS-1	-	33	13	23	22	9	NP	NP	NP	9	A-2-4 (O)
			4		13														
			5	15															
			6	50/4"	-	90	SS-2	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)	
		1030.0	7																
HARD, DARK GRAY, SANDY SILT, SOME CLAY, LITTLE STONE FRAGMENTS, RESIDUAL SHALE ROCK STRUCTURE, DAMP			8	16	29	89	44	SS-3	4.50	-	-	-	-	-	-	-	11	A-4a (V)	
			9	31															
		1027.5	10	50/4"	-	100	SS-4	-	-	-	-	-	-	-	-	-	6	Rock (V)	
SANDSTONE, GRAY AND BROWN AND DARK GRAY, SLIGHTLY TO MODERATELY WEATHERED, SLIGHTLY TO MODERATELY STRONG, FINE TO MEDIUM GRAINED, LAMINATED TO THIN FROM 10.4'-17.7', THIN TO THICK BEDDED FROM 17.7'-20.4', CONTAINS 1/4" CLAY SEAM AT 15.4', CONTAINS INTERBEDDED SHALE LAMINATIONS AND INCLUSIONS, BEDDING DISCONTINUITIES: LOW ANGLE, JOINT DISCONTINUITIES: HIGH ANGLE FROM 11.4'-11.6', HIGHLY FRACTURED TO SLIGHTLY FRACTURED, OPEN TO NARROW, SLIGHTLY ROUGH, BLOCKY/DISTURBED/SEAMY, GOOD TO POOR SURFACE CONDITION; RQD 34%, REC 95%. @13.7'-14.1'; Qu = 5900 PSI @ 1.4% STRAIN (DRY UNIT WEIGHT= 145.1 PCF)			11																
			12																
			13																
			14																
			15																
			16	34		95	NQ2-1											CORE	
			17																
			18																
			19																
		1016.6	20																
			EOB																
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. BORING OFFSET 20.0' SOUTHWEST DUE TO UTILITIES.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 1 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11 65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55TB		STATION / OFFSET: 502+23, 2' LT.		EXPLORATION ID											
TYPE: RETAINING WALL		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: RETAINING WALL 5		B-044-0-24											
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA / NQ2		CALIBRATION DATE: 7/30/24		ELEVATION: 1038.5 (MSL) EOB: 15.1 ft.		PAGE											
START: 9/25/24 END: 9/25/24		SAMPLING METHOD: SPT / NQ2		ENERGY RATIO (%): 89		LAT / LONG: 41.073034, -81.504437		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		1038.5							GR	CS	FS	SI	CL	LL	PL	PI	WC		
8.0" ASPHALT AND 10.0" BASE (DRILLERS DESCRIPTION)		1037.0																	
VERY DENSE, BROWN, COARSE AND FINE SAND, SOME STONE FRAGMENTS, LITTLE SILT, TRACE CLAY, RESIDUAL ROCK, DAMP		1035.5	TR	32 50/2"	-	63	SS-1	-	24	18	36	16	6	NP	NP	NP	5	A-3a (0)	
INTERBEDDED SANDSTONE (82%) AND SHALE (18%), CONTAINS A 0.5" CLAY SEAM AT 11.9', BEDDING DISCONTINUITIES: LOW ANGLE, JOINT DISCONTINUITIES: HIGH ANGLE FROM 9.1'-9.7', HIGHLY FRACTURED TO SLIGHTLY FRACTURED, OPEN TO NARROW, SLIGHTLY ROUGH, BLOCKY, FAIR TO POOR SURFACE CONDITION, RQD 78%, REC. 97%;				50/1"	-	100	SS-2	-	-	-	-	-	-	-	-	-	7	Rock (V)	
SANDSTONE, BROWN AND GRAY AND ORANGISH BROWN, SLIGHTLY TO MODERATELY WEATHERED, SLIGHTLY TO MODERATELY STRONG, FINE TO MEDIUM GRAINED, LAMINATED TO MEDIUM BEDDED;																			
SHALE, DARK GRAY, MODERATELY WEATHERED, WEAK, FISSILE.				78		97	NQ2-1											CORE	
@7.8'-8.2'; Qu = 6400 PSI @ 1.4% STRAIN (DRY UNIT WEIGHT= 143.4 PCF)																			
		1023.4	EOB																
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2.0 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																			

[illegible]

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. BORING OFFSET 25.0' NORTHWEST.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 2.0 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

[illegible]

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 3.0 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X11ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11 65-0-00 (114865)\GINT FILES\SUM-77-SR8-11 65-0-00 (114865)

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55TB		STATION / OFFSET: 603+45, 2' LT.		EXPLORATION ID												
TYPE: RETAINING WALL		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: RETAINING WALL 6		B-047-0-24												
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA / NQ2		CALIBRATION DATE: 7/30/24		ELEVATION: 1048.4 (MSL) EOB: 25.1 ft.		PAGE												
START: 9/5/24 END: 9/5/24		SAMPLING METHOD: SPT / NQ2		ENERGY RATIO (%): 89		LAT / LONG: 41.073232, -81.504892		1 OF 1												
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL	
		1048.4							GR	CS	FS	SI	CL	LL	PL	PI	WC			
5.0" TOPSOIL (DRILLERS DESCRIPTION)		1048.0																		
MEDIUM DENSE, BROWN AND ORANGISH BROWN BECOMING BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT, TRACE TO LITTLE CLAY, DAMP			1																	
			2																	
			3	3	12	72	SS-1	-	15	15	37	20	13	24	15	9	12	A-2-4 (O)		
			4																	
			5	4																
			6	6	16	39	SS-2	-	-	-	-	-	-	-	-	-	13	A-2-4 (V)		
			7																	
			8	4																
VERY DENSE, BROWN, STONE FRAGMENTS WITH SAND, TRACE SILT, TRACE CLAY, RESIDUAL ROCK, DAMP			9	7	30	89	SS-3	-	-	-	-	-	-	-	-	12	A-2-4 (V)			
			10																	
			11	5	32	102	50	SS-4	-	-	-	-	-	-	-	-	8	A-1-b (V)		
			12	3	37															
SANDSTONE, BROWN AND GRAY AND ORANGISH BROWN, SLIGHTLY TO MODERATELY WEATHERED, STRONG, FINE TO MEDIUM GRAINED, VERY THIN TO THICK BEDDED, FERRIFEROUS, CONTAINS SHALE LAMINATIONS AND INCLUSIONS, CONTAINS CONCRETIONS, BEDDING DISCONTINUITIES: LOW ANGLE, JOINT DISCONTINUITIES: HIGH ANGLE FROM 20.5'-21.0', HIGHLY FRACTURED TO SLIGHTLY FRACTURED, NARROW, SLIGHTLY ROUGH, BLOCKY, GOOD TO FAIR SURFACE CONDITION; RQD 71%, REC 98%.		1038.9																		
			13	3	50/4"	-	90	SS-5	-	-	-	-	-	-	-	-	11	A-1-b (V)		
		1035.4	TR																	
			14																	
			15	50/1"	-	100	SS-6	-	-	-	-	-	-	-	-	-	8	Rock (V)		
			16																	
			17																	
			18																	
			19																	
			20	71		98	NQ2-1												CORE	
			21																	
			22																	
			23																	
			24																	
			25																	
		1023.3	EOB																	
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.																				
ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 2.0 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																				

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55TB		STATION / OFFSET: 700+77, 10' RT.		EXPLORATION ID											
TYPE: RETAINING WALL		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: RETAINING WALL 7		B-048-0-24											
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA / NQ2		CALIBRATION DATE: 7/30/24		ELEVATION: 1037.8 (MSL) EOB: 16.2 ft.		PAGE											
START: 9/26/24 END: 9/26/24		SAMPLING METHOD: SPT / NQ2		ENERGY RATIO (%): 89		LAT / LONG: 41.073139, -81.504590		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		1037.8							GR	CS	FS	SI	CL	LL	PL	PI	WC		
5.0" ASPHALT (DRILLERS DESCRIPTION)		1037.4																	
VERY DENSE, BROWN, COARSE AND FINE SAND, SOME SILT, TRACE CLAY, TRACE GRAVEL, DAMP			1																
			2																
			3	20															
			4	25	73	61	SS-1	-	1	15	54	21	9	NP	NP	NP	7	A-3a (0)	
		1033.3		24															
VERY DENSE, BROWN, SANDY SILT, LITTLE CLAY, TRACE GRAVEL, RESIDUAL ROCK, DAMP			5																
			6	35	-	86	SS-2	-	6	13	44	23	14	NP	NP	NP	9	A-4a (0)	
		1031.6		20															
			7	50/2"															
SANDSTONE, BROWN AND GRAY AND ORANGISH BROWN, SLIGHTLY TO MODERATELY WEATHERED, SLIGHTLY TO MODERATELY STRONG, FINE TO MEDIUM GRAINED, VERY THIN TO MEDIUM BEDDED, CONTAINS SHALE SEAMS, LAMINATIONS, AND INCLUSIONS, VUGGY, BEDDING DISCONTINUITIES: LOW ANGLE, JOINT DISCONTINUITIES: HIGH ANGLE FROM 10.1'-11.1', HIGHLY FRACTURED TO SLIGHTLY FRACTURED, OPEN TO NARROW, SLIGHTLY ROUGH, BLOCKY/DISTURBED/SEAMY, GOOD TO FAIR SURFACE CONDITION; RQD 72%, REC 97%. @11.6'-12.0'; Qu = 6916 PSI @ 1.1% STRAIN (7.361 Gpa Elastic Modulus) (DRY UNIT WEIGHT= 144.1 PCF)			8																
			9																
			10																
			11																
			12	72		97	NQ2-1												
			13																
			14																
			15																
		1021.6	16																
			EOB																
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1.0 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55TB		STATION / OFFSET: 702+13, 1' RT.		EXPLORATION ID													
TYPE: RETAINING WALL		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: RETAINING WALL 7		B-049-0-24													
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA / NQ2		CALIBRATION DATE: 7/30/24		ELEVATION: 1040.4 (MSL) EOB: 15.1 ft.		PAGE													
START: 9/26/24 END: 6/26/24		SAMPLING METHOD: SPT / NQ2		ENERGY RATIO (%): 89		LAT / LONG: 41.073513, -81.504566		1 OF 1													
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL		
		1040.4							GR	CS	FS	SI	CL	LL	PL	PI	WC				
8.0" CONCRETE AND 12.0" BASE (DRILLERS DESCRIPTION)		1038.7	1																		
VERY DENSE, LIGHT BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, DAMP		1037.4	2																		
SANDSTONE, ORANGISH BROWN AND BROWN AND GRAY, MODERATELY TO HIGHLY WEATHERED, SLIGHTLY TO MODERATELY STRONG, FINE TO MEDIUM GRAINED, VERY THIN TO MEDIUM BEDDED, CONTAINS A SHALE SEAM FROM 9.4'-9.9', BEDDING DISCONTINUITIES: LOW ANGLE, JOINT DISCONTINUITIES: HIGH ANGLE FROM 5.2'-6.3' AND 14.3'-15.1', HIGHLY FRACTURED TO SLIGHTLY FRACTURED, OPEN TO NARROW, SLIGHTLY ROUGH, BLOCKY/DISTURBED/SEAMY, FAIR TO POOR SURFACE CONDITION; RQD 48%, REC 96%. @8.6'-9.0'; Qu = 6540 PSI @ 1.0% STRAIN (DRY UNIT WEIGHT= 133.0 PCF)			3	13	-	22	SS-1	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)			
			4																		
			5	50/2"	-	100	SS-2	-	-	-	-	-	-	-	-	-	-	8	Rock (V)		
			6																		
			7																		
			8																		
			9																		
			10	48		96	NQ2-1												CORE		
			11																		
			12																		
			13																		
			14																		
			15	1025.3	EOB																
		NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.																			
		ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2.0 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X11ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55TB		STATION / OFFSET: 800+19, 21' RT.		EXPLORATION ID												
TYPE: RETAINING WALL		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: RETAINING WALL 8		B-050-0-24												
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA / NQ2		CALIBRATION DATE: 7/30/24		ELEVATION: 1038.1 (MSL) EOB: 18.2 ft.		PAGE												
START: 9/4/24 END: 9/4/24		SAMPLING METHOD: SPT / NQ2		ENERGY RATIO (%): 89		LAT / LONG: 41.073021, -81.504721		1 OF 1												
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		1038.1								GR	CS	FS	SI	CL	LL	PL	PI	WC		
8.0" CONCRETE AND 11.0" BASE (DRILLERS DESCRIPTION)		1036.5	1																	
HARD, BROWN, SANDY SILT, LITTLE CLAY, LITTLE STONE FRAGMENTS, STONE FRAGMENTS ARE FRIABLE SANDSTONE, RESIDUAL ROCK, CONTAINS TRACE IRON STAINING, CONTAINS NO INTACT SOIL FOR HP READINGS, DAMP		1033.6	2		18	71	83	SS-1	-	13	18	20	34	15	25	17	8	8	A-4a (3)	
VERY DENSE, BROWN, STONE FRAGMENTS WITH SAND, TRACE SILT, TRACE CLAY, STONE FRAGMENTS ARE FRIABLE SANDSTONE, RESIDUAL ROCK, DAMP		1031.1	3		22															
VERY DENSE, BROWN, STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, STONE FRAGMENTS ARE FRIABLE SANDSTONE, RESIDUAL ROCK, DAMP		1030.1	4		26															
VERY DENSE, BROWN, STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, STONE FRAGMENTS ARE FRIABLE SANDSTONE, RESIDUAL ROCK, DAMP		1030.1	5		8	62	78	SS-2	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
VERY DENSE, BROWN, STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, STONE FRAGMENTS ARE FRIABLE SANDSTONE, RESIDUAL ROCK, DAMP		1030.1	6		12															
VERY DENSE, BROWN, STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, STONE FRAGMENTS ARE FRIABLE SANDSTONE, RESIDUAL ROCK, DAMP		1030.1	7		30															
VERY DENSE, BROWN, STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, STONE FRAGMENTS ARE FRIABLE SANDSTONE, RESIDUAL ROCK, DAMP		1030.1	8		24	-	63	SS-3	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)	
SANDSTONE, BROWN AND GRAY AND ORANGISH BROWN, SLIGHTLY TO MODERATELY WEATHERED, STRONG, FINE TO MEDIUM GRAINED, THIN TO MEDIUM BEDDED, FERRIFEROUS, CONTAINS SHALE LAMINATIONS AND INCLUSIONS, BEDDING DISCONTINUITIES: LOW ANGLE, JOINT DISCONTINUITIES: HIGH ANGLE FROM 8.4-8.7', 9.0-9.6', AND 10.5-11.0', FRACTURED TO SLIGHTLY FRACTURED, NARROW, SLIGHTLY ROUGH, VERY BLOCKY, GOOD TO FAIR SURFACE CONDITION; RQD 67%, REC 97%.		1019.9	9		50/2"															
		1019.9	10		67		97	NQ2-1												
		1019.9	11																	
		1019.9	12																	
		1019.9	13																	
		1019.9	14																	
		1019.9	15																	
		1019.9	16																	
		1019.9	17																	
		1019.9	18																	
		1019.9	EOB																	
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. BORING OFFSET 18.0' EAST.																				
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2.0 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																				

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55TB		STATION / OFFSET: 801+39, 10' RT.		EXPLORATION ID											
TYPE: RETAINING WALL		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: RETAINING WALL 8		B-051-0-24											
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA / NQ2		CALIBRATION DATE: 7/30/24		ELEVATION: 1038.7 (MSL) EOB: 18.0 ft.		PAGE											
START: 8/28/24 END: 8/28/24		SAMPLING METHOD: SPT / NQ2		ENERGY RATIO (%): 89		LAT / LONG: 41.073349, -81.504716		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		1038.7							GR	CS	FS	SI	CL	LL	PL	PI	WC		
8.0" CONCRETE (DRILLERS DESCRIPTION)		1038.0																	
VERY DENSE, GRAY, STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, DAMP			1																
			2																
			3	19 50/2"	-	63	SS-1	-	45	11	16	18	10	NP	NP	NP	6	A-2-4 (0)	
			4																
		1033.7	5	33 50/2"	-	88	SS-2	-	-	-	-	-	-	-	-	-	8	Rock (V)	
SANDSTONE, BROWN AND GRAY AND ORANGISH BROWN, SLIGHTLY TO MODERATELY WEATHERED, SLIGHTLY TO MODERATELY STRONG, FINE TO MEDIUM GRAINED, LAMINATED TO MEDIUM BEDDED, CONTAINS SHALE LAMINATIONS AND INCLUSIONS, VUGGY, BEDDING DISCONTINUITIES: LOW ANGLE, FRACTURED TO SLIGHTLY FRACTURED, NARROW, SLIGHTLY ROUGH, BLOCKY, GOOD TO FAIR SURFACE CONDITION; RQD 46%, REC 96%.			6																
			7																
			8	50/3"	-	100	SS-3	-	-	-	-	-	-	-	-	-	7	Rock (V)	
			9																
			10																
			11																
@11.3'-11.7'; Qu = 4620 PSI @ 1.1% STRAIN (DRY UNIT WEIGHT= 139.6 PCF)			12																
			13	46		96	NQ2-1											CORE	
			14																
			15																
			16																
		1020.7	17																
			18																
			EOB																
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																			

[illegible]

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

[illegible]

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 1 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55TB		STATION / OFFSET: 1001+29, 31' RT.				EXPLORATION ID												
TYPE: RETAINING WALL		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: RETAINING WALL 9				B-054-0-24												
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA / NQ2		CALIBRATION DATE: 7/30/24		ELEVATION: 1056.2 (MSL) EOB: 30.1 ft.				PAGE												
START: 8/29/24 END: 8/29/24		SAMPLING METHOD: SPT / NQ2		ENERGY RATIO (%): 89		LAT / LONG: 41.074829, -81.504710				1 OF 1												
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS		SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL		
8.0" CONCRETE (DRILLERS DESCRIPTION)		1056.2								GR	CS	FS	SI	CL	LL	PL	PI					
MEDIUM DENSE, BROWN, SANDY SILT, LITTLE CLAY, TRACE GRAVEL, TRACE IRON STAINING, DAMP		1055.5																				
			1																			
			2																			
			3		6	4	19	61	SS-1	-	-	-	-	-	-	-	-	-	14	A-4a (V)		
			4			9																
VERY DENSE, BROWN, SANDY SILT, LITTLE STONE FRAGMENTS, LITTLE CLAY, DAMP			5																			
			6		2	5	24	72	SS-2	-	8	9	31	38	14	NP	NP	NP	11	A-4a (3)		
			7																			
			8		4	7	59	72	SS-3	-	18	6	36	26	14	NP	NP	NP	11	A-4a (1)		
			9			33																
SANDSTONE BROWN AND GRAY AND ORANGISH BROWN, MODERATELY TO SLIGHTLY WEATHERED, SLIGHTLY TO MODERATELY STRONG, VERY FINE TO MEDIUM GRAINED, LAMINATED TO MEDIUM BEDDED, CONTAINS SHALE LAMINATIONS AND INCLUSIONS, CONTAINS CONCRETIONS, VUGGY, CONTAINS 1/8" CLAY SEAM AT 24.6', BEDDING DISCONTINUITIES: LOW ANGLE, HIGHLY FRACTURED TO SLIGHTLY FRACTURED, OPEN TO NARROW, SLIGHTLY ROUGH, BLOCKY, GOOD TO POOR SURFACE CONDITION; RQD 59%, REC 97%. @16.4'-16.8'; Qu = 4874 PSI @ 1.4% STRAIN (DRY UNIT WEIGHT= 138.0 PCF)		1049.2																				
			10		27	50/5"	-	109	SS-4	-	-	-	-	-	-	-	-	-	12	Rock (V)		
			11																			
			12																			
			13		50/4"		-	100	SS-5	-	-	-	-	-	-	-	-	-	7	Rock (V)		
			14																			
			15		50/1"		-	100	SS-6	-	-	-	-	-	-	-	-	-	7	Rock (V)		
			16																			
			17																			
			18																			
			19																			
			20		46			95	NQ2-1											CORE		
			21																			
			22																			
			23																			
			24																			
			25																			
			26																			
			27																			
			28		85			100	NQ2-2												CORE	
			29																			
			30																			
		1026.1																				

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (11 X 17) - OH DOT GDT - 5/31/26 11:27 - X:\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-0.00.GPJ

[illegible]

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2.0 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-5

PID: 114865	SFN:	PROJECT: SUM-IR-77/SR8-11.65/0.00	STATION / OFFSET: 1101+91, 51' LT.			START: 9/4/24		END: 9/4/24		PG 2 OF 2		B-056-0-24										
MATERIAL DESCRIPTION AND NOTES			ELEV. 1030.6	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL	
SANDSTONE , LIGHT BROWN AND MAROONISH BROWN, UNWEATHERED TO SLIGHTLY WEATHERED, SLIGHTLY TO MODERATELY STRONG, FINE TO MEDIUM GRAINED, VERY THIN TO VERY THICK BEDDED, CONTAINS CONCRETIONS, BEDDING DISCONTINUITIES: LOW ANGLE, HIGHLY FRACTURED TO INTACT, NARROW, SLIGHTLY ROUGH, INTACT, GOOD SURFACE CONDITION; RQD 72%, REC 98%. <i>(continued)</i>					69	100	NQ2-2													CORE		
				31																		
				32																		
				33																		
				34																		
			1025.5	EOB	35																	
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED. ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2.0 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																						

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/31/26 11:21 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77-SR8-11.65-0.00 (114865)\GINT FILES\SUM-77-SR8-11.65-

PROJECT: SUM-IR-77/SR8-11.65/0.00		DRILLING FIRM / OPERATOR: NEAS / JL		DRILL RIG: CME 55TB		STATION / OFFSET: 1103+45, 12' RT.		EXPLORATION ID											
TYPE: RETAINING WALL		SAMPLING FIRM / LOGGER: NEAS / JL		HAMMER: CME AUTOMATIC		ALIGNMENT: RETAINING WALL 10		B-057-0-24											
PID: 114865 SFN:		DRILLING METHOD: 3.25" HSA / NQ2		CALIBRATION DATE: 7/30/24		ELEVATION: 1039.3 (MSL) EOB: 20.1 ft.		PAGE											
START: 9/23/24 END: 9/23/24		SAMPLING METHOD: SPT / NQ2		ENERGY RATIO (%): 89		LAT / LONG: 41.076345, -81.504404		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		1039.3							GR	CS	FS	SI	CL	LL	PL	PI	WC		
7.0" ASPHALT AND 11.0" BASE (DRILLERS DESCRIPTION)		1037.8	1																
VERY DENSE, BROWN, STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, DAMP			2																
			3	16 37 36	108	50	SS-1	-	26	19	31	17	7	NP	NP	NP	7	A-2-4 (0)	
		1034.8	4																
SANDSTONE, BROWN AND LIGHT BROWN, UNWEATHERED TO SLIGHTLY WEATHERED, SLIGHTLY TO MODERATELY STRONG, FINE TO MEDIUM GRAINED, THIN TO MEDIUM BEDDED, CONTAINS SIDERITE INCLUSIONS, BEDDING DISCONTINUITIES: LOW ANGLE, FRACTURED TO SLIGHTLY FRACTURED, NARROW TO TIGHT, SLIGHTLY ROUGH, INTACT TO BLOCKY, GOOD SURFACE CONDITION; RQD 90%, REC 97%. @6.7'-7.1'; Qu = 5541 PSI @ 1.1% STRAIN (DRY UNIT WEIGHT= 133.5 PCF)			5	50/1"	-	100	SS-2	-	-	-	-	-	-	-	-	-	2	Rock (V)	
			6																
			7																
			8																
			9																
			10	85		96	NQ2-1											CORE	
			11																
			12																
			13																
			14																
			15																
			16																
			17																
			18	100		100	NQ2-2											CORE	
			19																
		1019.2	20																
			EOB																
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2.0 BAGS HOLE PLUG; PLACED SOIL CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 8/6/26 12:48 - X:\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\SUM-77 SR-8 CCTV BORING\GINT FILES\SUM-77 SR-8 CCTV.GP.

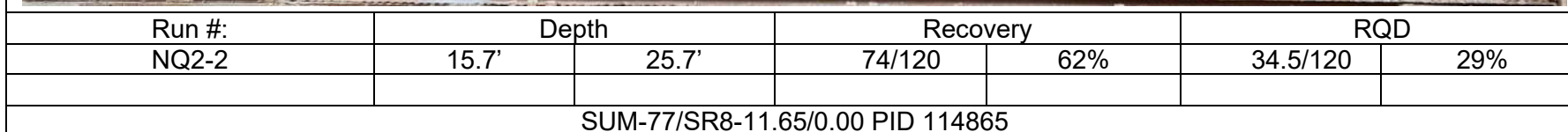
PROJECT: SUM-77 SR-8 CCTV TYPE: LIGHT TOWER PID: SFN: START: 5/27/26 END: 5/27/26		DRILLING FIRM / OPERATOR: NEAS INC. / J. HODGES SAMPLING FIRM / LOGGER: NEAS INC. / J. HODGES DRILLING METHOD: 3.25" HSA / NQ2 SAMPLING METHOD: SPT / NQ2		DRILL RIG: CME 55X HAMMER: CME AUTOMATIC CALIBRATION DATE: 3/25/26 ENERGY RATIO (%): 83		STATION / OFFSET: 354+07, 110' RT. ALIGNMENT: SR-8 ELEVATION: 1036.3 (MSL) EOB: 20.5 ft. LAT / LONG: 41.073072, -81.503844		EXPLORATION ID B-058-0-26 PAGE 1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV. 1036.3	DEPTH	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED
									GR	CS	FS	SI	CL	LL	PL	PI	WC		
DENSE, BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND, LITTLE SILT, TRACE CLAY, DAMP			1																
			2																
			3	13															
			4	17	42	100	SS-1	-	38	19	21	16	6	NP	NP	NP	8	A-1-b (0)	
		1031.8		13															
MEDIUM DENSE, BROWN, GRAVEL, SOME SAND, TRACE SILT, TRACE CLAY, MOIST			5	4															
			6	6	18	100	SS-2	-	68	11	10	8	3	NP	NP	NP	6	A-1-a (0)	
		1029.3		7															
VERY DENSE, BROWNISH GRAY, COARSE AND FINE SAND, LITTLE STONE FRAGMENTS, LITTLE SILT, TRACE CLAY, RESIDUAL SOIL, DAMP			7																
		1027.9		8															
SANDSTONE, GRAY, HIGHLY WEATHERED, WEAK TO SLIGHTLY STRONG.			8	50/5"	-	82	SS-3	-	-	-	-	-	-	-	-	-	7	A-3a (V)	
		1025.8	TR	9															
INTERBEDDED SANDSTONE (62%) AND SHALE (38%), CONTAINS LAMINATIONS AND INCLUSIONS OF SHALE, BEDDING DISCONTINUITIES: LOW ANGLE, HIGHLY FRACTURED TO MODERATELY FRACTURED, OPEN TO NARROW, SLIGHTLY ROUGH TO VERY ROUGH, BLOCKY/DISTURBED/SEAMY, GOOD TO FAIR SURFACE CONDITION, RQD 26%, REC. 96%;			10	50	-	83	SS-4	-	-	-	-	-	-	-	-	-	4	Rock (V)	
SANDSTONE, LIGHT GRAY, SLIGHTLY TO MODERATELY WEATHERED, SLIGHTLY STRONG, FINE TO MEDIUM GRAINED, LAMINATED TO VERY THIN BEDDED;			11																
SHALE, DARK GRAY, MODERATELY WEATHERED, VERY WEAK TO WEAK.			12																
			13																
			14																
			15																
			16	26		96	NQ2-1											CORE	
			17																
			18																
			19																
		1015.8	EOB	20															
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. BORING OFFSET 17.0' WEST (3.6' EAST OF PAVEMENT, IN FRONT OF WRONG WAY ROAD SIGN). ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 50.0 GAL. BENTONITE GROUT; POURED 2.0 BAGS HOLE PLUG																			

B-001-0-22

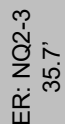


Run #:	Depth		Recovery		RQD	
NQ2-1	10.7'	15.7'	41.5/ 60	69%	21.75/60	36%
SUM-77/SR8-11.65/0.00 PID 114865						

B-001-0-22

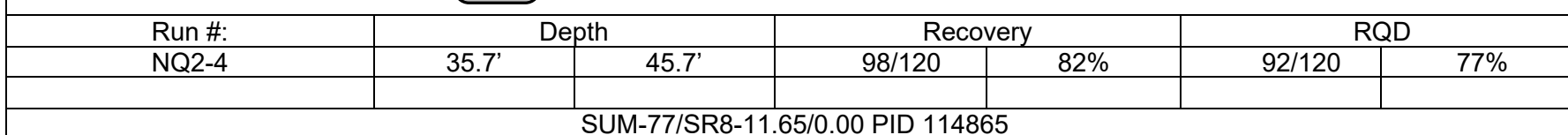


Run #:	Depth		Recovery		RQD	
NQ2-2	15.7'	25.7'	74/120	62%	34.5/120	29%
SUM-77/SR8-11.65/0.00 PID 114865						

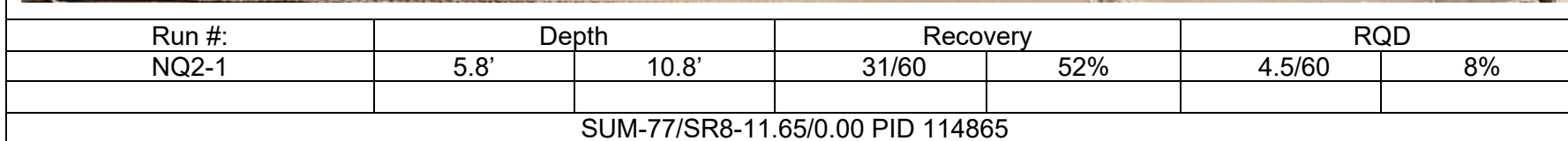
BR: NQ2-3
25.7'

SUM-77/SR8-11.65/0.00 PID 114865

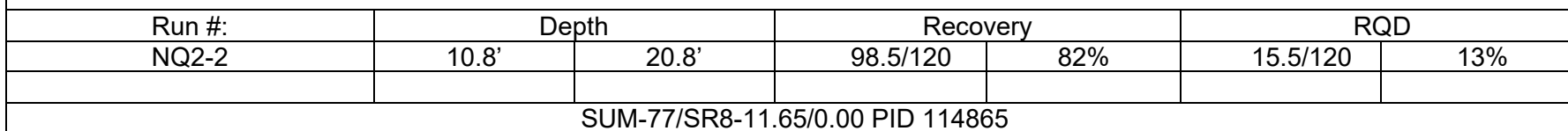
B-001-0-22



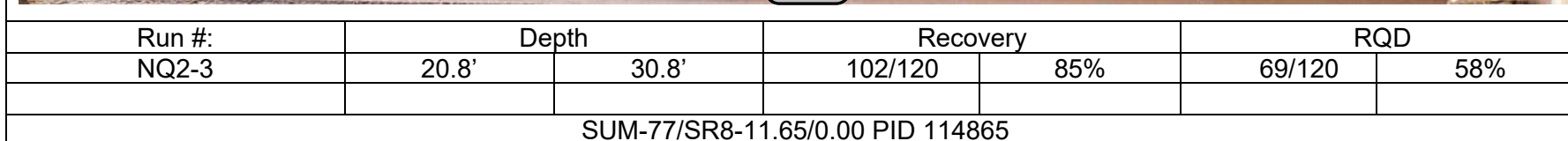
B-002-0-22



B-002-0-22



B-002-0-22

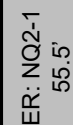


B-002-0-22



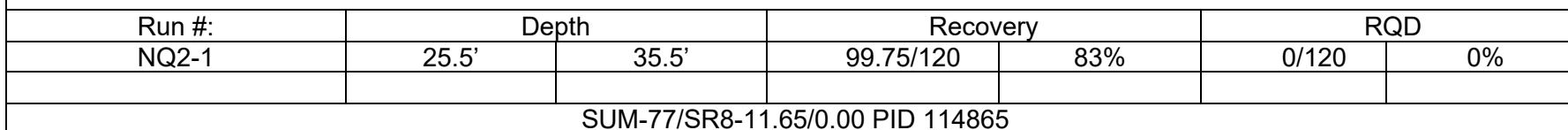
Run #:	Depth		Recovery		RQD	
NQ2-4	30.8'	40.8'	117.5/120	98%	115/120	96%
SUM-77/SR8-11.65/0.00 PID 114865						

B-003-0-22

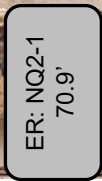


Run #:	Depth		Recovery		RQD	
NQ2-1	45.5'	55.5'	105.5/120	88%	74/120	62%
SUM-77/SR8-11.65/0.00 PID 114865						

B-004-0-22



B-005-0-22



Run #:	Depth		Recovery		RQD	
NQ2-1	60.9'	70.9'	109/120	91%	42/120	35%
SUM-77/SR8-11.65/0.00 PID 114865						

BR: NQ2-1
15.5'

ER: NQ2-1
25.5'

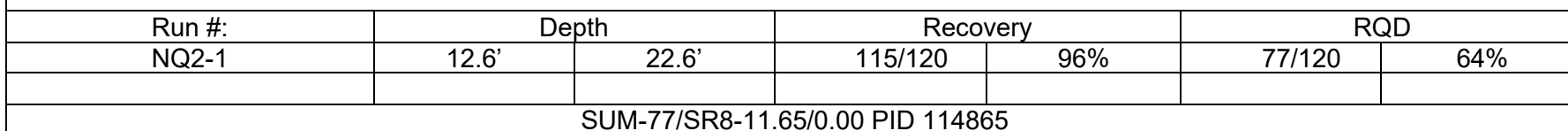
Run #:	Depth		Recovery		RQD	
NQ2-1	15.5'	25.5'	116/120	97%	51/120	43%
SUM-77/SR8-11.65/0.00 PID 114865						

BR: NQ2-1
25.5'

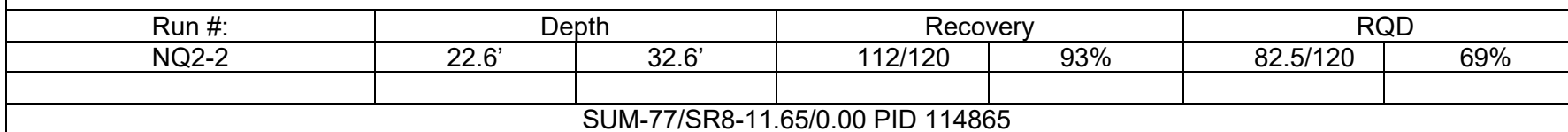
ER: NQ2-1
35.5'

Run #:	Depth		Recovery		RQD	
NQ2-1	25.5'	35.5'	120/120	100%	94/120	78%
SUM-77/SR8-11.65/0.00 PID 114865						

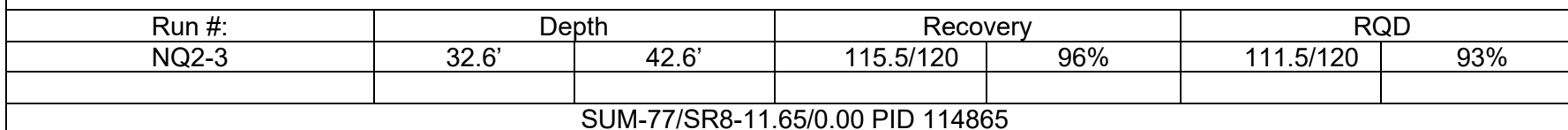
B-023-0-24



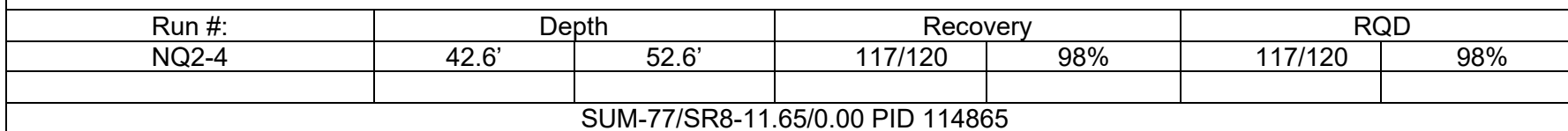
B-023-0-24



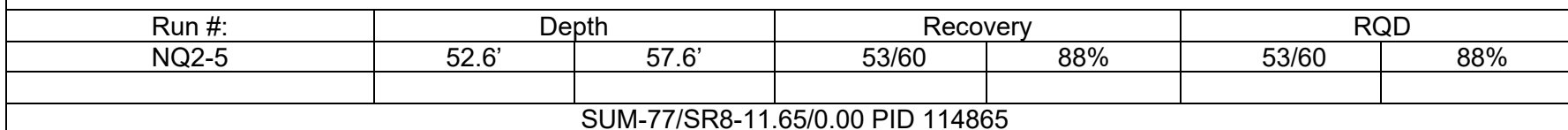
B-023-0-24



B-023-0-24



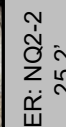
B-023-0-24



B-024-0-24

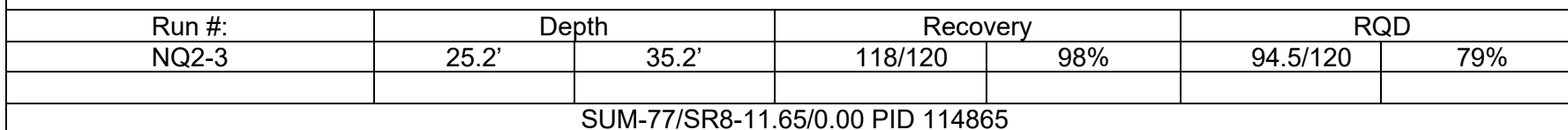


Run #:	Depth		Recovery		RQD	
NQ2-1	5.2'	15.2'	92.5/120	77%	54/120	45%
SUM-77/SR8-11.65/0.00 PID 114865						

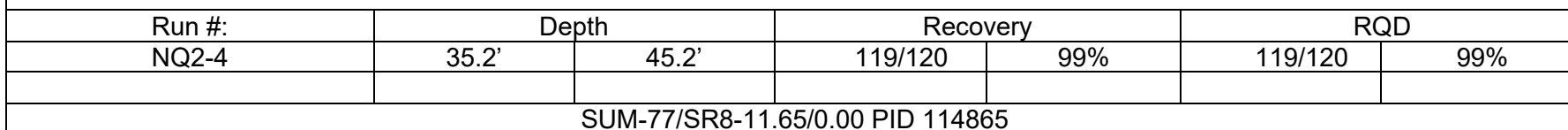
BR: NQ2-2
15.2'

Run #:	Depth		Recovery		RQD	
NQ2-2	15.2'	25.2'	119/120	99%	72.5/120	60%
SUM-77/SR8-11.65/0.00 PID 114865						

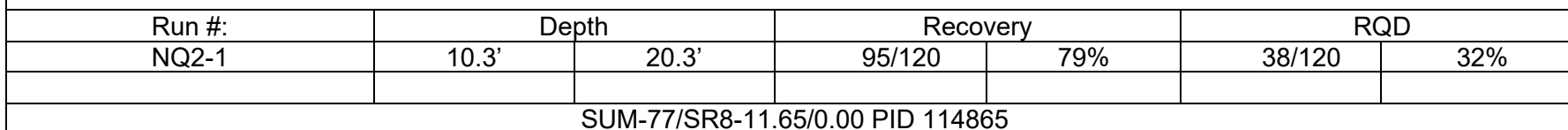
B-024-0-24



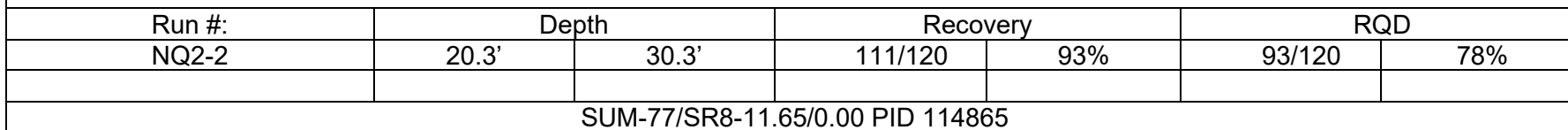
B-024-0-24



B-025-0-24



B-025-0-24

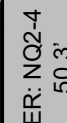


SUM-77/SR8-11.65/0.00 PID 114865

B-025-0-24

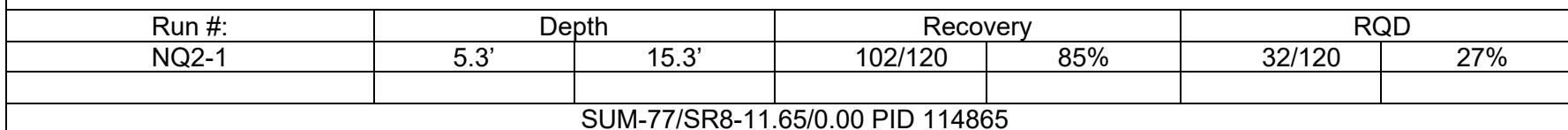


Run #:	Depth		Recovery		RQD	
NQ2-3	30.3'	40.3'	112.5/120	94%	99/120	83%
SUM-77/SR8-11.65/0.00 PID 114865						

BR: NQ2-4
40.3'

Run #:	Depth		Recovery		RQD	
NQ2-4	40.3'	50.3'	114/120	95%	110/120	92%
SUM-77/SR8-11.65/0.00 PID 114865						

B-026-0-22

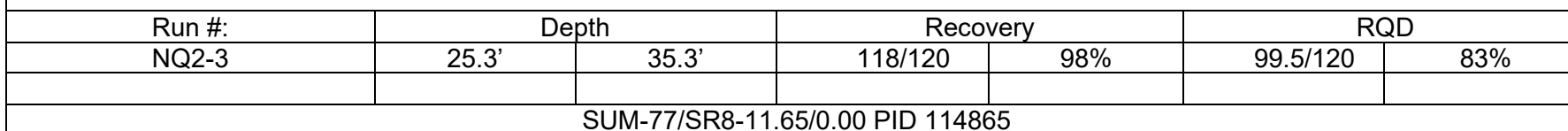


B-026-0-22

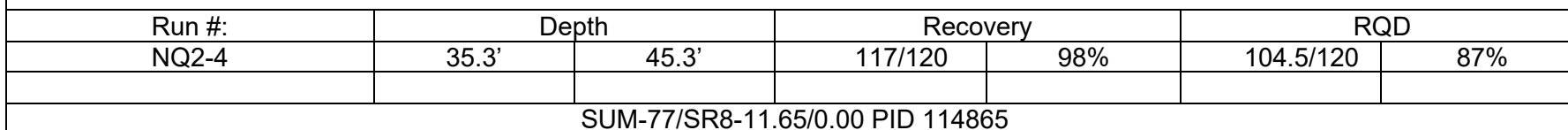


Run #:	Depth		Recovery		RQD	
NQ2-2	15.3'	25.3'	117/120	98%	65.25/120	54%
SUM-77/SR8-11.65/0.00 PID 114865						

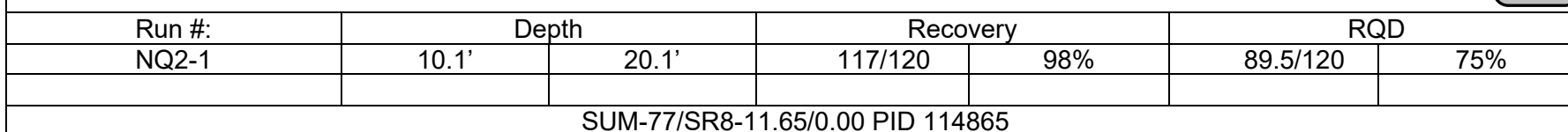
B-026-0-22



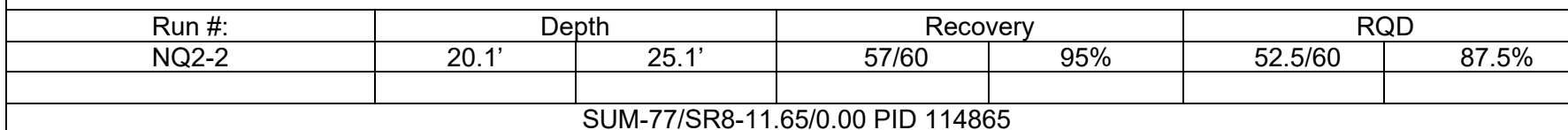
B-026-0-22



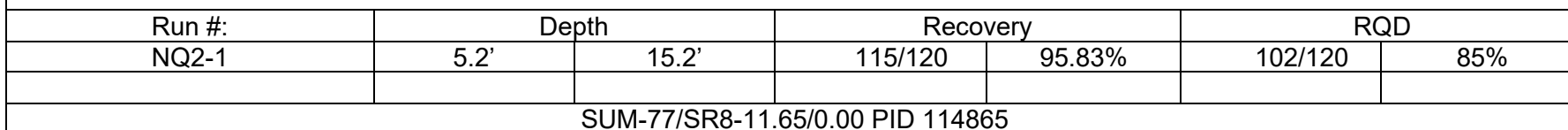
B-027-0-24



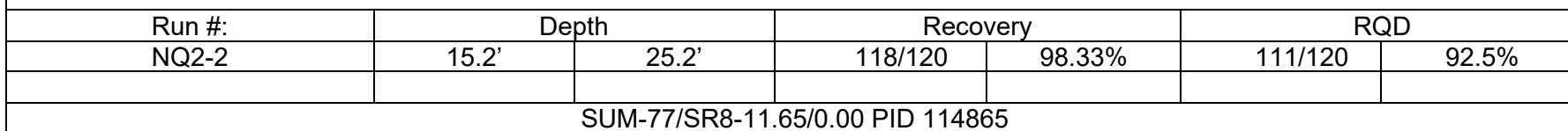
B-027-0-24



B-028-0-24



B-028-0-24

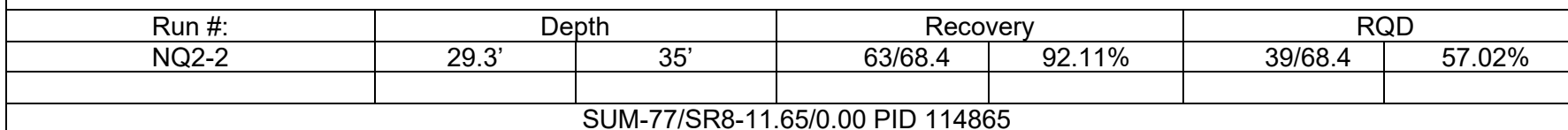


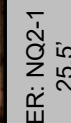
B-028-0-24



Run #:	Depth		Recovery		RQD	
NQ2-3	25.2'	27.7'	30/30	100%	30/30	100%
SUM-77/SR8-11.65/0.00 PID 114865						

B-029-0-24



BR: NQ2-1
15.5'

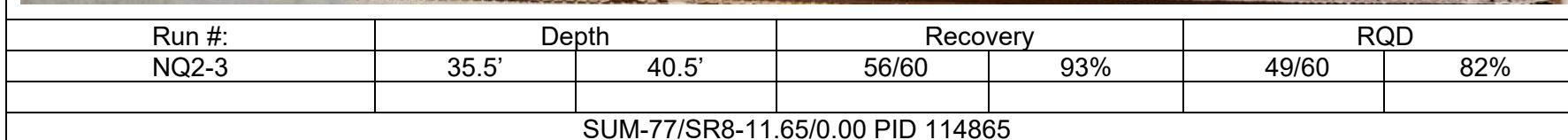
SUM-77/SR8-11.65/0.00 PID 114865

BR: NQ2-2
15.5'

ER: NQ2-2
25.5'

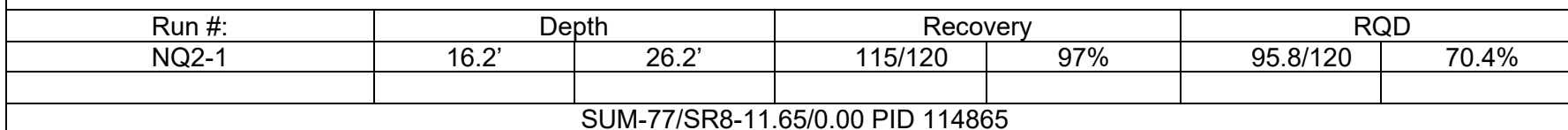
Run #:	Depth		Recovery		RQD	
NQ2-2	25.5'	35.5'	113.5/120	95%	54.5/120	45%
SUM-77/SR8-11.65/0.00 PID 114865						

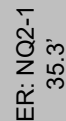
B-030-0-24



Run #:	Depth		Recovery		RQD	
NQ2-3	35.5'	40.5'	56/60	93%	49/60	82%
SUM-77/SR8-11.65/0.00 PID 114865						

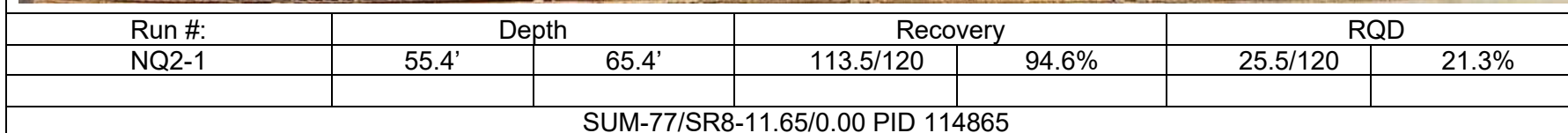
B-031-0-24



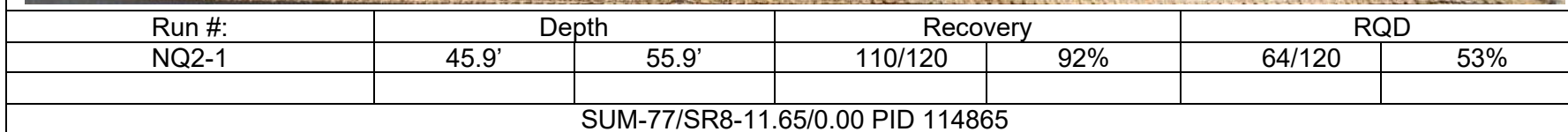
BR: NQ2-1
25.3'

Run #:	Depth		Recovery		RQD	
NQ2-1	25.3'	35.3'	120/120	100%	84/120	70%
SUM-77/SR8-11.65/0.00 PID 114865						

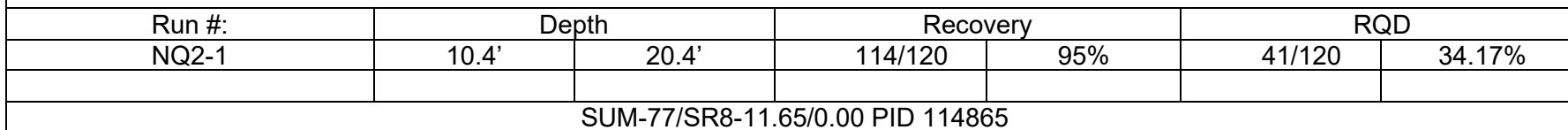
B-033-0-24



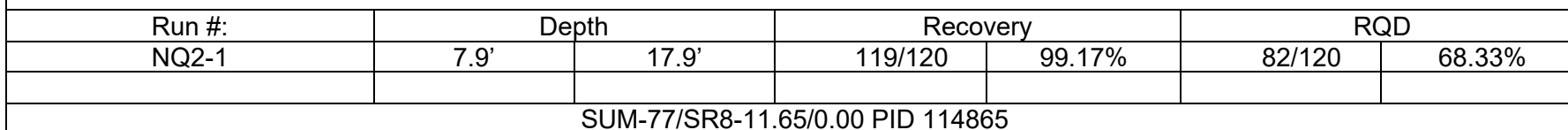
B-034-0-24



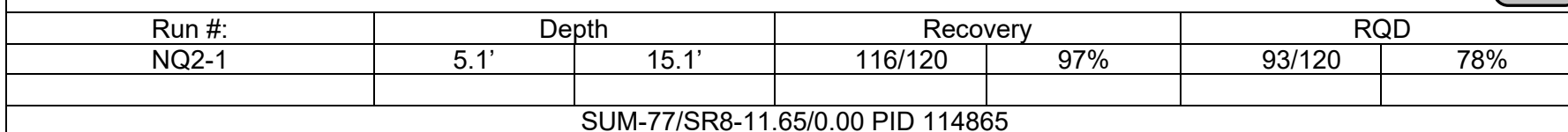
B-042-0-24



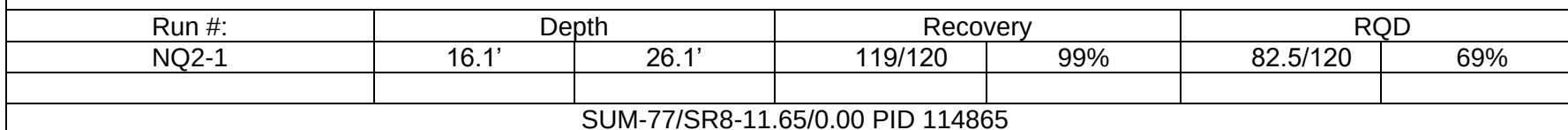
B-043-0-24



B-044-0-24



B-045-0-24

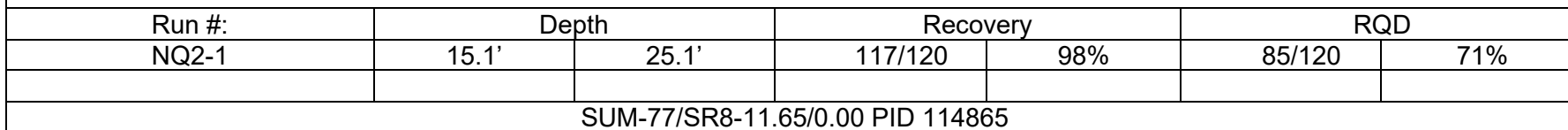


B-046-0-24

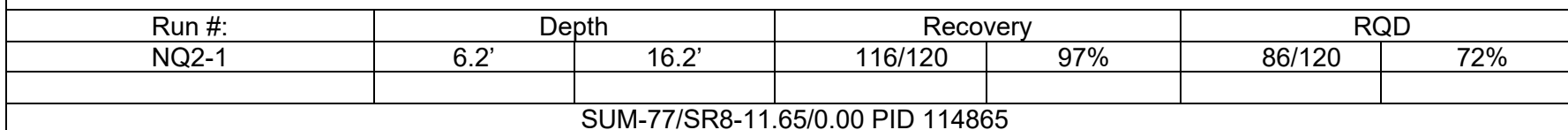


Run #:	Depth		Recovery		RQD	
NQ2-1	17.6'	27.6'	107.5/120	89.58%	66/120	55%
SUM-77/SR8-11.65/0.00 PID 114865						

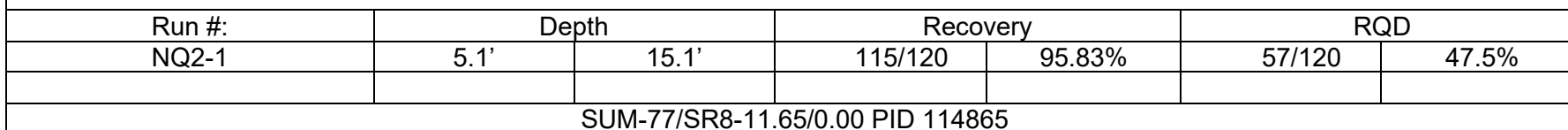
B-047-0-24



B-048-0-24



B-049-0-24



B-050-0-24



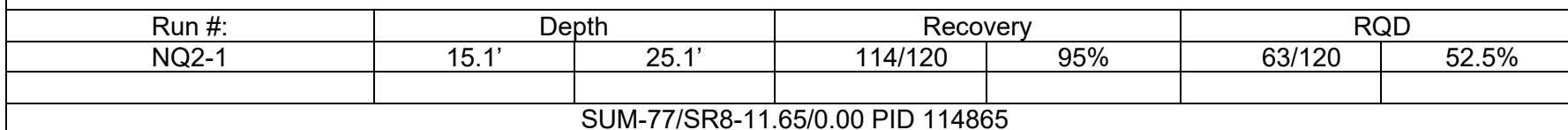
Run #:	Depth		Recovery		RQD	
NQ2-1	8.2'	18.2'	116.25/120	97%	80/120	67%
SUM-77/SR8-11.65/0.00 PID 114865						

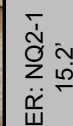
B-051-0-24



Run #:	Depth		Recovery		RQD	
NQ2-1	8'	18'	115.5/120	96.25%	55/120	45.83%
SUM-77/SR8-11.65/0.00 PID 114865						

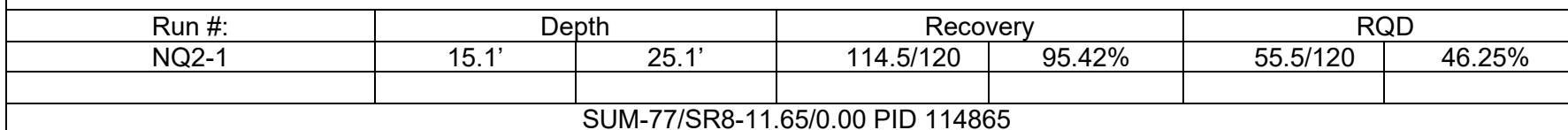
B-052-0-24



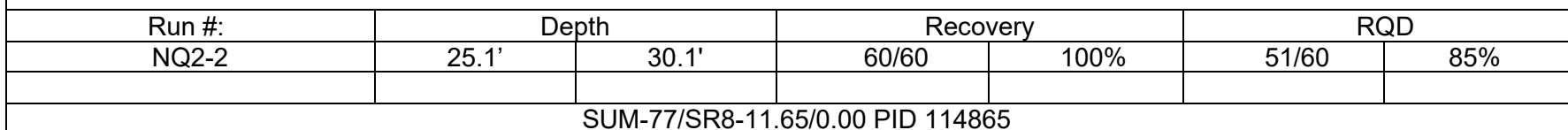
BR: NQ2-1
5.2'

Run #:	Depth		Recovery		RQD	
NQ2-1	5.2'	15.2'	118/120	98.33%	100/120	83.33%
SUM-77/SR8-11.65/0.00 PID 114865						

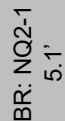
B-054-0-24



B-054-0-24



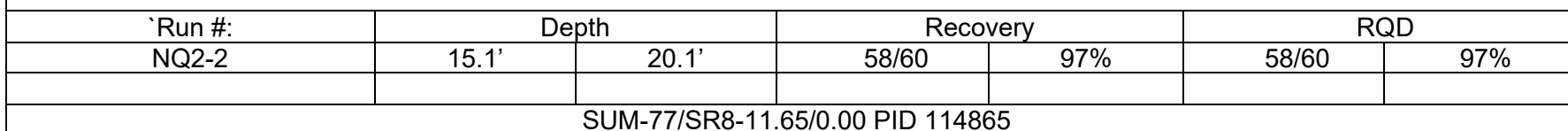
B-055-0-24



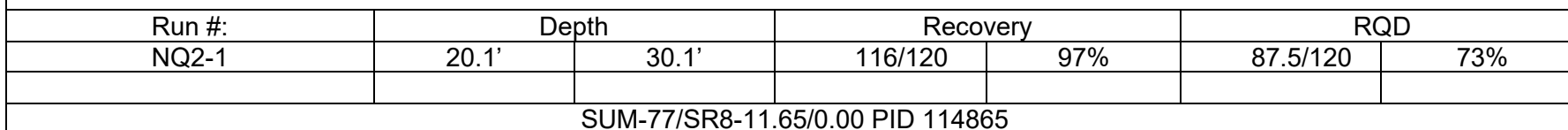
ER: NQ2-1
15.1'

Run #:	Depth		Recovery		RQD	
NQ2-1	5.1'	15.1'	118/120	98%	108/120	90%
SUM-77/SR8-11.65/0.00 PID 114865						

B-055-0-24



B-056-0-24

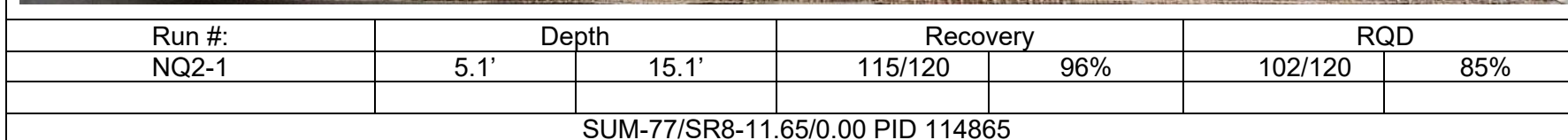


B-056-0-24



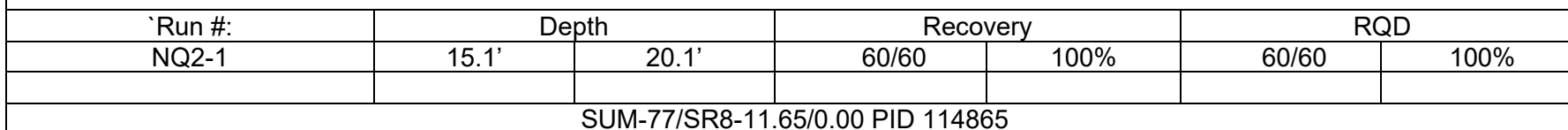
Run #:	Depth		Recovery		RQD	
NQ2-2	30.1'	35.1'	60/60	100%	41.5/60	69%
SUM-77/SR8-11.65/0.00 PID 114865						

B-057-0-24



Run #:	Depth		Recovery		RQD	
NQ2-1	5.1'	15.1'	115/120	96%	102/120	85%
SUM-77/SR8-11.65/0.00 PID 114865						

B-057-0-24



Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-022-0-24, NQ2-1, Depth: 32.7-33.1 ft)

Tested Date: 11/7/2024

Specimen Properties

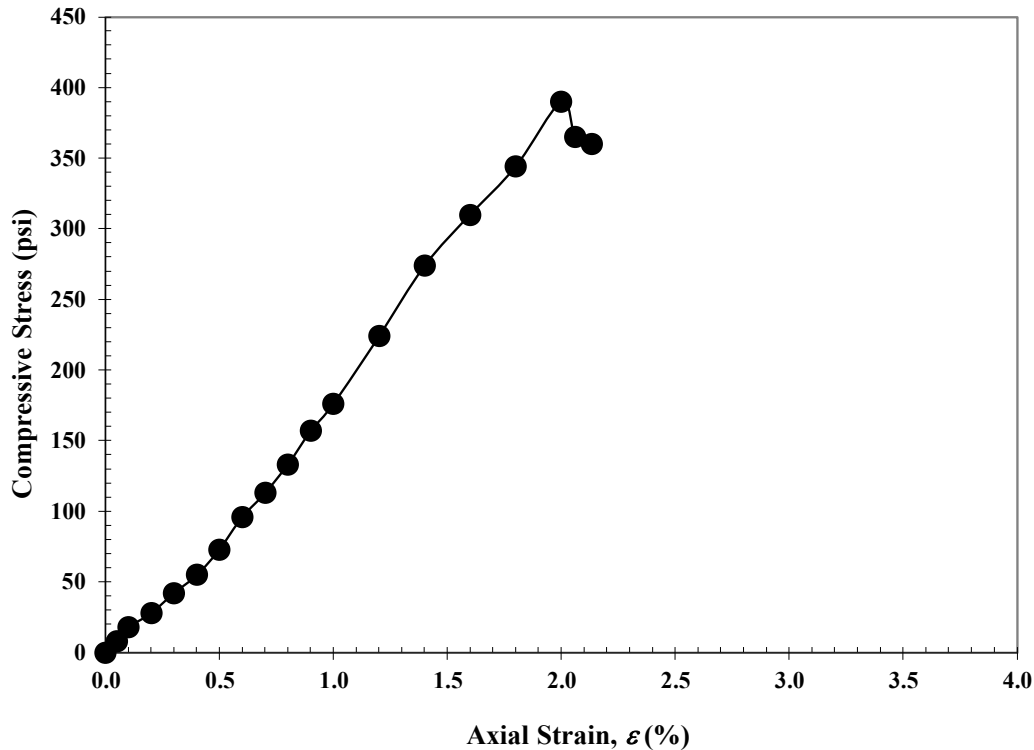
Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	4.12
Length to Diameter Ratio:	2.09
Area, A (in ²):	3.07
Volume, V (in ³):	12.65
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	5.5
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	154.7
Dry Unit Weight, γ_d (lb/ft ³):	146.6

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	390	3	(MPa)
Strain (%):	2.0		



Notes: Shale, dark gray, slightly weathered, very weak, fissile.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-023-0-24, NQ2-1, Depth: 14.9-15.3ft)

Tested Date: 10/22/2024

Specimen Properties

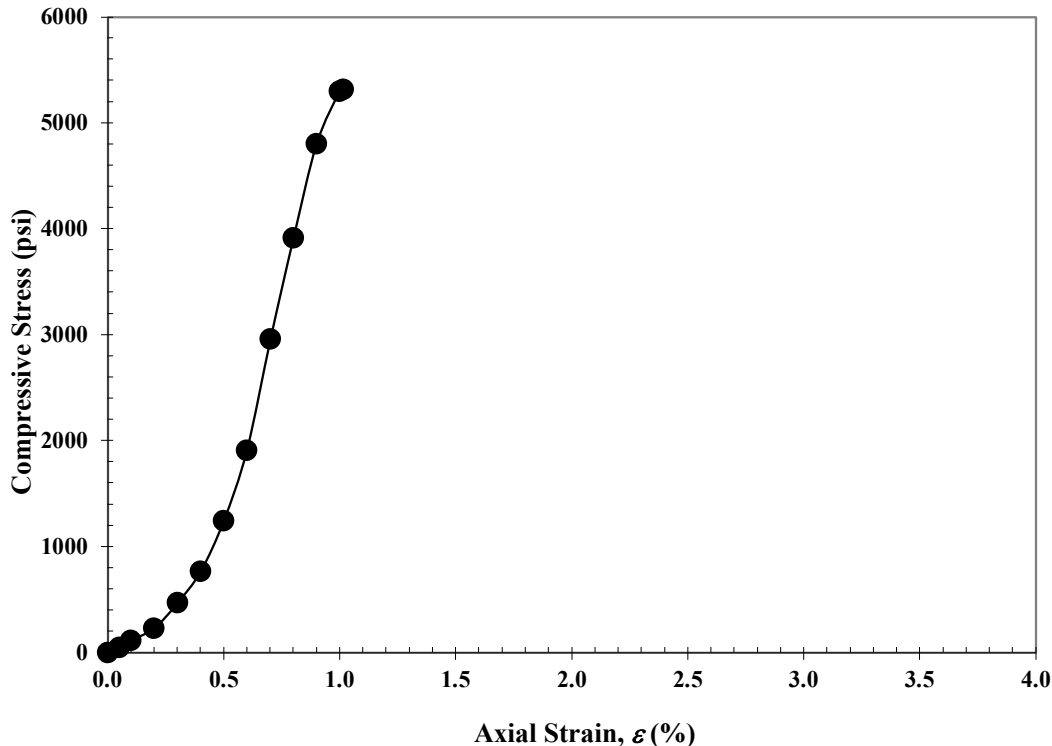
Average Dia., D_{avg} (in):	1.96
Average Height, H_{avg} (in):	4.23
Length to Diameter Ratio:	2.16
Area, A (in ²):	3.01
Volume, V (in ³):	12.76
Wet Mass of Specimen (lb):	1.0
Moisture Content (%):	2.9
Dry Mass of Specimen (lb):	1.0
Wet Unit Weight, γ (lb/ft ³):	140.1
Dry Unit Weight, γ_d (lb/ft ³):	136.2

Final Specimen Figure



Results

Unconfined Compressive Strength (psi): 5315 37 (MPa)
Strain (%): 1.0



Notes: Sandstone, brown and orangish brown, slightly weathered, moderately strong.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-024-0-24, NQ2-1, Depth: 7.4-7.8ft)

Tested Date: 10/22/24

Specimen Properties

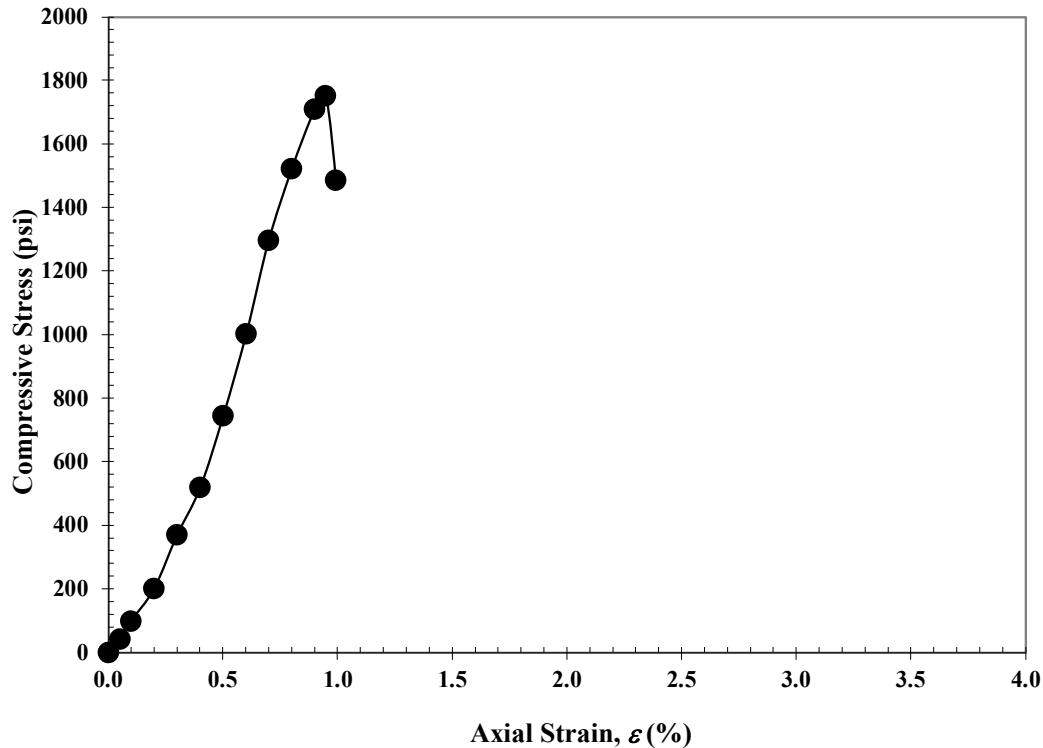
Average Dia., D_{avg} (in):	1.98
Average Height H_{avg} (in):	4.54
Length to Diameter Ratio:	2.29
Area, A (in ²):	3.07
Volume, V (in ³):	13.93
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	1.9
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	137.5
Dry Unit Weight, γ_d (lb/ft ³):	135.0

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	1752	12	(MPa)
Strain (%):	0.9		



Notes: Sandstone, brown and orangish brown, slightly weathered, slightly strong, contains shale inclusions, vuggy, slickensides noted after testing.

Sample trimming procedure does not conform to ASTM D4543 and the results reported may differ from the results obtained from a test specimen that meets the requirements of Practice D4543.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-025-0-24, NQ2-1, Depth: 15.6-16.0ft)

Tested Date: 10/23/2024

Specimen Properties

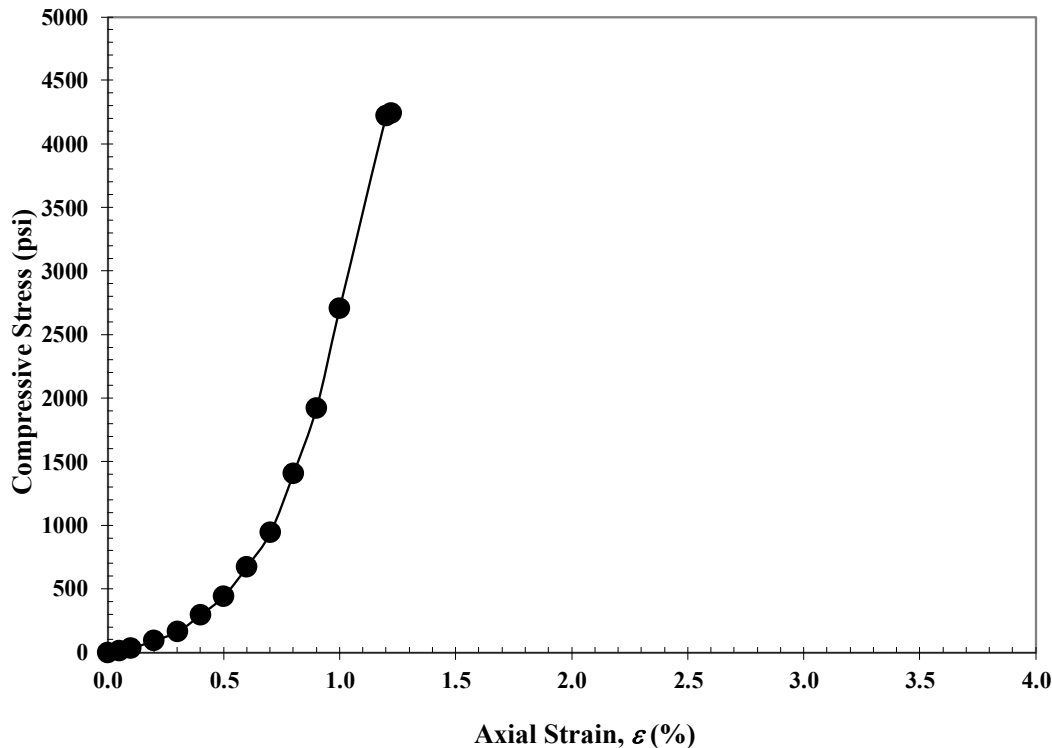
Average Dia., D_{avg} (in):	1.97
Average Height, H_{avg} (in):	4.50
Length to Diameter Ratio:	2.29
Area, A (in ²):	3.04
Volume, V (in ³):	13.69
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	2.0
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	140.8
Dry Unit Weight, γ_d (lb/ft ³):	138.0

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	4242	
Strain (%):	1.2	
		29 (MPa)



Notes: Sandstone, gray, unweathered, moderately strong.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-026-0-24, NQ2-1, Depth: 7.3 - 7.6ft)

Tested Date: 6/18/2024

Specimen Properties

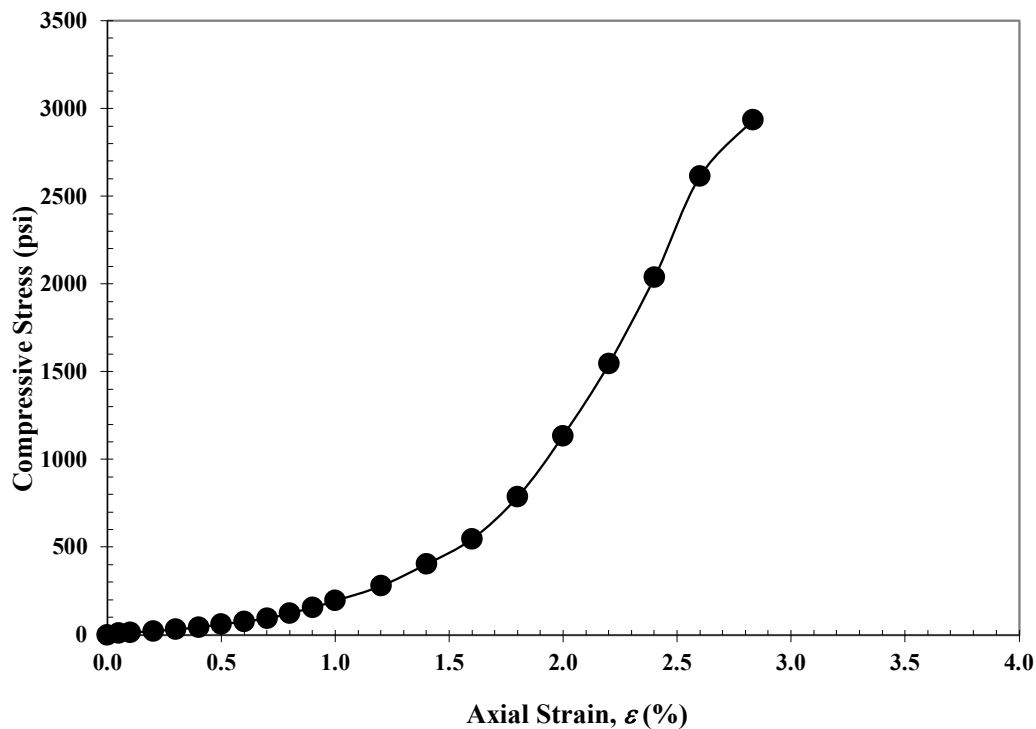
Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	3.78
Length to Diameter Ratio:	1.91
Area, A (in ²):	3.07
Volume, V (in ³):	11.59
Wet Mass of Specimen (lb):	1.0
Moisture Content (%):	2.3
Dry Mass of Specimen (lb):	1.0
Wet Unit Weight, γ (lb/ft ³):	151.2
Dry Unit Weight, γ_d (lb/ft ³):	147.8

Final Specimen Figure



Results

Unconfined Compressive Strength (psi): 2936 20 (MPa)
Strain (%): 2.8



Notes: SANDSTONE, light gray and black, moderately weathered, slightly strong, interbedded shale laminations. Please note that this specimen does not meet the L/D ratio of D7012 and the results reported may differ from a specimen that does meet the L/D requirements of D7012.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-026-0-24, NQ2-2, Depth: 15.8 - 16.1ft)

Tested Date: 6/18/2024

Specimen Properties

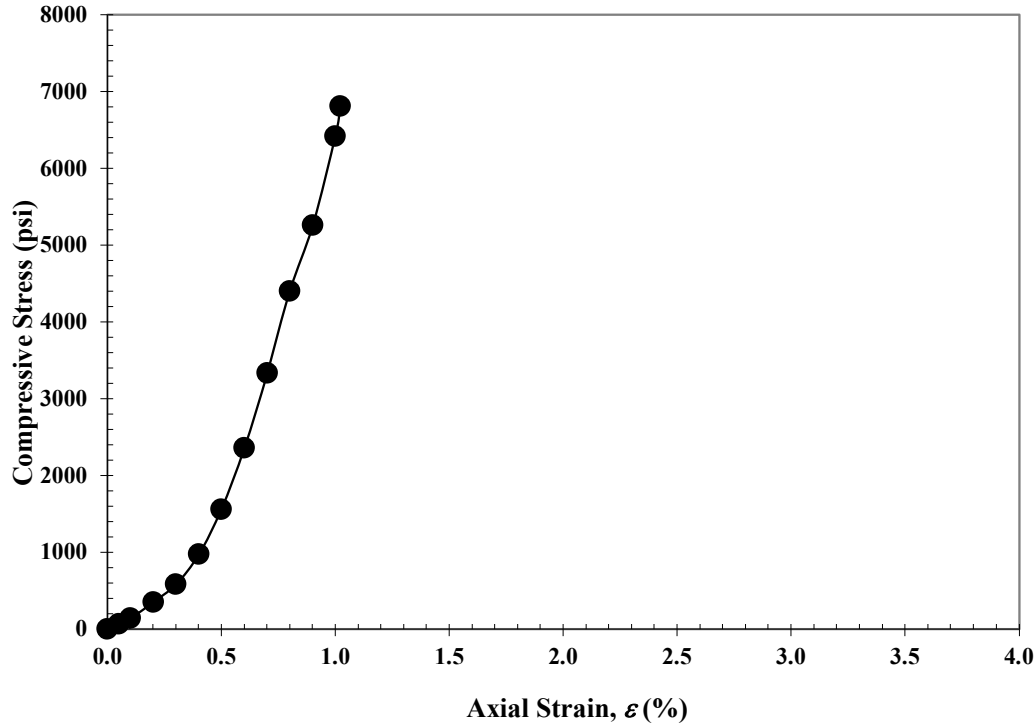
Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	3.82
Length to Diameter Ratio:	1.93
Area, A (in ²):	3.07
Volume, V (in ³):	11.72
Wet Mass of Specimen (lb):	0.9
Moisture Content (%):	0.9
Dry Mass of Specimen (lb):	0.9
Wet Unit Weight, γ (lb/ft ³):	139.9
Dry Unit Weight, γ_d (lb/ft ³):	138.7

Final Specimen Figure



Results

Unconfined Compressive Strength (psi): 6808 47 (MPa)
Strain (%): 1.0



Notes: SANDSTONE, brown, slightly weathered, moderately strong. Please note that this specimen does not meet the L/D ratio of D7012 and the results reported may differ from a specimen that does meet the L/D requirements of D7012.

Sample trimming procedure does not conform to ASTM D4543 and the results reported may differ from the results obtained from a test specimen that meets the requirements of Practice D4543.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-027-0-24, NQ2-1, Depth: 14.0-14.4ft)

Tested Date: 10/25/2024

Specimen Properties

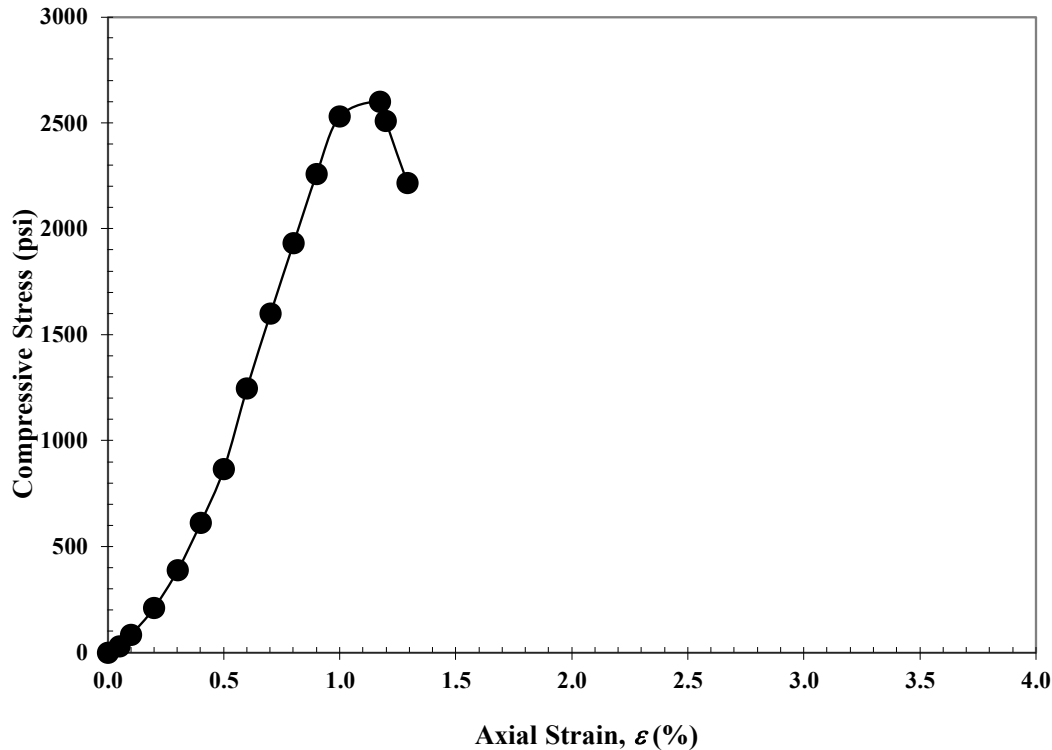
Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	4.26
Length to Diameter Ratio:	2.15
Area, A (in ²):	3.07
Volume, V (in ³):	13.08
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	1.5
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	141.7
Dry Unit Weight, γ_d (lb/ft ³):	139.7

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	2600	
Strain (%):	1.2	18 (MPa)



Notes: Sandstone, gray and brown, moderately weathered, slightly strong, fine to medium grained, contains shale laminations.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-028-0-24, NQ2-1, Depth: 6.3-6.7ft)

Tested Date: 10/25/2024

Specimen Properties

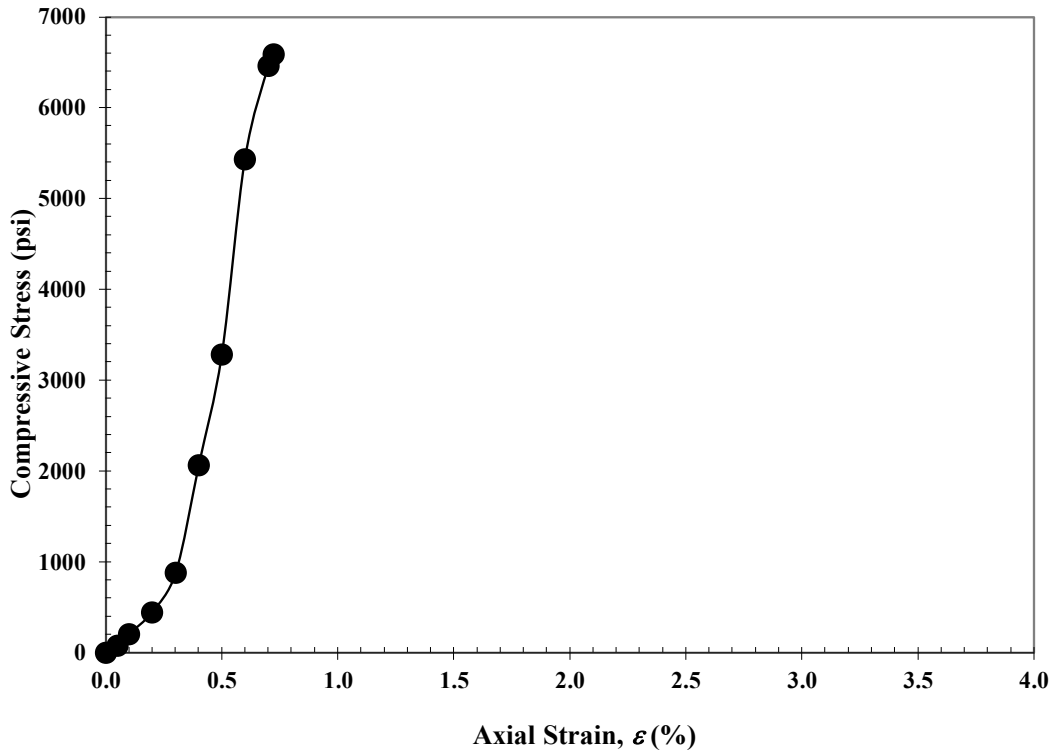
Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	4.43
Length to Diameter Ratio:	2.23
Area, A (in ²):	3.09
Volume, V (in ³):	13.69
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	1.7
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	135.8
Dry Unit Weight, γ_d (lb/ft ³):	133.5

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	6587	45	(MPa)
Strain (%):	0.7		



Notes: Sandstone, brown, slightly weathered, moderately strong, medium grained.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-029-0-24, NQ2-1, Depth: 19.4-19.8ft)

Tested Date: 10/25/2024

Specimen Properties

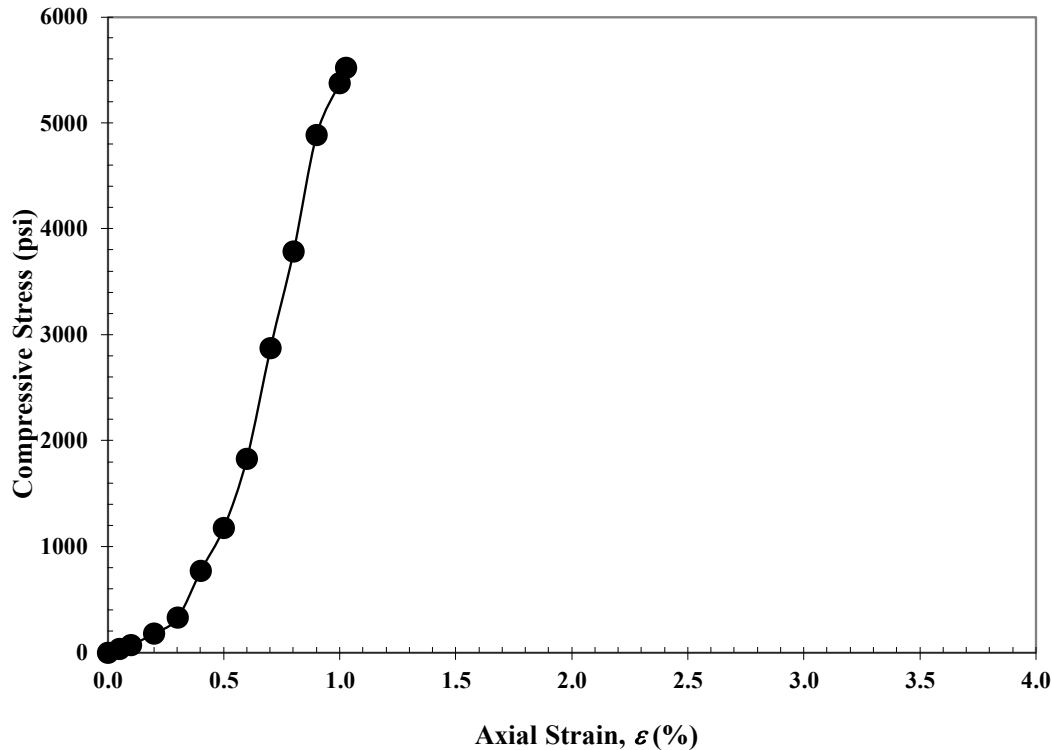
Average Dia., D_{avg} (in):	1.97
Average Height, H_{avg} (in):	4.38
Length to Diameter Ratio:	2.23
Area, A (in ²):	3.04
Volume, V (in ³):	13.32
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	0.9
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	138.8
Dry Unit Weight, γ_d (lb/ft ³):	137.5

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	5519	
Strain (%):	1.0	38 (MPa)



Notes: Sandstone, brown, slightly weathered, moderately strong, fine to medium grained.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-030-0-24, NQ2-1, Depth: 16.3-16.8 ft)

Tested Date: 11/7/2024

Specimen Properties

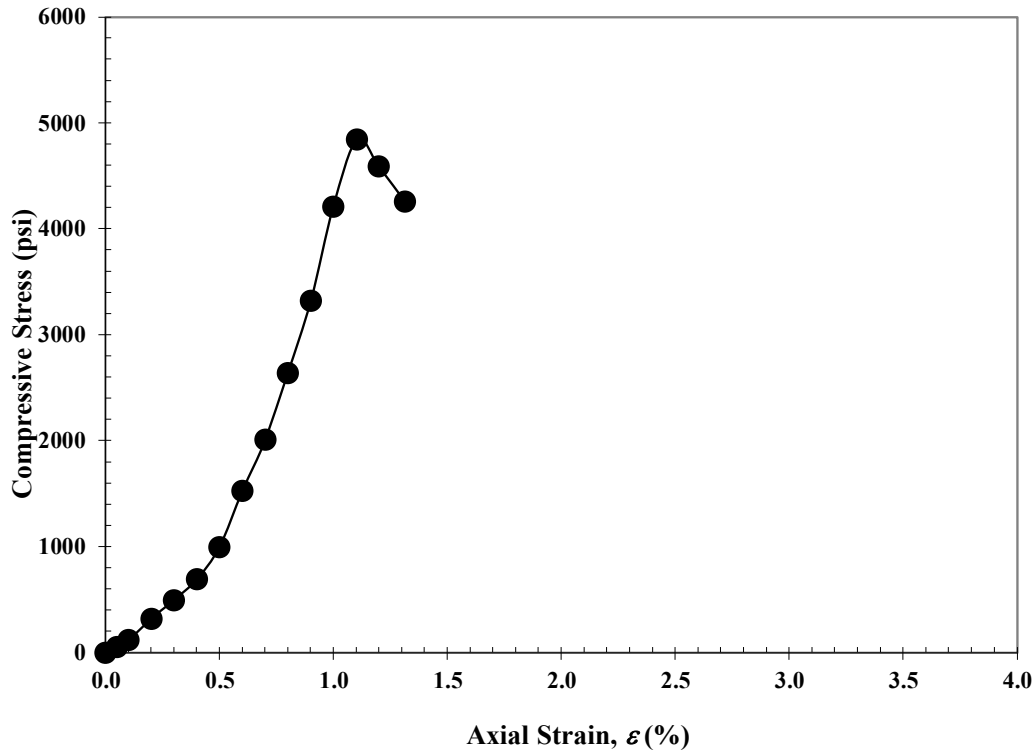
Average Dia., D_{avg} (in):	1.96
Average Height, H_{avg} (in):	4.26
Length to Diameter Ratio:	2.18
Area, A (in ²):	3.01
Volume, V (in ³):	12.83
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	2.9
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	152.7
Dry Unit Weight, γ_d (lb/ft ³):	148.4

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	4843	
Strain (%):	1.1	33 (MPa)



Notes: Siltstone, gray, slightly weathered, moderately strong, argillaceous.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-031-0-24, NQ2-1, Depth: 18.2-18.6 ft)

Tested Date: 11/7/2024

Specimen Properties

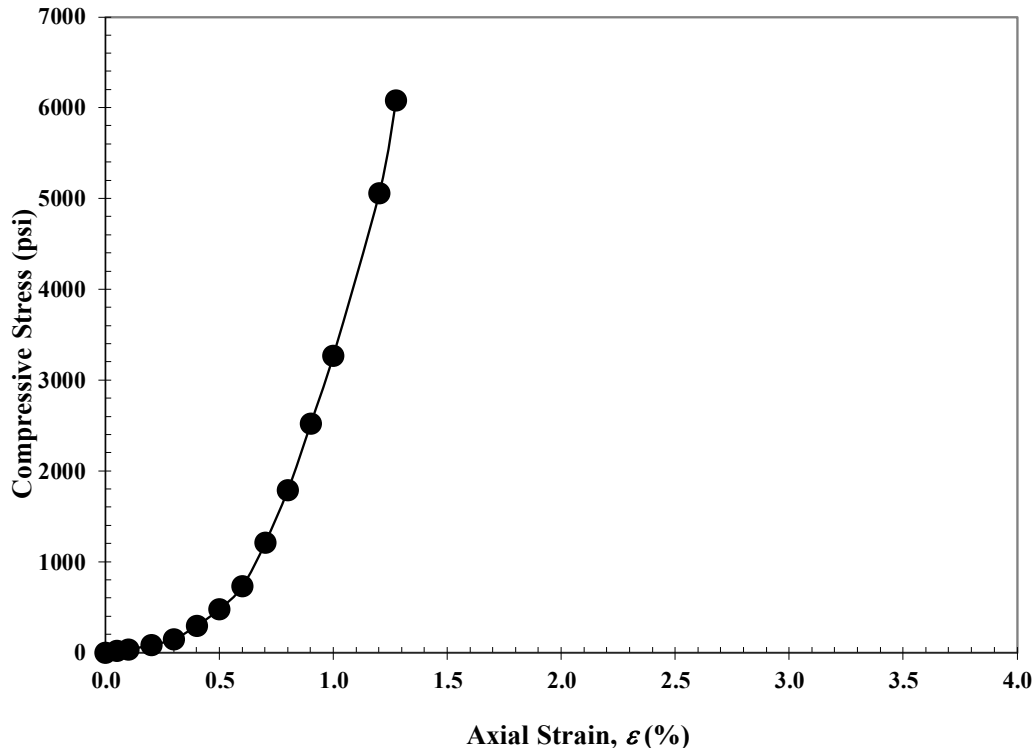
Average Dia., D_{avg} (in):	1.96
Average Height, H_{avg} (in):	4.40
Length to Diameter Ratio:	2.24
Area, A (in ²):	3.03
Volume, V (in ³):	13.32
Wet Mass of Specimen (lb):	1.2
Moisture Content (%):	2.3
Dry Mass of Specimen (lb):	1.2
Wet Unit Weight, γ (lb/ft ³):	159.8
Dry Unit Weight, γ_d (lb/ft ³):	156.2

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	6083	
Strain (%):	1.3	42 (MPa)



Notes: Shale, dark gray, slightly weathered, moderately strong, silty.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-042-0-24, NQ2-1, Depth: 13.7-14.1ft)

Tested Date: 10/25/2024

Specimen Properties

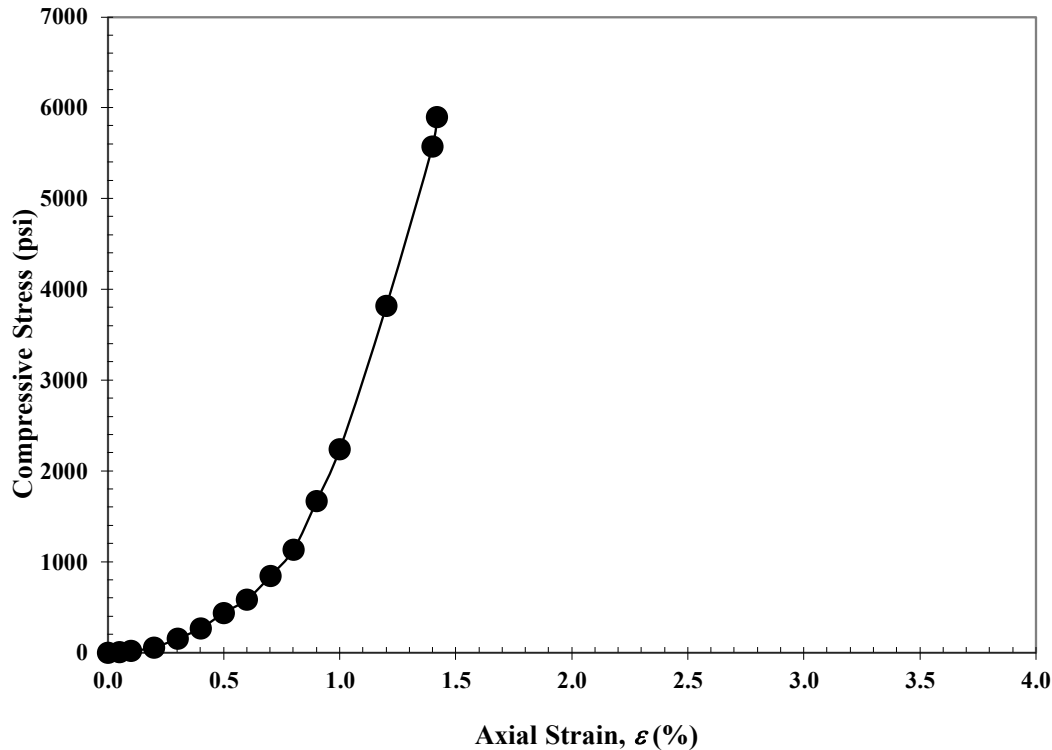
Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	4.51
Length to Diameter Ratio:	2.28
Area, A (in ²):	3.08
Volume, V (in ³):	13.88
Wet Mass of Specimen (lb):	1.2
Moisture Content (%):	0.7
Dry Mass of Specimen (lb):	1.2
Wet Unit Weight, γ (lb/ft ³):	146.1
Dry Unit Weight, γ_d (lb/ft ³):	145.1

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	5900	
Strain (%):	1.4	41 (MPa)



Notes: Sandstone, gray, slightly weathered, moderately strong, fine grained.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-043-0-24, NQ2-1, Depth: 8.9-9.3ft)

Tested Date: 10/25/2024

Specimen Properties

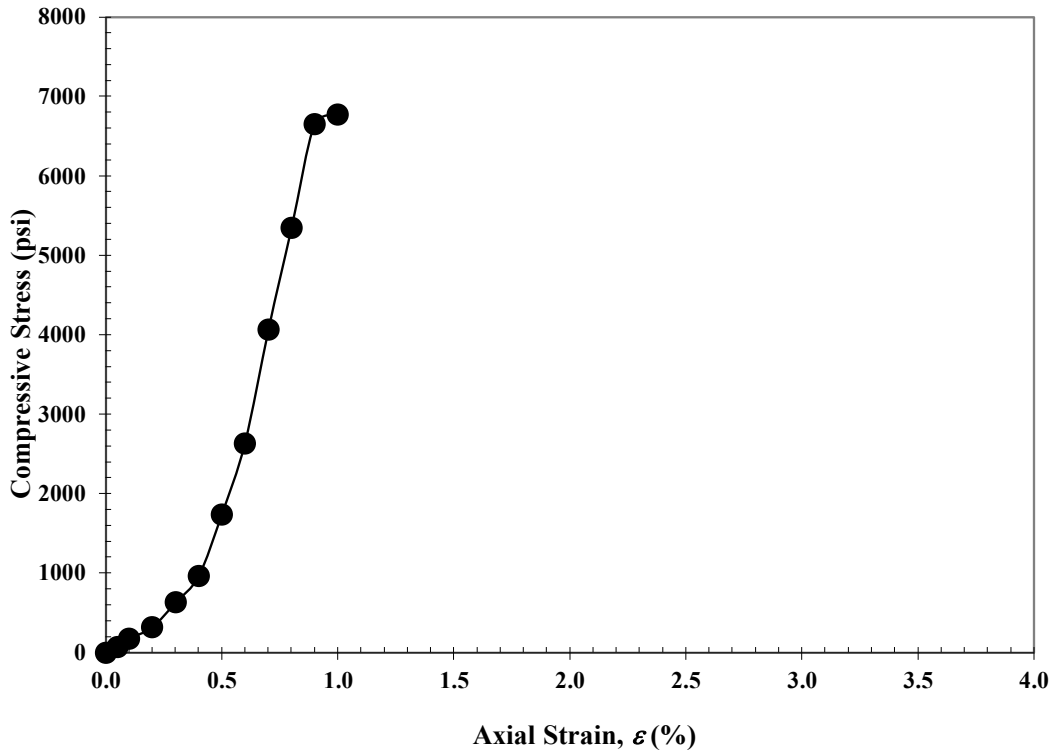
Average Dia., D_{avg} (in):	1.99
Average Height, H_{avg} (in):	4.53
Length to Diameter Ratio:	2.28
Area, A (in ²):	3.10
Volume, V (in ³):	14.03
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	1.4
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	133.2
Dry Unit Weight, γ_d (lb/ft ³):	131.4

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	6775	47	(MPa)
Strain (%):	1.0		



Notes: Sandstone, brown, slightly weathered, moderately strong, fine to medium grained.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-044-0-24, NQ2-1, Depth: 7.8-8.2 ft)

Tested Date: 11/7/2024

Specimen Properties

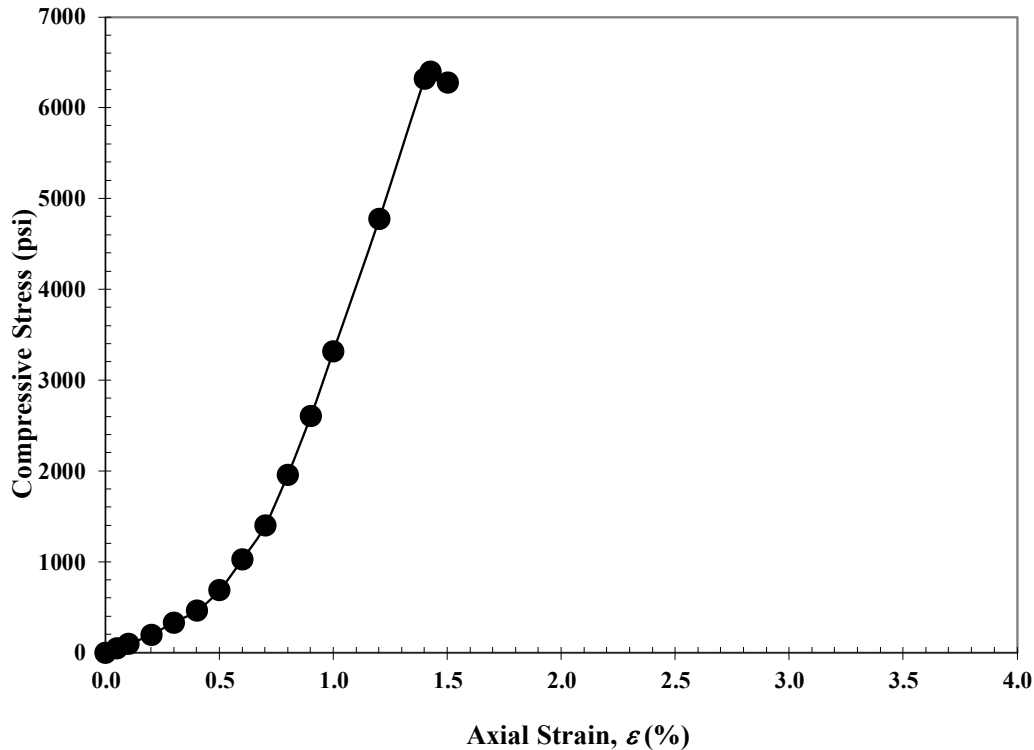
Average Dia., D_{avg} (in):	1.97
Average Height, H_{avg} (in):	3.86
Length to Diameter Ratio:	1.96
Area, A (in ²):	3.04
Volume, V (in ³):	11.74
Wet Mass of Specimen (lb):	1.0
Moisture Content (%):	1.0
Dry Mass of Specimen (lb):	1.0
Wet Unit Weight, γ (lb/ft ³):	144.9
Dry Unit Weight, γ_d (lb/ft ³):	143.4

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	6400		
Strain (%):	1.4		
		44	(MPa)



Notes: Sandstone, brown, slightly weathered, moderately strong, fine to medium grained.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-046-0-24, NQ2-1, Depth: 19.4-19.8ft)

Tested Date: 10/23/2024

Specimen Properties

Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	4.38
Length to Diameter Ratio:	2.21
Area, A (in ²):	3.09
Volume, V (in ³):	13.53
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	1.8
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	139.5
Dry Unit Weight, γ_d (lb/ft ³):	137.1

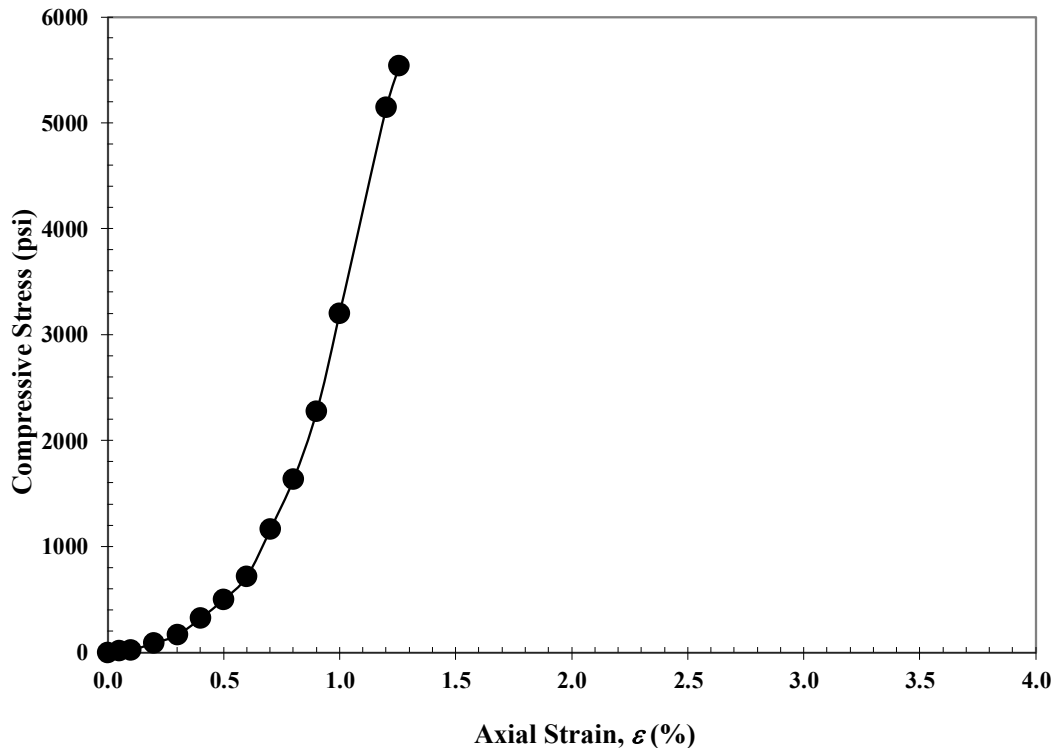
Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	5536
Strain (%):	1.3

38 (MPa)



Notes: Sandstone, brown, slightly weathered, moderately strong.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method D)

(Project: SUM-77-SR8-11.65, Boring Location: B-048-0-24, NQ2-1, Depth: 11.6-12.0 ft)

Tested Date: 11/7/2024

Specimen Properties

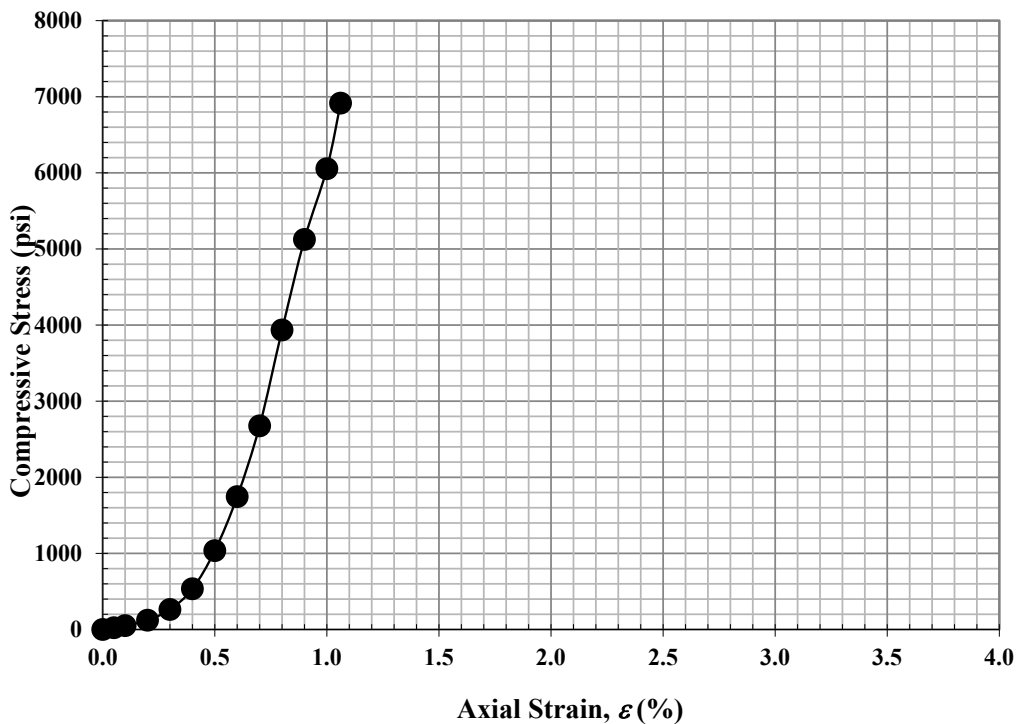
Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	4.33
Length to Diameter Ratio:	2.19
Area, A (in ²):	3.07
Volume, V (in ³):	13.33
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	1.1
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	145.6
Dry Unit Weight, γ_d (lb/ft ³):	144.1

After Testing



Results

Unconfined Compressive Strength (psi):	6916	48 (MPa)
Strain (%):	1.1	
Elastic Modulus (Gpa):	7.361	



Notes: Sandstone, brown and gray, slightly weathered, moderately strong, fine to medium grained, contains shale laminations.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-049-0-24, NQ2-1, Depth: 8.6-9.0ft)

Tested Date: 10/25/2024

Specimen Properties

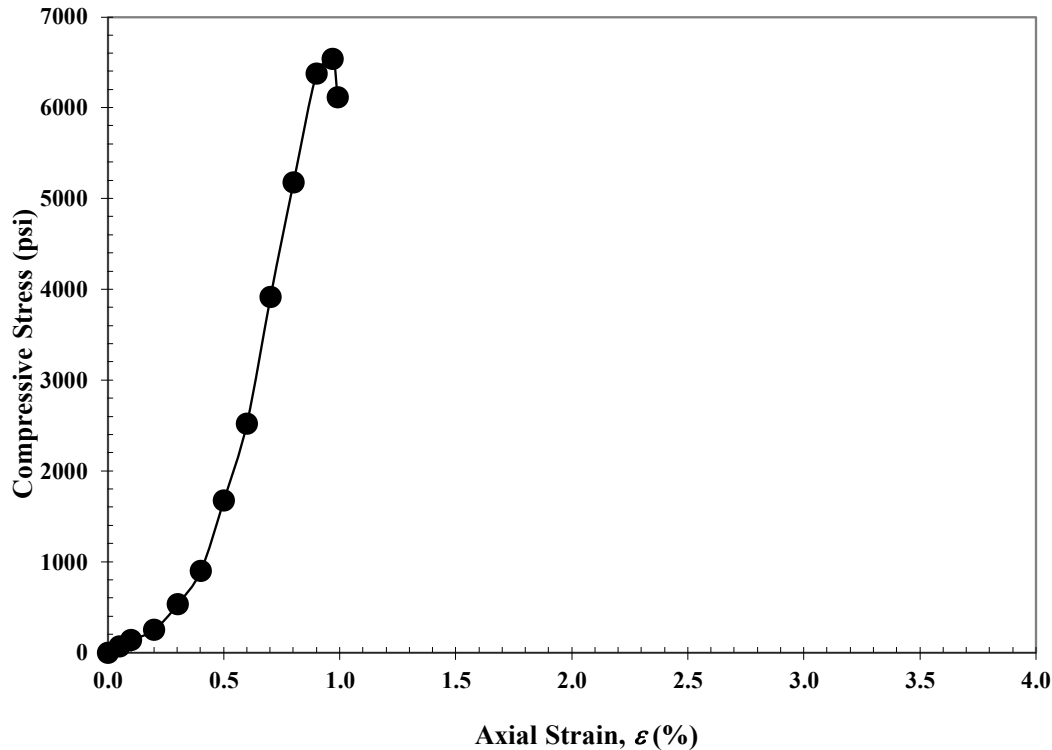
Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	4.55
Length to Diameter Ratio:	2.29
Area, A (in ²):	3.09
Volume, V (in ³):	14.06
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	1.1
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	134.5
Dry Unit Weight, γ_d (lb/ft ³):	133.0

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	6540	
Strain (%):	1.0	45 (MPa)



Notes: Sandstone, brown, moderately weathered, moderately strong, fine to medium grained.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-051-0-24, NQ2-1, Depth: 11.3-11.7ft)

Tested Date: 10/23/2024

Specimen Properties

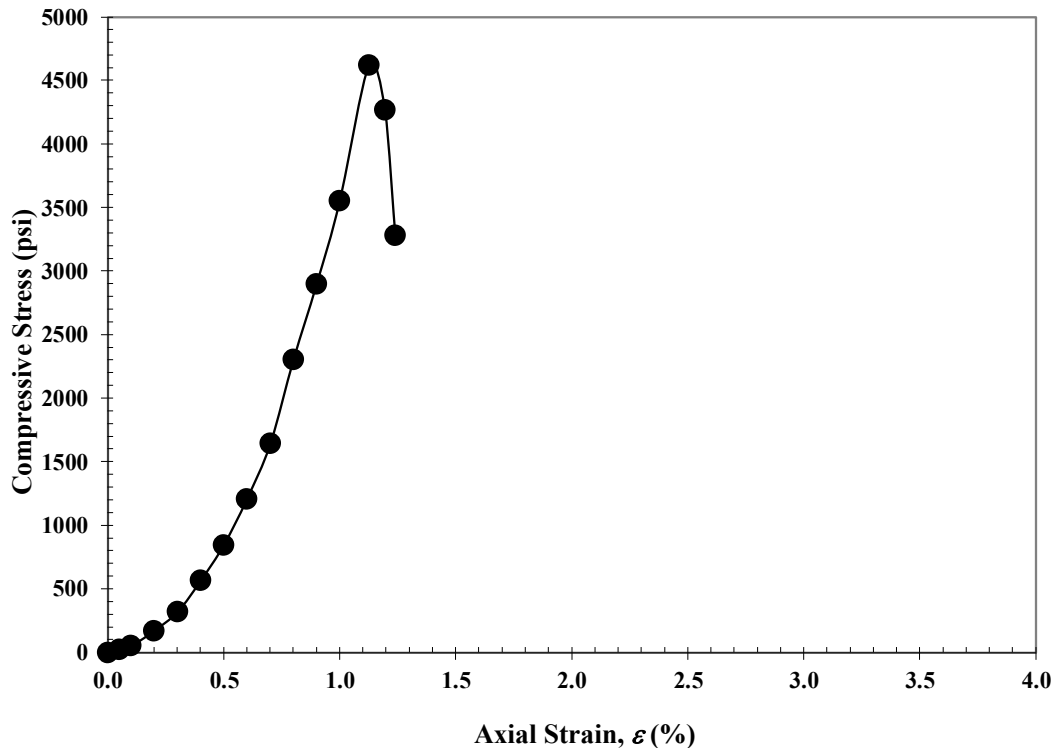
Average Dia., D_{avg} (in):	1.96
Average Height, H_{avg} (in):	4.44
Length to Diameter Ratio:	2.27
Area, A (in ²):	3.01
Volume, V (in ³):	13.37
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	0.9
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	140.8
Dry Unit Weight, γ_d (lb/ft ³):	139.6

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	4620	
Strain (%):	1.1	32 (MPa)



Notes: Sandstone, brown, slightly weathered, moderately strong, vuggy.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-052-0-24, NQ2-1, Depth: 17.9-18.3ft)

Tested Date: 10/23/2024

Specimen Properties

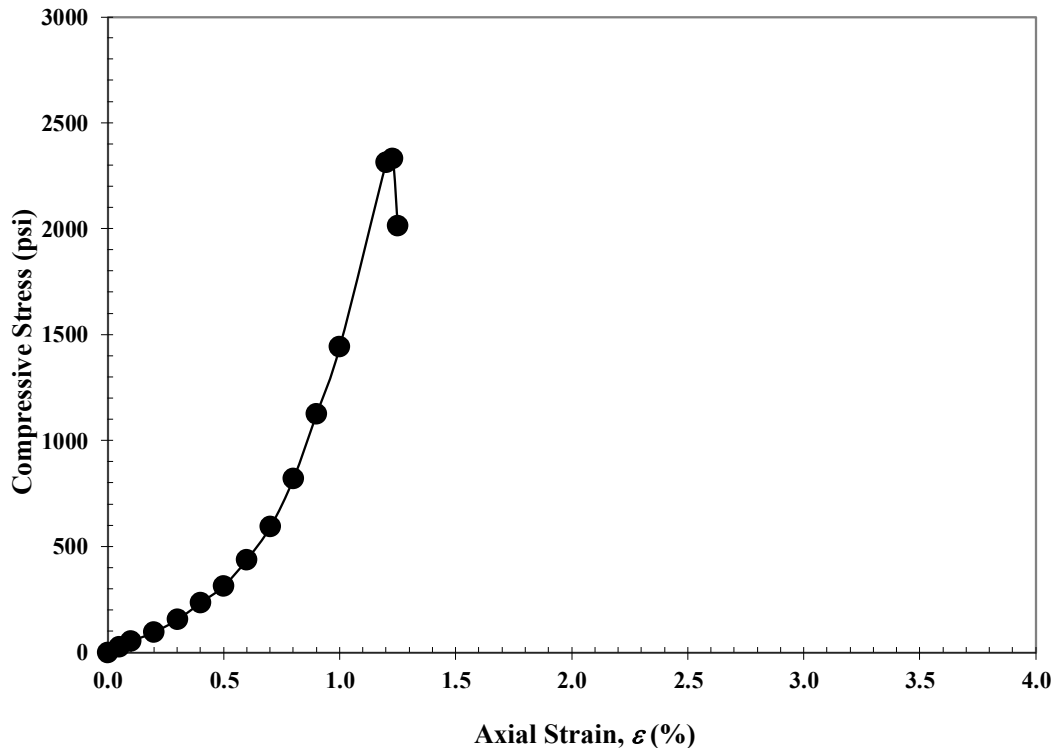
Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	4.64
Length to Diameter Ratio:	2.34
Area, A (in ²):	3.09
Volume, V (in ³):	14.34
Wet Mass of Specimen (lb):	1.2
Moisture Content (%):	0.9
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	138.9
Dry Unit Weight, γ_d (lb/ft ³):	137.6

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	2331	
Strain (%):	1.2	
		16 (MPa)



Notes: Sandstone, brown, slightly weathered, slightly strong, vuggy, contains shale laminations and inclusions.

Sample trimming procedure does not conform to ASTM D4543 and the results reported may differ from the results obtained from a test specimen that meets the requirements of Practice D4543.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-053-0-24, NQ2-1, Depth: 6.2-6.6ft)

Tested Date: 10/31/2024

Specimen Properties

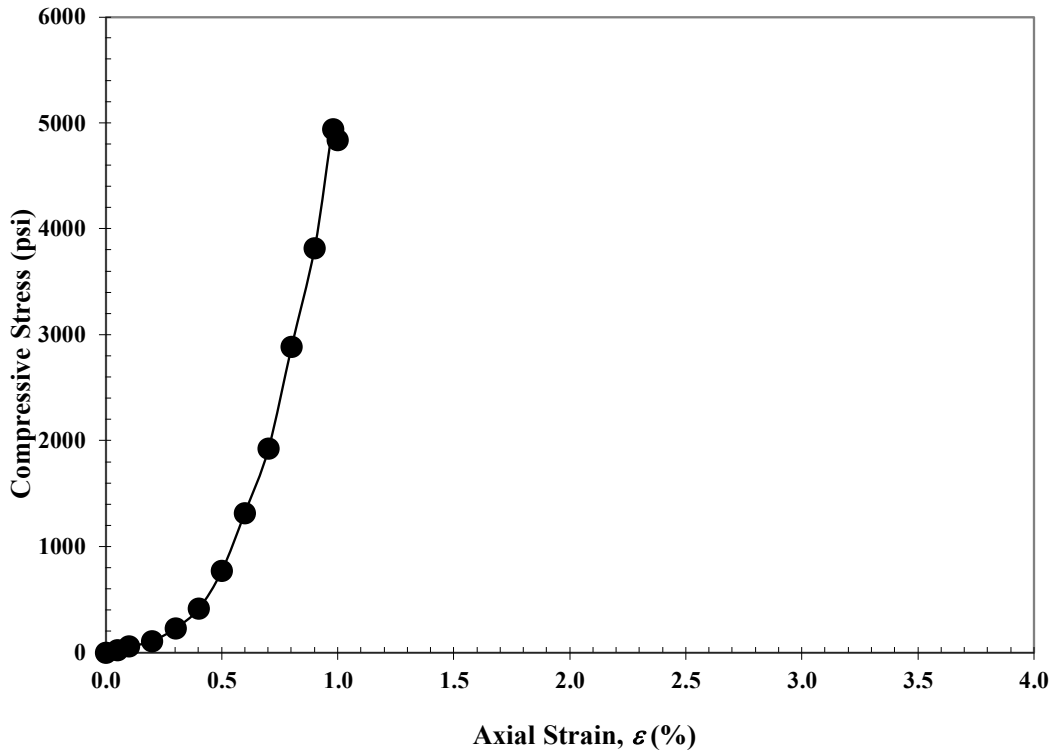
Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	4.60
Length to Diameter Ratio:	2.32
Area, A (in ²):	3.09
Volume, V (in ³):	14.22
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	0.8
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	136.5
Dry Unit Weight, γ_d (lb/ft ³):	135.4

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	4942	
Strain (%):	1.0	34 (MPa)



Notes: Sandstone, brown and orangish brown, slightly weathered, moderately strong.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-054-0-24, NQ2-1, Depth: 16.4-16.8ft)

Tested Date: 10/23/2024

Specimen Properties

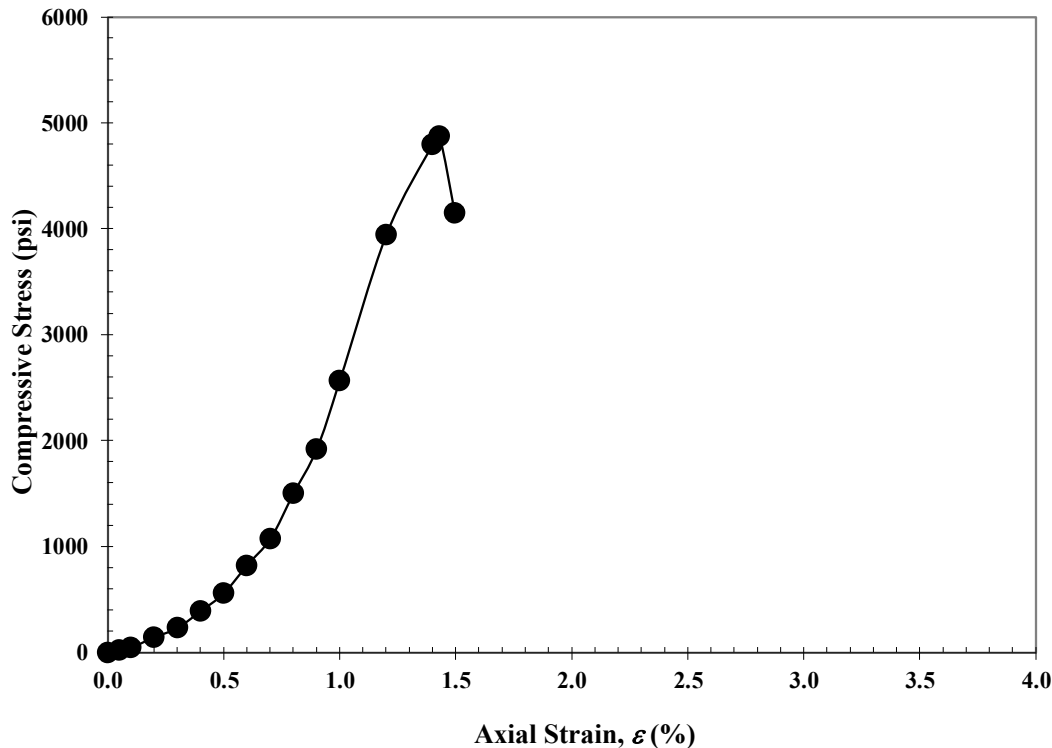
Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	4.48
Length to Diameter Ratio:	2.26
Area, A (in ²):	3.09
Volume, V (in ³):	13.82
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	1.9
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	140.7
Dry Unit Weight, γ_d (lb/ft ³):	138.0

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	4874	
Strain (%):	1.4	34 (MPa)



Notes: Sandstone, brown, slightly weathered, moderately strong, contains shale laminations and inclusions.

Sample trimming procedure does not conform to ASTM D4543 and the results reported may differ from the results obtained from a test specimen that meets the requirements of Practice D4543.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-055-0-24, NQ2-1, Depth: 6.5-6.9ft)

Tested Date: 10/31/2024

Specimen Properties

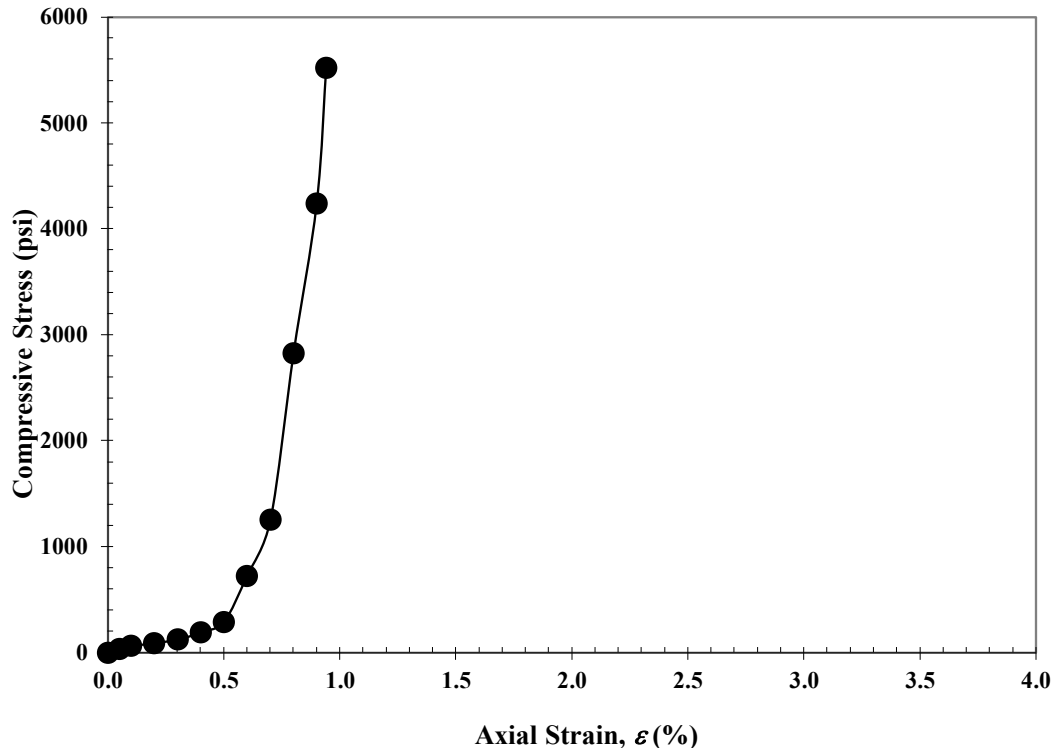
Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	4.46
Length to Diameter Ratio:	2.25
Area, A (in ²):	3.09
Volume, V (in ³):	13.78
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	1.1
Dry Mass of Specimen (lb):	1.1
Wet Unit Weight, γ (lb/ft ³):	134.4
Dry Unit Weight, γ_d (lb/ft ³):	132.9

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	5520	
Strain (%):	0.9	38 (MPa)



Notes: Sandstone, brown and orangish brown, slightly weathered, moderately strong.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: SUM-77-SR8-11.65, Boring Location: B-057-0-24, NQ2-1, Depth: 6.7-7.1ft)

Tested Date: 10/31/2024

Specimen Properties

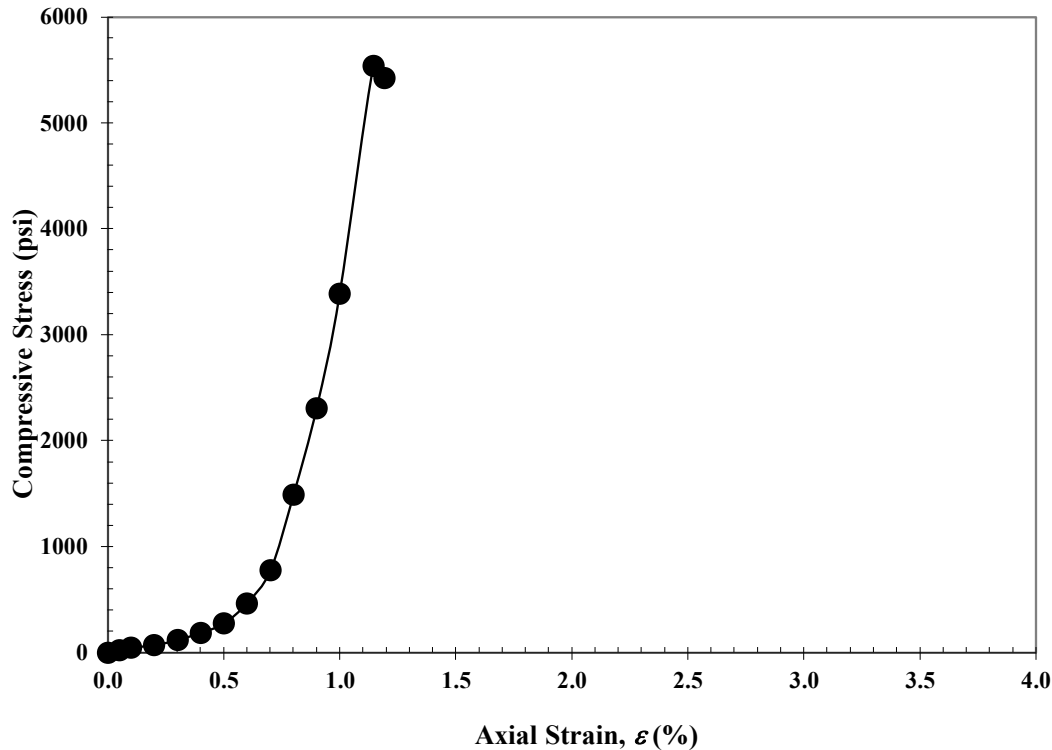
Average Dia., D_{avg} (in):	1.99
Average Height, H_{avg} (in):	4.19
Length to Diameter Ratio:	2.11
Area, A (in ²):	3.10
Volume, V (in ³):	12.99
Wet Mass of Specimen (lb):	1.0
Moisture Content (%):	1.2
Dry Mass of Specimen (lb):	1.0
Wet Unit Weight, γ (lb/ft ³):	135.1
Dry Unit Weight, γ_d (lb/ft ³):	133.5

Final Specimen Figure



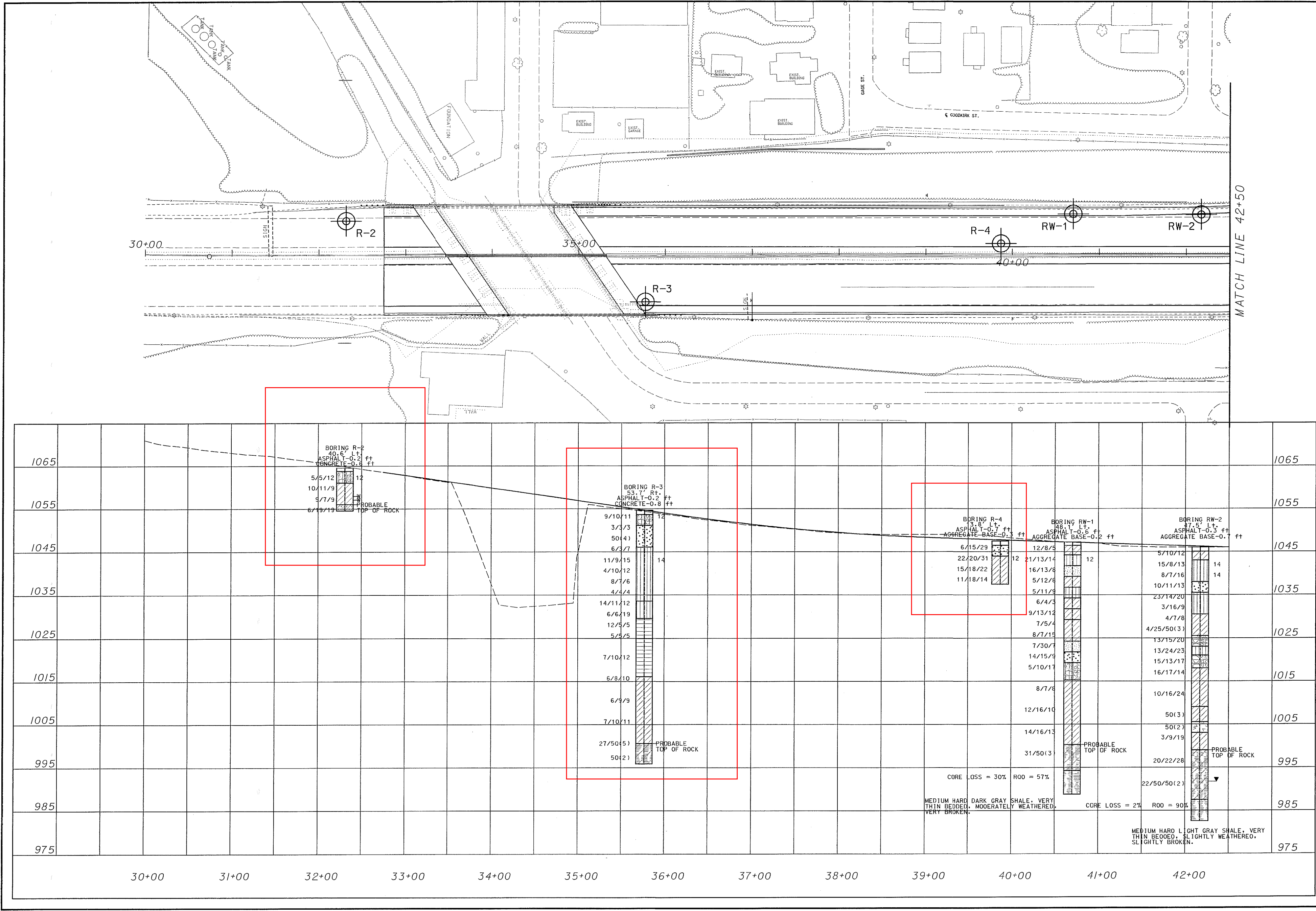
Results

Unconfined Compressive Strength (psi):	5541	38	(MPa)
Strain (%):	1.1		



Notes: Sandstone, brown, unweathered, moderately strong.

HISTORICAL BORINGS

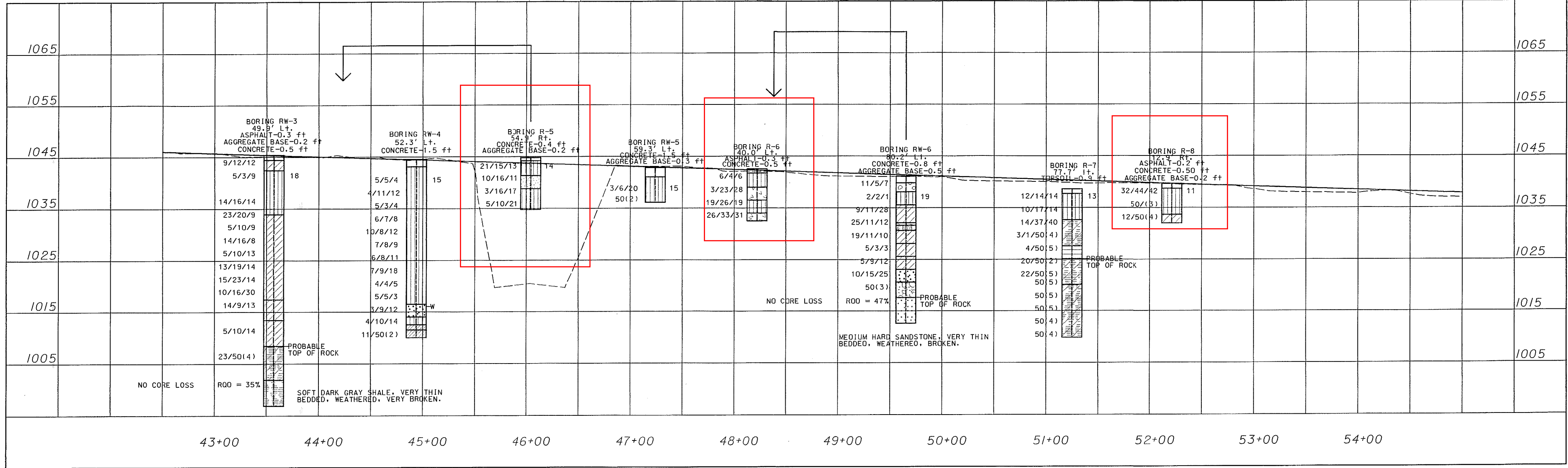
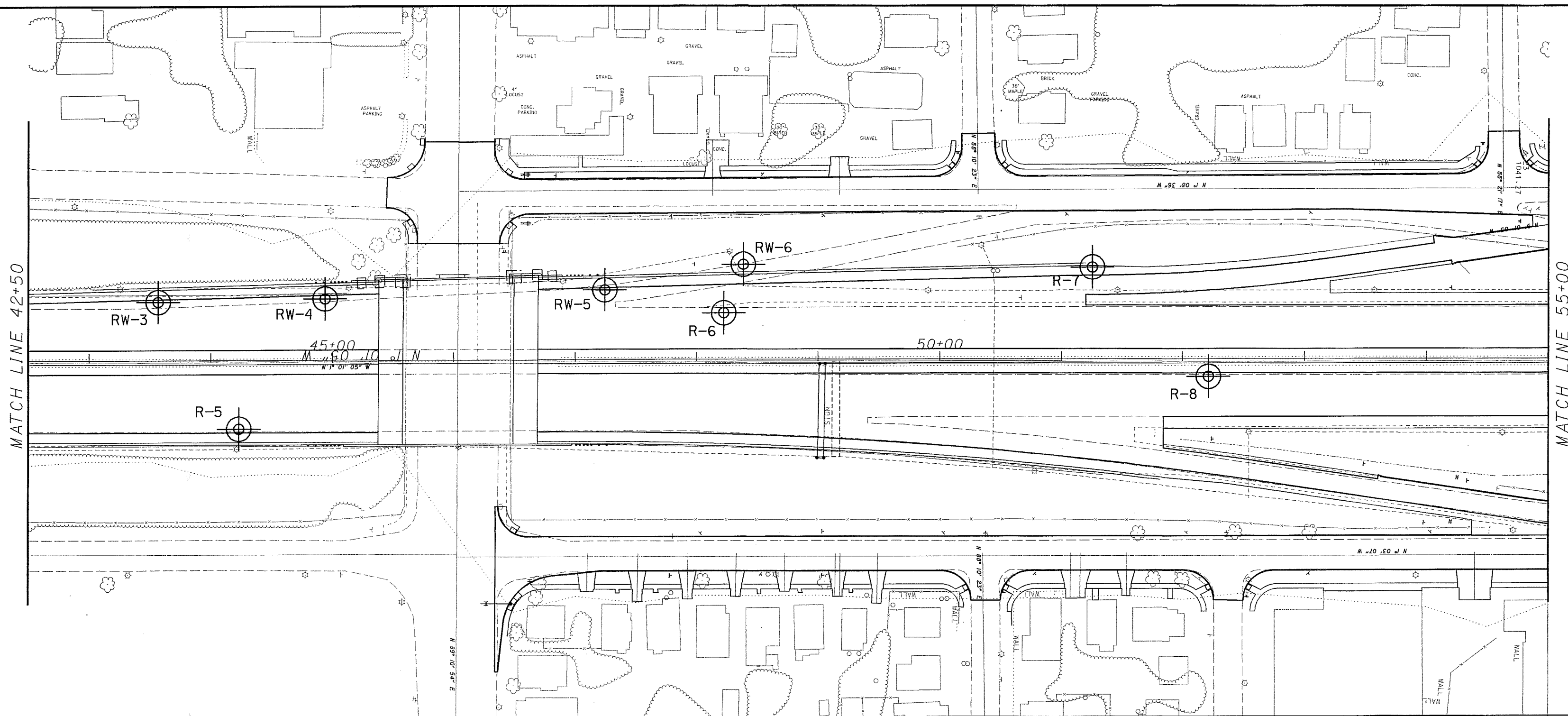


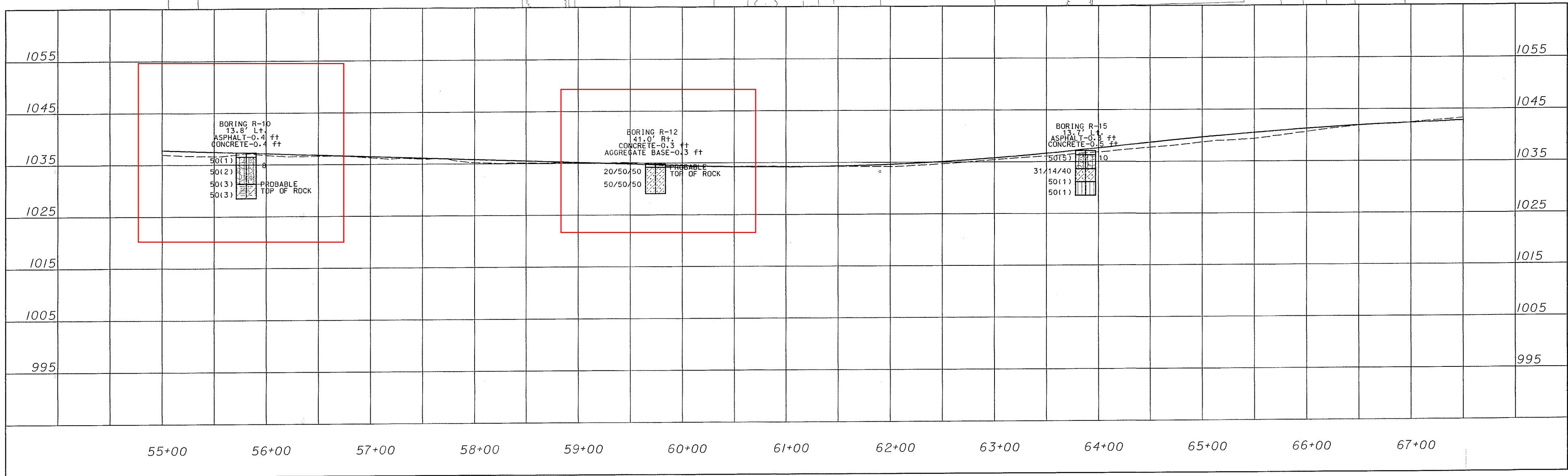
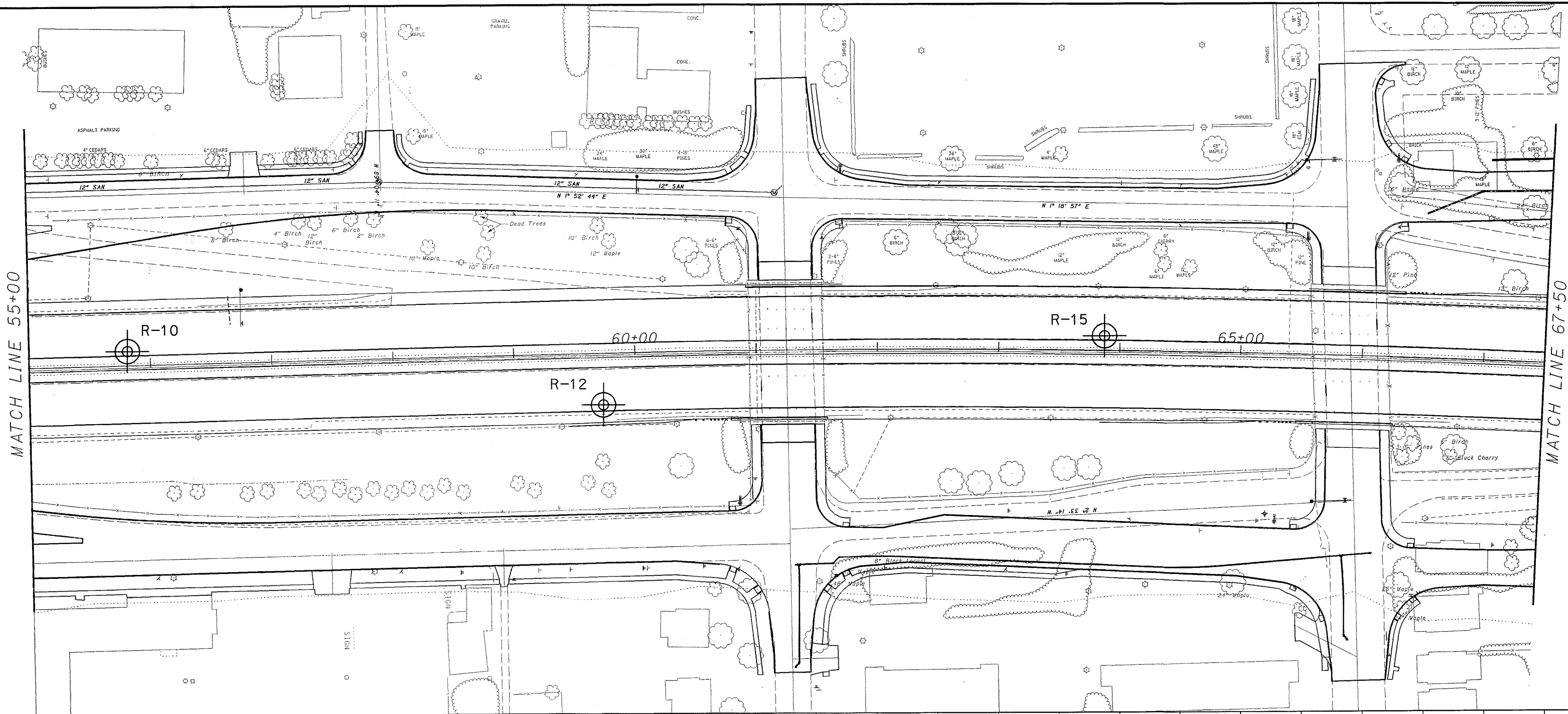
PID: 114865	SFN: _____	PROJECT: SUM-IR-77/SR8-11.65/0.00	STATION / OFFSET: 54' RT.	START: 3/24/99	END: 3/24/99	PG 2 OF 2	R-003-0-99
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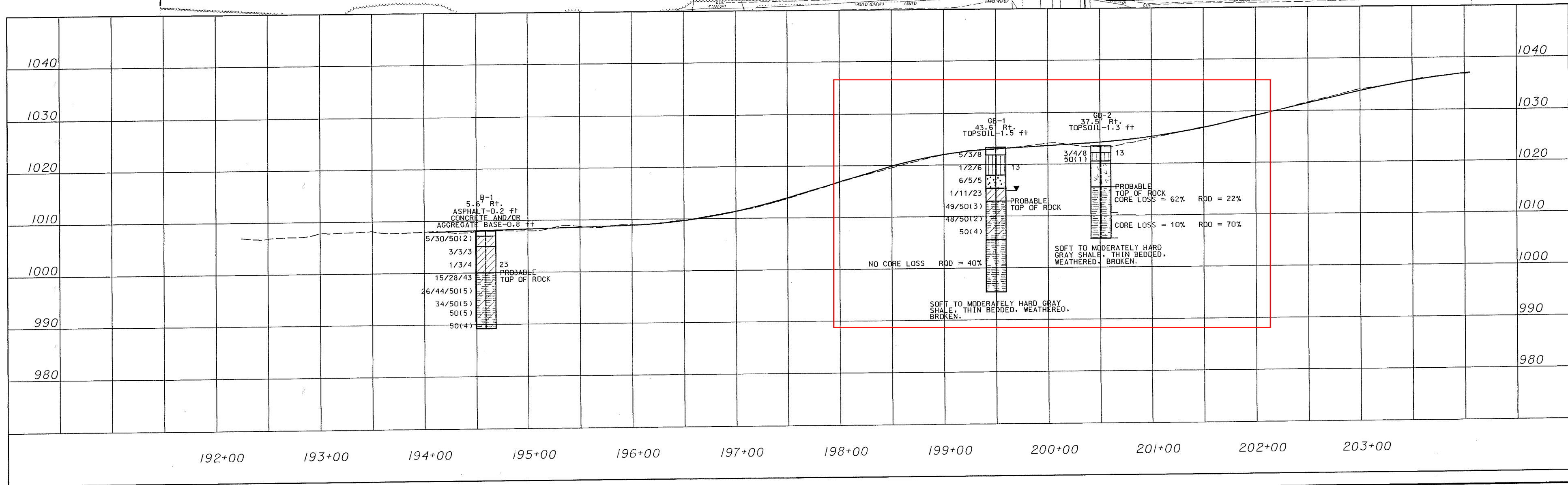
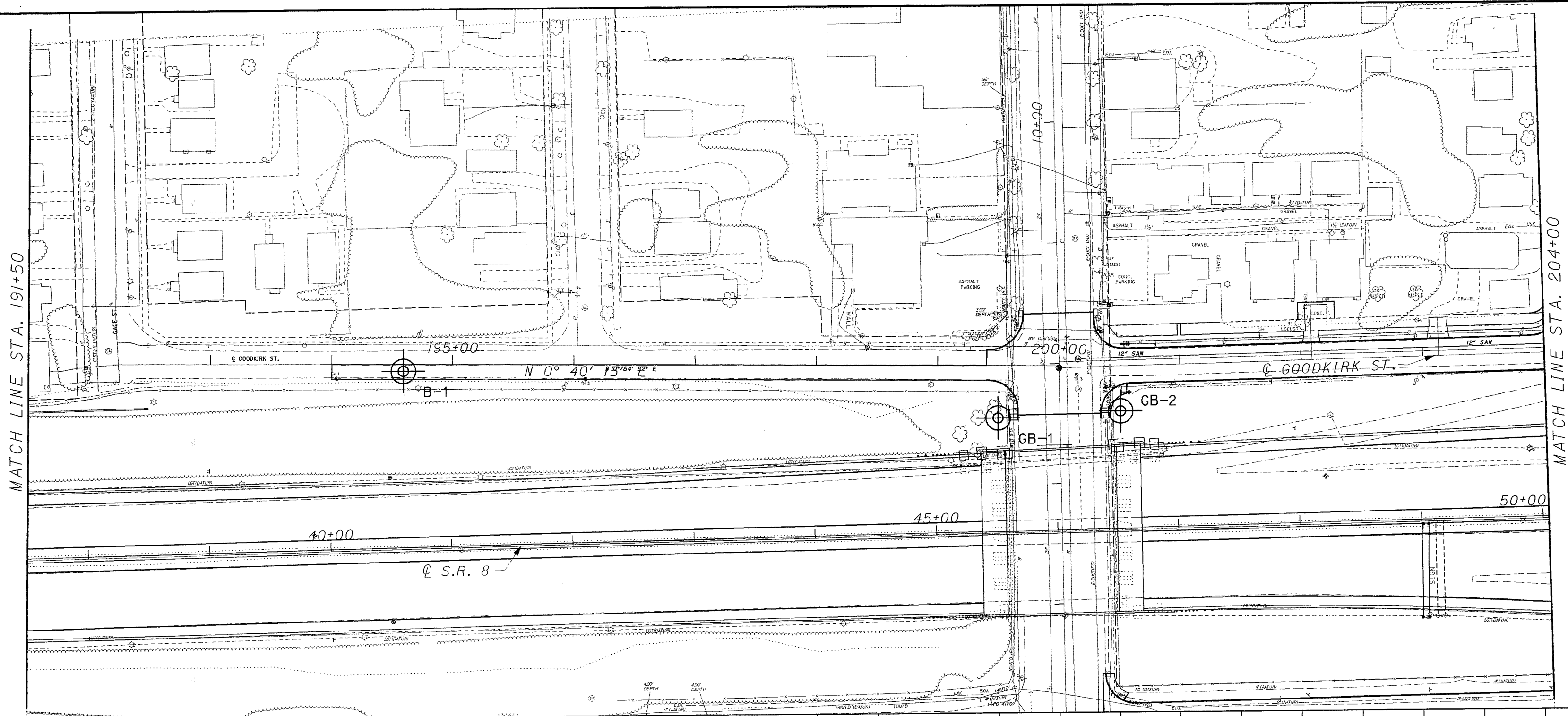
[illegible]

NOTES: NONE

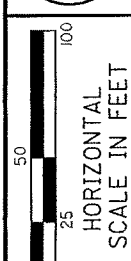
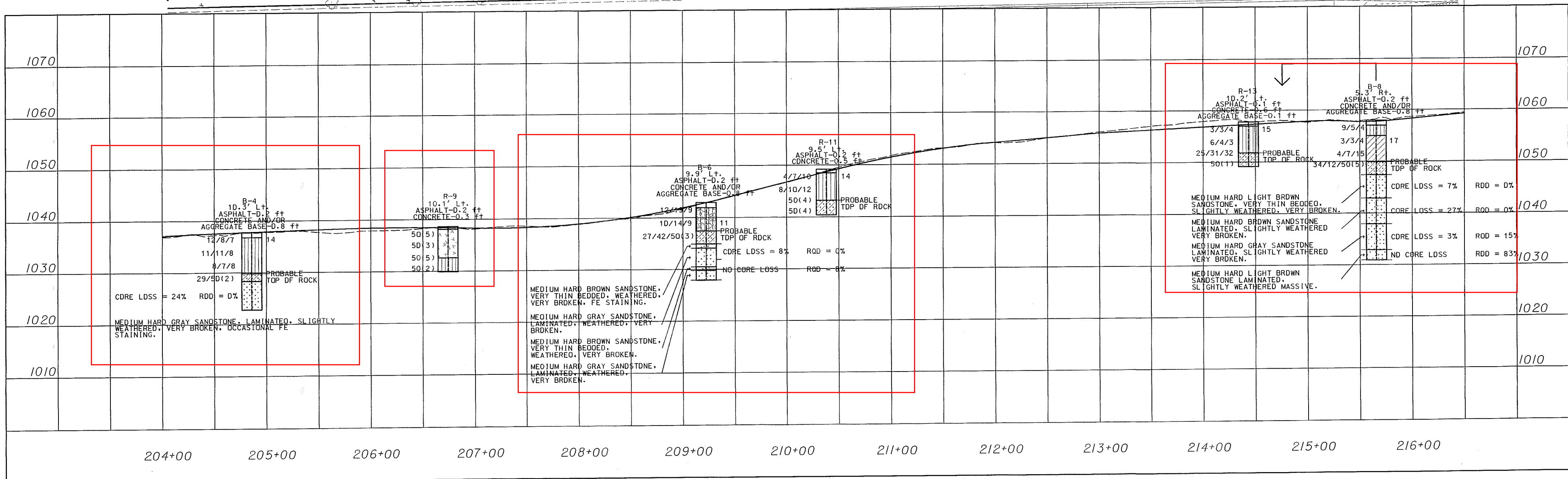
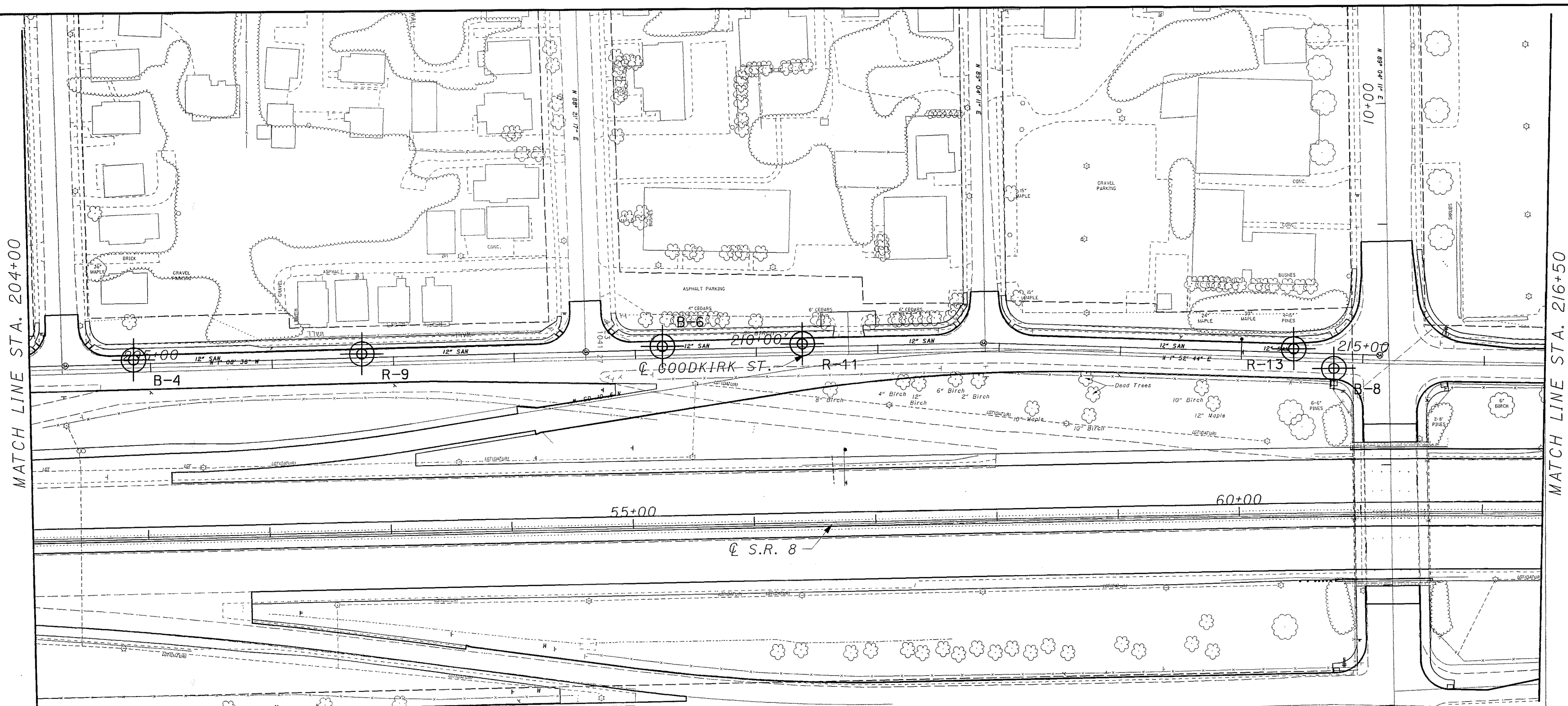
ABANDONMENT METHODS, MATERIALS, QUANTITIES: NOT RECORDED







Client: ODOT						Project: SUM - 8 - 0087						Job No. 9921-3077.00											
LOG OF: Boring GB-2						Location: Station 46+54, 98.9' Left						Date Drilled: 12/9/99						STANDARD PENETRATION (N)					
Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No.		Hand Penetro- meter (tsf)	WATER OBSERVATIONS: Water seepage at: None Water level at completion: ---	GRADATION						Moisture Content - %									
				Drive	Press			% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	PL	Natural	LL							
0	1023.5							DESCRIPTION															
1.3	1022.2							Topsoil - 15"															
		3			1			FILL: Medium dense brown SANDY SILT (A-4a); damp.															
3.0	1020.5	50/1	0					Began coring at 3.0'															
		Core 60"	Rec 18"					FILL: Cored through apparent random fill. Recovered material consisted of SHALE, SANDSTONE, CONCRETE fragments and gravel. Loss believed to be due to washing away of soil matrix.															
5																							
8.0	1015.5							Soft to moderately hard gray SHALE; thin bedded; moderately weathered to weathered; broken.															
		Core 60"	Rec 23"																				
10																							
		Core 60"	Rec 54"																				
15																							
18.0	1005.5							Bottom of Boring - 18.0'															
20																							
25																							



CALCULATED R.J.H. CHECKED B.E.W.

SOIL PROFILE - GOODKIRK STREET
STA. 204+00 TO STA. 216+50

SUM-8-0.63

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / BEAHR</u>		DRILL RIG: <u>'18 CME 55-TATV-265</u>		STATION / OFFSET: <u>333+51, 52' LT.</u>		EXPLORATION ID <u>B2-001-0-20</u>												
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / MORGAL</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT:		PAGE 1 OF 2												
PID: <u>102329</u> SFN: <u></u>		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>4/26/18</u>		ELEVATION: <u>1064.1 (MSL)</u> EOB: <u>40.0 ft.</u>														
START: <u>7/15/20</u> END: <u>7/15/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>95.2</u>		COORD: <u>511889.9690 N, 2242981.6090 E</u>														
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED	
									GR	CS	FS	SI	CL	LL	PL	PI	WC			
PAVEMENT AND BASE, 8" ASPHALT 4" GRANULAR BASE		1064.1																		
		1063.1	1	5	30	89	SS-1	-	8	8	23	37	24	25	17	8	11	A-4a (5)		
		1060.6	2	9 10																
MEDIUM DENSE, LIGHT TAN BROWN, FINE SAND, SOME GRAVEL, LITTLE SILT, DAMP			3																	
			4	2 8 13	33	78	SS-2	-	-	-	-	-	-	-	-	-	5	A-3 (V)		
			5																	
MEDIUM STIFF, BROWN, SANDY SILT, LITTLE FINE SAND, TRACE GRAVEL, SAND LENSES, DAMP			6	2																
			7	2 4	10	33	SS-3	-	-	-	-	-	-	-	-	-	13	A-3 (V)		
			8																	
MEDIUM STIFF, BROWN, SANDY SILT, LITTLE FINE SAND, TRACE GRAVEL, SAND LENSES, DAMP		1055.6																		
			9	3 4 5	14	83	SS-4	-	4	13	31	27	25	24	14	10	12	A-4a (3)		
			10																	
MEDIUM STIFF, BROWN, SANDY SILT, LITTLE TO SOME FINE SAND, DAMP			11	2																
			12	3 7	16	72	SS-5	-	-	-	-	-	-	-	-	-	10	A-4a (V)		
			13																	
MEDIUM STIFF, BROWN, SANDY SILT, LITTLE TO SOME FINE SAND, DAMP		1050.6																		
			14	4 6 7	21	56	SS-6	-	5	8	30	42	15	22	16	6	14	A-4a (4)		
			15																	
STIFF, BROWN, SILT AND CLAY, TRACE SAND, TRACE GRAVEL, QUARTZITE FRAGMENTS, DAMP		1048.1																		
			16	4 10 9	30	94	SS-7	-	6	3	12	51	28	30	18	12	18	A-6a (9)		
			17																	
VERY STIFF, BROWN, SILT AND CLAY, LITTLE GRAVEL, TRACE SAND, DAMP		1045.6																		
			18	3 4 4	13	83	SS-8	-	6	7	20	40	27	29	18	11	14	A-6a (7)		
			19																	
STIFF, BROWN, SILT AND CLAY, TRACE SAND, CONTAINS ROOT HAIRS, DAMP		1043.1																		
			20																	
			21	3 4 6	16	83	SS-9	-	-	-	-	-	-	-	-	-	-	17		A-6a (V)
STIFF, GREENISH GREY, SILT AND CLAY, CONTAINS SHALE FRAGMENTS AND GRAVEL, DAMP		1040.6																		
			22																	
			23	6 7 7	22	72	SS-10	-	-	-	-	-	-	-	-	-	-	11		A-6a (V)
			24	3 3 5	13	72	SS-11	-	-	-	-	-	-	-	-	-	16	A-6a (V)		
			25																	
			26																	
		1035.6																		
			27																	
			28																	
			29	30 50/4"	-	100	SS-12	-	-	-	-	-	-	-	-	-	10	UCF (V)		

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; TREMIED BENTONITE GROUT

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:29 - P:\22-0033 (13208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\13208-114865\ADMIN\CONTRACT\SSU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>		DRILL RIG: <u>'19 CME 75-079-797</u>		STATION / OFFSET: <u>4326+28, 12' LT.</u>		EXPLORATION ID												
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT:		P-011-0-20												
PID: <u>102329</u> SFN: _____		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>8/15/19</u>		ELEVATION: <u>1068.8 (MSL)</u> EOB: <u>9.1 ft.</u>		PAGE												
START: <u>6/23/20</u> END: <u>6/23/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>83.7</u>		COORD: <u>510595.6030 N, 2242987.9680 E</u>		1 OF 1												
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED	
		1068.8							GR	CS	FS	SI	CL	LL	PL	PI	WC			
PAVEMENT AND BASE, 4" ASPHALT		1067.7	TR	1																
MEDIUM DENSE, GREY AND BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT , DAMP				2	5 6 8	20	44	SS-1	- -	19 -	54 -	10 -	5 -	12 -	21 -	14 -	7 -	10 -	A-2-4 (0)	
		1065.3		3																
CLAYSTONE , GREY, DECOMPOSED TO HIGHLY WEATHERED, VERY WEAK, VERY FINE GRAINED, LAMINATED, ARENACEOUS.				4	6 50/4"	-	100	SS-2	-	4	22	32	34	8	20	NP	NP	6	Rock (V)	
				5																
				6	23 50/1"	-	100	SS-3	-	-	-	-	-	-	-	-	5	Rock (V)		
				7																
		1059.7		8																
			EOB	9	30 50/1"	-	100	SS-4	-	-	-	-	-	-	-	-	6	Rock (V)		
NOTES: NONE																				
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																				

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (113208-114865 SUM-J7 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\ADMIN\CONTRACT\SU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>				DRILL RIG: <u>'19 CME 75-079-797</u>				STATION / OFFSET: <u>4326+01, 91' LT.</u>				EXPLORATION ID							
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>				HAMMER: <u>CME AUTOMATIC</u>				ALIGNMENT:				P-012-0-20							
PID: <u>102329</u> SFN: _____		DRILLING METHOD: <u>3.25" HSA</u>				CALIBRATION DATE: <u>8/15/19</u>				ELEVATION: <u>1069.9 (MSL)</u> EOB: <u>10.0 ft.</u>				PAGE							
START: <u>6/23/20</u> END: <u>6/23/20</u>		SAMPLING METHOD: <u>SPT</u>				ENERGY RATIO (%): <u>83.7</u>				COORD: <u>510595.6030 N, 2242899.0430 E</u>				1 OF 1							
MATERIAL DESCRIPTION AND NOTES			ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
											GR	CS	FS	SI	CL	LL	PL	PI			
PAVEMENT AND BASE, 5.25" ASPHALT, 8.5" CONCRETE, 4" SUBBASE			1069.9																		
VERY STIFF, BROWN AND GREY, SANDY SILT , TRACE SAND AND GRAVEL, DAMP			1068.4	1																	
				2	3	6	17	72	SS-1	3.50	4	15	13	39	29	24	15	9	16	A-4a (7)	
				3																	
			1063.9	4	19																
				5	24	-	6	SS-2	-	14	17	32	26	11	17	NP	NP	9	A-4a (0)		
				6	50/5"																
MEDIUM DENSE, BROWN AND GREY, COARSE AND FINE SAND , LITTLE CLAY AND GRAVEL, DAMP			1061.4	7	10																
				8	13	39	50	SS-3	-	5	20	42	20	13	19	NP	NP	12	A-3a (0)		
				9	15																
HARD, GREY, CLAY , TRACE SAND AND GRAVEL, DAMP looks similar to grey claystone from other borings, though blow counts are under 50.			1059.9	10	10																
				EOB	30	88	56	SS-4	-	-	-	-	-	-	-	-	6	A-7-6 (V)			

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (13208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\13208-114865\ADMIN\CONTRACT\SU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>		DRILL RIG: <u>'19 CME 75-079-797</u>		STATION / OFFSET: <u>4327+97, 48' LT.</u>		EXPLORATION ID <u>P-013-0-20</u>												
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT: _____														
PID: <u>102329</u> SFN: _____		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>8/15/19</u>		ELEVATION: <u>1072.2 (MSL)</u> EOB: <u>8.9 ft.</u>		PAGE 1 OF 1												
START: <u>6/23/20</u> END: <u>6/23/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>83.7</u>		COORD: <u>510771.1720 N, 2242976.1200 E</u>														
MATERIAL DESCRIPTION AND NOTES		ELEV. <u>1072.2</u>	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED	
									GR	CS	FS	SI	CL	LL	PL	PI	WC			
PAVEMENT AND BASE , 2.25" ASPHALT, 8.75" CONCRETE, 4" SUBBASE		<u>1070.9</u>	TR	1	3															
HARD, GREY, SANDY SILT , TRACE SAND AND GRAVEL, MOIST TO DAMP				2	4 7	15	56	SS-1	4.00	0	19	30	40	11	24	16	8	14	A-4a (3)	<V>
		<u>1068.7</u>		3																<V>
CLAYSTONE , GREY, DECOMPOSED TO HIGHLY WEATHERED, VERY WEAK, VERY FINE GRAINED, LAMINATED, ARENACEOUS.				4	31 50/1"	-	100	SS-2	-	0	32	23	35	10	28	17	11	6	Rock (V)	<V>
				5															<V>	
				6	50/5"	-	100	SS-3	-	-	-	-	-	-	-	-	6	Rock (V)	<V>	
				7															<V>	
		<u>1063.3</u>	EOB	8	50/5"	-	100	SS-4	-	-	-	-	-	-	-	-	7	Rock (V)	<V>	
NOTES: NONE																				
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																				

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\ADMIN\CONTRACT\SU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>		DRILL RIG: <u>'19 CME 75-079-797</u>		STATION / OFFSET: <u>4329+22, 31' RT.</u>		EXPLORATION ID											
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT: _____		P-014-0-20											
PID: <u>102329</u> SFN: _____		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>8/15/19</u>		ELEVATION: <u>1075.0 (MSL)</u> EOB: <u>9.2 ft.</u>		PAGE											
START: <u>6/24/20</u> END: <u>6/24/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>83.7</u>		COORD: <u>510889.5650 N, 2243060.7870 E</u>		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED
		1075.0							GR	CS	FS	SI	CL	LL	PL	PI	WC		
PAVEMENT AND BASE, 3.5" ASPHALT, 8.5" CONCRETE		1073.9	TR	1				-	-	-	-	-	-	-	-	-	-		
DENSE, BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT, TRACE TO LITTLE CLAY, DAMP				2	6	50	78	SS-1	-	27	15	31	13	14	NP	NP	NP	12	A-2-4 (0)
		1071.5		3															
CLAYSTONE, GREY, DECOMPOSED TO HIGHLY WEATHERED, VERY WEAK, VERY FINE GRAINED, LAMINATED, ARENACEOUS.				4	7	-	82	SS-2	-	1	17	28	41	13	28	22	6	7	Rock (V)
				5															
				6	50/5"	-	80	SS-3	-	-	-	-	-	-	-	-	5	Rock (V)	
				7															
		1065.8	EOB	8															
				9	27	-	100	SS-4	-	-	-	-	-	-	-	-	4	Rock (V)	
					50/2"														
NOTES: NONE																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (13208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\13208-114865\ADMIN\CONTRACT\SU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>		DRILL RIG: <u>'19 CME 75-079-797</u>		STATION / OFFSET: <u>327+44, 9' RT.</u>		EXPLORATION ID <u>P-015-0-20</u>												
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT:		PAGE 1 OF 1												
PID: <u>102329</u> SFN: <u></u>		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>8/15/19</u>		ELEVATION: <u>1077.7 (MSL)</u> EOB: <u>8.75 ft.</u>														
START: <u>6/30/20</u> END: <u>6/30/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>83.7</u>		COORD: <u>511289.5650 N, 2243082.1500 E</u>														
MATERIAL DESCRIPTION AND NOTES		ELEV. <u>1077.7</u>	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED	
									GR	CS	FS	SI	CL	LL	PL	PI	WC			
PAVEMENT AND BASE, 3.75" ASPHALT, 8.5" CONCRETE, 3" GRAVEL BASE		<u>1076.4</u>	TR	1		-	90	SS-1	-	0	12	48	29	11	24	19	5	4	Rock (V)	
CLAYSTONE, GREY, DECOMPOSED TO HIGHLY WEATHERED, VERY WEAK, VERY FINE GRAINED, LAMINATED, ARENACEOUS.				2		-	100	SS-2	-	11	27	25	27	10	-	-	-	5	Rock (V)	
				3																
				4		-	33	SS-3	-	-	-	-	-	-	-	-	-	4	Rock (V)	
				5																
				6		-	100	SS-4	-	-	-	-	-	-	-	-	-	4	Rock (V)	
				7																
				8																
		<u>1068.9</u>	EOB		-	100	SS-4	-	-	-	-	-	-	-	-	-	4	Rock (V)		
NOTES: NONE																				
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																				

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (113208-114865 SUM-77 SR8-11.650,00 & SUM-77SR8-9.650,00)\113208-114865\ADMIN\CONTRACT\SU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>		DRILL RIG: <u>'19 CME 75-079-797</u>		STATION / OFFSET: <u>331+43, 8' RT.</u>		EXPLORATION ID											
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT:		P-017-0-20											
PID: <u>102329</u> SFN: <u></u>		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>8/15/19</u>		ELEVATION: <u>1070.3 (MSL)</u> EOB: <u>9.25 ft.</u>		PAGE											
START: <u>6/30/20</u> END: <u>6/30/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>83.7</u>		COORD: <u>511689.5650 N, 2243082.5290 E</u>		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED
		1070.3							GR	CS	FS	SI	CL	LL	PL	PI	WC		
PAVEMENT AND BASE, 4.75" ASPHALT, 8.25" CONCRETE, 4" GRAVEL BASE		1068.9	1	10				-	-	-	-	-	-	-	-	-	-		
HARD, BROWN, SILT, TRACE SAND AND GRAVEL, DAMP			2	13 18	43	78	SS-1	4.00	0	7	7	52	34	25	18	7	13	A-4b (8)	<V>
		1066.8	3																<V>
HARD, BROWN, SILT AND CLAY, TRACE SAND AND GRAVEL, DAMP			4	16 47 50/2"	-	71	SS-2	4.00	0	25	11	39	25	27	14	13	11	A-6a (7)	<V>
			5																<V>
		1064.0	6	17 50/5"	-	100	SS-3	-	-	-	-	-	-	-	-	-	8	Rock (V)	<V>
CLAYSTONE, GREY, DECOMPOSED TO HIGHLY WEATHERED, VERY WEAK, VERY FINE GRAINED, LAMINATED, ARENACEOUS.			7																<V>
			8																<V>
		1061.0	9	29 50/3"	-	100	SS-4	-	-	-	-	-	-	-	-	-	6	Rock (V)	<V>
			EOB																<V>
NOTES: NONE																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\ADMIN\CONTRACT\SU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>				DRILL RIG: <u>'19 CME 75-079-797</u>				STATION / OFFSET: <u>332+50, 11' LT.</u>				EXPLORATION ID							
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>				HAMMER: <u>CME AUTOMATIC</u>				ALIGNMENT:				P-018-0-20							
PID: <u>102329</u> SFN: _____		DRILLING METHOD: <u>3.25" HSA</u>				CALIBRATION DATE: <u>8/15/19</u>				ELEVATION: <u>1067.0 (MSL)</u> EOB: <u>10.0 ft.</u>				PAGE							
START: <u>6/23/20</u> END: <u>6/23/20</u>		SAMPLING METHOD: <u>SPT</u>				ENERGY RATIO (%): <u>83.7</u>				COORD: <u>511795.6030 N, 2243067.1070 E</u>				1 OF 1							
MATERIAL DESCRIPTION AND NOTES			ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
PAVEMENT AND BASE, 3" ASPHALT, 4.5" CONCRETE, 4" GRAVEL BASE (NOTE: CONCRETE IS BROKEN).			1067.0																		
			1065.7	1		3	11	56	SS-1	3.25	23	54	10	4	9	NP	NP	NP	8	A-1-b (0)	
LOOSE, BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND, TRACE CLAY, DAMP			1063.5	2		5															
				3																	
MEDIUM DENSE, BROWN, GRAVEL AND/OR STONE FRAGMENTS WITH SAND AND SILT, LITTLE CLAY, DAMP			1061.0	4		3	15	72	SS-2	-	21	15	32	19	13	17	15	2	11	A-2-4 (0)	
				5		6															
VERY STIFF, BROWN, SANDY SILT, DAMP			1058.5	6		3															
				7		13	42	67	SS-3	-	2	11	26	26	35	21	14	7	11	A-4a (5)	
			1057.0	8																	
SHALE, BROWN, HIGHLY WEATHERED, VERY WEAK TO WEAK, VERY FINE GRAINED, VERY THIN BEDDED, ARENACEOUS.				9		11															
			EOB	10		24	67	72	SS-4	-	-	-	-	-	-	-	-	9	Rock (V)		

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS

[illegible]

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\ADMIN\CONTRACT\SU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>				DRILL RIG: <u>'19 CME 75-079-797</u>				STATION / OFFSET: <u>336+52, 50' LT.</u>				EXPLORATION ID							
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>				HAMMER: <u>CME AUTOMATIC</u>				ALIGNMENT: _____				P-020-0-20							
PID: <u>102329</u> SFN: _____		DRILLING METHOD: <u>3.25" HSA</u>				CALIBRATION DATE: <u>8/15/19</u>				ELEVATION: <u>1056.0 (MSL)</u> EOB: <u>10.0 ft.</u>				PAGE							
START: <u>6/22/20</u> END: <u>6/22/20</u>		SAMPLING METHOD: <u>SPT</u>				ENERGY RATIO (%): <u>83.7</u>				COORD: <u>512195.6030 N, 2243013.5430 E</u>				1 OF 1							
MATERIAL DESCRIPTION AND NOTES			ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
			1056.0								GR	CS	FS	SI	CL	LL	PL	PI			
PAVEMENT AND BASE, 11.5" ASPHALT, 6" GRAVEL BASE			1054.5	1		17															
MEDIUM DENSE, BROWN, SANDY SILT, DAMP				2		11 10	29	50	SS-1	-	43	8	19	18	12	-	-	-	11	A-4a (V)	
			1052.5	3																	
LOOSE TO MEDIUM DENSE, ORANGISH BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, DAMP				4		7 5	14	11	SS-2	-	-	-	-	-	-	-	-	-	4	A-2-4 (V)	
			1050.0	5																	
STIFF, BROWN, SILT, DAMP				6		6 5	14	56	SS-3	-	0	10	8	55	27	28	20	8	11	A-4b (8)	
			1047.5	7																	
LOOSE, ORANGISH BROWN, SANDY SILT, TRACE CLAY, DAMP				8																	
			1046.0	9		4 5	13	67	SS-4	-	0	14	48	31	7	NP	NP	NP	15	A-4a (1)	
				10																	

[illegible]

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\ADMIN\CONTRACT\SU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>		DRILL RIG: <u>'19 CME 75-079-797</u>		STATION / OFFSET: <u>344+04, 55' LT.</u>		EXPLORATION ID <u>P-024-0-20</u>											
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT: _____													
PID: <u>102329</u> SFN: _____		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>8/15/19</u>		ELEVATION: <u>1044.9 (MSL)</u> EOB: <u>10.0 ft.</u>		PAGE 1 OF 1											
START: <u>6/22/20</u> END: <u>6/22/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>83.7</u>		COORD: <u>512947.3800 N, 2242995.1570 E</u>													
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED
		1044.9							GR	CS	FS	SI	CL	LL	PL	PI	WC		
PAVEMENT AND BASE, 12" ASPHALT, 5" GRAVEL BASE		1043.5	1	14															
MEDIUM DENSE, BROWN, SANDY SILT , CONTAINS ROCK FRAGMENTS, DAMP		1041.4	2	10 12	31	44	SS-1	-	0	17	45	28	10	18	17	1	9	A-4a (1)	
MEDIUM DENSE, ORANGISH BROWN, SILT , DAMP		1038.9	3																
			4	9 13 15	39	72	SS-2	-	1	4	16	56	23	21	18	3	13	A-4b (8)	
MEDIUM DENSE, BROWN, SANDY SILT , CONTAINS ROCK FRAGMENTS, DAMP		1036.4	5																
			6	4 6 21	38	72	SS-3	-	13	8	33	28	18	16	15	1	11	A-4a (2)	
MEDIUM DENSE, LIGHT BROWN, COARSE AND FINE SAND , LITTLE GRAVEL, DAMP		1034.9	7																
			8																
			9	4 13 9	31	72	SS-4	-	20	10	56	11	3	NP	NP	NP	5	A-3a (0)	
			10																
EOB																			
NOTES: NONE																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (13208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\13208-114865\ADMIN\CONTRACT\SU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>		DRILL RIG: <u>'19 CME 75-079-797</u>		STATION / OFFSET: <u>347+84, 59' RT.</u>		EXPLORATION ID <u>P-025-0-20</u>												
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT: _____														
PID: <u>102329</u> SFN: _____		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>8/15/19</u>		ELEVATION: <u>1042.7 (MSL)</u> EOB: <u>10.0 ft.</u>		PAGE 1 OF 1												
START: <u>6/25/20</u> END: <u>6/25/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>83.7</u>		COORD: <u>513322.2130 N, 2243106.1760 E</u>														
MATERIAL DESCRIPTION AND NOTES		ELEV. 1042.7	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED	
									GR	CS	FS	SI	CL	LL	PL	PI	WC			
PAVEMENT AND BASE, 11.75" ASPHALT, 5" GRAVEL BASE		1041.3	1	24																
MEDIUM DENSE, BROWN, COARSE AND FINE SAND , TRACE TO LITTLE GRAVEL, @3.5-5.5': LOOSE TO MEDIUM DENSITY, TRACE CLAY AS WELL, DAMP			2	14 12	36	50	SS-1	-	17	30	32	15	6	NP	NP	NP	7	A-3a (0)	<L> <L> <L>	
			3																<L> <L> <L>	
			4	8																<L> <L> <L>
			5	5	14	44	SS-2	-	10	25	35	22	8	14	NP	NP	9	A-3a (0)	<L> <L> <L>	
LOOSE, BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT , TRACE TO LITTLE CLAY, DAMP TO MOIST			6																<L> <L> <L>	
			7	2	3	7	17	SS-3	-	-	-	-	-	-	-	-	7	A-2-4 (V)	<L> <L> <L>	
			8																	<L> <L> <L>
			9	2																<L> <L> <L>
		1032.7	EOB	10	2	6	17	SS-4	-	-	-	-	-	-	-	-	9	A-2-4 (V)	<L> <L> <L>	
NOTES: NONE																				
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																				

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (13208-114865 SUM-77 SR8-11.650.00 - P\22-0033 (13208-114865 SUM-77 SR8-9.650.00)\13208-114865\ADMIN\CONTRACT\SU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>		DRILL RIG: <u>'19 CME 75-079-797</u>		STATION / OFFSET: <u>351+75, 80' RT.</u>		EXPLORATION ID <u>P-027-0-20</u>											
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT: _____													
PID: <u>102329</u> SFN: _____		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>8/15/19</u>		ELEVATION: <u>1039.1 (MSL)</u> EOB: <u>10.0 ft.</u>		PAGE 1 OF 1											
START: <u>6/30/20</u> END: <u>6/30/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>83.7</u>		COORD: <u>513722.2130 N, 2243120.1970 E</u>													
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED
		1039.1							GR	CS	FS	SI	CL	LL	PL	PI	WC		
PAVEMENT AND BASE, 12.25" ASPHALT, 3.75" GRAVEL BASE		1037.8	1	9															
MEDIUM DENSE, BROWN, COARSE AND FINE SAND, SOME CLAY, TRACE TO LITTLE GRAVEL, DAMP			2	12 13	35	50	SS-1	-	33	20	17	15	15	-	-	-	11	A-3a (V)	<V>
		1035.6	3																<V>
MEDIUM DENSE, GREY, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT, DAMP		1034.8	4	11 7	20	44	SS-2A	-	52	17	19	9	3	-	-	-	5	A-2-4 (V)	<V>
STIFF, BROWN, SILTY CLAY, TRACE TO LITTLE SAND, TRACE TO LITTLE GRAVEL, DAMP			5				SS-2B	1.75	-	-	-	-	-	-	-	-	12	A-6b (V)	<V>
			6	5															<V>
			7	5 6	15	44	SS-3	2.25	3	14	33	27	23	25	15	10	14	A-6b (V)	<V>
			8																<V>
			9	5															<V>
		1029.1	10	4 3	10	56	SS-4	3.75	-	-	-	-	-	-	-	-	14	A-6b (V)	<V>
			EOB																<V>
NOTES: NONE																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\ADMIN\CONTRACT\SSU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>		DRILL RIG: <u>'19 CME 75-079-797</u>		STATION / OFFSET: <u>352+04, 72' LT.</u>		EXPLORATION ID <u>P-028-0-20</u>												
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT:														
PID: <u>102329</u> SFN: <u></u>		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>8/15/19</u>		ELEVATION: <u>1039.3 (MSL)</u> EOB: <u>8.7 ft.</u>		PAGE 1 OF 1												
START: <u>6/22/20</u> END: <u>6/22/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>83.7</u>		COORD: <u>513747.3800 N, 2242961.4840 E</u>														
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED
		1039.3								GR	CS	FS	SI	CL	LL	PL	PI	WC		
PAVEMENT AND BASE , 13.75" ASPHALT, 4" GRAVEL BASE		1037.6	1		17				-	-	-	-	-	-	-	-	-	-		
DENSE, BROWN, COARSE AND FINE SAND , TRACE CLAY, TRACE GRAVEL, DAMP		1035.8	2		17	47	61	SS-1	-	27	16	28	17	12	NP	NP	NP	11	A-3a (V)	
			3		17															
STIFF, BROWN, SANDY SILT , TRACE GRAVEL, DAMP			4		2	3	10	56	SS-2	1.25	4	19	37	25	15	20	16	4	14	A-4a (1)
			5		4	4														
			6		4	6														
			7		50/5"	-	76	SS-3	4.00	0	17	24	35	24	22	15	7	13	A-4a (5)	
		1030.8	8																	
SANDSTONE , ORANGISH BROWN, DECOMPOSED TO HIGHLY WEATHERED, VERY WEAK TO WEAK, FINE GRAINED, VERY THIN BEDDED.		1030.6	TR		50/2"	-	100	SS-4	-	-	-	-	-	-	-	-	-	5	Rock (V)	
NOTES: NONE																				
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																				

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\ADMIN\CONTRACT\SSU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>		DRILL RIG: <u>'19 CME 75-079-797</u>		STATION / OFFSET: <u>354+06, 85' LT.</u>		EXPLORATION ID <u>P-029-0-20</u>											
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT:		PAGE 1 OF 1											
PID: <u>102329</u> SFN: <u></u>		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>8/15/19</u>		ELEVATION: <u>1038.0 (MSL)</u> EOB: <u>10.0 ft.</u>													
START: <u>6/22/20</u> END: <u>6/22/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>83.7</u>		COORD: <u>513949.7200 N, 2242949.4840 E</u>													
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED
		1038.0							GR	CS	FS	SI	CL	LL	PL	PI	WC		
PAVEMENT AND BASE, 13" ASPHALT, 5" GRAVEL BASE																			
		1036.5	1	15				-	-	-	-	-	-	-	-	-	-		
MEDIUM DENSE, BROWN, COARSE AND FINE SAND, DAMP			2	12 12	33	56	SS-1	-	8	3	58	15	16	NP	NP	NP	10	A-3a (0)	
		1034.5	3																
HARD, GREY, SANDY SILT, TRACE GRAVEL, DAMP			4	7															
		1032.0	5	9 15	33	39	SS-2	4.00	9	7	24	34	26	23	16	7	11	A-4a (5)	
			6																
MEDIUM DENSE, BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT, TRACE CLAY, DAMP			7	4 6 8	20	44	SS-3	-	32	7	28	17	16	19	16	3	11	A-2-4 (0)	
		1029.5	8																
DENSE TO VERY DENSE, BROWN AND GREY, COARSE AND FINE SAND, TRACE GRAVEL, SAMPLE IS MOTTLED, DAMP			9	5															
		1028.0	10	27 24	71	56	SS-4	-	2	2	61	18	17	17	16	1	9	A-3a (0)	
			EOB																
NOTES: NONE																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\ADMIN\CONTRACT\SU

PROJECT: INTERSTATE 76/77		DRILLING FIRM / OPERATOR: DLZ-AD / REINHART		DRILL RIG: '19 CME 75-079-797		STATION / OFFSET: 355+21, 111' RT.		EXPLORATION ID											
TYPE: ROADWAY		SAMPLING FIRM / LOGGER: DLZ / HUZINEC		HAMMER: CME AUTOMATIC		ALIGNMENT:		P-030-0-20											
PID: 102329 SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 8/15/19		ELEVATION: 1033.1 (MSL) EOB: 8.9 ft.		PAGE											
START: 6/30/20 END: 6/30/20		SAMPLING METHOD: SPT		ENERGY RATIO (%): 83.7		COORD: 514066.9780 N, 2243137.8480 E		1 OF 1											
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED
		1033.1							GR	CS	FS	SI	CL	LL	PL	PI	WC		
PAVEMENT AND BASE, 13.25" ASPHALT, 4" GRAVEL BASE		1031.7	1	12															
MEDIUM DENSE, GREYISH BROWN, COARSE AND FINE SAND, LITTLE GRAVEL, TRACE CLAY, DAMP			2	11	28	44	SS-1	-	48	42	3	5	2	-	-	-	8	A-3a (V)	
		1029.6	3																
MEDIUM DENSE, BROWN, SANDY SILT, TRACE CLAY, DAMP		1029.3	4	6			SS-2A	-	0	26	33	34	7	20	18	2	10	A-4a (1)	
VERY STIFF TO HARD, BROWN, CLAY, TRACE SAND AND GRAVEL, DAMP			5	6	29	33	SS-2B	4.00	-	-	-	-	-	-	-	-	13	A-7-6 (V)	
		1027.1	6	50/5"	-	20	SS-3	-	-	-	-	-	-	-	-	-	6	Rock (V)	
SANDSTONE, BROWN, HIGHLY WEATHERED, WEAK TO MODERATELY STRONG, MEDIUM GRAINED, VERY THIN TO THIN BEDDED, @8.5-8.9': ALSO PORTIONS OF LIGHT GREY TO GREY SANDSTONE, FRIABLE, SAME CHARACTERISTICS OTHERWISE.			7																
		1024.2	8	50/5"	-	20	SS-4	-	-	-	-	-	-	-	-	-	6	Rock (V)	
			EOB																
NOTES: NONE																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\ADMIN\CONTRACT\SU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>		DRILL RIG: <u>'19 CME 75-079-797</u>		STATION / OFFSET: <u>360+05, 53' LT.</u>		EXPLORATION ID <u>P-032-0-20</u>											
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT: _____													
PID: <u>102329</u> SFN: _____		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>8/15/19</u>		ELEVATION: <u>1034.6 (MSL)</u> EOB: <u>8.7 ft.</u>		PAGE 1 OF 1											
START: <u>6/22/20</u> END: <u>6/22/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>83.7</u>		COORD: <u>514547.3800 N, 2242971.2730 E</u>													
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED
		1034.6							GR	CS	FS	SI	CL	LL	PL	PI	WC		
PAVEMENT AND BASE, 12.75" ASPHALT, 4" GRAVEL BASE		1033.2	1	11															
VERY DENSE, BROWN, COARSE AND FINE SAND , CONTAINS ROCK FRAGMENTS, DAMP			2	42 29	99	61	SS-1	-	15	2	56	13	14	NP	NP	NP	9	A-3a (0)	
			3																
			4	6 50/2"	-	100	SS-2	1.75	0	16	49	20	15	20	17	3	12	A-3a (0)	
		1028.6	5																
VERY DENSE, BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT , DAMP			6	50/5"	-	100	SS-3	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)	
			7																
		1026.1	8																
SANDSTONE , BROWN, DECOMPOSED TO HIGHLY WEATHERED, VERY WEAK TO WEAK, FINE GRAINED, VERY THIN BEDDED.		1025.9	TR	50/2"	-	100	SS-4	-	-	-	-	-	-	-	-	-	8	Rock (V)	
NOTES: NONE																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:30 - P:\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\ADMIN\CONTRACT\SU

PROJECT: <u>INTERSTATE 76/77</u>		DRILLING FIRM / OPERATOR: <u>DLZ-AD / REINHART</u>		DRILL RIG: <u>'19 CME 75-079-797</u>		STATION / OFFSET: <u>360+37, 139' RT.</u>		EXPLORATION ID <u>P-033-0-20</u>											
TYPE: <u>ROADWAY</u>		SAMPLING FIRM / LOGGER: <u>DLZ / HUZINEC</u>		HAMMER: <u>CME AUTOMATIC</u>		ALIGNMENT:		PAGE 1 OF 1											
PID: <u>102329</u> SFN: <u></u>		DRILLING METHOD: <u>3.25" HSA</u>		CALIBRATION DATE: <u>8/15/19</u>		ELEVATION: <u>1045.3 (MSL)</u> EOB: <u>10.0 ft.</u>													
START: <u>6/30/20</u> END: <u>6/30/20</u>		SAMPLING METHOD: <u>SPT</u>		ENERGY RATIO (%): <u>83.7</u>		COORD: <u>514584.6860 N, 2243163.7210 E</u>													
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	HOLE SEALED
		1045.3							GR	CS	FS	SI	CL	LL	PL	PI	WC		
PAVEMENT AND BASE , 9" CONCRETE , 7" GRAVEL BASE		1044.0	1	18															
DENSE, GREY, SANDY SILT , DAMP			2	26 19	63	39	SS-1	-	0	27	25	38	10	18	NP	NP	8	A-4a (3)	<V>
		1041.8	3																<V>
VERY STIFF, BROWN AND GREY, SILT AND CLAY , TRACE SAND AND GRAVEL, @6-7.4': LITTLE SAND AND GRAVEL, DAMP			4	6															<V>
			5	5 6	15	44	SS-2	3.25	0	7	18	49	26	27	16	11	16	A-6a (8)	<V>
			6	5															<V>
		1038.0	7	9 10	27	56	SS-3A	3.00	-	-	-	-	-	-	-	-	16	A-6a (V)	<V>
MEDIUM DENSE, ORANGISH BROWN, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT , DAMP		1036.8	8			-	SS-3B	-	-	-	-	-	-	-	-	-	10	A-2-4 (V)	<V>
HARD, BROWN, SILT AND CLAY , TRACE SAND AND GRAVEL, DAMP		1035.3	9	9 6 10	22	44	SS-4	4.00	-	-	-	-	-	-	-	-	16	A-6a (V)	<V>
			10																<V>
			EOB																<V>
NOTES: NONE																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED ASPHALT PATCH; BACKFILLED WITH AUGER CUTTINGS																			

A large, empty rectangular box with a black border, occupying the upper half of the page. It is intended for a drawing or diagram.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: TREMIED BENTONITE GROUT

[illegible]

ABANDONMENT METHODS, MATERIALS, QUANTITIES: TREMIED BENTONITE GROUT

NOTES: NONE
ABANDONMENT METHODS, MATERIALS, QUANTITIES: TREMIED BENTONITE GROUT

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 5/16/24 11:29 - P:\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\ADMIN\CONTRACT\SSU

PID: 102329		SFN:	PROJECT: INTERSTATE 76/77	STATION / OFFSET: 4330+72, 89' LT.		START: 7/9/20	END: 7/9/20	PG 2 OF 2		E3-004-0-20											
MATERIAL DESCRIPTION AND NOTES			ELEV. 1068.3	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED	
										GR	CS	FS	SI	CL	LL	PL	PI				
SHALE , GREYISH BROWN, DECOMPOSED TO HIGHLY WEATHERED, WEAK TO SLIGHTLY STRONG, FINE GRAINED, THIN BEDDED; RQD 0%, REC 39%. <i>(continued)</i>			1062.3	31																	
				32																	
				33																	
				34																	
				35																	
SHALE , GREY, MODERATELY TO HIGHLY WEATHERED, WEAK, FINE GRAINED, LAMINATED TO VERY THIN BEDDED.			1048.3	36	50/5"	-	100	SS-15	-	-	-	-	-	-	-	-	-	-	Rock (V)		
				37																	
				38																	
				39	50/3"	-	100	SS-16	-	-	-	-	-	-	-	-	-	-	-	Rock (V)	
				40																	
				41	50/3"	-	67	SS-17	-	-	-	-	-	-	-	-	-	-	-	Rock (V)	
				42																	
				43																	
				44	50/3"	-	67	SS-18	-	-	-	-	-	-	-	-	-	-	-	Rock (V)	
				45																	
			1048.3	46	50/3"	-	67	SS-19	-	-	-	-	-	-	-	-	-	-	Rock (V)		
				47																	
				48																	
				49	50/2"	-	100	SS-20	-	-	-	-	-	-	-	-	-	-	-	Rock (V)	
				50																	
				EOB																	
NOTES: NONE																					
ABANDONMENT METHODS, MATERIALS, QUANTITIES: TREMIED BENTONITE GROUT																					

APPENDIX C

SUBGRADE ANALYSIS



OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES

Geotechnical Design Manual Section 600

**SUM-077/008-11.65/00.00
114865**

Entire Project

NEAS, Inc.

Prepared By: Zhao Mankoci
Date prepared: Tuesday, August 11, 2026

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che@neasinc.com**

NO. OF BORINGS: 61

NO. OF DCPS:



#	Boring ID	Alignment	Station	Offset	Dir	Add DCP Test Data Worksheets		Boring EL	Proposed Subgrade EL	Cut Fill
						Drill Rig	ER			
1	P-011-0-20	SR8	4326+28	14	LT	19 CME 75 079797	84	1068.8	1067.2	1.6 C
2	P-012-0-20	SR8	4326+11	103	LT	19 CME 75 079797	84	1069.9	1068.3	1.6 C
3	P-013-0-20	SR8	4327+94	44	LT	19 CME 75 079797	84	1072.2	1070.6	1.6 C
4	P-014-0-20	SR8	4329+20	29	RT	19 CME 75 079797	84	1075.0	1073.4	1.6 C
5	P-015-0-20	SR8	327+44	15	RT	19 CME 75 079797	84	1077.7	1076.1	1.6 C
6	P-016-0-20	SR8	328+48	46	LT	19 CME 75 079797	84	1076.9	1075.3	1.6 C
7	P-017-0-20	SR8	331+44	6	RT	19 CME 75 079797	84	1070.3	1068.7	1.6 C
8	P-018-0-20	SR8	332+50	7	LT	19 CME 75 079797	84	1067.0	1065.4	1.6 C
9	R-002-0-99	SR8	333+35	43	LT	CME 55T	63	1064.0	1062.5	1.5 C
10	B-003-0-22	SR8	333+93	48	LT	CME 55T	63	1063.0	1061.5	1.5 C
11	P-019-0-20	SR8	334+68	63	RT	19 CME 75 079797	84	1060.5	1058.9	1.6 C
12	B-005-0-22	SR8	336+46	58	LT	CME 55T	63	1054.8	1053.9	0.9 C
13	P-020-0-20	SR8	336+51	54	LT	19 CME 75 079797	84	1056.0	1054.4	1.6 C
14	P-022-0-20	SR8	340+03	41	LT	19 CME 75 079797	84	1048.3	1046.7	1.6 C
15	R-004-0-99	SR8	340+90	17	LT	CME 55T	63	1046.8	1045.3	1.5 C
16	P-024-0-20	SR8	344+03	59	LT	19 CME 75 079797	84	1044.9	1043.3	1.6 C
17	R-005-0-99	SR8	345+25	52	RT	CME 55T	63	1045.0	1043.5	1.5 C
18	P-025-0-20	SR8	347+76	59	RT	19 CME 75 079797	84	1042.7	1041.1	1.6 C
19	R-006-0-99	SR8	349+25	44	LT	CME 55T	63	1038.6	1040.6	2.0 F
20	P-027-0-20	SR8	351+76	80	RT	19 CME 75 079797	84	1039.1	1039.0	0.1 C
21	P-028-0-20	SR8	352+04	78	LT	19 CME 75 079797	84	1039.3	1038.7	0.6 C
22	R-008-0-99	SR8	353+24	10	RT	CME 55T	63	1037.3	1037.8	0.5 F
23	B-044-0-24	SR8	353+95	53	LT	CME 55TB	89	1038.5	1037.5	1.0 C
24	P-029-0-20	SR8	354+06	85	LT	19 CME 75 079797	84	1038.0	1037.4	0.6 C
25	P-030-0-20	SR8	355+20	104	RT	19 CME 75 079797	84	1033.1	1036.5	3.4 F
26	R-010-0-99	SR8	356+85	17	LT	CME 55T	63	1036.4	1034.9	1.5 C
27	B-006-0-22	SR8	357+15	56	RT	CME 55T	63	1036.1	1035.3	0.7 C
28	P-032-0-20	SR8	360+03	54	LT	19 CME 75 079797	84	1034.6	1033.0	1.6 C
29	R-012-0-99	SR8	360+76	38	RT	CME 55T	63	1035.0	1033.5	1.5 C
30	B-007-0-22	SR8	364+72	55	RT	CME 55T	63	1036.0	1035.3	0.7 C
31	B-008-0-22	SR8	368+33	54	LT	CME 55T	63	1041.8	1040.9	0.9 C
32	B-009-0-22	SR8	372+71	55	RT	CME 55T	63	1040.3	1039.5	0.8 C
33	B-018-0-22	SR8	373+97	98	LT	CME 55T	63	1038.0	1037.3	0.7 C
34	B-010-0-22	SR8	375+28	78	RT	CME 55T	63	1034.1	1034.5	0.4 F
35	B-011-0-22	SR8	376+76	72	LT	CME 55T	63	1030.7	1030.2	0.5 C
36	B-012-0-22	SR8	384+22	75	LT	CME 55T	63	1026.4	1026.8	0.4 F
37	B-019-0-22	SR8	387+43	102	LT	CME 55T	63	1031.7	1028.8	2.9 C
38	B-020-0-22	SR8	387+52	81	RT	CME 55T	63	1030.5	1028.9	1.6 C
39	B-013-0-22	SR8	390+97	57	LT	CME 55T	63	1031.4	1031.1	0.3 C
40	B-057-0-24	SR8	365+99	49	LT	CME 55TB	89	1039.3	1037.9	1.4 C
41	E3-001-0-20	Exit Ramp-I-76	102+32	23	LT	CME 55T	63	1089.6	1068.3	21.3 C
42	E3-002-0-20	Exit Ramp-I-76	103+59	18	LT	CME 55T	63	1091.7	1071.9	19.8 C
43	E3-003-0-20	Exit Ramp-I-76	104+82	19	LT	CME 55T	63	1093.2	1076.1	17.1 C
44	E3-004-0-20	Exit Ramp-I-76	106+33	13	LT	CME 55T	63	1098.3	1076.2	22.1 C



#	Boring ID	Alignment	Station	Offset	Dir	Add DCP Test Data Worksheets		Boring EL.	Proposed Subgrade EL.	Cut Fill
						Drill Rig	ER			
45	B-001-0-22	Exit Ramp-I-76	108+46	23	LT	CME 55T	63	1093.5	1076.3	17.2 C
46	B-002-0-22	Exit Ramp-I-76	109+81	16	LT	CME 55T	63	1090.6	1074.8	15.8 C
47	B-030-0-24	Exit Ramp-I-76	111+50	21	RT	CME 55T	63	1071.3	1069.8	1.5 C
48	B-031-0-24	Exit Ramp-I-76	113+29	21	RT	CME 55T	71	1066.7	1065.1	1.6 C
49	B-042-0-24	Exit Ramp-I-76	132+08	5	RT	CME 55TB	89	1037.0	1038.7	1.7 F
50	B-004-0-99	Exit Ramp-I-76	132+84	12	LT	CME 55T	63	1046.8	1038.0	8.8 C
51	R-009-0-99	Exit Ramp-I-76	134+72	13	LT	CME 55T	63	1039.0	1036.3	2.7 C
52	B-050-0-24	Exit Ramp-I-76	134+82	7	RT	CME 55TB	89	1038.1	1036.2	1.9 C
53	B-051-0-24	Exit Ramp-I-76	136+01	3	LT	CME 55TB	89	1038.7	1035.4	3.3 C
54	B-023-0-24	Exit Ramp-I-76	137+34	15	LT	CME 55TB	89	1043.3	1034.4	8.9 C
55	B-053-0-24	Exit Ramp-I-76	140+00	10	RT	CME 55TB	89	1034.5	1033.1	1.4 C
56	B-028-0-24	Exit Ramp-I-76	142+85	53	RT	CME 55TB	89	1033.8	1032.3	1.5 C
57	B-055-0-24	Exit Ramp-I-76	144+00	3	RT	CME 55TB	89	1034.4	1032.8	1.6 C
58	B-006-0-99	Goodkirk st	209+22	10	LT	CME 55T	63	1043.6	1042.1	1.5 C
59	R-011-0-99	Goodkirk st	210+38	10	LT	CME 55T	63	1047.3	1045.8	1.5 C
60	R-013-0-99	Goodkirk st	214+43	10	LT	CME 55T	63	1057.3	1055.3	2.0 C
61	B-008-0-99	Goodkirk st	214+76	5	RT	CME 55T	63	1057.4	1055.9	1.5 C



Add DCP Test Data Worksheets								Boring EL.	Proposed Subgrade EL.	Cut Fill
#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER			

#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics					Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)	
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class		GI	Unsuitable	Unstable	Unsuitable		Unstable
1	P 011-0 20	SS-1	1.0	2.5	-0.6	0.9	20	20		21	14	7	5	12	17	10	10	A-2-4	0						
		SS-2	3.5	4.3	1.9	2.7	50			20	NP	NP	34	8	42	6	0	Rock	0						
		SS-3	6.0	6.6	4.4	5.0	50								5	0	Rock	0							
		SS-4	8.5	9.1	6.9	7.5	50								6	0	Rock								
2	P 012-0 20	SS-1	1.2	2.7	-0.4	1.1	17	17	3.50	24	15	9	39	29	68	16	10	A-4a	7			Mc			
		SS-2	3.5	4.9	1.9	3.3	50			17	NP	NP	26	11	37	9	11	A-4a	0						
		SS-3	6.0	7.5	4.4	5.9	50			19	NP	NP	20	13	33	12	8	A-3a	0						
		SS-4	8.5	10.0	6.9	8.4	50								6	18	A-7-6								
3	P 013-0 20	SS-1	1.0	2.5	-0.6	0.9	15	15	4.00	24	16	8	40	11	51	14	11	A-4a	3			Mc			
		SS-2	3.5	4.1	1.9	2.5	50			28	17	11	35	10	45	6	0	Rock	0						
		SS-3	6.0	6.4	4.4	4.8	50								6	0	Rock	0							
		SS-4	8.5	8.9	6.9	7.3	50								7	0	Rock								
4	P 014-0 20	SS-1	1.1	2.6	-0.5	1.0	50	30		NP	NP	NP	13	14	27	12	10	A-2-4	0						
		SS-2	3.5	4.4	1.9	2.8	50			28	22	6	41	13	54	7	0	Rock	0						
		SS-3	6.0	6.4	4.4	4.8	50								5	0	Rock	0							
		SS-4	8.5	9.2	6.9	7.6	50								4	0	Rock								
5	P 015-0 20	SS-1	1.0	2.1	-0.6	0.5	50			24	19	5	29	11	40	4	0	Rock	0		Rock		6"		
		SS-2	3.5	3.9	1.9	2.3	50						27	10	37	5	0	Rock	0						
		SS-3	6.0	6.1	4.4	4.5	50								4	0	Rock	0							
		SS-4	8.5	8.8	6.9	7.2	50								4	0	Rock								
6	P 016-0 20	SS-1	1.2	2.7	-0.4	1.1	31	30	4.00	22	17	5	44	22	66	10	12	A-4a	6						
		SS-2	3.5	4.4	1.9	2.8	50			25	22	3	65	21	86	6	0	Rock	0						
		SS-3	6.0	6.9	4.4	5.3	50								6	0	Rock	0							
		SS-4	8.5	8.9	6.9	7.3	50								5	0	Rock								
7	P 017-0 20	SS-1	1.1	2.6	-0.5	1.0	43	30	4.00	25	18	7	52	34	86	13	13	A-4b	8		A-4b		23"		
		SS-2	3.5	4.9	1.9	3.3	50		4.00	27	14	13	39	25	64	11	14	A-6a	7						
		SS-3	6.0	6.9	4.4	5.3	50								8	0	Rock	0							
		SS-4	8.5	9.3	6.9	7.7	50								6	0	Rock								
8	P 018-0 20	SS-1	1.0	2.5	-0.6	0.9	11	11	3.25	NP	NP	NP	4	9	13	8	6	A-1-b	0	53					
		SS-2	3.5	5.0	1.9	3.4	15			17	15	2	19	13	32	11	10	A-2-4	0						
		SS-3	6.0	7.5	4.4	5.9	42			21	14	7	26	35	61	11	10	A-4a	5						
		SS-4	8.5	10.0	6.9	8.4	67								9	0	Rock								
9	R 002-0 99	SS-1	0.8	3.5	-0.7	2.0	17			25	17	8	24	11	35	12	10	A-2-4	0	133					
		SS-2	3.5	5.8	2.0	4.3	20									14	A-6a	10							
		SS-3	5.8	8.5	4.3	7.0	16									14	A-6a	10							



#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
		SS-4	8.5	10.0	7.0	8.5	38	16									0	Rock							
10	B 003-0 22	SS-1	1.5	3.0	0.0	1.5	29	12		NP	NP	NP	14	4	18	4	6	A-1-b	0						
		SS-2	3.0	4.5	1.5	3.0	13			23	15	8	25	11	36	10	10	A-4a	0						
		SS-3	4.5	6.0	3.0	4.5	13									11	10	A-4a	8						
		SS-4	6.0	7.5	4.5	6.0	12									11	6	A-1-b	0						
11	P 019-0 20	SS-1	1.0	2.5	-0.6	0.9	27	6		NP	NP	NP	21	16	37	8	11	A-4a	0	0					
		SS-2	3.5	5.0	1.9	3.4	8		1.25	21	16	5	30	16	46	12	11	A-4a	2			HP			
		SS-3	6.0	7.5	4.4	5.9	6			20	NP	NP	23	12	35	11	8	A-3a	0						
		SS-4	8.5	10.0	6.9	8.4	7									13	10	A-2-4							
12	B 005-0 22	SS-1	1.5	2.3	0.6	1.4	50	19		NP	NP	NP	19	5	24	4	6	A-1-b	0						
		SS-2	3.0	4.5	2.1	3.6	39									10	6	A-1-b	0						
		SS-3	4.5	6.0	3.6	5.1	19			NP	NP	NP	36	12	48	11	11	A-4a	3						
		SS-4	6.0	7.5	5.1	6.6	24									11	6	A-1-b							
13	P 020-0 20	SS-1	1.0	2.5	-0.6	0.9	29	14					18	12	30	11	10	A-4a	3						
		SS-2	3.5	5.0	1.9	3.4	14									4	10	A-2-4	0						
		SS-3	6.0	7.5	4.4	5.9	14			28	20	8	55	27	82	11	15	A-4b	8						
		SS-4	8.5	10.0	6.9	8.4	13			NP	NP	NP	31	7	38	15	11	A-4a							
14	P 022-0 20	SS-1	1.2	2.7	-0.4	1.1	36	30		18	15	3	28	19	47	13	10	A-4a	2			Mc			
		SS-2	3.5	5.0	1.9	3.4	35			17	15	2	39	16	55	11	10	A-4a	4						
		SS-3	6.0	7.5	4.4	5.9	45			16	14	2	32	12	44	10	10	A-4a	2						
		SS-4	8.5	10.0	6.9	8.4	15			15	NP	NP	28	14	42	11	11	A-4a							
15	R 004-0 99	SS-1	1.0	3.5	-0.5	2.0	44	30									8	A-3a	0						
		SS-2	3.5	5.6	2.0	4.1	51			30	15	15	48	15	63	12	14	A-6a	8						
		SS-3	5.6	7.7	4.1	6.2	40										14	A-6a	10						
		SS-4	7.7	10.0	6.2	8.5	32										14	A-6a							
16	P 024-0 20	SS-1	1.0	2.5	-0.6	0.9	31	30		18	17	1	28	10	38	9	12	A-4a	1						
		SS-2	3.5	5.0	1.9	3.4	39			21	18	3	56	23	79	13	13	A-4b	8			A-4b		36"	
		SS-3	6.0	7.5	4.4	5.9	38			16	15	1	28	18	46	11	10	A-4a	2						
		SS-4	8.5	10.0	6.9	8.4	31			NP	NP	NP	11	3	14	5	8	A-3a							
17	R 005-0 99	SS-1	0.0	3.5	-1.5	2.0	28	27								14	10	A-4a	8			Mc			
		SS-2	3.5	5.8	2.0	4.3	27										8	A-3a	0						
		SS-3	5.8	8.5	4.3	7.0	33										10	A-4a	8						
		SS-4	8.5	10.0	7.0	8.5	31										10	A-4a							
18	P 025-0	SS-1	1.0	2.5	-0.6	0.9	14			NP	NP	NP	15	6	21	7	8	A-3a	0						
		SS-2	3.5	5.0	1.9	3.4	7			14	NP	NP	22	8	30	9	8	A-3a	0						

#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
	20	SS-3	6.0	7.5	4.4	5.9	6	6								7	10	A-2-4	0						
		SS-4	8.5	10.0	6.9	8.4	35									9	10	A-2-4							
19	R 006-0 99	SS-1	0.0	3.5	2.0	5.5	10	10									10	A-4a	8						
		SS-2	3.5	5.8	5.5	7.8	51																		
		SS-3	5.8	8.5	7.8	10.5	45										10	A-4a							
		SS-4	8.5	10.0	10.5	12.0	64																		
20	P 027-0 20	SS-1	1.0	2.5	0.9	2.4	20	10					15	15	30	11	8	A-3a	0						
		SS-2	3.5	5.0	3.4	4.9	15		1.75				9	3	12	12	16	A-6b	0						
		SS-3	6.0	7.5	5.9	7.4	10		2.25	25	15	10	27	23	50	14	16	A-6b							
		SS-4	8.5	10.0	8.4	9.9	47		3.75							14	16	A-6b							
21	P 028-0 20	SS-1	1.2	2.7	0.6	2.1	10	10		NP	NP	NP	17	12	29	11	8	A-3a	0						
		SS-2	3.5	5.0	2.9	4.4	50		1.25	20	16	4	25	15	40	14	11	A-4a	1						
		SS-3	6.0	7.4	5.4	6.8	50		4.00	22	15	7	35	24	59	13	10	A-4a							
		SS-4	8.5	8.7	7.9	8.1	33									5	0	Rock							
22	R 008-0 99	SS-1	0.9	2.5	1.4	3.0	86	30		21	13	8	26	16	42	11	10	A-4a	1						
		SS-2	2.5	6.0	3.0	6.5	50										10	A-4a	8						
		SS-3	6.0	7.6	6.5	8.1	50										14	A-6a							
23	B 044-0 24	SS-1	2.5	3.1	1.5	2.1	50	30		NP	NP	NP	16	6	22	5	8	A-3a	0						
		SS-2	5.0	5.1	4.0	4.1	50									7	0	Rock	0						
24	P 029-0 20	SS-1	1.1	2.6	0.5	2.0	33	20		NP	NP	NP	15	16	31	10	8	A-3a	0						
		SS-2	3.5	5.0	2.9	4.4	20		4.00	23	16	7	34	26	60	11	11	A-4a	5						
		SS-3	6.0	7.5	5.4	6.9	71			19	16	3	17	16	33	11	10	A-2-4							
		SS-4	8.5	10.0	7.9	9.4	28			17	16	1	18	17	35	9	8	A-3a							
25	P 030-0 20	SS-1	1.0	2.5	4.4	5.9	29	29					5	2	7	8	8	A-3a	0						
		SS-2	3.5	5.0	6.9	8.4	50		4.00	20	18	2	34	7	41	13	18	A-7-6							
		SS-3	6.0	6.4	9.4	9.8	50									6	0	Rock							
		SS-4	8.5	8.9	11.9	12.3	99									6	0	Rock							
26	R 010-0 99	SS-1	1.5	3.0	0.0	1.5	50	30		NP	NP	NP	30		30	8	10	A-2-4	0						
		SS-2	3.0	4.5	1.5	3.0	50										10	A-2-4	0						
		SS-3	4.5	6.0	3.0	4.5	50										10	A-2-4	0						
		SS-4	6.0	8.8	4.5	7.3	50										0	Rock	0						
27	B	SS-1	1.5	2.2	0.8	1.4	50			NP	NP	NP	20	6	26	8	8	A-3a	0	0					



6/5/2026

[illegible]



6/5/2026

#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _c	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
36	B 012-0	SS-1	1.5	2.3	1.9	2.7	30	19		NP	NP	NP	17	5	22	7	6	A-1-b	0	133					
		SS-2	3.0	4.5	3.4	4.9	19			NP	NP	NP	20	5	25	7	6	A-1-b	0						
	22	SS-3	4.5	6.0	4.9	6.4	29								8	6	A-1-b	0							
		SS-4	6.0	7.5	6.4	7.9	27								8	6	A-1-b								
37	B 019-0	SS-1	1.5	3.0	-1.4	0.1	33	16		NP	NP	NP	10	2	12	5	8	A-3a	0	0					
		SS-2	3.0	4.5	0.1	1.6	16								8	8	A-3a	0							
	22	SS-3	4.5	5.4	1.6	2.5	50								10	0	Rock	0							
		SS-4	6.0	6.2	3.1	3.3	50								4	0	Rock	0							
38	B 020-0	SS-1	1.5	2.2	-0.1	0.6	50									7	0	Rock	0		Rock		6"		
		SS-2	3.0	3.4	1.4	1.8	50								6	0	Rock	0							
	22	SS-3	4.5	5.0	2.9	3.4	50								5	0	Rock	0							
		SS-4	6.0	6.2	4.4	4.6	50								5	0	Rock	0							
39	B 013-0	SS-1	1.5	1.9	1.2	1.6	50									6	0	Rock	0		Rock				
		SS-2	3.0	3.3	2.7	2.9	50								4	0	Rock	0							
	22	SS-3	4.5	4.8	4.2	4.4	50								5	0	Rock	0							
		SS-4	6.0	6.1	5.7	5.8	50									0	Rock								
40	B 057-0	SS-1	2.5	4.0	1.1	2.6	108	30		NP	NP	NP	17	7	24	7	10	A-2-4	0						
		SS-2	5.0	5.2	3.6	3.8	50								2	0	Rock	0							
	24																								
41	E -001-	SS-10	23.5	23.8	2.2	2.5	50										0	Rock	0		Rock				
		SS-11	26.0	26.4	4.7	5.1	50									0	Rock	0							
	-2	SS-12	28.5	29.4	7.2	8.1	50									0	Rock								
		SS-13	31.0	31.4	9.7	10.1	50									0	Rock								
42	E -002-	SS-9	21.0	21.4	1.2	1.6	50										0	Rock	0		Rock				
		SS-10	23.5	23.9	3.7	4.1	50									0	Rock	0							
	-2	SS-11	26.0	26.3	6.2	6.5	50									0	Rock								
		SS-12	28.5	29.3	8.7	9.5	50									0	Rock								
43	E -003-	SS-8	18.5	20.0	1.4	2.9	81										0	Rock	0		Rock				
		SS-9	21.0	21.4	3.9	4.3	50									0	Rock	0							
	-2	SS-10	23.5	23.9	6.4	6.8	50									0	Rock								
		SS-11	26.0	26.4	8.9	9.3	50									0	Rock								
44	E -004-	RC-2	16.0	23.0	-6.1	0.9											0	Rock	0		Rock		6"		
		RC-3	23.0	26.0	0.9	3.9																			
	-2																								



#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
45	B	NQ2-2	15.7	25.7	-1.5	8.5											0	Rock	0		Rock		6"		
	001-0																								
	22																								
46	B	NQ2-2	10.8	20.8	-5.0	5.0											0	Rock	0		Rock		6"		
	002-0																								
	22																								
47	B	SS-1	2.5	2.7	1.0	1.2	50			NP	NP	NP	11	6	17	3	6	A-1-b	0						
	030-0	SS-2	5.0	5.4	3.5	3.9	50									6	6	A-1-b	0						
	24	SS-3	7.5	8.1	6.0	6.6	50									5	0	Rock							
		SS-4	10.0	10.8	8.5	9.3	50	30								5	0	Rock							
48	B	SS-1	2.5	4.0	0.9	2.4	9			27	18	9	41	21	62	14	13	A-4a	5			N ₆₀		12"	
	031-0	SS-2	5.0	6.5	3.4	4.9	20			26	17	9	37	22	59	12	12	A-4a	5						
	24	SS-3	7.5	9.0	5.9	7.4	20									14	10	A-4a							
		SS-4	10.0	11.5	8.4	9.9	5	9								17	14	A-6a							
49	B	SS-1	2.5	4.0	4.2	5.7	28			NP	NP	NP	22	9	31	9	10	A-2-4	0						
	042-0	SS-2	5.0	5.9	6.7	7.6	50									9	10	A-2-4							
	24	SS-3	7.5	9.0	9.2	10.7	89		4.50							11	10	A-4a							
		SS-4	10.0	10.3	11.7	12.0	50	28								6	0	Rock							
50	B	SS-4	8.0	9.5	-0.8	0.7	50										0	Rock	0		Rock		6"		
	004-0	SS-5	9.5	15.0	0.7	6.2											0	Rock	0						
	99																								
51	R	SS-1	1.5	3.0	-1.2	0.3	50																		
	009-0	SS-2	3.0	4.5	0.3	1.8	50																		
	99	SS-3	4.5	6.0	1.8	3.3	50										10	A-4a	8						
		SS-4	6.0	8.7	3.3	6.0	50	30									10	A-4a	8						
52	B	SS-1	2.5	4.0	0.6	2.1	71			25	17	8	34	15	49	8	12	A-4a	3						
	050-0	SS-2	5.0	6.5	3.1	4.6	62									8	6	A-1-b	0						
	24	SS-3	7.5	8.1	5.6	6.2	50									9	10	A-2-4							
								30									0	Rock							
53	B	SS-1	2.5	3.1	-0.8	-0.2	50			NP	NP	NP	18	10	28	6	10	A-2-4	0						
	051-0	SS-2	5.0	5.6	1.7	2.3	50									8	0	Rock	0						



#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
	24	SS-3	7.5	7.7	4.2	4.4	50	30							7	0	Rock	0							
54	B	SS-3	7.5	7.9	-1.4	-1.0	50									0	Rock	0							
	023-0	SS-4	10.0	11.6	1.1	2.7	50									0	Rock	0							
	24	SS-5	12.5	12.6	3.6	3.7	50									0	Rock	0							
55	B	SS-1A	2.5	3.0	1.1	1.6	50			NP	NP	NP	15	7	22	9	8	A-3a	0						
	053-0	SS-1B	3.0	3.4	1.6	2.0	50									9	0	Rock	0						
	24	SS-2	5.0	5.1	3.6	3.7	50									6	0	Rock	0						
56	B	SS-1	2.5	2.8	1.0	1.3	50									3	0	Rock	0		Rock				
	028-0	SS-2	5.0	5.2	3.5	3.7	50									7	0	Rock	0						
	24																								
57	B	SS-1	2.5	2.7	0.9	1.1	50									3	0	Rock	0		Rock				
	055-0	SS-2	5.0	5.1	3.4	3.5	50									8	0	Rock	0						
	24																								
58	B	SS-1	1.0	2.5	-0.5	1.0	22										10	A-2-4	0						
	006-0	SS-2	2.5	5.5	1.0	4.0	23			NP	NP	NP				11	10	A-2-4	0						
	99	SS-3	5.5	8.0	4.0	6.5	50										0	Rock	0						
		SS-4	8.0	8.8	6.5	7.3	50										0	Rock							
59	R	SS-1	0.7	3.5	-0.8	2.0	17			18	14	4	39	13	52	14	10	A-4a	3			Mc			
	011-0	SS-2	3.5	6.0	2.0	4.5	22										10	A-4a	8						
	99	SS-3	6.0	7.5	4.5	6.0	50										0	Rock	0						
		SS-4	7.8	8.8	6.3	7.3	50										0	Rock							
60	R	SS-1	0.8	3.5	-1.2	1.5	7			20	15	5	27	12	39	15	10	A-4a	1			Mc			
	013-0	SS-2	3.5	6.0	1.5	4.0	7										10	A-4a	8						
	99	SS-3	6.0	7.5	4.0	5.5	63										0	Rock	0						
		SS-4	7.5	8.6	5.5	6.6	50										0	Rock							
61	B	SS-1	1.0	3.0	-0.5	1.5	9										10	A-4a	8			N ₆₀		12"	
	008-0	SS-2	3.0	5.5	1.5	4.0	7			27	12	15	31	13	44	17	14	A-6a	3						
	99	SS-3	5.5	8.0	4.0	6.5	22										0	Rock	0						
		SS-4	8.0	10.5	6.5	9.0	50										0	Rock							

PID: 114865

###

County-Route-Section: SUM-077/008-11.65/00.00

No. of Borings: 61

Geotechnical Consultant: NEAS, Inc.

Prepared By: Zhao Mankoci

Date prepared: 8/11/2026

Chemical Stabilization Options		
320	Rubblize & Roll	Option
206	Cement Stabilization	Option
	Lime Stabilization	No
206	Depth	14"

Excavate and Replace Stabilization Options	
Global Geotextile Average(N60L):	0"
Average(HP):	0"
Global Geogrid Average(N60L):	0"
Average(HP):	0"

Design CBR	11
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% Samples within 3 feet of subgrade			
N ₆₀ ≤ 5	0%	HP ≤ 0.5	0%
N ₆₀ < 12	6%	0.5 < HP ≤ 1	0%
12 ≤ N ₆₀ < 15	3%	1 < HP ≤ 2	2%
N ₆₀ ≥ 20	45%	HP > 2	4%
M+	3%		
Rock	33%		
Unsuitable Soil	2%		

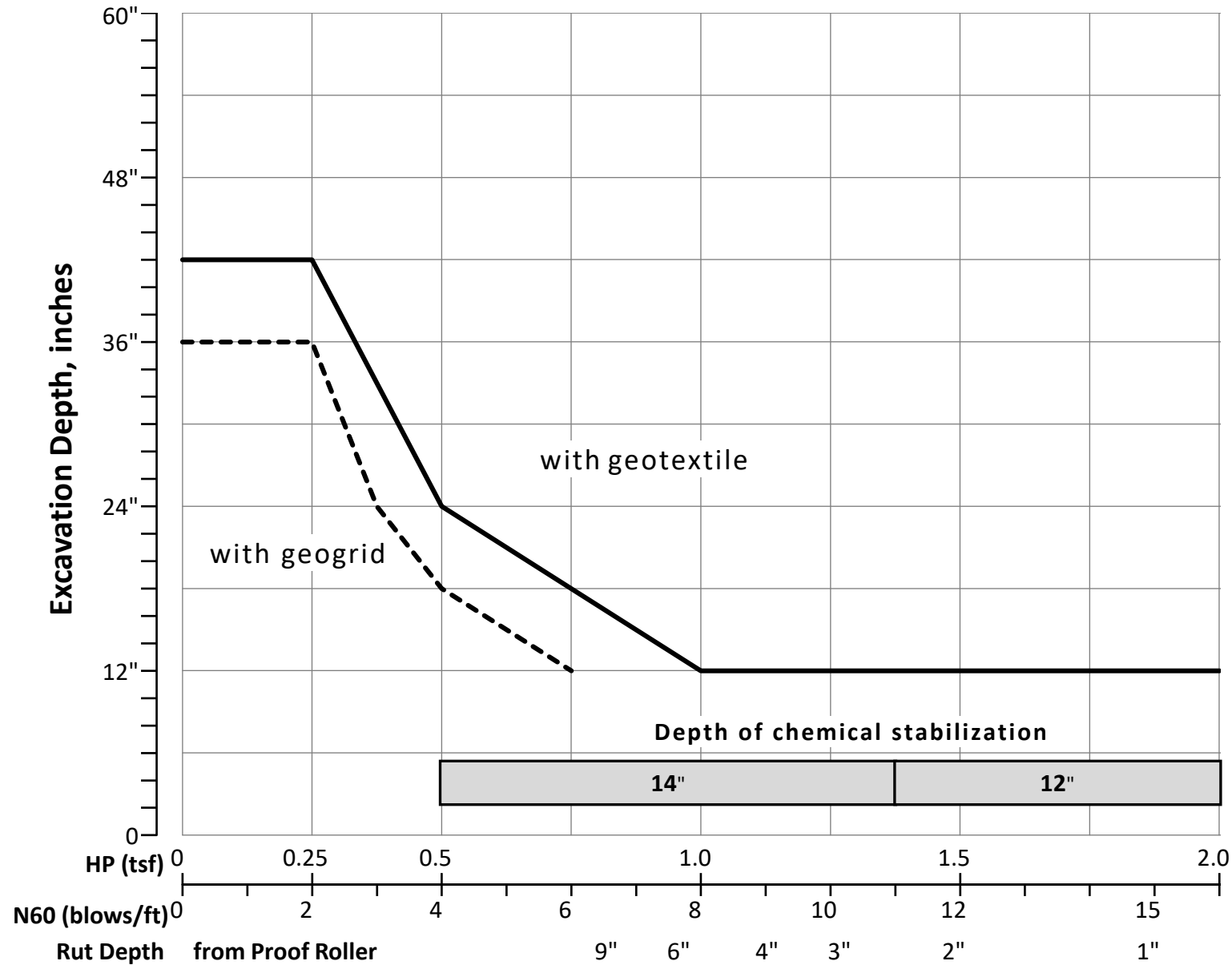
Excavate and Replace at Surface	
Average	0"
Maximum	0"
Minimum	0"

% Proposed Subgrade Surface	
Unstable & Unsuitable	27%
Unstable	9%
Unsuitable (Soil & Rock)	17%

	N ₆₀	N _{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M _C	M _{OPT}	GI
Average	41	22	3.20	22	16	6	26	13	39	9	6	1
Maximum	108	30	4.50	30	22	15	65	35	86	17	18	10
Minimum	5	6	1.25	14	12	1	4	2	7	2	0	0

Classification Counts by Sample																					
ODOT Class	UCF	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals	
Count	0	94	0	14	20	0	0	0	0	25	41	3	0	9	3	0	2	0	0	211	
Percent	0%	45%	0%	7%	9%	0%	0%	0%	0%	12%	19%	1%	0%	4%	1%	0%	1%	0%	0%	100%	
% Rock Granular Cohesive	0%	45%	47%									8%									100%
Surface Class Count	0	32	0	7	12	0	0	0	0	19	22	2	0	4	0	0	0	0	0	98	
Surface Class Percent	0%	33%	0%	7%	12%	0%	0%	0%	0%	19%	22%	2%	0%	4%	0%	0%	0%	0%	0%	100%	

Fig. 600-1 – Subgrade Stabilization



VERRIDE TABLE

Calculated Average	New Values	Check to Override
3.20	0.50	☐ HP
21.96	7.00	☐ N60L

Average HP



Average N_{60L}





OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES

Geotechnical Design Manual Section 600

**SUM-077/008-11.65/00.00
114865**

SR-8

NEAS, Inc.

Prepared By: Zhao Mankoci
Date prepared: Tuesday, August 11, 2026

**Chunmei (Melinda) He, Ph.D., P.E.
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Suite 240
Columbus, OH 43231
614.714.0299 Ext 111
che@neasinc.com**

NO. OF BORINGS: 42

NO. OF DCPS:



#	Boring ID	Alignment	Station	Offset	Dir	Add DCP Test Data Worksheets		Boring EL.	Proposed Subgrade EL.	Cut Fill
						Drill Rig	ER			
1	P-011-0-20	SR8	4326+28	14	LT	19 CME 75 079797	84	1068.8	1067.2	1.6 C
2	P-012-0-20	SR8	4326+11	103	LT	19 CME 75 079797	84	1069.9	1068.3	1.6 C
3	P-013-0-20	SR8	4327+94	44	LT	19 CME 75 079797	84	1072.2	1070.6	1.6 C
4	P-014-0-20	SR8	4329+20	29	RT	19 CME 75 079797	84	1075.0	1073.4	1.6 C
5	P-015-0-20	SR8	327+44	15	RT	19 CME 75 079797	84	1077.7	1076.1	1.6 C
6	P-016-0-20	SR8	328+48	46	LT	19 CME 75 079797	84	1076.9	1075.3	1.6 C
7	P-017-0-20	SR8	331+44	6	RT	19 CME 75 079797	84	1070.3	1068.7	1.6 C
8	P-018-0-20	SR8	332+50	7	LT	19 CME 75 079797	84	1067.0	1065.4	1.6 C
9	R-002-0-99	SR8	333+35	43	LT	CME 55T	63	1064.0	1062.5	1.5 C
10	B-003-0-22	SR8	333+93	48	LT	CME 55T	63	1063.0	1061.5	1.5 C
11	P-019-0-20	SR8	334+68	63	RT	19 CME 75 079797	84	1060.5	1058.9	1.6 C
12	B-005-0-22	SR8	336+46	58	LT	CME 55T	63	1054.8	1053.9	0.9 C
13	P-020-0-20	SR8	336+51	54	LT	19 CME 75 079797	84	1056.0	1054.4	1.6 C
14	P-022-0-20	SR8	340+03	41	LT	19 CME 75 079797	84	1048.3	1046.7	1.6 C
15	R-004-0-99	SR8	340+90	17	LT	CME 55T	63	1046.8	1045.3	1.5 C
16	P-024-0-20	SR8	344+03	59	LT	19 CME 75 079797	84	1044.9	1043.3	1.6 C
17	R-005-0-99	SR8	345+25	52	RT	CME 55T	63	1045.0	1043.5	1.5 C
18	P-025-0-20	SR8	347+76	59	RT	19 CME 75 079797	84	1042.7	1041.1	1.6 C
19	R-006-0-99	SR8	349+25	44	LT	CME 55T	63	1038.6	1040.6	2.0 F
20	P-027-0-20	SR8	351+76	80	RT	19 CME 75 079797	84	1039.1	1039.0	0.1 C
21	P-028-0-20	SR8	352+04	78	LT	19 CME 75 079797	84	1039.3	1038.7	0.6 C
22	R-008-0-99	SR8	353+24	10	RT	CME 55T	63	1037.3	1037.8	0.5 F
23	B-044-0-24	SR8	353+95	53	LT	CME 55TB	89	1038.5	1037.5	1.0 C
24	P-029-0-20	SR8	354+06	85	LT	19 CME 75 079797	84	1038.0	1037.4	0.6 C
25	P-030-0-20	SR8	355+20	104	RT	19 CME 75 079797	84	1033.1	1036.5	3.4 F
26	R-010-0-99	SR8	356+85	17	LT	CME 55T	63	1036.4	1034.9	1.5 C
27	B-006-0-22	SR8	357+15	56	RT	CME 55T	63	1036.1	1035.3	0.7 C
28	P-032-0-20	SR8	360+03	54	LT	19 CME 75 079797	84	1034.6	1033.0	1.6 C
29	R-012-0-99	SR8	360+76	38	RT	CME 55T	63	1035.0	1033.5	1.5 C
30	B-028-0-24	SR8	361+87	5	LT	CME 55TB	89	1033.8	1032.5	1.3 C
31	B-055-0-24	SR8	363+04	50	LT	CME 55TB	89	1034.4	1032.9	1.5 C
32	B-057-0-24	SR8	365+99	49	LT	CME 55TB	89	1039.3	1037.9	1.4 C
33	B-007-0-22	SR8	364+72	55	RT	CME 55T	63	1036.0	1035.3	0.7 C
34	B-008-0-22	SR8	368+33	54	LT	CME 55T	63	1041.8	1040.9	0.9 C
35	B-009-0-22	SR8	372+71	55	RT	CME 55T	63	1040.3	1039.5	0.8 C
36	B-018-0-22	SR8	373+97	98	LT	CME 55T	63	1038.0	1037.3	0.7 C
37	B-010-0-22	SR8	375+28	78	RT	CME 55T	63	1034.1	1034.5	0.4 F
38	B-011-0-22	SR8	376+76	72	LT	CME 55T	63	1030.7	1030.2	0.5 C
39	B-012-0-22	SR8	384+22	75	LT	CME 55T	63	1026.4	1026.8	0.4 F
40	B-019-0-22	SR8	387+43	102	LT	CME 55T	63	1031.7	1028.8	2.9 C
41	B-020-0-22	SR8	387+52	81	RT	CME 55T	63	1030.5	1028.9	1.6 C
42	B-013-0-22	SR8	390+97	57	LT	CME 55T	63	1031.4	1031.1	0.3 C



#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
1	P 011-0 20	SS-1	1.0	2.5	-0.6	0.9	20	20		21	14	7	5	12	17	10	10	A-2-4	0						
		SS-2	3.5	4.3	1.9	2.7	50			20	NP	NP	34	8	42	6	0	Rock	0						
		SS-3	6.0	6.6	4.4	5.0	50								5	0	Rock	0							
		SS-4	8.5	9.1	6.9	7.5	50								6	0	Rock								
2	P 012-0 20	SS-1	1.2	2.7	-0.4	1.1	17	17	3.50	24	15	9	39	29	68	16	10	A-4a	7			Mc			
		SS-2	3.5	4.9	1.9	3.3	50			17	NP	NP	26	11	37	9	11	A-4a	0						
		SS-3	6.0	7.5	4.4	5.9	50			19	NP	NP	20	13	33	12	8	A-3a	0						
		SS-4	8.5	10.0	6.9	8.4	50								6	18	A-7-6								
3	P 013-0 20	SS-1	1.0	2.5	-0.6	0.9	15	15	4.00	24	16	8	40	11	51	14	11	A-4a	3			Mc			
		SS-2	3.5	4.1	1.9	2.5	50			28	17	11	35	10	45	6	0	Rock	0						
		SS-3	6.0	6.4	4.4	4.8	50								6	0	Rock	0							
		SS-4	8.5	8.9	6.9	7.3	50								7	0	Rock								
4	P 014-0 20	SS-1	1.1	2.6	-0.5	1.0	50	30		NP	NP	NP	13	14	27	12	10	A-2-4	0						
		SS-2	3.5	4.4	1.9	2.8	50			28	22	6	41	13	54	7	0	Rock	0						
		SS-3	6.0	6.4	4.4	4.8	50								5	0	Rock	0							
		SS-4	8.5	9.2	6.9	7.6	50								4	0	Rock								
5	P 015-0 20	SS-1	1.0	2.1	-0.6	0.5	50			24	19	5	29	11	40	4	0	Rock	0		Rock		6"		
		SS-2	3.5	3.9	1.9	2.3	50						27	10	37	5	0	Rock	0						
		SS-3	6.0	6.1	4.4	4.5	50								4	0	Rock	0							
		SS-4	8.5	8.8	6.9	7.2	50								4	0	Rock								
6	P 016-0 20	SS-1	1.2	2.7	-0.4	1.1	31	30	4.00	22	17	5	44	22	66	10	12	A-4a	6						
		SS-2	3.5	4.4	1.9	2.8	50			25	22	3	65	21	86	6	0	Rock	0						
		SS-3	6.0	6.9	4.4	5.3	50								6	0	Rock	0							
		SS-4	8.5	8.9	6.9	7.3	50								5	0	Rock								
7	P 017-0 20	SS-1	1.1	2.6	-0.5	1.0	43	30	4.00	25	18	7	52	34	86	13	13	A-4b	8		A-4b		23"		
		SS-2	3.5	4.9	1.9	3.3	50		4.00	27	14	13	39	25	64	11	14	A-6a	7						
		SS-3	6.0	6.9	4.4	5.3	50								8	0	Rock	0							
		SS-4	8.5	9.3	6.9	7.7	50								6	0	Rock								
8	P 018-0 20	SS-1	1.0	2.5	-0.6	0.9	11	11	3.25	NP	NP	NP	4	9	13	8	6	A-1-b	0	53					
		SS-2	3.5	5.0	1.9	3.4	15			17	15	2	19	13	32	11	10	A-2-4	0						
		SS-3	6.0	7.5	4.4	5.9	42			21	14	7	26	35	61	11	10	A-4a	5						
		SS-4	8.5	10.0	6.9	8.4	67								9	0	Rock								
9	R 002-0 99	SS-1	0.8	3.5	-0.7	2.0	17			25	17	8	24	11	35	12	10	A-2-4	0	133					
		SS-2	3.5	5.8	2.0	4.3	20									14	A-6a	10							
		SS-3	5.8	8.5	4.3	7.0	16									14	A-6a	10							

#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
		SS-4	8.5	10.0	7.0	8.5	38	16									0	Rock							
10	B 003-0 22	SS-1	1.5	3.0	0.0	1.5	29	12		NP	NP	NP	14	4	18	4	6	A-1-b	0						
		SS-2	3.0	4.5	1.5	3.0	13			23	15	8	25	11	36	10	10	A-4a	0						
		SS-3	4.5	6.0	3.0	4.5	13									11	10	A-4a	8						
		SS-4	6.0	7.5	4.5	6.0	12									11	6	A-1-b	0						
11	P 019-0 20	SS-1	1.0	2.5	-0.6	0.9	27	6		NP	NP	NP	21	16	37	8	11	A-4a	0	0					
		SS-2	3.5	5.0	1.9	3.4	8		1.25	21	16	5	30	16	46	12	11	A-4a	2			HP			
		SS-3	6.0	7.5	4.4	5.9	6			20	NP	NP	23	12	35	11	8	A-3a	0						
		SS-4	8.5	10.0	6.9	8.4	7									13	10	A-2-4							
12	B 005-0 22	SS-1	1.5	2.3	0.6	1.4	50	19		NP	NP	NP	19	5	24	4	6	A-1-b	0						
		SS-2	3.0	4.5	2.1	3.6	39									10	6	A-1-b	0						
		SS-3	4.5	6.0	3.6	5.1	19			NP	NP	NP	36	12	48	11	11	A-4a	3						
		SS-4	6.0	7.5	5.1	6.6	24									11	6	A-1-b							
13	P 020-0 20	SS-1	1.0	2.5	-0.6	0.9	29	14					18	12	30	11	10	A-4a	3						
		SS-2	3.5	5.0	1.9	3.4	14									4	10	A-2-4	0						
		SS-3	6.0	7.5	4.4	5.9	14			28	20	8	55	27	82	11	15	A-4b	8						
		SS-4	8.5	10.0	6.9	8.4	13			NP	NP	NP	31	7	38	15	11	A-4a							
14	P 022-0 20	SS-1	1.2	2.7	-0.4	1.1	36	30		18	15	3	28	19	47	13	10	A-4a	2			Mc			
		SS-2	3.5	5.0	1.9	3.4	35			17	15	2	39	16	55	11	10	A-4a	4						
		SS-3	6.0	7.5	4.4	5.9	45			16	14	2	32	12	44	10	10	A-4a	2						
		SS-4	8.5	10.0	6.9	8.4	15			15	NP	NP	28	14	42	11	11	A-4a							
15	R 004-0 99	SS-1	1.0	3.5	-0.5	2.0	44	30									8	A-3a	0						
		SS-2	3.5	5.6	2.0	4.1	51			30	15	15	48	15	63	12	14	A-6a	8						
		SS-3	5.6	7.7	4.1	6.2	40										14	A-6a	10						
		SS-4	7.7	10.0	6.2	8.5	32										14	A-6a							
16	P 024-0 20	SS-1	1.0	2.5	-0.6	0.9	31	30		18	17	1	28	10	38	9	12	A-4a	1						
		SS-2	3.5	5.0	1.9	3.4	39			21	18	3	56	23	79	13	13	A-4b	8			A-4b		36"	
		SS-3	6.0	7.5	4.4	5.9	38			16	15	1	28	18	46	11	10	A-4a	2						
		SS-4	8.5	10.0	6.9	8.4	31			NP	NP	NP	11	3	14	5	8	A-3a							
17	R 005-0 99	SS-1	0.0	3.5	-1.5	2.0	28	27								14	10	A-4a	8			Mc			
		SS-2	3.5	5.8	2.0	4.3	27										8	A-3a	0						
		SS-3	5.8	8.5	4.3	7.0	33										10	A-4a	8						
		SS-4	8.5	10.0	7.0	8.5	31										10	A-4a							
18	P 025-0	SS-1	1.0	2.5	-0.6	0.9	14			NP	NP	NP	15	6	21	7	8	A-3a	0						
		SS-2	3.5	5.0	1.9	3.4	7			14	NP	NP	22	8	30	9	8	A-3a	0						

#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
	20	SS-3	6.0	7.5	4.4	5.9	6	6								7	10	A-2-4	0						
		SS-4	8.5	10.0	6.9	8.4	35									9	10	A-2-4							
19	R 006-0 99	SS-1	0.0	3.5	2.0	5.5	10	10									10	A-4a	8						
		SS-2	3.5	5.8	5.5	7.8	51																		
		SS-3	5.8	8.5	7.8	10.5	45										10	A-4a							
		SS-4	8.5	10.0	10.5	12.0	64																		
20	P 027-0 20	SS-1	1.0	2.5	0.9	2.4	20	10					15	15	30	11	8	A-3a	0						
		SS-2	3.5	5.0	3.4	4.9	15		1.75				9	3	12	12	16	A-6b	0						
		SS-3	6.0	7.5	5.9	7.4	10		2.25	25	15	10	27	23	50	14	16	A-6b							
		SS-4	8.5	10.0	8.4	9.9	47		3.75							14	16	A-6b							
21	P 028-0 20	SS-1	1.2	2.7	0.6	2.1	10	10		NP	NP	NP	17	12	29	11	8	A-3a	0						
		SS-2	3.5	5.0	2.9	4.4	50		1.25	20	16	4	25	15	40	14	11	A-4a	1						
		SS-3	6.0	7.4	5.4	6.8	50		4.00	22	15	7	35	24	59	13	10	A-4a							
		SS-4	8.5	8.7	7.9	8.1	33									5	0	Rock							
22	R 008-0 99	SS-1	0.9	2.5	1.4	3.0	86	30		21	13	8	26	16	42	11	10	A-4a	1						
		SS-2	2.5	6.0	3.0	6.5	50										10	A-4a	8						
		SS-3	6.0	7.6	6.5	8.1	50										14	A-6a							
23	B 044-0 24	SS-1	2.5	3.1	1.5	2.1	50	30		NP	NP	NP	16	6	22	5	8	A-3a	0						
		SS-2	5.0	5.1	4.0	4.1	50									7	0	Rock	0						
24	P 029-0 20	SS-1	1.1	2.6	0.5	2.0	33	20		NP	NP	NP	15	16	31	10	8	A-3a	0						
		SS-2	3.5	5.0	2.9	4.4	20		4.00	23	16	7	34	26	60	11	11	A-4a	5						
		SS-3	6.0	7.5	5.4	6.9	71			19	16	3	17	16	33	11	10	A-2-4							
		SS-4	8.5	10.0	7.9	9.4	28			17	16	1	18	17	35	9	8	A-3a							
25	P 030-0 20	SS-1	1.0	2.5	4.4	5.9	29	29					5	2	7	8	8	A-3a	0						
		SS-2	3.5	5.0	6.9	8.4	50		4.00	20	18	2	34	7	41	13	18	A-7-6							
		SS-3	6.0	6.4	9.4	9.8	50									6	0	Rock							
		SS-4	8.5	8.9	11.9	12.3	99									6	0	Rock							
26	R 010-0 99	SS-1	1.5	3.0	0.0	1.5	50	30		NP	NP	NP	30		30	8	10	A-2-4	0						
		SS-2	3.0	4.5	1.5	3.0	50										10	A-2-4	0						
		SS-3	4.5	6.0	3.0	4.5	50										10	A-2-4	0						
		SS-4	6.0	8.8	4.5	7.3	50										0	Rock	0						
27	B	SS-1	1.5	2.2	0.8	1.4	50			NP	NP	NP	20	6	26	8	8	A-3a	0	0					



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#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
	006-022	SS-2	3.0	3.6	2.3	2.9	50	30							8	8	A-3a	0							
		SS-3	4.5	5.4	3.8	4.7	50			NP	NP	NP	27	13	40	11	11	A-4a	1						
		SS-4	6.0	6.5	5.3	5.8	34									11	0	Rock							
28	P032-020	SS-1	1.0	2.5	-0.6	0.9	99	30		NP	NP	NP	13	14	27	9	8	A-3a	0						
		SS-2	3.5	3.7	1.9	2.1	50		1.75	20	17	3	20	15	35	12	8	A-3a	0						
		SS-3	6.0	6.4	4.4	4.8	50									9	10	A-2-4	0						
		SS-4	8.5	8.7	6.9	7.1	50									8	0	Rock							
29	R012-099	SS-1	0.6	3.1	-0.9	1.6	50									0		Rock	0		Rock		6"		
		SS-2	3.1	5.7	1.6	4.2	50										0		Rock	0					
30	B028-024	SS-1	2.5	2.8	1.2	1.5	50								3	0	Rock	0		Rock					
		SS-2	5.0	5.2	3.7	3.9	50									7	0	Rock	0						
31	B055-024	SS-1	2.5	2.7	1.0	1.2	50								3	0	Rock	0		Rock					
		SS-2	5.0	5.1	3.5	3.6	50									8	0	Rock	0						
32	B057-024	SS-1	2.5	4.0	1.1	2.6	108	30		NP	NP	NP	17	7	24	7	10	A-2-4	0						
		SS-2	5.0	5.2	3.6	3.8	50									2	0	Rock	0						
33	B007-022	SS-1	1.5	3.0	0.8	2.3	34	30		NP	NP	NP	13	5	18	10	8	A-3a	0	120					
		SS-2	3.0	4.0	2.3	3.3	50									11	8	A-3a	0						
		SS-3	4.5	4.9	3.8	4.2	50									9	0	Rock	0						
		SS-4	6.0	6.1	5.3	5.4	50									8	0	Rock							
34	B008-022	SS-1	1.5	3.0	0.6	2.2	39	30		NP	NP	NP	23	8	31	6	10	A-2-4	0	327					
		SS-2	3.0	4.5	2.2	3.7	37			NP	NP	NP	14	5	19	7	8	A-3a	0						
		SS-3	4.5	6.0	3.7	5.2	44									9	8	A-3a	0						
		SS-4	6.0	6.8	5.2	5.9	50									9	0	Rock							
35	B009-022	SS-1	1.5	3.0	0.7	2.2	50	30		NP	NP	NP	18	6	24	9	8	A-3a	0	13					
		SS-2	3.0	4.5	2.2	3.7	50									8	0	Rock	0						
		SS-3	4.5	6.0	3.7	5.2	50									8	0	Rock	0						
		SS-4	6.0	7.5	5.2	6.7	50									9	0	Rock							



#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
36	B 018-0 22	SS-1	1.5	1.8	0.8	1.1	50	30								3	0	Rock	0		Rock				
		SS-2	3.0	3.1	2.3	2.4	50									6	0	Rock	0						
		SS-3	4.5	4.8	3.8	4.1	50									5	0	Rock	0						
		SS-4	6.0	6.2	5.3	5.5	50																		
37	B 010-0 22	SS-1	1.5	1.8	1.9	2.2	50									4	0	Rock	0		Rock				
		SS-2	3.0	3.3	3.4	3.7	50									4	0	Rock	0						
		SS-3	4.5	4.8	4.9	5.2	50									4	0	Rock	0						
		SS-4	6.0	6.3	6.4	6.7	50									4	0	Rock							
38	B 011-0 22	SS-1	1.5	2.3	1.0	1.8	23	13		NP	NP	NP	18	5	23	7	6	A-1-b	0	53					
		SS-2	3.0	4.5	2.5	4.0	13									8	0	Rock	0						
		SS-3	4.5	6.0	4.0	5.5	39									8	0	Rock	0						
		SS-4	6.0	6.8	5.5	6.3	100									8	0	Rock							
39	B 012-0 22	SS-1	1.5	2.3	1.9	2.7	30	19		NP	NP	NP	17	5	22	7	6	A-1-b	0	133					
		SS-2	3.0	4.5	3.4	4.9	19			NP	NP	NP	20	5	25	7	6	A-1-b	0						
		SS-3	4.5	6.0	4.9	6.4	29									8	6	A-1-b	0						
		SS-4	6.0	7.5	6.4	7.9	27									8	6	A-1-b							
40	B 019-0 22	SS-1	1.5	3.0	-1.4	0.1	33	16		NP	NP	NP	10	2	12	5	8	A-3a	0	0					
		SS-2	3.0	4.5	0.1	1.6	16									8	8	A-3a	0						
		SS-3	4.5	5.4	1.6	2.5	50									10	0	Rock	0						
		SS-4	6.0	6.2	3.1	3.3	50									4	0	Rock	0						
41	B 020-0 22	SS-1	1.5	2.2	-0.1	0.6	50									7	0	Rock	0		Rock		6"		
		SS-2	3.0	3.4	1.4	1.8	50									6	0	Rock	0						
		SS-3	4.5	5.0	2.9	3.4	50									5	0	Rock	0						
		SS-4	6.0	6.2	4.4	4.6	50									5	0	Rock	0						
42	B 013-0 22	SS-1	1.5	1.9	1.2	1.6	50									6	0	Rock	0		Rock				
		SS-2	3.0	3.3	2.7	2.9	50									4	0	Rock	0						
		SS-3	4.5	4.8	4.2	4.4	50									5	0	Rock	0						
		SS-4	6.0	6.1	5.7	5.8	50										0	Rock							

PID: 114865

###

County-Route-Section: SUM-077/008-11.65/00.00

No. of Borings: 42

Geotechnical Consultant: NEAS, Inc.

Prepared By: Zhao Mankoci

Date prepared: 8/11/2026

Chemical Stabilization Options		
320	Rubblize & Roll	Option
206	Cement Stabilization	Option
	Lime Stabilization	No
206	Depth	14"

Excavate and Replace Stabilization Options	
Global Geotextile Average(N60L):	0"
Average(HP):	0"
Global Geogrid Average(N60L):	0"
Average(HP):	0"

Design CBR	11
------------	----

% Samples within 3 feet of subgrade			
N ₆₀ ≤ 5	0%	HP ≤ 0.5	0%
N ₆₀ < 12	4%	0.5 < HP ≤ 1	0%
12 ≤ N ₆₀ < 15	4%	1 < HP ≤ 2	2%
N ₆₀ ≥ 20	49%	HP > 2	5%
M+	3%		
Rock	28%		
Unsuitable Soil	3%		

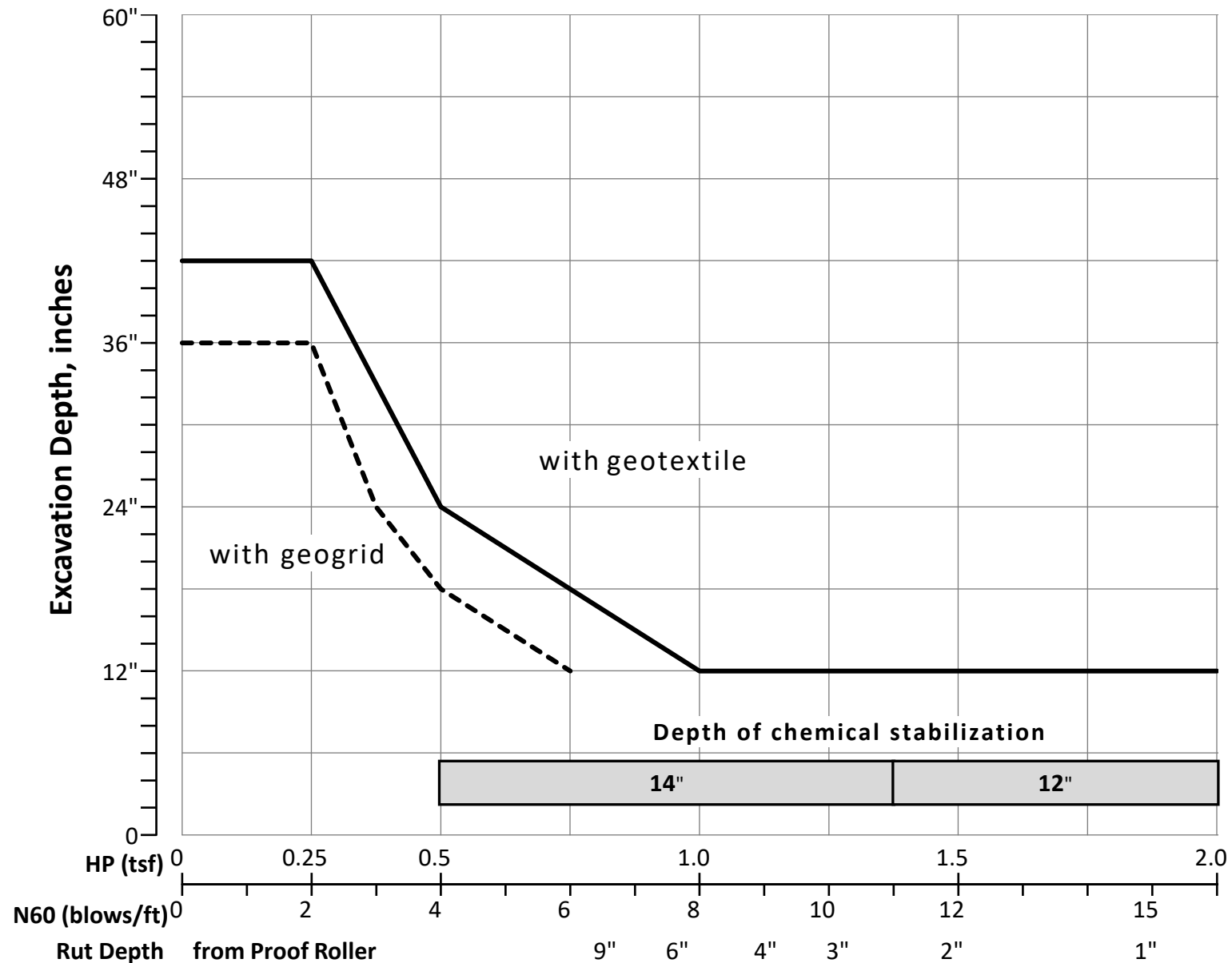
Excavate and Replace at Surface	
Average	0"
Maximum	0"
Minimum	0"

% Proposed Subgrade Surface	
Unstable & Unsuitable	21%
Unstable	7%
Unsuitable (Soil & Rock)	14%

	N ₆₀	N _{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M _C	M _{OPT}	GI
Average	41	22	3.12	21	16	6	26	13	38	8	6	1
Maximum	108	30	4.00	30	22	15	65	35	86	16	18	10
Minimum	6	6	1.25	14	13	1	4	2	7	2	0	0

Classification Counts by Sample																				
ODOT Class	UCF	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals
Count	0	60	0	11	15	0	0	0	0	24	29	3	0	7	3	0	2	0	0	154
Percent	0%	39%	0%	7%	10%	0%	0%	0%	0%	16%	19%	2%	0%	5%	2%	0%	1%	0%	0%	100%
% Rock Granular Cohesive	0%	39%	51%									10%								100%
Surface Class Count	0	20	0	6	9	0	0	0	0	18	14	2	0	3	0	0	0	0	0	72
Surface Class Percent	0%	28%	0%	8%	13%	0%	0%	0%	0%	25%	19%	3%	0%	4%	0%	0%	0%	0%	0%	100%

Fig. 600-1 – Subgrade Stabilization



VERRIDE TABLE

Calculated Average	New Values	Check to Override
3.12	0.50	☐ HP
22.00	7.00	☐ N60L

Average HP

Average N_{60L}





OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES

Geotechnical Design Manual Section 600

**SUM-077/008-11.65/00.00
114865**

Entire Project

NEAS, Inc.

Prepared By: Zhao Mankoci
Date prepared: Tuesday, August 11, 2026

**Chunmei (Melinda) He, Ph.D., P.E.
2800 Corporate Exchange Drive
Suite 240
Columbus, OH 43231
614.714.0299 Ext 111
che@neasinc.com**

NO. OF BORINGS: 19

NO. OF DCPS:



#	Boring ID	Alignment	Station	Offset	Dir	Add DCP Test Data Worksheets		Boring EL.	Proposed Subgrade EL.	Cut Fill
						Drill Rig	ER			
1	P-012-0-20	Exit Ramp-I-76	101+34	6	rt	CME 55T	63	1069.9	1067.8	2.1 C
2	E3-001-0-20	Exit Ramp-I-76	102+32	23	LT	CME 55T	63	1089.6	1068.3	21.3 C
3	E3-002-0-20	Exit Ramp-I-76	103+59	18	LT	CME 55T	63	1091.7	1071.9	19.8 C
4	E3-003-0-20	Exit Ramp-I-76	104+82	19	LT	CME 55T	63	1093.2	1076.1	17.1 C
5	E3-004-0-20	Exit Ramp-I-76	106+33	13	LT	CME 55T	63	1098.3	1076.2	22.1 C
6	B-001-0-22	Exit Ramp-I-76	108+46	23	LT	CME 55T	63	1093.5	1076.3	17.2 C
7	B-002-0-22	Exit Ramp-I-76	109+81	16	LT	CME 55T	63	1090.6	1074.8	15.8 C
8	B-030-0-24	Exit Ramp-I-76	111+50	21	RT	CME 55T	63	1071.3	1069.8	1.5 C
9	B-031-0-24	Exit Ramp-I-76	113+29	21	RT	CME 55T	71	1066.7	1065.1	1.6 C
10	B-042-0-24	Exit Ramp-I-76	132+08	5	RT	CME 55TB	89	1037.0	1038.7	1.7 F
11	B-004-0-99	Exit Ramp-I-76	132+84	12	LT	CME 55T	63	1046.8	1038.0	8.8 C
12	R-009-0-99	Exit Ramp-I-76	134+72	13	LT	CME 55T	63	1039.0	1036.3	2.7 C
13	B-050-0-24	Exit Ramp-I-76	134+82	7	RT	CME 55TB	89	1038.1	1036.2	1.9 C
14	B-051-0-24	Exit Ramp-I-76	136+01	3	LT	CME 55TB	89	1038.7	1035.4	3.3 C
15	B-023-0-24	Exit Ramp-I-76	137+34	15	LT	CME 55TB	89	1043.3	1034.4	8.9 C
16	B-053-0-24	Exit Ramp-I-76	140+00	10	RT	CME 55TB	89	1034.5	1033.1	1.4 C
17	P-032-0-20	Exit Ramp-I-76	141+00	17	RT	19 CME 75 079797	84	1034.6	1032.8	1.8 C
18	B-028-0-24	Exit Ramp-I-76	142+85	53	RT	CME 55TB	89	1033.8	1032.3	1.5 C
19	B-055-0-24	Exit Ramp-I-76	144+00	3	RT	CME 55TB	89	1034.4	1032.8	1.6 C



Add DCP Test Data Worksheets								Boring EL.	Proposed Subgrade EL.	Cut Fill
#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER			



6/5/2026

#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics					Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class		GI	Unsuitable	Unstable	Unsuitable	
1	P 012-0 20	SS-1	1.2	2.7	-0.9	0.6	17	17	3.50	24	15	9	39	29	68	16	10	A-4a	7			Mc		
		SS-2	3.5	4.9	1.4	2.8	50			17	NP	NP	26	11	37	9	11	A-4a	0					
		SS-3	6.0	7.5	3.9	5.4	50			19	NP	NP	20	13	33	12	8	A-3a	0					
		SS-4	8.5	10.0	6.4	7.9	50								6	18	A-7-6							
2	E -001- -2	SS-10	23.5	23.8	2.2	2.5	50									0		Rock	0		Rock			
		SS-11	26.0	26.4	4.7	5.1	50									0		Rock	0					
		SS-12	28.5	29.4	7.2	8.1	50									0		Rock						
		SS-13	31.0	31.4	9.7	10.1	50									0		Rock						
3	E -002- -2	SS-9	21.0	21.4	1.2	1.6	50									0		Rock	0		Rock			
		SS-10	23.5	23.9	3.7	4.1	50									0		Rock	0					
		SS-11	26.0	26.3	6.2	6.5	50									0		Rock						
		SS-12	28.5	29.3	8.7	9.5	50									0		Rock						
4	E -003- -2	SS-8	18.5	20.0	1.4	2.9	81									0		Rock	0		Rock			
		SS-9	21.0	21.4	3.9	4.3	50									0		Rock	0					
		SS-10	23.5	23.9	6.4	6.8	50									0		Rock						
		SS-11	26.0	26.4	8.9	9.3	50									0		Rock						
5	E -004- -2	RC-2	16.0	23.0	-6.1	0.9										0		Rock	0		Rock		6"	
		RC-3	23.0	26.0	0.9	3.9																		
6	B 001-0 22	NQ2-2	15.7	25.7	-1.5	8.5										0		Rock	0		Rock		6"	
7	B 002-0 22	NQ2-2	10.8	20.8	-5.0	5.0										0		Rock	0		Rock		6"	
8	B 030-0 24	SS-1	2.5	2.7	1.0	1.2	50	30		NP	NP	NP	11	6	17	3	6	A-1-b	0					
		SS-2	5.0	5.4	3.5	3.9	50									6	6	A-1-b	0					
		SS-3	7.5	8.1	6.0	6.6	50									5	0	Rock						
		SS-4	10.0	10.8	8.5	9.3	50									5	0	Rock						
9	B 031-0 24	SS-1	2.5	4.0	0.9	2.4	9			27	18	9	41	21	62	14	13	A-4a	5			N ₆₀		12"
		SS-2	5.0	6.5	3.4	4.9	20			26	17	9	37	22	59	12	12	A-4a	5					
		SS-3	7.5	9.0	5.9	7.4	20									14	10	A-4a						



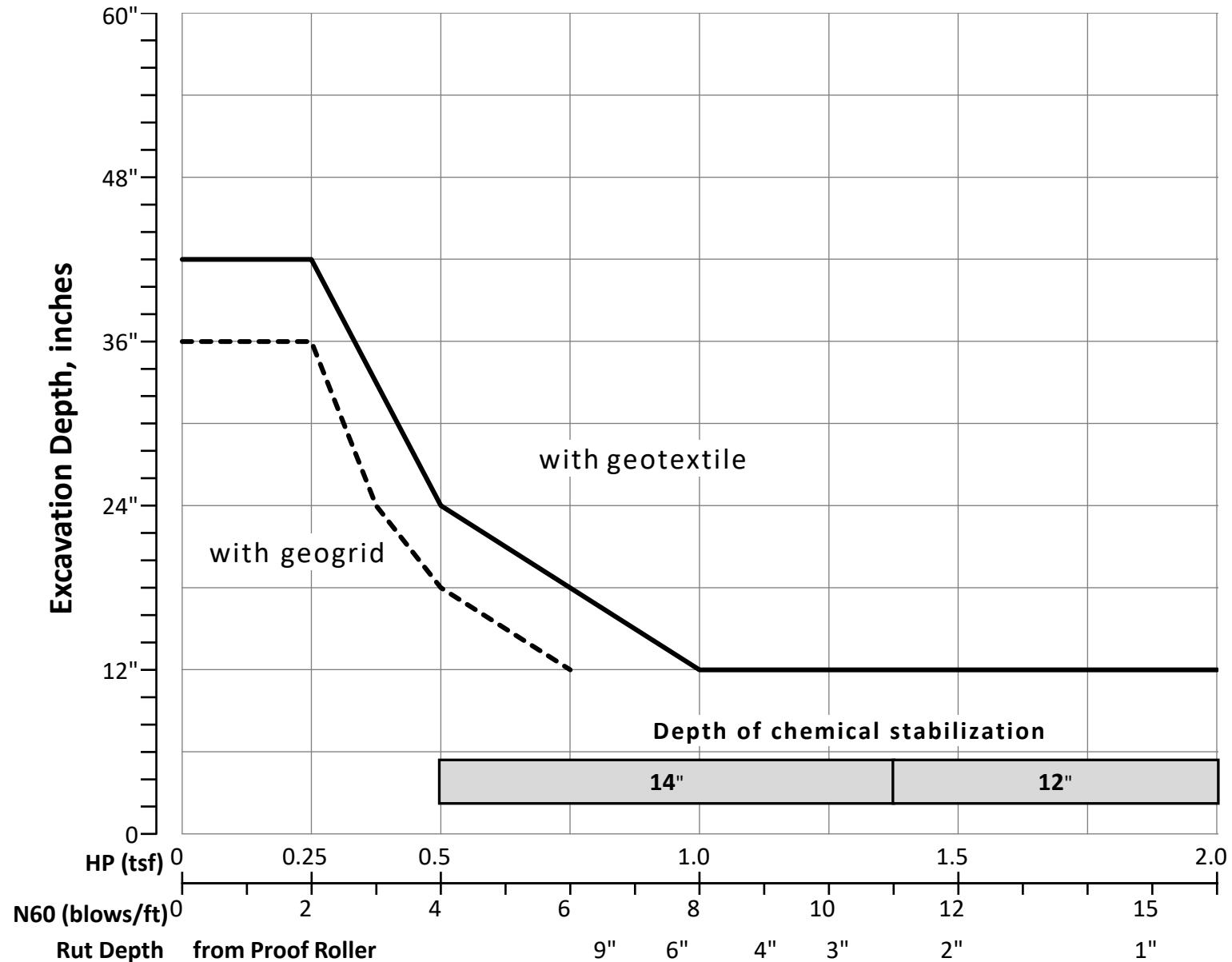
#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
		SS-4	10.0	11.5	8.4	9.9	5	9								17	14	A-6a							
10	B 042-0 24	SS-1	2.5	4.0	4.2	5.7	28	28		NP	NP	NP	22	9	31	9	10	A-2-4	0						
		SS-2	5.0	5.9	6.7	7.6	50									9	10	A-2-4							
		SS-3	7.5	9.0	9.2	10.7	89		4.50							11	10	A-4a							
		SS-4	10.0	10.3	11.7	12.0	50									6	0	Rock							
11	B 004-0 99	SS-4	8.0	9.5	-0.8	0.7	50										0	Rock	0		Rock		6"		
		SS-5	9.5	15.0	0.7	6.2											0	Rock	0						
12	R 009-0 99	SS-1	1.5	3.0	-1.2	0.3	50	30																	
		SS-2	3.0	4.5	0.3	1.8	50																		
		SS-3	4.5	6.0	1.8	3.3	50										10	A-4a	8						
		SS-4	6.0	8.7	3.3	6.0	50										10	A-4a	8						
13	B 050-0 24	SS-1	2.5	4.0	0.6	2.1	71	30		25	17	8	34	15	49	8	12	A-4a	3						
		SS-2	5.0	6.5	3.1	4.6	62									8	6	A-1-b	0						
		SS-3	7.5	8.1	5.6	6.2	50									9	10	A-2-4							
																	0	Rock							
14	B 051-0 24	SS-1	2.5	3.1	-0.8	-0.2	50	30		NP	NP	NP	18	10	28	6	10	A-2-4	0						
		SS-2	5.0	5.6	1.7	2.3	50									8	0	Rock	0						
		SS-3	7.5	7.7	4.2	4.4	50									7	0	Rock	0						
15	B 023-0 24	SS-3	7.5	7.9	-1.4	-1.0	50										0	Rock	0						
		SS-4	10.0	11.6	1.1	2.7	50										0	Rock	0						
		SS-5	12.5	12.6	3.6	3.7	50										0	Rock	0						
16	B 053-0 24	SS-1A	2.5	3.0	1.1	1.6	50	30		NP	NP	NP	15	7	22	9	8	A-3a	0						
		SS-1B	3.0	3.4	1.6	2.0	50									9	0	Rock	0						
		SS-2	5.0	5.1	3.6	3.7	50									6	0	Rock	0						
17	P 032-0 20	SS-1	1.0	2.5	-0.8	0.7	99	30		NP	NP	NP	13	14	27	9	8	A-3a	0						
		SS-2	3.5	3.7	1.7	1.9	50		1.75	20	17	3	20	15	35	12	8	A-3a	0						
		SS-3	6.0	6.4	4.2	4.6	50									9	10	A-2-4	0						
		SS-4	8.5	8.7	6.7	6.9	50									8	0	Rock							
18	B 028-0	SS-1	2.5	2.8	1.0	1.3	50									3	0	Rock	0		Rock				
		SS-2	5.0	5.2	3.5	3.7	50									7	0	Rock	0						



6/5/2026

[illegible]

Fig. 600-1 – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
3.25	0.50	☐ HP
26.00	7.00	☐ N60L

Average HP

Average N_{60L}





OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES

Geotechnical Design Manual Section 600

**SUM-077/008-11.65/00.00
114865**

Entire Project

NEAS, Inc.

Prepared By: Zhao Mankoci
Date prepared: Tuesday, August 11, 2026

**Chunmei (Melinda) He, Ph.D., P.E.
2800 Corporate Exchange Drive
Suite 240
Columbus, OH 43231
614.714.0299 Ext 111
che@neasinc.com**

NO. OF BORINGS: 6

NO. OF DCPS:



#	Boring ID	Alignment	Station	Add DCP Test Data Worksheets				Boring EL.	Proposed Subgrade EL.	Cut Fill
				Offset	Dir	Drill Rig	ER			
1	B-004-0-99	Goodkirk st	204+86	10	LT	CME 55T	63	1038.2	1036.7	1.5 C
2	R-009-0-99	Goodkirk st	206+74	10	LT	CME 55T	63	1039.0	1037.5	1.5 C
3	B-006-0-99	Goodkirk st	209+22	10	LT	CME 55T	63	1043.6	1042.1	1.5 C
4	R-011-0-99	Goodkirk st	210+38	10	LT	CME 55T	63	1047.3	1045.8	1.5 C
5	R-013-0-99	Goodkirk st	214+43	10	LT	CME 55T	63	1057.3	1055.3	2.0 C
6	B-008-0-99	Goodkirk st	214+76	5	RT	CME 55T	63	1057.4	1055.9	1.5 C



Add DCP Test Data Worksheets								Boring EL.	Proposed Subgrade EL	Cut Fill
#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER			



#	Boring ID	Sample #	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
1	B 004-0 99	SS-4	8.0	9.5	6.5	8.0	50										0	Rock			Rock				
		SS-5	9.5	15.0	8.0	13.5											0	Rock							
2	R 009-0 99	SS-1	1.5	3.0	0.0	1.5	50																		
		SS-2	3.0	4.5	1.5	3.0	50																		
		SS-3	4.5	6.0	3.0	4.5	50									10		A-4a	8						
		SS-4	6.0	8.7	4.5	7.2	50	30								10		A-4a	8						
3	B 006-0 99	SS-1	1.0	2.5	-0.5	1.0	22									10		A-2-4	0						
		SS-2	2.5	5.5	1.0	4.0	23			NP	NP	NP				11	10	A-2-4	0						
		SS-3	5.5	8.0	4.0	6.5	50									0		Rock	0						
		SS-4	8.0	8.8	6.5	7.3	50	22								0		Rock							
4	R 011-0 99	SS-1	0.7	3.5	-0.8	2.0	17			18	14	4	39	13	52	14	10	A-4a	3			Mc			
		SS-2	3.5	6.0	2.0	4.5	22										10	A-4a	8						
		SS-3	6.0	7.5	4.5	6.0	50										0	Rock	0						
		SS-4	7.8	8.8	6.3	7.3	50	17									0	Rock							
5	R 013-0 99	SS-1	0.8	3.5	-1.2	1.5	7			20	15	5	27	12	39	15	10	A-4a	1			Mc			
		SS-2	3.5	6.0	1.5	4.0	7										10	A-4a	8						
		SS-3	6.0	7.5	4.0	5.5	63										0	Rock	0						
		SS-4	7.5	8.6	5.5	6.6	50	7									0	Rock							
6	B 008-0 99	SS-1	1.0	3.0	-0.5	1.5	9										10	A-4a	8			N ₆₀		12"	
		SS-2	3.0	5.5	1.5	4.0	7			27	12	15	31	13	44	17	14	A-6a	3						
		SS-3	5.5	8.0	4.0	6.5	22										0	Rock	0						
		SS-4	8.0	10.5	6.5	9.0	50	7									0	Rock							

PID: 114865

###

County-Route-Section: SUM-077/008-11.65/00.00

No. of Borings: 6

Geotechnical Consultant: NEAS, Inc.

Prepared By: Zhao Mankoci

Date prepared: 8/11/2026

Chemical Stabilization Options		
320	Rubblize & Roll	Option
206	Cement Stabilization	Option
	Lime Stabilization	No
206	Depth	12"

Excavate and Replace Stabilization Options	
Global Geotextile Average(N60L):	0"
Average(HP):	0"
Global Geogrid Average(N60L):	0"
Average(HP):	0"

Design CBR	9
------------	---

% Samples within 3 feet of subgrade			
N ₆₀ ≤ 5	0%	HP ≤ 0.5	0%
N ₆₀ < 12	24%	0.5 < HP ≤ 1	0%
12 ≤ N ₆₀ < 15	0%	1 < HP ≤ 2	0%
N ₆₀ ≥ 20	35%	HP > 2	0%
M+	12%		
Rock	0%		
Unsuitable Soil	0%		

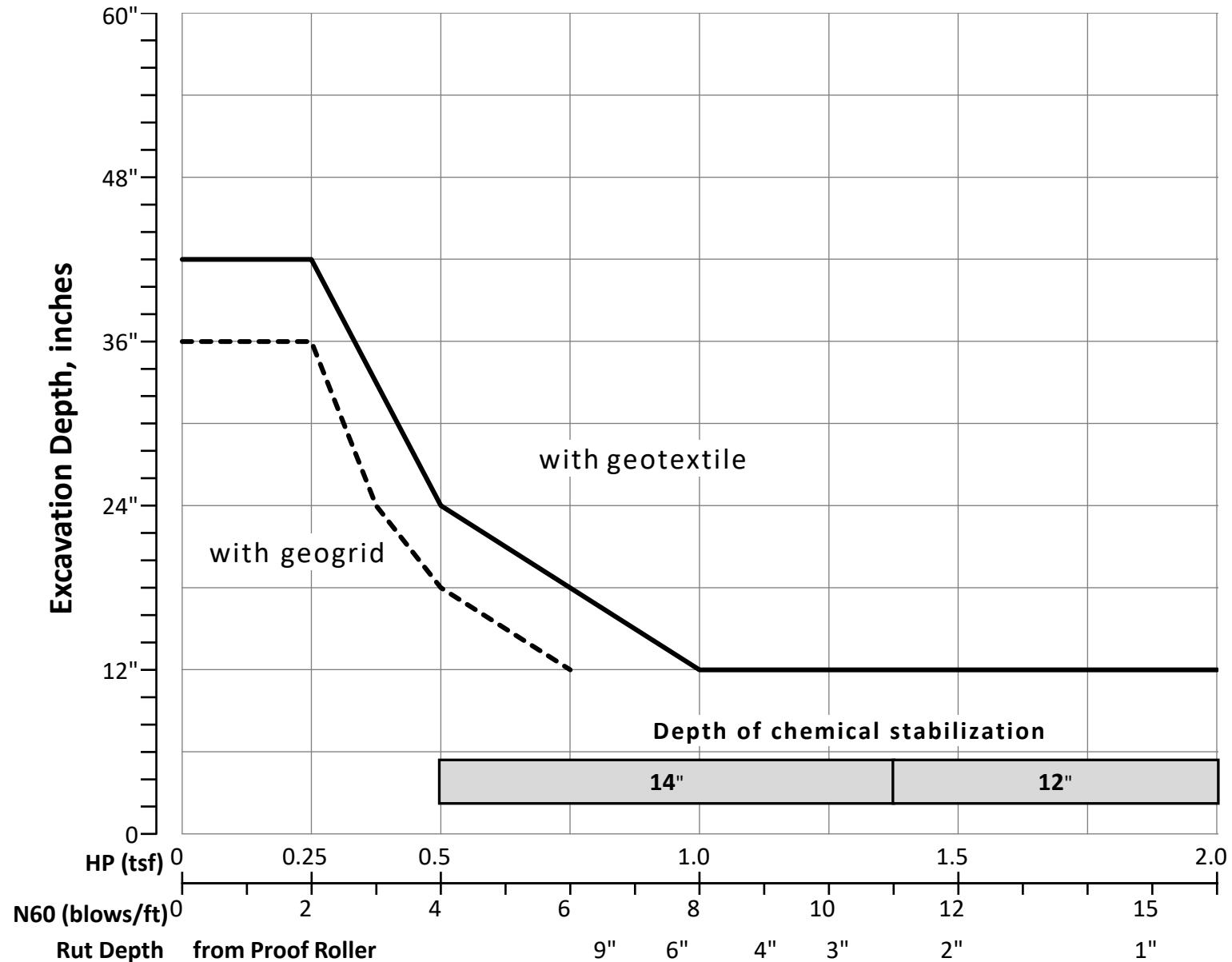
Excavate and Replace at Surface	
Average	0"
Maximum	0"
Minimum	0"

% Proposed Subgrade Surface	
Unstable & Unsuitable	50%
Unstable	38%
Unsuitable (Soil & Rock)	13%

	N ₆₀	N _{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M _C	M _{OPT}	GI
Average	36	17	NP	22	14	8	32	13	45	14	5	3
Maximum	63	30	NP	27	15	15	39	13	52	17	14	8
Minimum	7	7	NP	18	12	4	27	12	39	11	0	0

Classification Counts by Sample																					
ODOT Class	UCF	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals	
Count	0	10	0	0	2	0	0	0	0	0	7	0	0	1	0	0	0	0	0	20	
Percent	0%	50%	0%	0%	10%	0%	0%	0%	0%	0%	35%	0%	0%	5%	0%	0%	0%	0%	0%	100%	
% Rock Granular Cohesive	0%	50%	45%									5%									100%
Surface Class Count	0	0	0	0	2	0	0	0	0	0	5	0	0	1	0	0	0	0	0	8	
Surface Class Percent	0%	0%	0%	0%	25%	0%	0%	0%	0%	0%	63%	0%	0%	13%	0%	0%	0%	0%	0%	100%	

Fig. 600-1 – Subgrade Stabilization



VERRIDE TABLE

Calculated Average	New Values	Check to Override
NP	0.50	☐ HP
16.60	7.00	☐ N60L

Average HP



Average N_{60L}



APPENDIX D

BRIDGE FOUNDATION ANALYSES

Objective: To evaluate the bearing resistance of shallow foundation on level soil.
Method: In accordance with ODOT Bridge Design Manual, 2024 [Sect. 204.6.2.2] and LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 10.6.3.1.2].

Givens:

Soil Design Parameters (Average Below Footing):

Drained Conditions (Effective Stress):

$\phi'_{fd} := 25 \text{ deg}$ Effective angle of internal friction

$\gamma_{fd} := 112 \frac{\text{lbf}}{\text{ft}^3}$ Unit weight

$c'_{fd} := 200 \frac{\text{lbf}}{\text{ft}^2}$ Effective Cohesion

Undrained Conditions (Total Stress):

$\phi_{fdu} := 0 \text{ deg}$ Angle of internal friction (Same as Drained Conditions if Sand)

$Su_{fdu} := 1800 \frac{\text{lbf}}{\text{ft}^2}$ Undrained Shear Strength

Undercut & Replacement Design Parameters:

$\phi_{Re} := 34 \text{ deg}$ Angle of internal friction for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$c_{Re} := 0 \frac{\text{lbf}}{\text{ft}^2}$ Cohesion for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$\gamma_{Re} := 130 \frac{\text{lbf}}{\text{ft}^3}$ Unit Weight for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$\delta_{Re} := 0.67 \cdot \phi_{Re}$ $\delta_{Re} = 22.8 \text{ deg}$ Friction angle between Replacement soils and footing taken as specified in LRFD BDS C3.11.5.3 (degrees)

$D_{undercut} := 0 \text{ ft}$ Depth of Undercut below bottom of footing

Foundation Surcharge Soil Parameters:

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

Footing Geometry:

$D_f := 0 \text{ ft}$ Footing cover at Toe
Note: Unless on rock, top of footing should be at least 1-ft from soil surface and bottom of footing at least 5-ft from nearest soil surface per BDM 202.2.3.1a

$D_F := D_f + D_{undercut}$ Embenment Depth at bottom of Undercut

$B := 17 \text{ ft}$ Footing base width

$B' := B$ Footing effective base width

$L' := 231 \text{ ft}$ Footing effective length (assumed to equal actual length)

$d_w := 0 \text{ ft}$ Depth of groundwater below ground surface

Compute Bearing Resistance:

Drained Conditions (Effective Stress):

$$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right) \quad N_q = 10.66$$

$$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right) \quad N_c = 20.72$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd}) \quad N_\gamma = 10.9$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right) \quad s_c = 1.038$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right) \quad s_q = 1.034$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right) \quad s_\gamma = 0.971$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see LRFD BDS C10.6.3.1.2a.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5) \quad C_{wq} = 1$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \quad C_{w\gamma} = 0.5$$

Depth Correction Factor per ODOT BDM 305.2.1:

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'} \right)$$

$$d_q = 1$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 21.505$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 11.028$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 10.556$$

Compute nominal bearing resistance at bottom of Undercut & Replacement, LRFD [Eq 10.6.3.1.2a-1]:

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nd} = 9325.8 \frac{\text{lbf}}{\text{ft}^2}$$

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

$$\phi_b := 0.5$$

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 4.7 \text{ ksf}$$

Bearing resistance factor **LRFD Table 11.5.5.2.2-1.**
 $\phi_b = 0.5$ in clay; $\phi_b = 0.45$ in sand

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

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

$$s_c = 1.015$$

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

$$s_q = 1$$

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

$$s_\gamma = 1$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a.**
"Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

Depth Correction Factor per ODOT BDM 305.2.1:

$$d_q := 1 + 2 \cdot \tan(\phi_{fdu}) \cdot (1 - \sin(\phi_{fdu}))^2 \cdot \tan \left(\frac{D_F}{B'} \right)$$

$$d_q = 1$$

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

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

$$N_{cm} = 5.216$$

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

$$N_{qm} = 1$$

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

$$N_{\gamma m} = 0$$

Compute nominal bearing resistance at bottom of Undercut & Replacement. **LRFD [Eq 10.6.3.1.2a-1]:**

$$q_{nu} := S_{ufdu} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nu} = 9388.2 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.5$$

Bearing resistance factor LRFD Table 11.5.5.2.2-1.

$\phi_b = 0.5$ in clay; $\phi_b = 0.45$ in sand

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 4.7 \text{ ksf}$$

Factored bearing resistance Undrained
Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 4.7 \text{ ksf}$

Undrained Conditions: $q_{Ru} = 4.7 \text{ ksf}$

Objective: To evaluate the bearing resistance of shallow foundation on level soil.
Method: In accordance with ODOT Bridge Design Manual, 2024 [Sect. 204.6.2.2] and LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 10.6.3.1.2].

Givens:

Soil Design Parameters (Average Below Footing):

Drained Conditions (Effective Stress):

$\phi'_{fd} := 26 \text{ deg}$ Effective angle of internal friction

$\gamma_{fd} := 112 \frac{\text{lbf}}{\text{ft}^3}$ Unit weight

$c'_{fd} := 200 \frac{\text{lbf}}{\text{ft}^2}$ Effective Cohesion

Undrained Conditions (Total Stress):

$\phi_{fdu} := 0 \text{ deg}$ Angle of internal friction (Same as Drained Conditions if Sand)

$Su_{fdu} := 2250 \frac{\text{lbf}}{\text{ft}^2}$ Undrained Shear Strength

Undercut & Replacement Design Parameters:

$\phi_{Re} := 34 \text{ deg}$ Angle of internal friction for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$c_{Re} := 0 \frac{\text{lbf}}{\text{ft}^2}$ Cohesion for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$\gamma_{Re} := 130 \frac{\text{lbf}}{\text{ft}^3}$ Unit Weight for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$\delta_{Re} := 0.67 \cdot \phi_{Re}$ $\delta_{Re} = 22.8 \text{ deg}$ Friction angle between Replacement soils and footing taken as specified in LRFD BDS C3.11.5.3 (degrees)

$D_{undercut} := 0 \text{ ft}$ Depth of Undercut below bottom of footing

Foundation Surcharge Soil Parameters:

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

Footing Geometry:

$D_f := 0 \text{ ft}$ Footing cover at Toe
Note: Unless on rock, top of footing should be at least 1-ft from soil surface and bottom of footing at least 5-ft from nearest soil surface per BDM 202.2.3.1a

$D_F := D_f + D_{undercut}$ Embenment Depth at bottom of Undercut

$B := 17 \text{ ft}$ Footing base width

$B' := B$ Footing effective base width

$L' := 231 \text{ ft}$ Footing effective length (assumed to equal actual length)

$d_w := 0 \text{ ft}$ Depth of groundwater below ground surface

Compute Bearing Resistance:

Drained Conditions (Effective Stress):

$$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right) \quad N_q = 11.85$$

$$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right) \quad N_c = 22.25$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd}) \quad N_\gamma = 12.5$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right) \quad s_c = 1.039$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right) \quad s_q = 1.036$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right) \quad s_\gamma = 0.971$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see LRFD BDS C10.6.3.1.2a.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5) \quad C_{wq} = 1$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \quad C_{w\gamma} = 0.5$$

Depth Correction Factor per ODOT BDM 305.2.1:

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'} \right)$$

$$d_q = 1$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 23.127$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 12.28$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 12.17$$

Compute nominal bearing resistance at bottom of Undercut & Replacement, LRFD [Eq 10.6.3.1.2a-1]:

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nd} = 10418.1 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.5$$

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 5.2 \text{ ksf}$$

Bearing resistance factor LRFD Table 11.5.5.2.2-1.

$\phi_b = 0.5$ in clay; $\phi_b = 0.45$ in sand

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

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

$$s_c = 1.015$$

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

$$s_q = 1$$

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

$$s_\gamma = 1$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see LRFD BDS C10.6.3.1.2a.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

Depth Correction Factor per ODOT BDM 305.2.1:

$$d_q := 1 + 2 \cdot \tan(\phi_{fdu}) \cdot (1 - \sin(\phi_{fdu}))^2 \cdot \tan \left(\frac{D_F}{B'} \right)$$

$$d_q = 1$$

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

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

$$N_{cm} = 5.216$$

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

$$N_{qm} = 1$$

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

$$N_{\gamma m} = 0$$

Compute nominal bearing resistance at bottom of Undercut & Replacement, LRFD [Eq 10.6.3.1.2a-1]:

$$q_{nu} := S_{u_{fdu}} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nu} = 11735.2 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.5$$

Bearing resistance factor LRFD Table 11.5.5.2.2-1.

$\phi_b = 0.5$ in clay; $\phi_b = 0.45$ in sand

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 5.9 \text{ ksf}$$

Factored bearing resistance Undrained
Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 5.2 \text{ ksf}$

Undrained Conditions: $q_{Ru} = 5.9 \text{ ksf}$

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[illegible]

INPUT DATA FOR CONSOLIDATION — $\alpha = 1/2$

[illegible]

IMMEDIATE SETTLEMENT, Si

Node #	Settlement along section:		Layer (k)	Young's Modulus, E [lb/ft ²]	Poisson's Ratio, μ	Settlement of each layer, Si(k) [ft.]	Initial Z [ft.]	Final Z * [ft.]	Total Settlement Sum of Si(k), [ft.]
	X	Y							
	[ft.]	[ft.]							
1	-150.00	0.00	1	650000	0.3500	-0.0000	1029.10	1029.10	0.00
			2	500000	0.3500	-0.0002			
			3	200000	0.3000	-0.0005			
			4	300000	0.3000	0.0041			
2	-128.57	0.00	1	650000	0.3500	-0.0000	1029.10	1029.08	0.02
			2	500000	0.3500	-0.0005			
			3	200000	0.3000	-0.0010			
			4	300000	0.3000	0.0194			
3	-107.14	0.00	1	650000	0.3500	-0.0006	1029.10	1029.00	0.10
			2	500000	0.3500	0.0015			
			3	200000	0.3000	0.0154			
			4	300000	0.3000	0.0791			
4	-85.71	0.00	1	650000	0.3500	0.0071	1029.10	1028.84	0.26
			2	500000	0.3500	0.0297			
			3	200000	0.3000	0.0626			
			4	300000	0.3000	0.1654			
5	-64.29	0.00	1	650000	0.3500	0.0071	1029.10	1028.81	0.29
			2	500000	0.3500	0.0291			
			3	200000	0.3000	0.0628			
			4	300000	0.3000	0.1913			
6	-42.86	0.00	1	650000	0.3500	0.0071	1029.10	1028.80	0.30
			2	500000	0.3500	0.0289			
			3	200000	0.3000	0.0625			
			4	300000	0.3000	0.1982			
7	-21.43	0.00	1	650000	0.3500	0.0071	1029.10	1028.80	0.30
			2	500000	0.3500	0.0289			
			3	200000	0.3000	0.0624			
			4	300000	0.3000	0.2001			
8	0.00	0.00	1	650000	0.3500	0.0071	1029.10	1028.80	0.30
			2	500000	0.3500	0.0289			
			3	200000	0.3000	0.0623			
			4	300000	0.3000	0.2005			
9	21.43	0.00	1	650000	0.3500	0.0071	1029.10	1028.80	0.30
			2	500000	0.3500	0.0289			
			3	200000	0.3000	0.0624			
			4	300000	0.3000	0.2001			
10	42.86	0.00	1	650000	0.3500	0.0071	1029.10	1028.80	0.30
			2	500000	0.3500	0.0289			
			3	200000	0.3000	0.0625			
			4	300000	0.3000	0.1982			
11	64.29	0.00	1	650000	0.3500	0.0071	1029.10	1028.81	0.29
			2	500000	0.3500	0.0291			
			3	200000	0.3000	0.0628			
			4	300000	0.3000	0.1913			
12	85.71	0.00	1	650000	0.3500	0.0071	1029.10	1028.84	0.26
			2	500000	0.3500	0.0297			
			3	200000	0.3000	0.0626			
			4	300000	0.3000	0.1654			

*Note: Final Z is calculated assuming only 'Immediate Settlement' exists.

[illegible]

IMMEDIATE SETTLEMENT, Si

Node #	Settlement along section:		Layer	Young's Modulus, E	Poisson's Ratio, μ	Settlement of each layer, Si(k)	Initial Z	Final Z *	Total Settlement Sum of Si(k),
	X	Y	(k)	[lb/ft ²]		[ft.]	[ft.]	[ft.]	[ft.]
13	107.14	0.00	1	650000	0.3500	-0.0006	1029.10	1029.00	0.10
			2	500000	0.3500	0.0015			
			3	200000	0.3000	0.0154			
			4	300000	0.3000	0.0791			
14	128.57	0.00	1	650000	0.3500	-0.0000	1029.10	1029.08	0.02
			2	500000	0.3500	-0.0005			
			3	200000	0.3000	-0.0010			
			4	300000	0.3000	0.0193			
15	150.00	0.00	1	650000	0.3500	-0.0000	1029.10	1029.10	0.00
			2	500000	0.3500	-0.0002			
			3	200000	0.3000	-0.0005			
			4	300000	0.3000	0.0041			

*Note: Final Z is calculated assuming only 'Immediate Settlement' exists.

ULTIMATE SETTLEMENT, Sc

Approximately one end of culvert

*Note: Final Z is calculated assuming only 'Ultimate Settlement' exists.

TABULATED GEOMETRY: INPUT OF FOUNDATION SOILS

Found. Soil #	Point #	Coordinates (X, Z) :		DESCRIPTION
		(X) [ft.]	(Z) [ft.]	
1	1	-62.50	1029.10	A-4b
	2	0.00	1029.10	
	3	62.50	1029.10	
2	1	-62.50	1026.30	A-4a
	2	0.00	1026.30	
	3	62.50	1026.30	
3	1	-62.50	1019.30	A-1-b
	2	0.00	1019.30	
	3	62.50	1019.30	
4	1	-62.50	1013.30	Bedrock
	2	0.00	1013.30	
	3	62.50	1013.30	

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Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT BDM[Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range -- uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	OR 0		OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5		
		General Conditions	OR Completely Dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 0 + 3 + 20 + 12 + 7 = 42$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 10$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 200 \text{ psi}$$

Unconfined compressive strength of rock core, GDM Table 400-6

$$\gamma_q := 120 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**)

$$\gamma_r := 140 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (**Hoek and Brown, 1988**):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.126005645$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.0158932728$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{jd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$c'_{jd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{jd} = 26 \text{ deg}$$

$$c'_{jd} = 4368 \text{ psf}$$

Abutment Loading from bridge designer:

Service I loads:

$$V_{Ia} := 73400 \frac{\text{lbf}}{\text{ft}}$$

Service I Vertical Load

$$H_{Ia} := 15300 \frac{\text{lbf}}{\text{ft}}$$

Service I Horizontal Load

$$M_{Ia} := 190500 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Service I Moment

Strength I loads:

$$V_{Ib} := 101300 \frac{\text{lbf}}{\text{ft}}$$

Strength I Vertical Load

$$H_{Ib} := 23500 \frac{\text{lbf}}{\text{ft}}$$

Strength I Horizontal Load

$$M_{Ib} := 291700 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Strength I Moment

Footing Geometry:

Preliminary Wall Dimensioning:

$$B := 12 \text{ ft}$$

Footing base width

$$A := 2 \text{ ft}$$

Toe projection

$$D := 4 \text{ ft}$$

Footing thickness

$$D_f := 12.45 \text{ ft}$$

Depth to bottom of footing below proposed ground surface

$$L' := 114 \text{ ft}$$

Footing Length

$$d_w := 0 \text{ ft}$$

Depth of groundwater below proposed ground surface

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{jd} = 26 \text{ deg}$$

Effective angle of internal friction, from above RMR estimation

$$\gamma_{jd} := \gamma_r = 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{jd} = 4368 \text{ psf}$$

Effective Cohesion, from above RMR estimation

$$\delta_{jd} := \frac{2}{3} \cdot \phi'_{jd} \quad \delta_{jd} = 17.3 \text{ deg}$$

Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Bearing Resistance for Weak and Flat Bedded Rock:

$$e_{lb} := \frac{M_{lb}}{V_{lb}} = 2.9 \text{ ft}$$

$$B' := B - 2 \cdot e_{lb} = 6.2 \text{ ft}$$

Eccentricity.

The effective bearing width is equal to B-2e.

Drained Conditions (Effective Stress):

$$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right) \quad N_q = 11.85$$

$$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right) \quad N_c = 22.25$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd}) \quad N_\gamma = 12.5$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right) \quad s_c = 1.029$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right) \quad s_q = 1.027$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right) \quad s_\gamma = 0.978$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wg} := \text{if} (d_w \geq D_f, 1.0, 0.5) \quad C_{wg} = 0.5$$

$$C_{wy} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \quad C_{wy} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_f}{B'} \right)$$

$$d_q = 1.34$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 22.903$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 16.313$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 12.264$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 114906.6 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 51.7 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 51.7 \text{ ksf}$

Evaluate External Stability of Wall:

Compute the factored bearing stress:

$$\sigma_V := \frac{V_{lb}}{B'}$$

$$\sigma_V = 16.232 \text{ ksf}$$

Factored bearing resistance

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions:

$$CDR_{Bearing_D} := \frac{q_{Rd}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{Bearing_D} = 3.19$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 \cdot B}{20}$$

$$e_{max} = 5.4 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
within middle 9B/10 for rock.

$$e_{Ia} := \frac{M_{Ia}}{V_{Ia}} = 2.6 \text{ ft}$$

Eccentricity.

$$B' := B - 2 \cdot e_{Ia} = 6.8 \text{ ft}$$

The effective bearing width is equal to B-2e.

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{Ia}}$$

Is the CDR > or = to 1.0?

$$CDR_{Eccentricity} = 2.08$$

Calculations:

Earth Pressure Coefficients:

$$\beta' := 0 \text{ deg}$$

Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, $\beta' = 0$ deg. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

Foundation Soil Passive Earth:

Drained Conditions assuming ($\phi'_{fd} > 0$):

Input Parameters for **LRFD Figure 3.11.5.4-2**, assumes $\theta = 90$ degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0$$

$$\frac{-\delta_{fd}}{\phi'_{fd}} = -0.667$$

$$k'_p := 4.5$$

Passive Earth Pressure Coefficient
from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.86$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p$$

$$k_{pd} = 3.87$$

Passive Earth Pressure Coefficient for
Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

$$y_1 := 3.25 \text{ ft}$$

$$y_1 = 3.3 \text{ ft}$$

Depth to where passive pressure may begin to be utilized in front of wall.

$$y_2 := D_f$$

$$y_2 = 12.5 \text{ ft}$$

Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

$$\phi'_s := 29 \text{ deg}$$

Friction angle at bedrock, GDM 1303.3.5

Factored Sliding Force (Strength Ia):

$$R_u := H_{Ia}$$

$$R_u = 15300 \frac{\text{lbf}}{\text{ft}}$$

Drained Conditions (Effective Stress):

Compute passive resistance throughout the design life of the wall **LRFD [Eq 3.11.5.4-1]:**

$$r_{ep1} := (k_{pd} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}) \cdot \cos(\delta_{fd})$$

Nominal passive pressure at y1

$$r_{ep2} := (k_{pd} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}) \cdot \cos(\delta_{fd})$$

Nominal passive pressure at y2

$$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot (y_2 - y_1)$$

$$R_{ep} = 188280.5 \frac{\text{lbf}}{\text{ft}}$$

Nominal passive resistance Drained Conditions

416 Note: Passive Resistance shall be neglected in stability computations, unless the base of the wall extends below the depth of maximum scour, freeze-thaw or other disturbances. In the latter case, only the embedment below the greater of these depths shall be considered effective **LRFD [11.6.3.5]**.

Compute sliding resistance between soil and foundation:

$$c := 1.0$$

c = 1.0 for Cast-in-Place
c = 0.8 for Precast

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 73400 \frac{\text{lbf}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{\tau} := c \cdot \Sigma V \cdot \tan(\phi'_s)$$

$$R_{\tau} = 40686.3 \frac{\text{lbf}}{\text{ft}}$$

Nominal sliding resistance Under Drained Condition

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]**:

$$\phi_{ep} := 0.5$$

Resistance factor for passive resistance specified in
LRFD Table 10.5.5.2.2-1

$$\phi_{\tau} := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau} + \phi_{ep} \cdot R_{ep}$$

$$R_R := \phi R_n$$

Factored Sliding Resistance to be used in CDR Calculations:

$$R_R = 134826.551 \frac{\text{lbf}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding} = 8.81$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT BDM[Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range -- uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	OR 0		OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5		
		General Conditions	OR Completely Dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 0 + 3 + 20 + 12 + 7 = 42$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 10$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 1125 \text{ psi}$$

Unconfined compressive strength of rock core, GDM Table 400-6

$$\gamma_q := 120 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**)

$$\gamma_r := 140 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (**Hoek and Brown, 1988**):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.126005645$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.0158932728$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{jd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$c'_{jd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{jd} = 26 \text{ deg}$$

$$c'_{jd} = 4368 \text{ psf}$$

Abutment Loading from bridge designer:

Service I loads:

$$V_{Ia} := 73400 \frac{\text{lbf}}{\text{ft}}$$

Service I Vertical Load

$$H_{Ia} := 15300 \frac{\text{lbf}}{\text{ft}}$$

Service I Horizontal Load

$$M_{Ia} := 190500 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Service I Moment

Strength I loads:

$$V_{Ib} := 101300 \frac{\text{lbf}}{\text{ft}}$$

Strength I Vertical Load

$$H_{Ib} := 23500 \frac{\text{lbf}}{\text{ft}}$$

Strength I Horizontal Load

$$M_{Ib} := 291700 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Strength I Moment

Footing Geometry:

Preliminary Wall Dimensioning:

$$B := 12 \text{ ft}$$

Footing base width

$$A := 2 \text{ ft}$$

Toe projection

$$D := 4 \text{ ft}$$

Footing thickness

$$D_f := 10.2 \text{ ft}$$

Depth to bottom of footing below proposed ground surface

$$L' := 114 \text{ ft}$$

Footing Length

$$d_w := 0 \text{ ft}$$

Depth of groundwater below proposed ground surface

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{jd} = 26 \text{ deg}$$

Effective angle of internal friction, from above RMR estimation

$$\gamma_{jd} := \gamma_r = 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{jd} = 4368 \text{ psf}$$

Effective Cohesion, from above RMR estimation

$$\delta_{jd} := \frac{2}{3} \cdot \phi'_{jd} \quad \delta_{jd} = 17.3 \text{ deg}$$

Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Bearing Resistance for Weak and Flat Bedded Rock:

$$e_{lb} := \frac{M_{lb}}{V_{lb}} = 2.9 \text{ ft}$$

$$B' := B - 2 \cdot e_{lb} = 6.2 \text{ ft}$$

Eccentricity.

The effective bearing width is equal to B-2e.

Drained Conditions (Effective Stress):

$$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right) \quad N_q = 11.85$$

$$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right) \quad N_c = 22.25$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd}) \quad N_\gamma = 12.5$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right) \quad s_c = 1.029$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right) \quad s_q = 1.027$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right) \quad s_\gamma = 0.978$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5) \quad C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \quad C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_f}{B'} \right)$$

$$d_q = 1.31$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 22.903$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 15.997$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 12.264$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 112510.8 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 50.6 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 50.6 \text{ ksf}$$

Evaluate External Stability of Wall:

Compute the factored bearing stress:

$$\sigma_V := \frac{V_{lb}}{B'}$$

$$\sigma_V = 16.232 \text{ ksf}$$

Factored bearing resistance

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions:

$$CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 3.12$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 \cdot B}{20}$$

$$e_{max} = 5.4 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
within middle 9B/10 for rock.

$$e_{Ia} := \frac{M_{Ia}}{V_{Ia}} = 2.6 \text{ ft}$$

Eccentricity.

$$B' := B - 2 \cdot e_{Ia} = 6.8 \text{ ft}$$

The effective bearing width is equal to B-2e.

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{\text{Eccentricity}} := \frac{e_{max}}{e_{Ia}}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Eccentricity}} = 2.08$$

Calculations:

Earth Pressure Coefficients:

$$\beta' := 0 \text{ deg}$$

Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, $\beta' = 0$ deg. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

Foundation Soil Passive Earth:

Drained Conditions assuming ($\phi'_{fd} > 0$):

Input Parameters for **LRFD Figure 3.11.5.4-2**, assumes $\theta = 90$ degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0$$

$$\frac{-\delta_{fd}}{\phi'_{fd}} = -0.667$$

$$k'_p := 4.5$$

Passive Earth Pressure Coefficient
from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.86$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p$$

$$k_{pd} = 3.87$$

Passive Earth Pressure Coefficient for
Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

$$y_1 := 3.25 \text{ ft}$$

$$y_1 = 3.3 \text{ ft}$$

Depth to where passive pressure may begin to be utilized in front of wall.

$$y_2 := D_f$$

$$y_2 = 10.2 \text{ ft}$$

Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

$$\phi'_s := 29 \text{ deg}$$

Friction angle at bedrock, GDM 1303.3.5

Factored Sliding Force (Strength Ia):

$$R_u := H_{Ia}$$

$$R_u = 15300 \frac{\text{lbf}}{\text{ft}}$$

Drained Conditions (Effective Stress):

Compute passive resistance throughout the design life of the wall **LRFD [Eq 3.11.5.4-1]**:

$$r_{ep1} := (k_{pd} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}) \cdot \cos(\delta_{fd})$$

Nominal passive pressure at y1

$$r_{ep2} := (k_{pd} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}) \cdot \cos(\delta_{fd})$$

Nominal passive pressure at y2

$$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot (y_2 - y_1)$$

$$R_{ep} = 138189.8 \frac{\text{lbf}}{\text{ft}}$$

Nominal passive resistance Drained Conditions

416 Note: Passive Resistance shall be neglected in stability computations, unless the base of the wall extends below the depth of maximum scour, freeze-thaw or other disturbances. In the latter case, only the embedment below the greater of these depths shall be considered effective **LRFD [11.6.3.5]**.

Compute sliding resistance between soil and foundation:

$$c := 1.0$$

c = 1.0 for Cast-in-Place
c = 0.8 for Precast

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 73400 \frac{\text{lbf}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{\tau} := c \cdot \Sigma V \cdot \tan(\phi'_s)$$

$$R_{\tau} = 40686.3 \frac{\text{lbf}}{\text{ft}}$$

Nominal sliding resistance Under Drained Condition

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]**:

$$\phi_{ep} := 0.5$$

Resistance factor for passive resistance specified in
LRFD Table 10.5.5.2.2-1

$$\phi_{\tau} := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau} + \phi_{ep} \cdot R_{ep}$$

$$R_R := \phi R_n$$

Factored Sliding Resistance to be used in CDR Calculations:

$$R_R = 109781.204 \frac{\text{lbf}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding} = 7.18$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT BDM[Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	0		0.0 to 0.2	0.2 to 0.5	>0.5		
		General Conditions	Completely Dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 2 + 17 + 20 + 12 + 7 = 58$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 2600 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_q := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**)

$$\gamma_r := 141.7 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (**Hoek and Brown, 1988**):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.2231301601$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.1410534383$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 34 \text{ deg}$$

$$c'_{fd} = 6032 \text{ psf}$$

Abutment Loading from bridge designer:

Service I loads:

$$V_{Ia} := 26900 \frac{\text{lbf}}{\text{ft}}$$

Service I Vertical Load

$$M_{Ia} := 67800 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Service I Moment

Strength I loads:

$$V_{Ib} := 35600 \frac{\text{lbf}}{\text{ft}}$$

Strength I Vertical Load

$$H_{Ib} := 13900 \frac{\text{lbf}}{\text{ft}}$$

Strength I Horizontal Load

$$M_{Ib} := 106000 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Strength I Moment

Footing Geometry:

Preliminary Wall Dimensioning:

$$B := 9.75 \text{ ft}$$

Footing base width

$$A := 1.29 \text{ ft}$$

Toe projection

$$D := 3 \text{ ft}$$

Footing thickness

$$D_f := 5 \text{ ft}$$

Depth to bottom of footing below proposed ground surface

$$L' := 28 \text{ ft}$$

Footing Length

$$d_w := 0 \text{ ft}$$

Depth of groundwater below proposed ground surface

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 34 \text{ deg}$$

Effective angle of internal friction, from
above RMR estimation

$$\gamma_{fd} := \gamma_r = 141.7 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 6032 \text{ psf}$$

Effective Cohesion, from
above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd} \quad \delta_{fd} = 22.7 \text{ deg}$$

Friction angle between foundation soils and footing
taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Bearing Resistance for Weak and Flat Bedded Rock:

$$e_{fb} := \frac{M_{fb}}{V_{fb}} = 3 \text{ ft}$$

$$B' := B - 2 \cdot e_{fb} = 3.8 \text{ ft}$$

Eccentricity.

The effective bearing width is equal to B-2e.

Drained Conditions (Effective Stress):

$$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right) \quad N_q = 29.44$$

$$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right) \quad N_c = 42.16$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd}) \quad N_\gamma = 41.1$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right) \quad s_c = 1.095$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right) \quad s_q = 1.091$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right) \quad s_\gamma = 0.946$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5) \quad C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \quad C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_f}{B'} \right)$$

$$d_q = 1.24$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 46.154$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 39.893$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 38.838$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$
$$q_{nd} = 295588.9 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$
$$q_{Rd} = 133 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 133 \text{ ksf}$

Evaluate External Stability of Wall:

Compute the factored bearing stress :

Factored bearing resistance

$$\sigma_V := \frac{V_{lb}}{B'}$$
$$\sigma_V = 9.381 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$ Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 14.18$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 \cdot B}{20}$$
$$e_{max} = 4.4 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
within middle 9B/10 for rock.

$$e_{Ia} := \frac{M_{Ia}}{V_{Ia}} = 2.5 \text{ ft}$$

Eccentricity.

$$B' := B - 2 \cdot e_{Ia} = 4.7 \text{ ft}$$

The effective bearing width is equal to B-2e.

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{\text{Eccentricity}} := \frac{e_{max}}{e_{Ia}}$ Is the CDR > or = to 1.0?

$$CDR_{\text{Eccentricity}} = 1.74$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT BDM[Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	0		0.0 to 0.2	0.2 to 0.5	>0.5		
		General Conditions	Completely Dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 2 + 17 + 20 + 12 + 7 = 58$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 2600 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_q := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**)

$$\gamma_r := 141.7 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (**Hoek and Brown, 1988**):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.2231301601$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.1410534383$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 34 \text{ deg}$$

$$c'_{fd} = 6032 \text{ psf}$$

Abutment Loading from bridge designer:

Service I loads:

$$V_{Ia} := 49100 \frac{\text{lbf}}{\text{ft}}$$

Service I Vertical Load

$$M_{Ia} := 159500 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Service I Moment

Strength I loads:

$$V_{Ib} := 67100 \frac{\text{lbf}}{\text{ft}}$$

Strength I Vertical Load

$$H_{Ib} := 19000 \frac{\text{lbf}}{\text{ft}}$$

Strength I Horizontal Load

$$M_{Ib} := 223100 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Strength I Moment

Footing Geometry:

Preliminary Wall Dimensioning:

$$B := 8.75 \text{ ft}$$

Footing base width

$$A := 1.54 \text{ ft}$$

Toe projection

$$D := 3 \text{ ft}$$

Footing thickness

$$D_f := 5 \text{ ft}$$

Depth to bottom of footing below proposed ground surface

$$L' := 68 \text{ ft}$$

Footing Length

$$d_w := 0 \text{ ft}$$

Depth of groundwater below proposed ground surface

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 34 \text{ deg}$$

Effective angle of internal friction, from
above RMR estimation

$$\gamma_{fd} := \gamma_r = 141.7 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 6032 \text{ psf}$$

Effective Cohesion, from
above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd} \quad \delta_{fd} = 22.7 \text{ deg}$$

Friction angle between foundation soils and footing
taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Bearing Resistance for Weak and Flat Bedded Rock:

$$e_{fb} := \frac{M_{fb}}{V_{fb}} = 3.3 \text{ ft}$$

$$B' := B - 2 \cdot e_{fb} = 2.1 \text{ ft}$$

Eccentricity.

The effective bearing width is equal to B-2e.

Drained Conditions (Effective Stress):

$$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right) \quad N_q = 29.44$$

$$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right) \quad N_c = 42.16$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd}) \quad N_\gamma = 41.1$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right) \quad s_c = 1.022$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right) \quad s_q = 1.021$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right) \quad s_\gamma = 0.988$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5) \quad C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \quad C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_f}{B'} \right)$$

$$d_q = 1.31$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 43.073$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 39.295$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 40.556$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$
$$q_{nd} = 274622.1 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$
$$q_{Rd} = 123.6 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 123.6 \text{ ksf}$

Evaluate External Stability of Wall:

Compute the factored bearing stress :

Factored bearing resistance

$$\sigma_V := \frac{V_{lb}}{B'}$$
$$\sigma_V = 31.949 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$ Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 3.87$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 \cdot B}{20}$$
$$e_{max} = 3.9 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
within middle 9B/10 for rock.

$$e_{Ia} := \frac{M_{Ia}}{V_{Ia}} = 3.2 \text{ ft}$$

Eccentricity.

$$B' := B - 2 \cdot e_{Ia} = 2.3 \text{ ft}$$

The effective bearing width is equal to B-2e.

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{\text{Eccentricity}} := \frac{e_{max}}{e_{Ia}}$ Is the CDR > or = to 1.0?

$$CDR_{\text{Eccentricity}} = 1.21$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT BDM[Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	0		0.0 to 0.2	0.2 to 0.5	>0.5		
		General Conditions	Completely Dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 4 + 17 + 20 + 12 + 7 = 60$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 6587 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_q := 120 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**)

$$\gamma_r := 136 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (**Hoek and Brown, 1988**):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.2396510364$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.1761544269$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 35 \text{ deg}$$

$$c'_{fd} = 6240 \text{ psf}$$

Abutment Loading from bridge designer:

Service I loads:

$$V_{Ia} := 461700 \frac{\text{lbf}}{\text{ft}}$$

Service I Vertical Load

$$H_{Ia} := 23000 \frac{\text{lbf}}{\text{ft}}$$

Service I Horizontal Load

$$M_{Ia} := 309120 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Service I Moment

Strength I loads:

$$V_{Ib} := 624340 \frac{\text{lbf}}{\text{ft}}$$

Strength I Vertical Load

$$H_{Ib} := 28600 \frac{\text{lbf}}{\text{ft}}$$

Strength I Horizontal Load

$$M_{Ib} := 447760 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Strength I Moment

Footing Geometry:

Preliminary Wall Dimensioning:

$$B := 12 \text{ ft}$$

Footing base width

$$A := 4.5 \text{ ft}$$

Toe projection

$$D := 3 \text{ ft}$$

Footing thickness

$$D_f := 6 \text{ ft}$$

Depth to bottom of footing below proposed ground surface

$$L' := 12 \text{ ft}$$

Footing Length

$$d_w := 0 \text{ ft}$$

Depth of groundwater below proposed ground surface

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 35 \text{ deg}$$

Effective angle of internal friction, from
above RMR estimation

$$\gamma_{fd} := \gamma_r = 136 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 6240 \text{ psf}$$

Effective Cohesion, from
above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd}$$

$$\delta_{fd} = 23.3 \text{ deg}$$

Friction angle between foundation soils and footing
taken as specified in LRFD BDS C3.11.5.3 (degrees)

Bearing Resistance for Weak and Flat Bedded Rock:

$$e_{lb} := \frac{M_{lb}}{V_{lb}} = 0.7 \text{ ft}$$

$$B' := B - 2 \cdot e_{lb} = 10.6 \text{ ft}$$

Eccentricity.

The effective bearing width is equal to B-2e.

Drained Conditions (Effective Stress):

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

$$N_q = 33.3$$

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

$$N_c = 46.12$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 48$$

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

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

$$s_c = 1.636$$

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

$$s_q = 1.617$$

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

$$s_\gamma = 0.648$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_f}{B'} \right)$$

$$d_q = 1.13$$

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

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

$$N_{cm} = 75.44$$

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

$$N_{qm} = 60.902$$

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

$$N_{\gamma m} = 31.114$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 503846.4 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 226.7 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 226.7 \text{ ksf}$

Evaluate External Stability of Wall:

Compute the factored bearing stress:

Factored bearing resistance

$$\sigma_V := \frac{V_{lb}}{B'}$$

$$\sigma_V = 59.091 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions:

$$CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 3.84$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 \cdot B}{20}$$

$$e_{max} = 5.4 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
within middle 9B/10 for rock.

$$e_{Ia} := \frac{M_{Ia}}{V_{Ia}} = 0.7 \text{ ft}$$

Eccentricity.

$$B' := B - 2 \cdot e_{Ia} = 10.7 \text{ ft}$$

The effective bearing width is equal to B-2e.

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{\text{Eccentricity}} := \frac{e_{max}}{e_{Ia}}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Eccentricity}} = 8.07$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT BDM[Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	0		0.0 to 0.2	0.2 to 0.5	>0.5		
		General Conditions	Completely Dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 4 + 17 + 20 + 12 + 7 = 60$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 6587 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_q := 120 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**)

$$\gamma_r := 136 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (**Hoek and Brown, 1988**):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.2396510364$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.1761544269$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 35 \text{ deg}$$

$$c'_{fd} = 6240 \text{ psf}$$

Abutment Loading from bridge designer:

Service I loads:

$$V_{Ia} := 727780 \frac{\text{lbf}}{\text{ft}}$$

Service I Vertical Load

$$H_{Ia} := 9700 \frac{\text{lbf}}{\text{ft}}$$

Service I Horizontal Load

$$M_{Ia} := 308110 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Service I Moment

Strength I loads:

$$V_{Ib} := 1066790 \frac{\text{lbf}}{\text{ft}}$$

Strength I Vertical Load

$$H_{Ib} := 12700 \frac{\text{lbf}}{\text{ft}}$$

Strength I Horizontal Load

$$M_{Ib} := 438300 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Strength I Moment

Footing Geometry:

Preliminary Wall Dimensioning:

$$B := 9 \text{ ft}$$

Footing base width

$$A := 3 \text{ ft}$$

Toe projection

$$D := 3 \text{ ft}$$

Footing thickness

$$D_f := 6 \text{ ft}$$

Depth to bottom of footing below proposed ground surface

$$L' := 9 \text{ ft}$$

Footing Length

$$d_w := 0 \text{ ft}$$

Depth of groundwater below proposed ground surface

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 35 \text{ deg}$$

Effective angle of internal friction, from
above RMR estimation

$$\gamma_{fd} := \gamma_r = 136 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 6240 \text{ psf}$$

Effective Cohesion, from
above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd}$$

$$\delta_{fd} = 23.3 \text{ deg}$$

Friction angle between foundation soils and footing
taken as specified in LRFD BDS C3.11.5.3 (degrees)

Bearing Resistance for Weak and Flat Bedded Rock:

$$e_{lb} := \frac{M_{lb}}{V_{lb}} = 0.4 \text{ ft}$$

$$B' := B - 2 \cdot e_{lb} = 8.2 \text{ ft}$$

Eccentricity.

The effective bearing width is equal to B-2e.

Drained Conditions (Effective Stress):

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

$$N_q = 33.3$$

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

$$N_c = 46.12$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 48$$

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

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

$$s_c = 1.656$$

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

$$s_q = 1.636$$

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

$$s_\gamma = 0.637$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see LRFD BDS C10.6.3.1.2a. "Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_f}{B'} \right)$$

$$d_q = 1.16$$

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

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

$$N_{cm} = 76.38$$

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

$$N_{qm} = 63.263$$

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

$$N_{\gamma m} = 30.571$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 507884.7 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 228.5 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 228.5 \text{ ksf}$$

Evaluate External Stability of Wall:

Compute the factored bearing stress:

Factored bearing resistance

$$\sigma_V := \frac{V_{lb}}{B'}$$

$$\sigma_V = 130.442 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions:

$$CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 1.75$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 \cdot B}{20}$$

$$e_{max} = 4.1 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
within middle 9B/10 for rock.

$$e_{Ia} := \frac{M_{Ia}}{V_{Ia}} = 0.4 \text{ ft}$$

Eccentricity.

$$B' := B - 2 \cdot e_{Ia} = 8.2 \text{ ft}$$

The effective bearing width is equal to B-2e.

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{\text{Eccentricity}} := \frac{e_{max}}{e_{Ia}}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Eccentricity}} = 9.57$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT BDM[Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	0		0.0 to 0.2	0.2 to 0.5	>0.5		
		General Conditions	Completely Dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 4 + 17 + 20 + 12 + 7 = 60$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 5519 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_q := 120 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**)

$$\gamma_r := 138.8 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (**Hoek and Brown, 1988**):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.2396510364$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.1761544269$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 35 \text{ deg}$$

$$c'_{fd} = 6240 \text{ psf}$$

Abutment Loading from bridge designer:

Service I loads:

$$V_{Ia} := 26900 \frac{\text{lbf}}{\text{ft}}$$

Service I Vertical Load

$$M_{Ia} := 67800 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Service I Moment

Strength I loads:

$$V_{Ib} := 35600 \frac{\text{lbf}}{\text{ft}}$$

Strength I Vertical Load

$$H_{Ib} := 13900 \frac{\text{lbf}}{\text{ft}}$$

Strength I Horizontal Load

$$M_{Ib} := 106000 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Strength I Moment

Footing Geometry:

Preliminary Wall Dimensioning:

$$B := 9.75 \text{ ft}$$

Footing base width

$$A := 1.29 \text{ ft}$$

Toe projection

$$D := 3 \text{ ft}$$

Footing thickness

$$D_f := 5 \text{ ft}$$

Depth to bottom of footing below proposed ground surface

$$L' := 28 \text{ ft}$$

Footing Length

$$d_w := 0 \text{ ft}$$

Depth of groundwater below proposed ground surface

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 35 \text{ deg}$$

Effective angle of internal friction, from
above RMR estimation

$$\gamma_{fd} := \gamma_r = 138.8 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 6240 \text{ psf}$$

Effective Cohesion, from
above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd} \quad \delta_{fd} = 23.3 \text{ deg}$$

Friction angle between foundation soils and footing
taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Bearing Resistance for Weak and Flat Bedded Rock:

$$e_{fb} := \frac{M_{fb}}{V_{fb}} = 3 \text{ ft}$$

$$B' := B - 2 \cdot e_{fb} = 3.8 \text{ ft}$$

Eccentricity.

The effective bearing width is equal to B-2e.

Drained Conditions (Effective Stress):

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

$$N_q = 33.3$$

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

$$N_c = 46.12$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 48$$

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

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

$$s_c = 1.098$$

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

$$s_q = 1.095$$

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

$$s_\gamma = 0.946$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_f}{B'} \right)$$

$$d_q = 1.23$$

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

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

$$N_{cm} = 50.636$$

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

$$N_{qm} = 45.011$$

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

$$N_{\gamma m} = 45.425$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$
$$q_{nd} = 335455.9 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$
$$q_{Rd} = 151 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 151 \text{ ksf}$

Evaluate External Stability of Wall:

Compute the factored bearing stress :

Factored bearing resistance

$$\sigma_V := \frac{V_{lb}}{B'}$$
$$\sigma_V = 9.381 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$ Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 16.09$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 \cdot B}{20}$$
$$e_{max} = 4.4 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
within middle 9B/10 for rock.

$$e_{Ia} := \frac{M_{Ia}}{V_{Ia}} = 2.5 \text{ ft}$$

Eccentricity.

$$B' := B - 2 \cdot e_{Ia} = 4.7 \text{ ft}$$

The effective bearing width is equal to B-2e.

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{\text{Eccentricity}} := \frac{e_{max}}{e_{Ia}}$ Is the CDR > or = to 1.0?

$$CDR_{\text{Eccentricity}} = 1.74$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT BDM[Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range -- uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	OR 0		OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5		
		General Conditions	OR Completely Dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 4 + 17 + 20 + 12 + 7 = 60$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 5519 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_q := 120 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**)

$$\gamma_r := 138.8 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (**Hoek and Brown, 1988**):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.2396510364$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.1761544269$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 35 \text{ deg}$$

$$c'_{fd} = 6240 \text{ psf}$$

Abutment Loading from bridge designer:

Service I loads:

$$V_{Ia} := 41600 \frac{\text{lbf}}{\text{ft}}$$

Service I Vertical Load

$$M_{Ia} := 113000 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Service I Moment

Strength I loads:

$$V_{Ib} := 57100 \frac{\text{lbf}}{\text{ft}}$$

Strength I Vertical Load

$$H_{Ib} := 14600 \frac{\text{lbf}}{\text{ft}}$$

Strength I Horizontal Load

$$M_{Ib} := 155800 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Strength I Moment

Footing Geometry:

Preliminary Wall Dimensioning:

$$B := 8.75 \text{ ft}$$

Footing base width

$$A := 1.54 \text{ ft}$$

Toe projection

$$D := 3 \text{ ft}$$

Footing thickness

$$D_f := 5 \text{ ft}$$

Depth to bottom of footing below proposed ground surface

$$L' := 68 \text{ ft}$$

Footing Length

$$d_w := 0 \text{ ft}$$

Depth of groundwater below proposed ground surface

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 35 \text{ deg}$$

Effective angle of internal friction, from
above RMR estimation

$$\gamma_{fd} := \gamma_r = 138.8 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 6240 \text{ psf}$$

Effective Cohesion, from
above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd} \quad \delta_{fd} = 23.3 \text{ deg}$$

Friction angle between foundation soils and footing
taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Bearing Resistance for Weak and Flat Bedded Rock:

$$e_{fb} := \frac{M_{fb}}{V_{fb}} = 2.7 \text{ ft}$$

$$B' := B - 2 \cdot e_{fb} = 3.3 \text{ ft}$$

Eccentricity.

The effective bearing width is equal to B-2e.

Drained Conditions (Effective Stress):

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

$$N_q = 33.3$$

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

$$N_c = 46.12$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 48$$

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

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

$$s_c = 1.035$$

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

$$s_q = 1.034$$

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

$$s_\gamma = 0.981$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_f}{B'} \right)$$

$$d_q = 1.25$$

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

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

$$N_{cm} = 47.736$$

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

$$N_{qm} = 43.09$$

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

$$N_{\gamma m} = 47.098$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 316181 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 142.3 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 142.3 \text{ ksf}$

Evaluate External Stability of Wall:

Compute the factored bearing stress :

Factored bearing resistance

$$\sigma_V := \frac{V_{lb}}{B'}$$

$$\sigma_V = 17.34 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$

Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 8.21$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 \cdot B}{20}$$

$$e_{max} = 3.9 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
within middle 9B/10 for rock.

$$e_{Ia} := \frac{M_{Ia}}{V_{Ia}} = 2.7 \text{ ft}$$

Eccentricity.

$$B' := B - 2 \cdot e_{Ia} = 3.3 \text{ ft}$$

The effective bearing width is equal to
B-2e.

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{\text{Eccentricity}} := \frac{e_{max}}{e_{Ia}}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Eccentricity}} = 1.45$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT BDM[Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	0		0.0 to 0.2	0.2 to 0.5	>0.5		
		General Conditions	Completely Dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 4 + 17 + 20 + 12 + 7 = 60$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 5315 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_q := 120 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**)

$$\gamma_r := 136.2 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (**Hoek and Brown, 1988**):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.2396510364$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.1761544269$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 35 \text{ deg}$$

$$c'_{fd} = 6240 \text{ psf}$$

Abutment Loading from bridge designer:

Service I loads:

$$V_{Ia} := 48900 \frac{\text{lbf}}{\text{ft}}$$

Service I Vertical Load

$$M_{Ia} := 109300 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Service I Moment

Strength I loads:

$$V_{Ib} := 68300 \frac{\text{lbf}}{\text{ft}}$$

Strength I Vertical Load

$$H_{Ib} := 17600 \frac{\text{lbf}}{\text{ft}}$$

Strength I Horizontal Load

$$M_{Ib} := 148900 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Strength I Moment

Footing Geometry:

Preliminary Wall Dimensioning:

$$B := 9 \text{ ft}$$

Footing base width

$$A := 1.83 \text{ ft}$$

Toe projection

$$D := 3 \text{ ft}$$

Footing thickness

$$D_f := 5 \text{ ft}$$

Depth to bottom of footing below proposed ground surface

$$L' := 267.25 \text{ ft}$$

Footing Length

$$d_w := 0 \text{ ft}$$

Depth of groundwater below proposed ground surface

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 35 \text{ deg}$$

Effective angle of internal friction, from
above RMR estimation

$$\gamma_{fd} := \gamma_r = 136.2 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 6240 \text{ psf}$$

Effective Cohesion, from
above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd} \quad \delta_{fd} = 23.3 \text{ deg}$$

Friction angle between foundation soils and footing
taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Bearing Resistance for Weak and Flat Bedded Rock:

$$e_{fb} := \frac{M_{fb}}{V_{fb}} = 2.2 \text{ ft}$$

$$B' := B - 2 \cdot e_{fb} = 4.6 \text{ ft}$$

Eccentricity.

The effective bearing width is equal to B-2e.

Drained Conditions (Effective Stress):

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

$$N_q = 33.3$$

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

$$N_c = 46.12$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 48$$

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

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

$$s_c = 1.013$$

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

$$s_q = 1.012$$

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

$$s_\gamma = 0.993$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_f}{B'} \right)$$

$$d_q = 1.21$$

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

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

$$N_{cm} = 46.702$$

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

$$N_{qm} = 40.762$$

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

$$N_{\gamma m} = 47.695$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$
$$q_{nd} = 311182 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$
$$q_{Rd} = 140 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 140 \text{ ksf}$

Evaluate External Stability of Wall:

Compute the factored bearing stress :

Factored bearing resistance

$$\sigma_V := \frac{V_{lb}}{B'}$$
$$\sigma_V = 14.72 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$ Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 9.51$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 \cdot B}{20}$$
$$e_{max} = 4.1 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
within middle 9B/10 for rock.

$$e_{Ia} := \frac{M_{Ia}}{V_{Ia}} = 2.2 \text{ ft}$$

Eccentricity.

$$B' := B - 2 \cdot e_{Ia} = 4.5 \text{ ft}$$

The effective bearing width is equal to B-2e.

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{\text{Eccentricity}} := \frac{e_{max}}{e_{Ia}}$ Is the CDR > or = to 1.0?

$$CDR_{\text{Eccentricity}} = 1.81$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT BDM[Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	0		0.0 to 0.2	0.2 to 0.5	>0.5		
		General Conditions	Completely Dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 2 + 13 + 20 + 12 + 7 = 54$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 1752 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_q := 120 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**)

$$\gamma_r := 135 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (**Hoek and Brown, 1988**):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.1934266046$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.0904406983$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 32 \text{ deg}$$

$$c'_{fd} = 5616 \text{ psf}$$

Abutment Loading from bridge designer:

Service I loads:

$$V_{Ia} := 48900 \frac{\text{lbf}}{\text{ft}}$$

Service I Vertical Load

$$M_{Ia} := 109300 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Service I Moment

Strength I loads:

$$V_{Ib} := 68300 \frac{\text{lbf}}{\text{ft}}$$

Strength I Vertical Load

$$H_{Ib} := 17600 \frac{\text{lbf}}{\text{ft}}$$

Strength I Horizontal Load

$$M_{Ib} := 148900 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Strength I Moment

Footing Geometry:

Preliminary Wall Dimensioning:

$$B := 9 \text{ ft}$$

Footing base width

$$A := 1.83 \text{ ft}$$

Toe projection

$$D := 3 \text{ ft}$$

Footing thickness

$$D_f := 5 \text{ ft}$$

Depth to bottom of footing below proposed ground surface

$$L' := 267.25 \text{ ft}$$

Footing Length

$$d_w := 0 \text{ ft}$$

Depth of groundwater below proposed ground surface

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 32 \text{ deg}$$

Effective angle of internal friction, from
above RMR estimation

$$\gamma_{fd} := \gamma_r = 135 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 5616 \text{ psf}$$

Effective Cohesion, from
above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd} \quad \delta_{fd} = 21.3 \text{ deg}$$

Friction angle between foundation soils and footing
taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Bearing Resistance for Weak and Flat Bedded Rock:

$$e_{fb} := \frac{M_{fb}}{V_{fb}} = 2.2 \text{ ft}$$

$$B' := B - 2 \cdot e_{fb} = 4.6 \text{ ft}$$

Eccentricity.

The effective bearing width is equal to B-2e.

Drained Conditions (Effective Stress):

$$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right) \quad N_q = 23.18$$

$$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right) \quad N_c = 35.49$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd}) \quad N_\gamma = 30.2$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right) \quad s_c = 1.011$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right) \quad s_q = 1.011$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right) \quad s_\gamma = 0.993$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5) \quad C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \quad C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_f}{B'} \right)$$

$$d_q = 1.23$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 35.893$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 28.751$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 30.005$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$
$$q_{nd} = 214897.1 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$
$$q_{Rd} = 96.7 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 96.7 \text{ ksf}$

Evaluate External Stability of Wall:

Compute the factored bearing stress :

Factored bearing resistance

$$\sigma_V := \frac{V_{lb}}{B'}$$
$$\sigma_V = 14.72 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$ Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 6.57$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 \cdot B}{20}$$
$$e_{max} = 4.1 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
within middle 9B/10 for rock.

$$e_{Ia} := \frac{M_{Ia}}{V_{Ia}} = 2.2 \text{ ft}$$

Eccentricity.

$$B' := B - 2 \cdot e_{Ia} = 4.5 \text{ ft}$$

The effective bearing width is equal to B-2e.

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{\text{Eccentricity}} := \frac{e_{max}}{e_{Ia}}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Eccentricity}} = 1.81$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT BDM[Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range -- uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	OR 0		OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5		
		General Conditions	OR Completely Dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 4 + 13 + 20 + 12 + 7 = 56$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 4242 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_q := 120 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**)

$$\gamma_r := 138 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (**Hoek and Brown, 1988**):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.2077481871$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.1129467638$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 33 \text{ deg}$$

$$c'_{fd} = 5824 \text{ psf}$$

Abutment Loading from bridge designer:

Service I loads:

$$V_{Ia} := 48900 \frac{\text{lbf}}{\text{ft}}$$

Service I Vertical Load

$$M_{Ia} := 109300 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Service I Moment

Strength I loads:

$$V_{Ib} := 68300 \frac{\text{lbf}}{\text{ft}}$$

Strength I Vertical Load

$$H_{Ib} := 17600 \frac{\text{lbf}}{\text{ft}}$$

Strength I Horizontal Load

$$M_{Ib} := 148900 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Strength I Moment

Footing Geometry:

Preliminary Wall Dimensioning:

$$B := 9 \text{ ft}$$

Footing base width

$$A := 1.83 \text{ ft}$$

Toe projection

$$D := 3 \text{ ft}$$

Footing thickness

$$D_f := 5 \text{ ft}$$

Depth to bottom of footing below proposed ground surface

$$L' := 273.92 \text{ ft}$$

Footing Length

$$d_w := 0 \text{ ft}$$

Depth of groundwater below proposed ground surface

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 33 \text{ deg}$$

Effective angle of internal friction, from
above RMR estimation

$$\gamma_{fd} := \gamma_r = 138 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 5824 \text{ psf}$$

Effective Cohesion, from
above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd} \quad \delta_{fd} = 22 \text{ deg}$$

Friction angle between foundation soils and footing
taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Bearing Resistance for Weak and Flat Bedded Rock:

$$e_{fb} := \frac{M_{fb}}{V_{fb}} = 2.2 \text{ ft}$$

$$B' := B - 2 \cdot e_{fb} = 4.6 \text{ ft}$$

Eccentricity.

The effective bearing width is equal to B-2e.

Drained Conditions (Effective Stress):

$$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right) \quad N_q = 26.09$$

$$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right) \quad N_c = 38.64$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd}) \quad N_\gamma = 35.2$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right) \quad s_c = 1.011$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right) \quad s_q = 1.011$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right) \quad s_\gamma = 0.993$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5) \quad C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \quad C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_f}{B'} \right)$$

$$d_q = 1.22$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 39.08$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 32.224$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 34.949$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 242865.1 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 109.3 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 109.3 \text{ ksf}$

Evaluate External Stability of Wall:

Compute the factored bearing stress :

Factored bearing resistance

$$\sigma_V := \frac{V_{lb}}{B'}$$

$$\sigma_V = 14.72 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{Bearing_D} := \frac{q_{Rd}}{\sigma_V}$

Is the CDR > or = to 1.0?

$$CDR_{Bearing_D} = 7.42$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 \cdot B}{20}$$

$$e_{max} = 4.1 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
within middle 9B/10 for rock.

$$e_{Ia} := \frac{M_{Ia}}{V_{Ia}} = 2.2 \text{ ft}$$

Eccentricity.

$$B' := B - 2 \cdot e_{Ia} = 4.5 \text{ ft}$$

The effective bearing width is equal to B-2e.

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{Ia}}$$

Is the CDR > or = to 1.0?

$$CDR_{Eccentricity} = 1.81$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT BDM[Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 Mpa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	0		0.0 to 0.2	0.2 to 0.5	>0.5		
		General Conditions	Completely Dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 2 + 13 + 20 + 12 + 7 = 54$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 2936 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_q := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**)

$$\gamma_r := 147.8 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (**Hoek and Brown, 1988**):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.1934266046$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.0904406983$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 32 \text{ deg}$$

$$c'_{fd} = 5616 \text{ psf}$$

Abutment Loading from bridge designer:

Service I loads:

$$V_{Ia} := 48900 \frac{\text{lbf}}{\text{ft}}$$

Service I Vertical Load

$$M_{Ia} := 109300 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Service I Moment

Strength I loads:

$$V_{Ib} := 68300 \frac{\text{lbf}}{\text{ft}}$$

Strength I Vertical Load

$$H_{Ib} := 17600 \frac{\text{lbf}}{\text{ft}}$$

Strength I Horizontal Load

$$M_{Ib} := 148900 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Strength I Moment

Footing Geometry:

Preliminary Wall Dimensioning:

$$B := 9 \text{ ft}$$

Footing base width

$$A := 1.83 \text{ ft}$$

Toe projection

$$D := 3 \text{ ft}$$

Footing thickness

$$D_f := 5 \text{ ft}$$

Depth to bottom of footing below proposed ground surface

$$L' := 273.92 \text{ ft}$$

Footing Length

$$d_w := 0 \text{ ft}$$

Depth of groundwater below proposed ground surface

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 32 \text{ deg}$$

Effective angle of internal friction, from
above RMR estimation

$$\gamma_{fd} := \gamma_r = 147.8 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 5616 \text{ psf}$$

Effective Cohesion, from
above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd} \quad \delta_{fd} = 21.3 \text{ deg}$$

Friction angle between foundation soils and footing
taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Bearing Resistance for Weak and Flat Bedded Rock:

$$e_{fb} := \frac{M_{fb}}{V_{fb}} = 2.2 \text{ ft}$$

$$B' := B - 2 \cdot e_{fb} = 4.6 \text{ ft}$$

Eccentricity.

The effective bearing width is equal to B-2e.

Drained Conditions (Effective Stress):

$$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right)^2, 1.0 \right) \quad N_q = 23.18$$

$$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right) \quad N_c = 35.49$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd}) \quad N_\gamma = 30.2$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right) \quad s_c = 1.011$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right) \quad s_q = 1.011$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right) \quad s_\gamma = 0.993$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5) \quad C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \quad C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_f}{B'} \right)$$

$$d_q = 1.23$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 35.883$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 28.744$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 30.01$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$
$$q_{nd} = 215286.1 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$
$$q_{Rd} = 96.9 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 96.9 \text{ ksf}$

Evaluate External Stability of Wall:

Compute the factored bearing stress :

Factored bearing resistance

$$\sigma_V := \frac{V_{lb}}{B'}$$
$$\sigma_V = 14.72 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$ Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 6.58$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 \cdot B}{20}$$
$$e_{max} = 4.1 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
within middle 9B/10 for rock.

$$e_{Ia} := \frac{M_{Ia}}{V_{Ia}} = 2.2 \text{ ft}$$

Eccentricity.

$$B' := B - 2 \cdot e_{Ia} = 4.5 \text{ ft}$$

The effective bearing width is equal to B-2e.

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{\text{Eccentricity}} := \frac{e_{max}}{e_{Ia}}$ Is the CDR > or = to 1.0?

$$CDR_{\text{Eccentricity}} = 1.81$$

APPENDIX E

RETAINING WALLS ANALYSES

RETAINING WALL 1

RW1 Boring E3-004-0-20 for STA. 2+00:

Active Earth Pressure Coefficient:

$$\phi'_f := 29 \text{ deg}$$

Effective angle of internal friction

$$\delta := 0 \cdot \phi'_f$$

$\delta = 0$ (degrees) based on ODOT GDM 1501.1.2 for nongravity cantilever walls

$$\theta := 90 \cdot \text{deg}$$

Angle of back face of wall to horizontal

$$\beta := 26.565 \text{ deg}$$

Inclination of ground slope behind face of wall. Horizontal backfill behind CIP wall, 0 degree

$$\gamma := 125 \text{ pcf}$$

Unit Weight of backfill

$$K := \left(1 + \sqrt{\frac{(\sin(\phi'_f + \delta) \cdot \sin(\phi'_f - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 1.327$$

Active earth pressure coefficient (LRFD Sect. 3.11.5.3.)

$$K_{af} := \left(\frac{(\sin(\theta + \phi'_f))^2}{(K \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) = 0.577$$

Triangular Distribution Load:

$$H := 3.11 \text{ ft}$$

1090.41 ft - 1087.3 ft (top of rock) = 3.11 ft

$$CTC := 10 \text{ ft}$$

Center-to-center spacing between shafts

$$\text{Surcharge} := 0 \text{ psf}$$

Live load surcharge

$$EH := K_{af} \cdot \gamma \cdot H \cdot CTC = 186.814 \frac{\text{lbf}}{\text{in}}$$

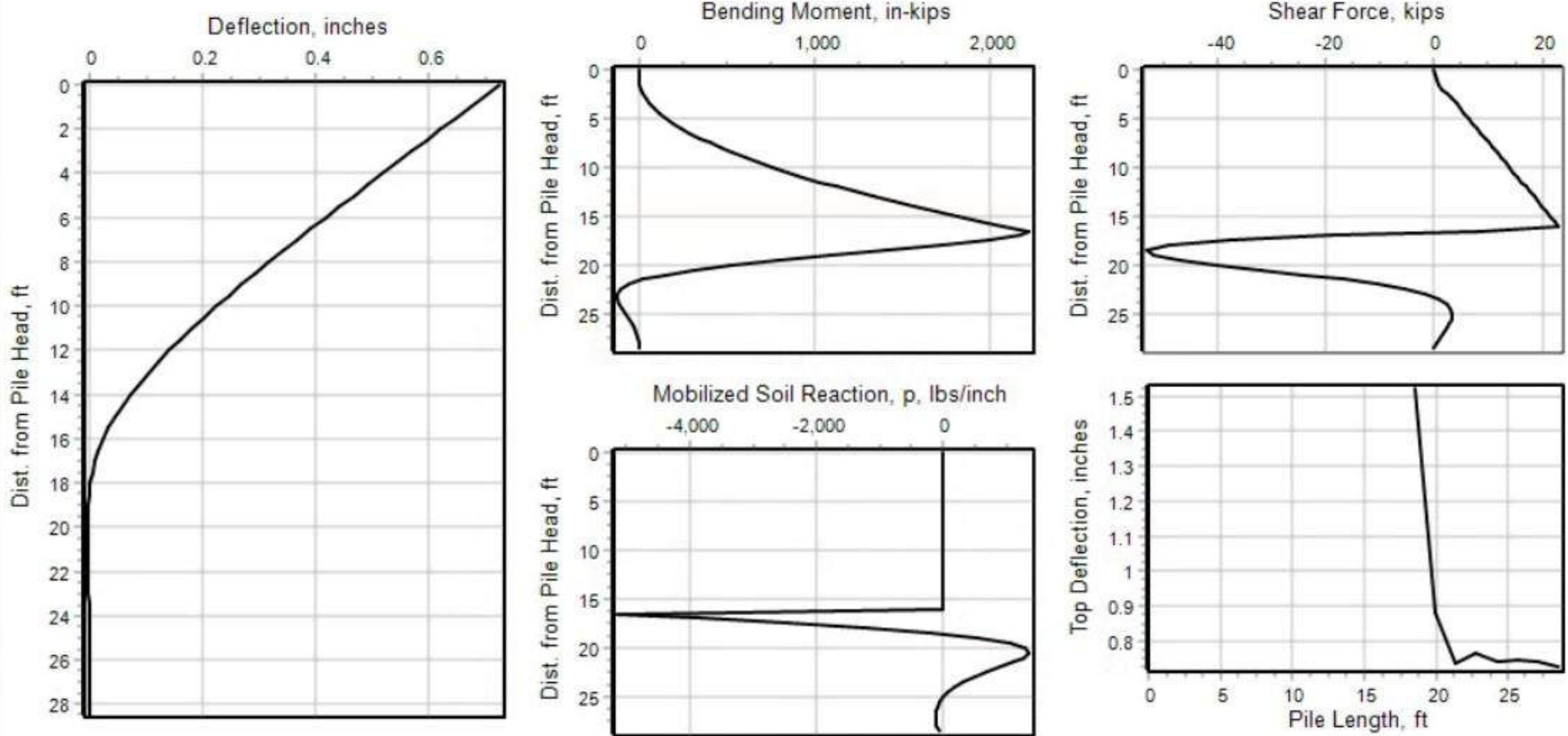
$$\text{FactEH} := 1.5 \cdot EH = 280.221 \frac{\text{lbf}}{\text{in}}$$

$$LL_H := K_{af} \cdot \text{Surcharge} \cdot CTC = 0 \frac{\text{lbf}}{\text{in}}$$

$$\text{FactLL}_H := 1.75 \cdot LL_H = 0 \frac{\text{lbf}}{\text{in}}$$

$$\text{ServiceI} := EH + LL_H = 186.814 \frac{\text{lbf}}{\text{in}}$$

$$\text{StrengthI} := \text{FactEH} + \text{FactLL}_H = 280.221 \frac{\text{lbf}}{\text{in}}$$



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Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Zmankoci

Files Used for Analysis

Path to file locations:
\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\geotechnical\retaining
walls\Retaining Wall X (LocationDescription)\analysis\RW1\Lpile\Files\

Name of input data file:
RW 1_STA. 2+00_W24x62_Service I_E3-004-20.lp12d

Name of output report file:
RW 1_STA. 2+00_W24x62_Service I_E3-004-20.lp12o

Name of plot output file:

RW 1_STA. 2+00_W24x62_Service I_E3-004-20.lp12p

Name of runtime message file:

RW 1_STA. 2+00_W24x62_Service I_E3-004-20.lp12r

Date and Time of Analysis

Date: August 10, 2026

Time: 12:10:52

Problem Title

Project Name: SUM-77 SR-8

Job Number: RW1 Soldier Pile Lagging Wall

Client: ODOT / Arcadis

Engineer: ZM

Description: STA. 2+00

Program Options and Settings

Computational Options:

- Conventional Analysis

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 57

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- Analysis includes loading by one distributed lateral load acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 2
Total length of pile = 28.500 ft
Depth of ground surface below top of pile = 16.5000 ft

Pile diameters used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	7.0400
2	16.500	7.0400
3	16.500	42.0000
4	28.500	42.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile
Cross-sectional Shape = Strong H-Pile
Length of section = 16.500000 ft
Flange Width = 7.040000 in
Section Depth = 23.700000 in
Flange Thickness = 0.590000 in
Web Thickness = 0.430000 in
Section Area = 18.200000 sq. in

Moment of Inertia	=	1550. in^4
Elastic Modulus	=	29000000. psi

Pile Section No. 2:

Section 2 is an elastic pile	
Cross-sectional Shape	= Circular Pile
Length of section	= 12.000000 ft
Width of top of section	= 42.000000 in
Width of bottom of section	= 42.000000 in
Top Area	= 18.200000 sq. in
Bottom Area	= 47.700000 sq. in
Moment of Inertia at Top	= 1550. in^4
Moment of Inertia at Bottom	= 5170. in^4
Elastic Modulus	= 29000000. psi

Soil and Rock Layering Information

The soil profile is modelled using 3 layers

Layer 1 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	16.500000 ft
Distance from top of pile to bottom of layer	=	18.110000 ft
Effective unit weight at top of layer	=	139.000000 pcf
Effective unit weight at bottom of layer	=	139.000000 pcf
Uniaxial compressive strength at top of layer	=	1125. psi
Uniaxial compressive strength at bottom of layer	=	1125. psi
Initial modulus of rock at top of layer	=	3000. psi
Initial modulus of rock at bottom of layer	=	3000. psi
RQD of rock at top of layer	=	0.0000 %
RQD of rock at bottom of layer	=	0.0000 %
k _{rm} of rock at top of layer	=	0.0001600
k _{rm} of rock at bottom of layer	=	0.0001600

Layer 2 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	18.110000	ft
Distance from top of pile to bottom of layer	=	28.110000	ft
Effective unit weight at top of layer	=	139.000000	pcf
Effective unit weight at bottom of layer	=	139.000000	pcf
Uniaxial compressive strength at top of layer	=	1125.	psi
Uniaxial compressive strength at bottom of layer	=	1125.	psi
Initial modulus of rock at top of layer	=	3000.	psi
Initial modulus of rock at bottom of layer	=	3000.	psi
RQD of rock at top of layer	=	0.0000	%
RQD of rock at bottom of layer	=	0.0000	%
k _{rm} of rock at top of layer	=	0.0001600	
k _{rm} of rock at bottom of layer	=	0.0001600	

Layer 3 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	28.110000	ft
Distance from top of pile to bottom of layer	=	42.110000	ft
Effective unit weight at top of layer	=	136.000000	pcf
Effective unit weight at bottom of layer	=	136.000000	pcf
Uniaxial compressive strength at top of layer	=	750.000000	psi
Uniaxial compressive strength at bottom of layer	=	750.000000	psi
Initial modulus of rock at top of layer	=	2040.	psi
Initial modulus of rock at bottom of layer	=	2040.	psi
RQD of rock at top of layer	=	0.0000	%
RQD of rock at bottom of layer	=	0.0000	%
k _{rm} of rock at top of layer	=	0.0002500	
k _{rm} of rock at bottom of layer	=	0.0002500	

(Depth of the lowest soil layer extends 13.610 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Uniaxial qu psi	RQD %	E50 or krm	Rock Mass Modulus psi
1	Weak Rock	16.5000 18.1100	139.0000 139.0000	1125. 1125.	0.00 0.00	1.60E-04 1.60E-04	3000. 3000.
2	Weak Rock	18.1100 28.1100	139.0000 139.0000	1125. 1125.	0.00 0.00	1.60E-04 1.60E-04	3000. 3000.
3	Weak Rock	28.1100 42.1100	136.0000 136.0000	750.0000 750.0000	0.00 0.00	2.50E-04 2.50E-04	2040. 2040.

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Distributed Lateral Loading Used For All Load Cases

Distributed lateral load intensity defined using 4 points

Point No.	Depth X ft	Dist. Load lb/in
1	0.000	0.000
2	3.110	186.814
3	3.110	125.000
4	16.500	125.000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Analysis No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run
1	1	V = 0.0000 lbs	M = 0.0000 in-lbs	0.0000000	Yes	

Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

File Section No. 2:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	16.5000	0.00	N.A.	Yes	N.A.	N.A.
2	18.1100	1.6100	No	Yes	N.A.	N.A.
3	28.1100	11.6100	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head	=	0.0 lbs
Applied moment at pile head	=	0.0 in-lbs
Axial thrust load on pile head	=	0.0 lbs

Depth	Deflect.	Bending	Shear	Slope	Total	Bending	Soil Res.	Soil Spr.	Distrib.
X	y	Moment	Force	S	Stress	Stiffness	p	Es*H	Lat.
Load									
feet	inches	in-lbs	lbs	radians	psi*	lb-in^2	lb/inch	lb/inch	lb/inch

0.00	0.7260	-8.32E-07	0.00	-0.00431	1.89E-09	4.50E+10	0.00	0.00	
7.5086									
0.5000	0.7001	135.1548	112.6290	-0.00431	0.3069	4.50E+10	0.00	0.00	
30.0344									
1.0000	0.6743	1352.	382.9387	-0.00431	3.0693	4.50E+10	0.00	0.00	
60.0688									
1.5000	0.6484	4730.	833.4547	-0.00431	10.7426	4.50E+10	0.00	0.00	
90.1032									
2.0000	0.6225	11353.	1464.	-0.00431	25.7823	4.50E+10	0.00	0.00	
120.1376									
2.5000	0.5967	22301.	2275.	-0.00431	50.6438	4.50E+10	0.00	0.00	
150.1720									
3.0000	0.5709	38654.	3211.	-0.00430	87.7826	4.50E+10	0.00	0.00	
161.7212									
3.5000	0.5451	60830.	4071.	-0.00430	138.1429	4.50E+10	0.00	0.00	
125.0000									
4.0000	0.5193	87506.	4821.	-0.00429	198.7226	4.50E+10	0.00	0.00	
125.0000									
4.5000	0.4936	118681.	5571.	-0.00427	269.5216	4.50E+10	0.00	0.00	
125.0000									
5.0000	0.4681	154357.	6321.	-0.00425	350.5399	4.50E+10	0.00	0.00	
125.0000									
5.5000	0.4426	194533.	7071.	-0.00423	441.7776	4.50E+10	0.00	0.00	
125.0000									
6.0000	0.4173	239208.	7821.	-0.00420	543.2347	4.50E+10	0.00	0.00	

125.0000								
6.5000	0.3922	288384.	8571.	-0.00417	654.9111	4.50E+10	0.00	0.00
125.0000								
7.0000	0.3673	342060.	9321.	-0.00412	776.8069	4.50E+10	0.00	0.00
125.0000								
7.5000	0.3427	400236.	10071.	-0.00407	908.9220	4.50E+10	0.00	0.00
125.0000								
8.0000	0.3184	462911.	10821.	-0.00402	1051.	4.50E+10	0.00	0.00
125.0000								
8.5000	0.2945	530087.	11571.	-0.00395	1204.	4.50E+10	0.00	0.00
125.0000								
9.0000	0.2710	601763.	12321.	-0.00388	1367.	4.50E+10	0.00	0.00
125.0000								
9.5000	0.2480	677938.	13071.	-0.00379	1540.	4.50E+10	0.00	0.00
125.0000								
10.0000	0.2255	758614.	13821.	-0.00369	1723.	4.50E+10	0.00	0.00
125.0000								
10.5000	0.2037	843790.	14571.	-0.00359	1916.	4.50E+10	0.00	0.00
125.0000								
11.0000	0.1825	933465.	15321.	-0.00347	2120.	4.50E+10	0.00	0.00
125.0000								
11.5000	0.1620	1027641.	16071.	-0.00334	2334.	4.50E+10	0.00	0.00
125.0000								
12.0000	0.1424	1126317.	16821.	-0.00319	2558.	4.50E+10	0.00	0.00
125.0000								
12.5000	0.1237	1229492.	17571.	-0.00304	2792.	4.50E+10	0.00	0.00
125.0000								
13.0000	0.1060	1337168.	18321.	-0.00287	3037.	4.50E+10	0.00	0.00
125.0000								
13.5000	0.08934	1449344.	19071.	-0.00268	3291.	4.50E+10	0.00	0.00
125.0000								
14.0000	0.07385	1566020.	19821.	-0.00248	3556.	4.50E+10	0.00	0.00
125.0000								
14.5000	0.05961	1687195.	20571.	-0.00226	3832.	4.50E+10	0.00	0.00
125.0000								
15.0000	0.04672	1812871.	21321.	-0.00203	4117.	4.50E+10	0.00	0.00
125.0000								
15.5000	0.03528	1943047.	22071.	-0.00178	4413.	4.50E+10	0.00	0.00

125.0000								
16.0000	0.02540	2077722.	22821.	-0.00151	4718.	4.50E+10	0.00	0.00
125.0000								
16.5000	0.01718	2216898.	7919.	-0.00122	5035.	4.50E+10	-5155.	1800000.
62.5000								
17.0000	0.01074	2172755.	-18865.	-9.37E-04	27775.	4.76E+10	-3836.	2142857.
0.00								
17.5000	0.00594	1990515.	-37758.	-6.82E-04	24028.	5.05E+10	-2461.	2485714.
0.00								
18.0000	0.00256	1719665.	-48765.	-4.67E-04	19618.	5.34E+10	-1208.	2828571.
0.00								
18.5000	3.43E-04	1405331.	-52932.	-2.95E-04	15163.	5.64E+10	-181.102	3171429.
0.00								
19.0000	-9.81E-04	1084477.	-51753.	-1.66E-04	11076.	5.96E+10	574.3237	3514286.
0.00								
19.5000	-0.00165	784299.	-46849.	-7.40E-05	7587.	6.30E+10	1060.	3857143.
0.00								
20.0000	-0.00187	522284.	-39744.	-1.31E-05	4789.	6.64E+10	1308.	4200000.
0.00								
20.5000	-0.00181	307367.	-31718.	2.37E-05	2674.	7.00E+10	1367.	4542857.
0.00								
21.0000	-0.00158	141672.	-23745.	4.26E-05	1170.	7.38E+10	1290.	4885714.
0.00								
21.5000	-0.00129	22427.	-16491.	4.93E-05	175.9063	7.76E+10	1128.	5228571.
0.00								
22.0000	-9.93E-04	-56216.	-10340.	4.81E-05	419.0976	8.17E+10	922.4975	5571429.
0.00								
22.5000	-7.17E-04	-101648.	-5450.	4.24E-05	720.7561	8.59E+10	707.2169	5914286.
0.00								
23.0000	-4.84E-04	-121621.	-1814.	3.49E-05	820.7226	9.02E+10	504.8267	6257143.
0.00								
23.5000	-2.99E-04	-123421.	687.5210	2.69E-05	793.1141	9.48E+10	329.1309	6600000.
0.00								
24.0000	-1.61E-04	-113371.	2235.	1.96E-05	694.1741	9.95E+10	186.5581	6942857.
0.00								
24.5000	-6.43E-05	-96605.	3028.	1.34E-05	563.9445	1.04E+11	78.0435	7285714.
0.00								
25.0000	-6.56E-07	-77030.	3265.	8.49E-06	428.9526	1.09E+11	0.8338	7628571.

0.00	25.5000	3.76E-05	-57425.	3118.	4.87E-06	305.2120	1.15E+11	-49.958	7971429.
0.00	26.0000	5.78E-05	-39618.	2727.	2.38E-06	201.0862	1.20E+11	-80.121	8314286.
0.00	26.5000	6.61E-05	-24696.	2201.	7.98E-07	119.7642	1.26E+11	-95.443	8657143.
0.00	27.0000	6.74E-05	-13210.	1611.	-9.34E-08	61.2392	1.31E+11	-101.098	9000000.
0.00	27.5000	6.50E-05	-5363.	1015.	-5.12E-07	23.7789	1.37E+11	-97.543	9000000.
0.00	28.0000	6.13E-05	-1028.	446.9135	-6.51E-07	4.3602	1.44E+11	-91.879	9000000.
0.00	28.5000	5.72E-05	0.00	0.00	-6.72E-07	0.00	1.50E+11	-58.364	3060000.
0.00									

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection	=	0.72597921 inches
Computed slope at pile head	=	-0.0043100 radians
Maximum bending moment	=	2216898. inch-lbs
Maximum shear force	=	-52932. lbs
Depth of maximum bending moment	=	16.50000000 feet below pile head
Depth of maximum shear force	=	18.50000000 feet below pile head
Number of iterations	=	6
Number of zero deflection points	=	2
Pile deflection at ground	=	0.01718227 inches

Pile-head Deflection vs. Pile Length for Load Case 1

Boundary Condition Type 1, Shear and Moment

Shear = 0. lbs
 Moment = 0. in-lbs
 Axial Load = 0. lbs

Pile Length feet	Pile Head Deflection inches	Maximum Moment ln-lbs	Maximum Shear lbs
28.50000	0.72597921	2216898.	-52932.
27.07500	0.74140220	2252563.	-53807.
25.65000	0.74557842	2259524.	-54318.
24.22500	0.73855721	2238265.	-53774.
22.80000	0.76640825	2301914.	-55827.
21.37500	0.73764219	2217016.	-59124.
19.95000	0.87965883	2301862.	-90387.
18.52500	1.52244106	2238348.	-133609.

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	0.00	M, in-lb	0.00	0.00	0.7260	-0.00431	-52932.	2216898.

Maximum pile-head deflection = 0.7259792130 inches

Maximum pile-head rotation = -0.0043100411 radians = -0.246947 deg.

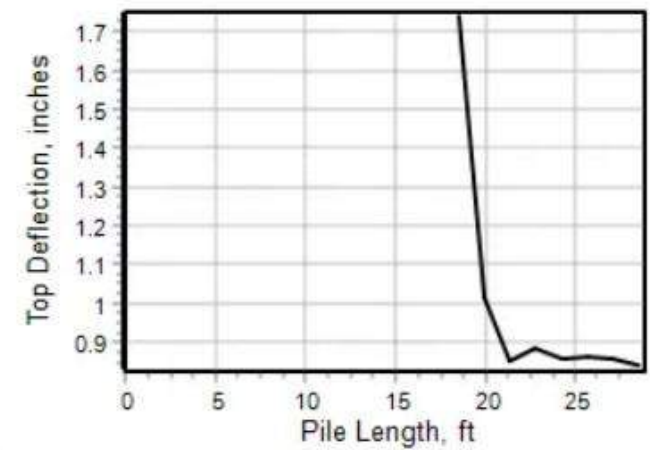
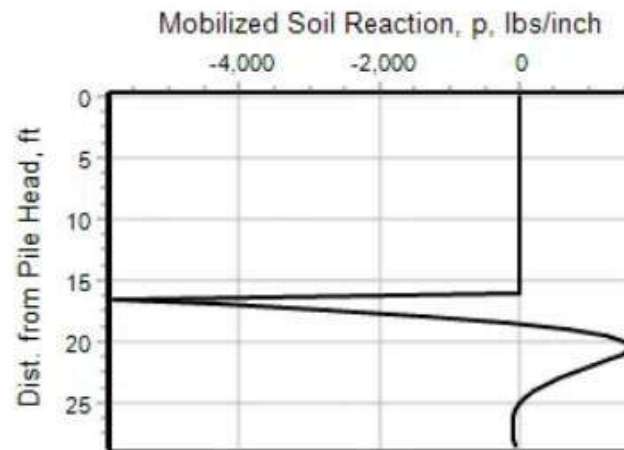
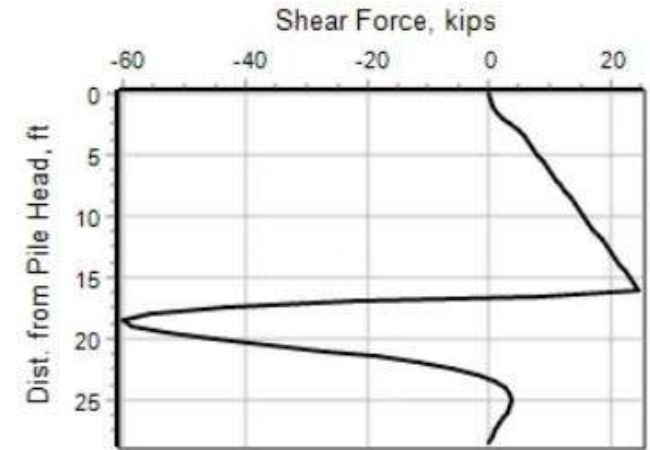
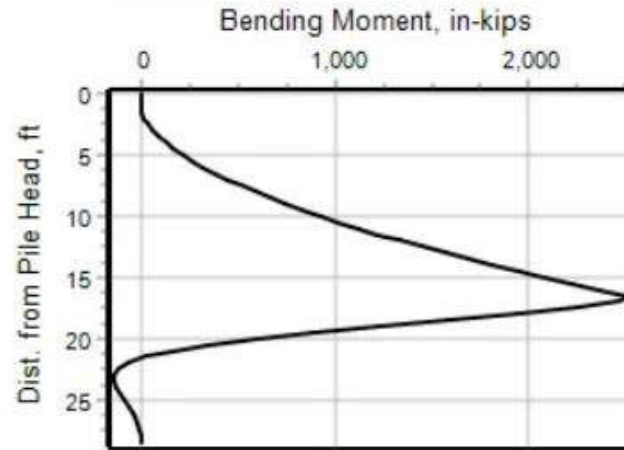
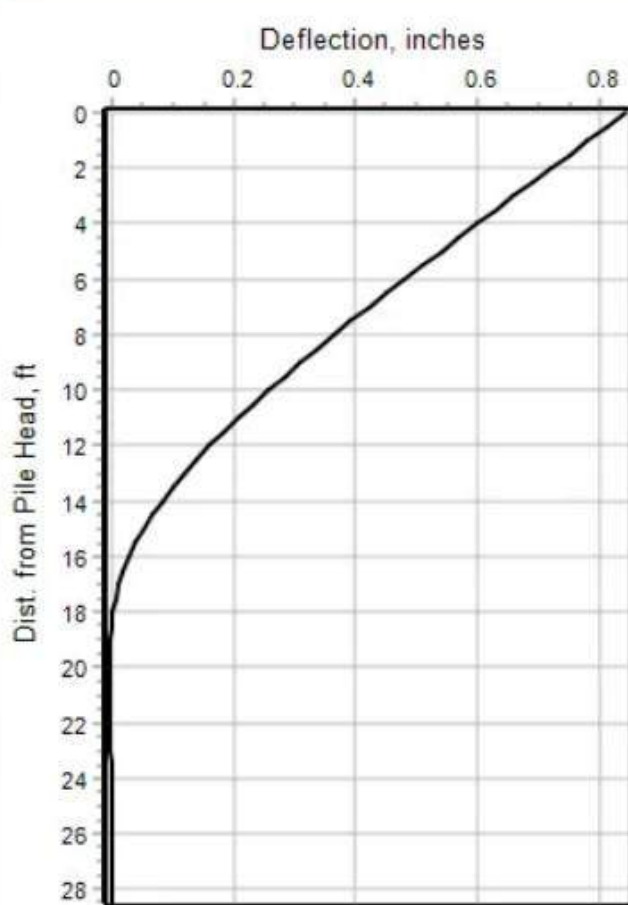
Summary of Warning Messages

The following warning was reported 984 times

**** Warning ****

An unreasonable input value for unconfined compressive strength has been specified for a soil defined using the weak rock criteria. The input value is greater than 500 psi. Please check your input data for correctness.

The analysis ended normally.



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Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Columbus, OH, USA

=====

This model was prepared by:
Zmankoci

Files Used for Analysis

Path to file locations:
\\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\\113208-114865\\geotechnical\\retaining
walls\\Retaining Wall X (LocationDescription)\\analysis\\RW1\\Lpile\\Files\\

Name of input data file:
RW 1_STA. 2+00_W24x62_Strength I_E3-004-20.lp12d

Name of output report file:
RW 1_STA. 2+00_W24x62_Strength I_E3-004-20.lp12o

Name of plot output file:

RW 1_STA. 2+00_W24x62_Strength I_E3-004-20.lp12p

Name of runtime message file:

RW 1_STA. 2+00_W24x62_Strength I_E3-004-20.lp12r

Date and Time of Analysis

Date: August 10, 2026

Time: 12:16:59

Problem Title

Project Name: SUM-77 SR-8

Job Number: RW1 Soldier Pile Lagging Wall

Client: ODOT / Arcadis

Engineer: ZM

Description: STA. 2+00

Program Options and Settings

Computational Options:

- Conventional Analysis

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 57

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- Analysis includes loading by one distributed lateral load acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 2
Total length of pile = 28.500 ft
Depth of ground surface below top of pile = 16.5000 ft

Pile diameters used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	7.0400
2	16.500	7.0400
3	16.500	42.0000
4	28.500	42.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile
Cross-sectional Shape = Strong H-Pile
Length of section = 16.500000 ft
Flange Width = 7.040000 in
Section Depth = 23.700000 in
Flange Thickness = 0.590000 in
Web Thickness = 0.430000 in
Section Area = 18.200000 sq. in

Moment of Inertia	=	1550. in^4
Elastic Modulus	=	29000000. psi

Pile Section No. 2:

Section 2 is an elastic pile	
Cross-sectional Shape	= Circular Pile
Length of section	= 12.000000 ft
Width of top of section	= 42.000000 in
Width of bottom of section	= 42.000000 in
Top Area	= 18.200000 sq. in
Bottom Area	= 47.700000 sq. in
Moment of Inertia at Top	= 1550. in^4
Moment of Inertia at Bottom	= 5170. in^4
Elastic Modulus	= 29000000. psi

Soil and Rock Layering Information

The soil profile is modelled using 3 layers

Layer 1 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	16.500000 ft
Distance from top of pile to bottom of layer	=	18.110000 ft
Effective unit weight at top of layer	=	139.000000 pcf
Effective unit weight at bottom of layer	=	139.000000 pcf
Uniaxial compressive strength at top of layer	=	1125. psi
Uniaxial compressive strength at bottom of layer	=	1125. psi
Initial modulus of rock at top of layer	=	3000. psi
Initial modulus of rock at bottom of layer	=	3000. psi
RQD of rock at top of layer	=	0.0000 %
RQD of rock at bottom of layer	=	0.0000 %
k _{rm} of rock at top of layer	=	0.0001600
k _{rm} of rock at bottom of layer	=	0.0001600

Layer 2 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	18.110000	ft
Distance from top of pile to bottom of layer	=	28.110000	ft
Effective unit weight at top of layer	=	139.000000	pcf
Effective unit weight at bottom of layer	=	139.000000	pcf
Uniaxial compressive strength at top of layer	=	1125.	psi
Uniaxial compressive strength at bottom of layer	=	1125.	psi
Initial modulus of rock at top of layer	=	3000.	psi
Initial modulus of rock at bottom of layer	=	3000.	psi
RQD of rock at top of layer	=	0.0000	%
RQD of rock at bottom of layer	=	0.0000	%
k _{rm} of rock at top of layer	=	0.0001600	
k _{rm} of rock at bottom of layer	=	0.0001600	

Layer 3 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	28.110000	ft
Distance from top of pile to bottom of layer	=	42.110000	ft
Effective unit weight at top of layer	=	136.000000	pcf
Effective unit weight at bottom of layer	=	136.000000	pcf
Uniaxial compressive strength at top of layer	=	750.000000	psi
Uniaxial compressive strength at bottom of layer	=	750.000000	psi
Initial modulus of rock at top of layer	=	2040.	psi
Initial modulus of rock at bottom of layer	=	2040.	psi
RQD of rock at top of layer	=	0.0000	%
RQD of rock at bottom of layer	=	0.0000	%
k _{rm} of rock at top of layer	=	0.0002500	
k _{rm} of rock at bottom of layer	=	0.0002500	

(Depth of the lowest soil layer extends 13.610 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Uniaxial qu psi	RQD %	E50 or krm	Rock Mass Modulus psi
1	Weak Rock	16.5000 18.1100	139.0000 139.0000	1125. 1125.	0.00 0.00	1.60E-04 1.60E-04	3000. 3000.
2	Weak Rock	18.1100 28.1100	139.0000 139.0000	1125. 1125.	0.00 0.00	1.60E-04 1.60E-04	3000. 3000.
3	Weak Rock	28.1100 42.1100	136.0000 136.0000	750.0000 750.0000	0.00 0.00	2.50E-04 2.50E-04	2040. 2040.

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Distributed Lateral Loading Used For All Load Cases

Distributed lateral load intensity defined using 4 points

Point No.	Depth X ft	Dist. Load lb/in
1	0.000	0.000
2	3.110	280.221
3	3.110	125.000
4	16.500	125.000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Analysis No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run
1	1	V = 0.0000 lbs	M = 0.0000 in-lbs	0.0000000	Yes	

Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

File Section No. 2:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	16.5000	0.00	N.A.	Yes	N.A.	N.A.
2	18.1100	1.6100	No	Yes	N.A.	N.A.
3	28.1100	11.6100	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head	=	0.0 lbs
Applied moment at pile head	=	0.0 in-lbs
Axial thrust load on pile head	=	0.0 lbs

Depth	Deflect.	Bending	Shear	Slope	Total	Bending	Soil Res.	Soil Spr.	Distrib.
X	y	Moment	Force	S	Stress	Stiffness	p	Es*H	Lat.
Load									
feet	inches	in-lbs	lbs	radians	psi*	lb-in^2	lb/inch	lb/inch	lb/inch
0.00	0.8408	2.77E-07	0.00	-0.00505	6.30E-10	4.50E+10	0.00	0.00	
11.2629									
0.5000	0.8105	202.7322	168.9435	-0.00505	0.4604	4.50E+10	0.00	0.00	
45.0516									
1.0000	0.7802	2027.	574.4080	-0.00505	4.6040	4.50E+10	0.00	0.00	
90.1032									
1.5000	0.7499	7096.	1250.	-0.00505	16.1139	4.50E+10	0.00	0.00	
135.1548									
2.0000	0.7196	17030.	2196.	-0.00505	38.6735	4.50E+10	0.00	0.00	
180.2064									
2.5000	0.6894	33451.	3413.	-0.00504	75.9657	4.50E+10	0.00	0.00	
225.2580									
3.0000	0.6591	57981.	4764.	-0.00504	131.6739	4.50E+10	0.00	0.00	
225.0817									
3.5000	0.6289	90615.	5814.	-0.00503	205.7837	4.50E+10	0.00	0.00	
125.0000									
4.0000	0.5988	127749.	6564.	-0.00501	290.1127	4.50E+10	0.00	0.00	
125.0000									
4.5000	0.5687	169382.	7314.	-0.00499	384.6612	4.50E+10	0.00	0.00	
125.0000									
5.0000	0.5388	215516.	8064.	-0.00497	489.4290	4.50E+10	0.00	0.00	
125.0000									
5.5000	0.5091	266149.	8814.	-0.00494	604.4161	4.50E+10	0.00	0.00	
125.0000									
6.0000	0.4796	321283.	9564.	-0.00490	729.6226	4.50E+10	0.00	0.00	

125.0000								
6.5000	0.4504	380916.	10314.	-0.00485	865.0485	4.50E+10	0.00	0.00
125.0000								
7.0000	0.4214	445050.	11064.	-0.00479	1011.	4.50E+10	0.00	0.00
125.0000								
7.5000	0.3928	513683.	11814.	-0.00473	1167.	4.50E+10	0.00	0.00
125.0000								
8.0000	0.3646	586817.	12564.	-0.00466	1333.	4.50E+10	0.00	0.00
125.0000								
8.5000	0.3369	664450.	13314.	-0.00457	1509.	4.50E+10	0.00	0.00
125.0000								
9.0000	0.3098	746584.	14064.	-0.00448	1695.	4.50E+10	0.00	0.00
125.0000								
9.5000	0.2832	833217.	14814.	-0.00437	1892.	4.50E+10	0.00	0.00
125.0000								
10.0000	0.2573	924351.	15564.	-0.00426	2099.	4.50E+10	0.00	0.00
125.0000								
10.5000	0.2321	1019985.	16314.	-0.00413	2316.	4.50E+10	0.00	0.00
125.0000								
11.0000	0.2077	1120118.	17064.	-0.00398	2544.	4.50E+10	0.00	0.00
125.0000								
11.5000	0.1843	1224752.	17814.	-0.00383	2781.	4.50E+10	0.00	0.00
125.0000								
12.0000	0.1618	1333885.	18564.	-0.00366	3029.	4.50E+10	0.00	0.00
125.0000								
12.5000	0.1404	1447519.	19314.	-0.00347	3287.	4.50E+10	0.00	0.00
125.0000								
13.0000	0.1202	1565652.	20064.	-0.00327	3556.	4.50E+10	0.00	0.00
125.0000								
13.5000	0.1012	1688286.	20814.	-0.00305	3834.	4.50E+10	0.00	0.00
125.0000								
14.0000	0.08352	1815419.	21564.	-0.00282	4123.	4.50E+10	0.00	0.00
125.0000								
14.5000	0.06734	1947053.	22314.	-0.00257	4422.	4.50E+10	0.00	0.00
125.0000								
15.0000	0.05271	2083186.	23064.	-0.00230	4731.	4.50E+10	0.00	0.00
125.0000								
15.5000	0.03975	2223820.	23814.	-0.00201	5050.	4.50E+10	0.00	0.00

125.0000								
16.0000	0.02857	2368954.	24564.	-0.00171	5380.	4.50E+10	0.00	0.00
125.0000								
16.5000	0.01929	2518587.	7769.	-0.00138	5720.	4.50E+10	-5786.	1800000.
62.5000								
17.0000	0.01202	2462180.	-22281.	-0.00106	31474.	4.76E+10	-4293.	2142857.
0.00								
17.5000	0.00662	2251213.	-43385.	-7.67E-04	27175.	5.05E+10	-2741.	2485714.
0.00								
18.0000	0.00282	1941556.	-55597.	-5.24E-04	22149.	5.34E+10	-1329.	2828571.
0.00								
18.5000	3.31E-04	1584054.	-60108.	-3.31E-04	17091.	5.64E+10	-174.760	3171429.
0.00								
19.0000	-0.00115	1220260.	-58616.	-1.85E-04	12462.	5.96E+10	672.1766	3514286.
0.00								
19.5000	-0.00189	880665.	-52956.	-8.16E-05	8520.	6.30E+10	1214.	3857143.
0.00								
20.0000	-0.00213	584790.	-44845.	-1.32E-05	5363.	6.64E+10	1489.	4200000.
0.00								
20.5000	-0.00205	342519.	-35727.	2.79E-05	2979.	7.00E+10	1551.	4542857.
0.00								
21.0000	-0.00179	156072.	-26695.	4.89E-05	1289.	7.38E+10	1460.	4885714.
0.00								
21.5000	-0.00146	22182.	-18494.	5.61E-05	173.9808	7.76E+10	1274.	5228571.
0.00								
22.0000	-0.00112	-65859.	-11554.	5.45E-05	490.9951	8.17E+10	1040.	5571429.
0.00								
22.5000	-8.07E-04	-116466.	-6047.	4.80E-05	825.8234	8.59E+10	795.6777	5914286.
0.00								
23.0000	-5.43E-04	-138428.	-1960.	3.94E-05	934.1378	9.02E+10	566.6838	6257143.
0.00								
23.5000	-3.35E-04	-139990.	844.5960	3.03E-05	899.5886	9.48E+10	368.2817	6600000.
0.00								
24.0000	-1.79E-04	-128293.	2572.	2.20E-05	785.5411	9.95E+10	207.5745	6942857.
0.00								
24.5000	-7.04E-05	-109124.	3451.	1.50E-05	637.0203	1.04E+11	85.4919	7285714.
0.00								
25.0000	9.19E-07	-86877.	3704.	9.50E-06	483.7825	1.09E+11	-1.168	7628571.

0.00	25.5000	4.36E-05	-64672.	3527.	5.43E-06	343.7263	1.15E+11	-57.985	7971429.
0.00	26.0000	6.61E-05	-44554.	3078.	2.62E-06	226.1370	1.20E+11	-91.529	8314286.
0.00	26.5000	7.51E-05	-27732.	2479.	8.44E-07	134.4845	1.26E+11	-108.346	8657143.
0.00	27.0000	7.62E-05	-14810.	1811.	-1.57E-07	68.6556	1.31E+11	-114.271	9000000.
0.00	27.5000	7.32E-05	-6001.	1139.	-6.26E-07	26.6095	1.37E+11	-109.818	9000000.
0.00	28.0000	6.87E-05	-1147.	500.1136	-7.81E-07	4.8647	1.44E+11	-103.005	9000000.
0.00	28.5000	6.38E-05	0.00	0.00	-8.05E-07	0.00	1.50E+11	-65.118	3060000.
0.00									

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection	=	0.84081577 inches
Computed slope at pile head	=	-0.0050498 radians
Maximum bending moment	=	2518587. inch-lbs
Maximum shear force	=	-60108. lbs
Depth of maximum bending moment	=	16.50000000 feet below pile head
Depth of maximum shear force	=	18.50000000 feet below pile head
Number of iterations	=	6
Number of zero deflection points	=	2
Pile deflection at ground	=	0.01928616 inches

Pile-head Deflection vs. Pile Length for Load Case 1

Boundary Condition Type 1, Shear and Moment

Shear = 0. lbs
 Moment = 0. in-lbs
 Axial Load = 0. lbs

Pile Length feet	Pile Head Deflection inches	Maximum Moment ln-lbs	Maximum Shear lbs
28.50000	0.84081577	2518587.	-60108.
27.07500	0.85824803	2557104.	-61087.
25.65000	0.86279619	2564301.	-61527.
24.22500	0.85498026	2541601.	-60983.
22.80000	0.88621461	2609896.	-63227.
21.37500	0.85392161	2518764.	-66911.
19.95000	1.01354708	2609822.	-102127.
18.52500	1.74017916	2541733.	-151478.

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	0.00	M, in-lb	0.00	0.00	0.8408	-0.00505	-60108.	2518587.

Maximum pile-head deflection = 0.8408157733 inches

Maximum pile-head rotation = -0.0050498358 radians = -0.289334 deg.

Summary of Warning Messages

The following warning was reported 984 times

**** Warning ****

An unreasonable input value for unconfined compressive strength has been specified for a soil defined using the weak rock criteria. The input value is greater than 500 psi. Please check your input data for correctness.

The analysis ended normally.

SUM-77/SR-8 Retaining Wall 1 STA. 2+00		
Depth (ft)	Shear Force from Lpile(lb)	Average Shear (lb) based on ODOT GDM 1501.7.9
0	0	0
0.5	168.9435	247.7838333
1	574.408	837.8703
1.5	1250	1766.621643
2	2196	2597.193071
2.5	3413	3510.772571
3	4764	4473.571429
3.5	5814	5447
4	6564	6392.428571
4.5	7314	7271.142857
5	8064	8064
5.5	8814	8814
6	9564	9564
6.5	10314	10314
7	11064	11064
7.5	11814	11814
8	12564	12564
8.5	13314	13314
9	14064	14064
9.5	14814	14814
10	15564	15564
10.5	16314	16314.0000
11	17064	17064.0000
11.5	17814	17814.0000
12	18564	18564.0000
12.5	19314	19314.0000
13	20064	20064.0000
13.5	20814	20814.0000
14	21564	21564.0000
14.5	22314	22314.0000
15	23064	20557.5714
15.5	23814	14401.1429
16	24564	5122.7143
16.5	7769	-6007.4286
17	-22281	-17889.1429
17.5	-43385	-29664.8571
18	-55597	-40739.1429
18.5	-60108	-48255.4286
19	-58616	-50176.2857
19.5	-52956	-47792.0000
20	-44845	-42491.5714
20.5	-35727	-35555.2857
21	-26695	-28045.4286
21.5	-18494	-20760.2857
22	-11554	-14233.2006
22.5	-6047	-8761.9149
23	-1960	-4455.3434
23.5	844.596	-1284.2006
24	2572	870.2280
24.5	3451	2173.7994
25	3704	2807.9423
25.5	3527	2946.0000
26	3078	2741.2857
26.5	2479	2319.7305
27	1811	1790.5877
27.5	1139	1185.8227
28	500.1136	546.3712
28.5	0	0.0000

RW1 B-001-0-22 for STA. 2+50:

Active Earth Pressure Coefficient:

$$\phi'_f := 30 \text{ deg}$$

Effective angle of internal friction
(Average)

$$\delta := 0 \cdot \phi'_f$$

Friction angle between backfill and
wall taken as specified in LRFD
BDS C3.11.5.3 (degrees)

$$\theta := 90 \cdot \text{deg}$$

Angle of back face of wall to horizontal

$$\beta := 26.565 \text{ deg}$$

Inclination of ground slope behind
face of wall. Horizontal backfill
behind CIP wall, 0 degree

$$\gamma := 125 \text{ pcf}$$

Unit Weight of backfill

$$K := \left(1 + \sqrt{\frac{(\sin(\phi'_f + \delta) \cdot \sin(\phi'_f - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 1.4$$

Active earth pressure coefficient
(LRFD Sect. 3.11.5.3.)

$$K_{af} := \left(\frac{(\sin(\theta + \phi'_f))^2}{(K \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) = 0.536$$

Triangular Distribution Load:

$$H := 7.41 \text{ ft}$$

1090.41 ft - 1083.0 (top of bedrock) = 7.41 ft

$$CTC := 10 \text{ ft}$$

Center-to-center spacing between shafts

$$\text{Surcharge} := 0 \text{ psf}$$

Live load surcharge

$$EH := K_{af} \cdot \gamma \cdot H \cdot CTC = 413.646 \frac{\text{lbf}}{\text{in}}$$

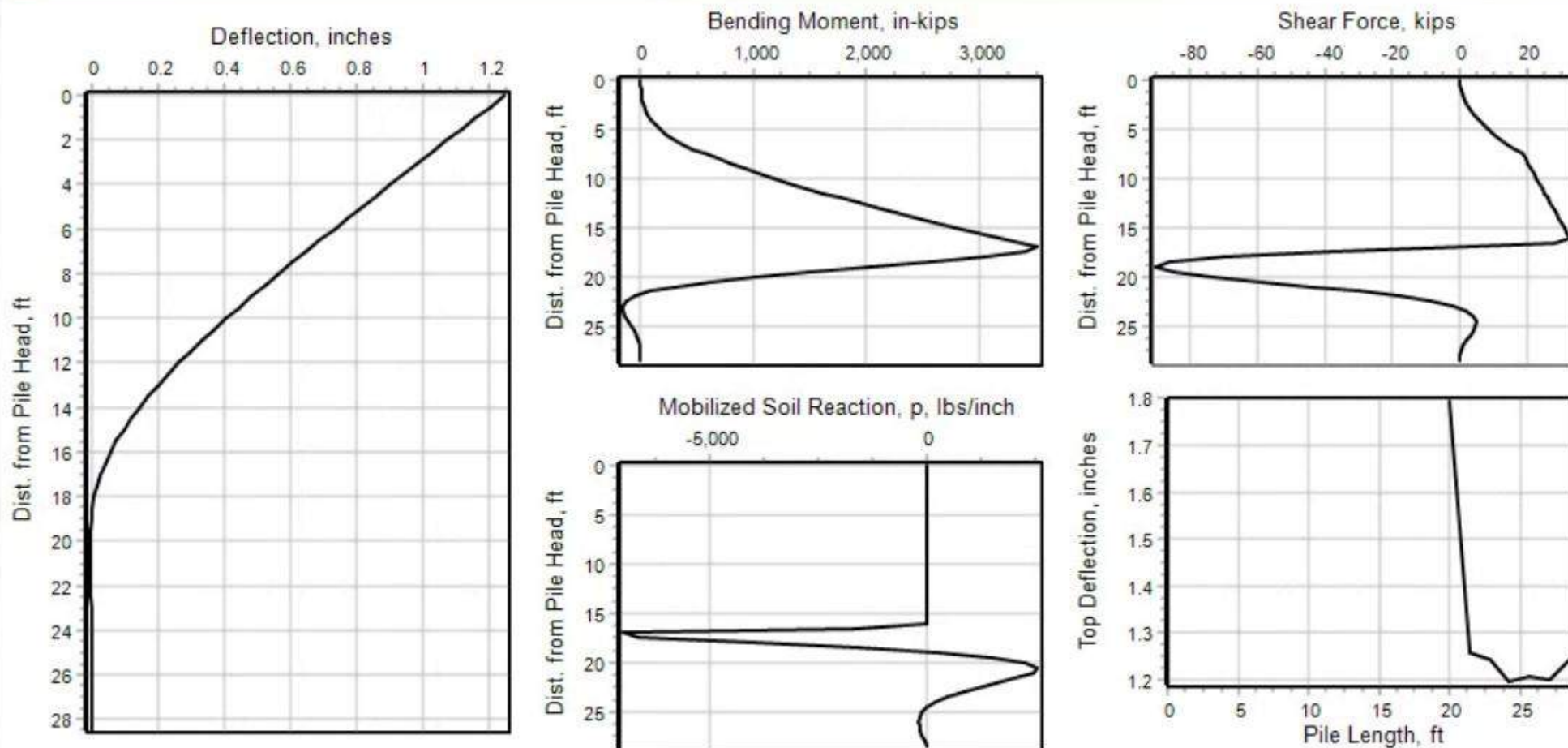
$$\text{FactEH} := 1.5 \cdot EH = 620.468 \frac{\text{lbf}}{\text{in}}$$

$$LL_H := K_{af} \cdot \text{Surcharge} \cdot CTC = 0 \frac{\text{lbf}}{\text{in}}$$

$$\text{FactLL}_H := 1.75 \cdot LL_H = 0 \frac{\text{lbf}}{\text{in}}$$

$$\text{ServiceI} := EH + LL_H = 413.646 \frac{\text{lbf}}{\text{in}}$$

$$\text{StrengthI} := \text{FactEH} + \text{FactLL}_H = 620.468 \frac{\text{lbf}}{\text{in}}$$



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Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Columbus, OH, USA

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This model was prepared by:
Zmankoci

Files Used for Analysis

Path to file locations:
\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\geotechnical\retaining
walls\Retaining Wall X (LocationDescription)\analysis\RW1\Lpile\Files\

Name of input data file:
RW 1_STA. 2+50_W24x62_Service I_B-001-0-22.lp12d

Name of output report file:
RW 1_STA. 2+50_W24x62_Service I_B-001-0-22.lp12o

Name of plot output file:

RW 1_STA. 2+50_W24x62_Service I_B-001-0-22.lp12p

Name of runtime message file:

RW 1_STA. 2+50_W24x62_Service I_B-001-0-22.lp12r

Date and Time of Analysis

Date: August 10, 2026

Time: 12:00:07

Problem Title

Project Name: SUM-77 SR-8

Job Number: RW1 Soldier Pile Lagging Wall

Client: ODOT / Arcadis

Engineer: ZM

Description: STA. 2+50

Program Options and Settings

Computational Options:

- Conventional Analysis

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 57

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- Analysis includes loading by one distributed lateral load acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 2
Total length of pile = 28.500 ft
Depth of ground surface below top of pile = 16.5000 ft

Pile diameters used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	7.0400
2	16.500	7.0400
3	16.500	42.0000
4	28.500	42.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile
Cross-sectional Shape = Strong H-Pile
Length of section = 16.500000 ft
Flange Width = 7.040000 in
Section Depth = 23.700000 in
Flange Thickness = 0.590000 in
Web Thickness = 0.430000 in
Section Area = 18.200000 sq. in

Moment of Inertia	=	1550. in^4
Elastic Modulus	=	29000000. psi

Pile Section No. 2:

Section 2 is an elastic pile	
Cross-sectional Shape	= Circular Pile
Length of section	= 12.000000 ft
Width of top of section	= 42.000000 in
Width of bottom of section	= 42.000000 in
Top Area	= 18.200000 sq. in
Bottom Area	= 20.100000 sq. in
Moment of Inertia at Top	= 1550. in^4
Moment of Inertia at Bottom	= 1830. in^4
Elastic Modulus	= 29000000. psi

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

Layer 1 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	16.500000 ft
Distance from top of pile to bottom of layer	=	45.700000 ft
Effective unit weight at top of layer	=	132.000000 pcf
Effective unit weight at bottom of layer	=	132.000000 pcf
Uniaxial compressive strength at top of layer	=	360.000000 psi
Uniaxial compressive strength at bottom of layer	=	360.000000 psi
Initial modulus of rock at top of layer	=	2880. psi
Initial modulus of rock at bottom of layer	=	2880. psi
RQD of rock at top of layer	=	42.000000 %
RQD of rock at bottom of layer	=	42.000000 %
k _{rm} of rock at top of layer	=	0.0005000
k _{rm} of rock at bottom of layer	=	0.0005000

(Depth of the lowest soil layer extends 17.200 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Uniaxial qu psi	RQD %	E50 or krm	Rock Mass Modulus psi
1	Weak Rock	16.5000 45.7000	132.0000 132.0000	360.0000 360.0000	42.0000 42.0000	5.00E-04 5.00E-04	2880. 2880.

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Distributed Lateral Loading Used For All Load Cases

Distributed lateral load intensity defined using 4 points

Point No.	Depth X ft	Dist. Load lb/in
1	0.000	0.000
2	7.410	413.646
3	7.410	137.882

4

16.500

137.882

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Analysis No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run
1	1	V = 0.0000 lbs	M = 0.0000 in-lbs	0.0000000	Yes	

Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Pile Section No. 2:

Moment-curvature properties were derived from elastic section properties

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head = 0.0 lbs
Applied moment at pile head = 0.0 in-lbs
Axial thrust load on pile head = 0.0 lbs

Depth	Deflect.	Bending	Shear	Slope	Total	Bending	Soil Res.	Soil Spr.	Distrib.
X	y	Moment	Force	S	Stress	Stiffness	p	Es*H	Lat.
Load									
feet	inches	in-lbs	lbs	radians	psi*	lb-in^2	lb/inch	lb/inch	lb/inch
-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
0.00	1.2449	4.16E-06	-2.31E-08	-0.00716	9.44E-09	4.50E+10	0.00	0.00	
6.9778									
0.5000	1.2019	125.6010	104.6675	-0.00716	0.2852	4.50E+10	0.00	0.00	
27.9113									
1.0000	1.1590	1256.	355.8695	-0.00716	2.8524	4.50E+10	0.00	0.00	
55.8227									
1.5000	1.1161	4396.	774.5396	-0.00715	9.9833	4.50E+10	0.00	0.00	

83.7340								
2.0000	1.0731	10550.	1361.	-0.00715	23.9598	4.50E+10	0.00	0.00
111.6453								
2.5000	1.0302	20724.	2114.	-0.00715	47.0639	4.50E+10	0.00	0.00
139.5567								
3.0000	0.9873	35922.	3035.	-0.00715	81.5775	4.50E+10	0.00	0.00
167.4680								
3.5000	0.9444	57148.	4124.	-0.00714	129.7823	4.50E+10	0.00	0.00
195.3794								
4.0000	0.9016	85409.	5380.	-0.00713	193.9604	4.50E+10	0.00	0.00
223.2907								
4.5000	0.8589	121707.	6803.	-0.00712	276.3935	4.50E+10	0.00	0.00
251.2020								
5.0000	0.8162	167049.	8394.	-0.00710	379.3637	4.50E+10	0.00	0.00
279.1134								
5.5000	0.7737	222439.	10153.	-0.00707	505.1527	4.50E+10	0.00	0.00
307.0247								
6.0000	0.7313	288882.	12079.	-0.00704	656.0424	4.50E+10	0.00	0.00
334.9360								
6.5000	0.6892	367383.	14172.	-0.00700	834.3149	4.50E+10	0.00	0.00
362.8474								
7.0000	0.6474	458946.	16433.	-0.00694	1042.	4.50E+10	0.00	0.00
390.7587								
7.5000	0.6059	564577.	18279.	-0.00687	1282.	4.50E+10	0.00	0.00
224.6974								
8.0000	0.5649	678296.	19367.	-0.00679	1540.	4.50E+10	0.00	0.00
137.8820								
8.5000	0.5245	796979.	20194.	-0.00669	1810.	4.50E+10	0.00	0.00
137.8820								
9.0000	0.4846	920626.	21021.	-0.00658	2091.	4.50E+10	0.00	0.00
137.8820								
9.5000	0.4455	1049237.	21849.	-0.00644	2383.	4.50E+10	0.00	0.00
137.8820								
10.0000	0.4073	1182812.	22676.	-0.00630	2686.	4.50E+10	0.00	0.00
137.8820								
10.5000	0.3700	1321350.	23503.	-0.00613	3001.	4.50E+10	0.00	0.00
137.8820								
11.0000	0.3338	1464852.	24331.	-0.00594	3327.	4.50E+10	0.00	0.00

137.8820								
11.5000	0.2987	1613318.	25158.	-0.00574	3664.	4.50E+10	0.00	0.00
137.8820								
12.0000	0.2649	1766748.	25985.	-0.00551	4012.	4.50E+10	0.00	0.00
137.8820								
12.5000	0.2326	1925141.	26813.	-0.00526	4372.	4.50E+10	0.00	0.00
137.8820								
13.0000	0.2017	2088498.	27640.	-0.00500	4743.	4.50E+10	0.00	0.00
137.8820								
13.5000	0.1726	2256819.	28467.	-0.00471	5125.	4.50E+10	0.00	0.00
137.8820								
14.0000	0.1453	2430103.	29294.	-0.00439	5519.	4.50E+10	0.00	0.00
137.8820								
14.5000	0.1199	2608352.	30122.	-0.00406	5923.	4.50E+10	0.00	0.00
137.8820								
15.0000	0.09657	2791564.	30949.	-0.00370	6340.	4.50E+10	0.00	0.00
137.8820								
15.5000	0.07550	2979740.	31776.	-0.00331	6767.	4.50E+10	0.00	0.00
137.8820								
16.0000	0.05682	3172879.	32604.	-0.00290	7206.	4.50E+10	0.00	0.00
137.8820								
16.5000	0.04068	3370983.	28177.	-0.00246	7655.	4.50E+10	-1682.	248099.
68.9410								
17.0000	0.02725	3511005.	2423.	-0.00201	47234.	4.53E+10	-6971.	1535198.
0.00								
17.5000	0.01660	3400064.	-38295.	-0.00155	45420.	4.56E+10	-6602.	2386286.
0.00								
18.0000	0.00864	3051467.	-69826.	-0.00113	40478.	4.59E+10	-3909.	2715429.
0.00								
18.5000	0.00307	2562151.	-86223.	-7.62E-04	33749.	4.62E+10	-1557.	3044571.
0.00								
19.0000	-5.06E-04	2016791.	-90040.	-4.66E-04	26380.	4.66E+10	284.5406	3373714.
0.00								
19.5000	-0.00252	1481674.	-84519.	-2.41E-04	19245.	4.69E+10	1556.	3702857.
0.00								
20.0000	-0.00340	1002559.	-73003.	-8.25E-05	12932.	4.72E+10	2283.	4032000.
0.00								
20.5000	-0.00351	605638.	-58500.	1.95E-05	7758.	4.75E+10	2551.	4361143.

0.00									
21.0000	-0.00316	300565.	-43426.	7.65E-05	3823.	4.79E+10	2473.	4690286.	
0.00									
21.5000	-0.00259	84531.	-29501.	1.01E-04	1068.	4.82E+10	2168.	5019429.	
0.00									
22.0000	-0.00196	-53442.	-17763.	1.03E-04	670.4158	4.85E+10	1744.	5348571.	
0.00									
22.5000	-0.00136	-128622.	-8666.	9.14E-05	1602.	4.89E+10	1288.	5677714.	
0.00									
23.0000	-8.60E-04	-157431.	-2218.	7.39E-05	1948.	4.92E+10	861.2938	6006857.	
0.00									
23.5000	-4.75E-04	-155235.	1870.	5.49E-05	1907.	4.96E+10	501.1841	6336000.	
0.00									
24.0000	-2.02E-04	-134996.	4045.	3.74E-05	1647.	4.99E+10	224.0099	6665143.	
0.00									
24.5000	-2.61E-05	-106692.	4808.	2.29E-05	1293.	5.03E+10	30.4059	6994286.	
0.00									
25.0000	7.31E-05	-77294.	4632.	1.19E-05	930.3356	5.06E+10	-89.166	7323429.	
0.00									
25.5000	1.17E-04	-51105.	3916.	4.35E-06	610.9230	5.09E+10	-149.472	7652571.	
0.00									
26.0000	1.25E-04	-30298.	2968.	-4.34E-07	359.7201	5.13E+10	-166.580	7981714.	
0.00									
26.5000	1.12E-04	-15488.	2003.	-3.11E-06	182.6299	5.16E+10	-155.114	8310857.	
0.00									
27.0000	8.80E-05	-6262.	1158.	-4.37E-06	73.3334	5.20E+10	-126.650	8640000.	
0.00									
27.5000	5.96E-05	-1595.	520.4081	-4.82E-06	18.5497	5.24E+10	-85.801	8640000.	
0.00									
28.0000	3.01E-05	-16.636	132.8893	-4.91E-06	0.1922	5.27E+10	-43.372	8640000.	
0.00									
28.5000	6.44E-07	0.00	0.00	-4.91E-06	0.00	5.31E+10	-0.927	4320000.	
0.00									

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 1.24485237 inches
 Computed slope at pile head = -0.0071553 radians
 Maximum bending moment = 3511005. inch-lbs
 Maximum shear force = -90040. lbs
 Depth of maximum bending moment = 17.00000000 feet below pile head
 Depth of maximum shear force = 19.00000000 feet below pile head
 Number of iterations = 9
 Number of zero deflection points = 3
 Pile deflection at ground = 0.04068440 inches

Pile-head Deflection vs. Pile Length for Load Case 1

Boundary Condition Type 1, Shear and Moment

Shear = 0. lbs
 Moment = 0. in-lbs
 Axial Load = 0. lbs

Pile Length feet	Pile Head Deflection inches	Maximum Moment ln-lbs	Maximum Shear lbs
28.50000	1.24485237	3511005.	-90040.
27.07500	1.20027948	3421902.	-86564.
25.65000	1.20806359	3432223.	-87972.
24.22500	1.19633556	3409431.	-87247.
22.80000	1.24420593	3496550.	-89935.
21.37500	1.25846261	3487183.	-100073.
19.95000	1.79410714	3491594.	-159717.

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

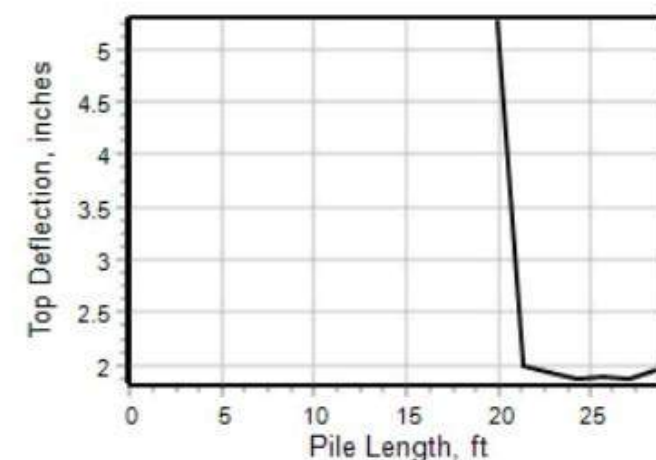
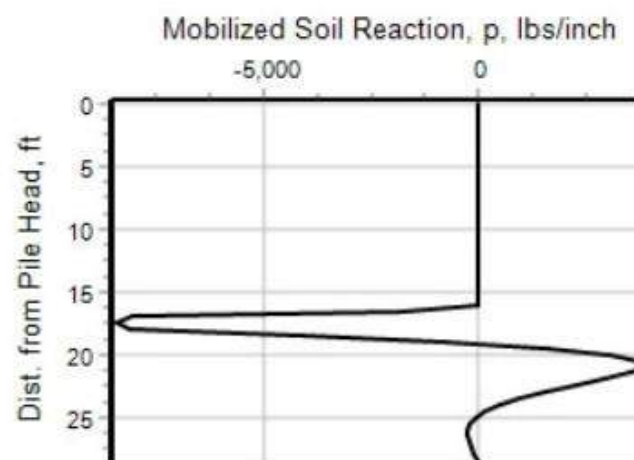
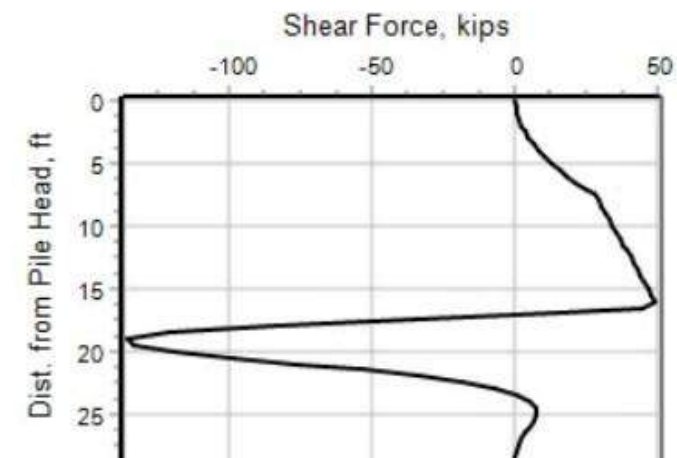
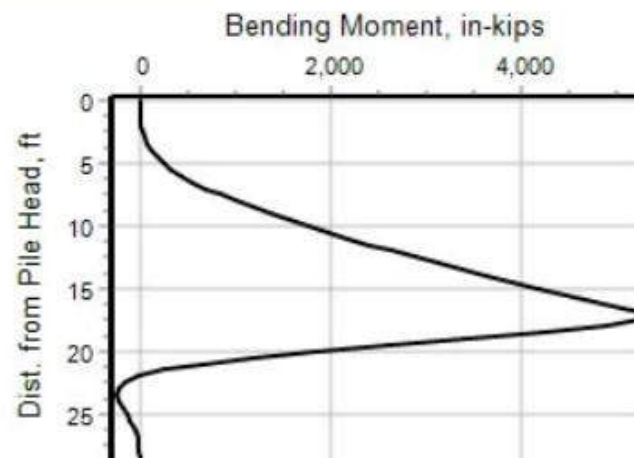
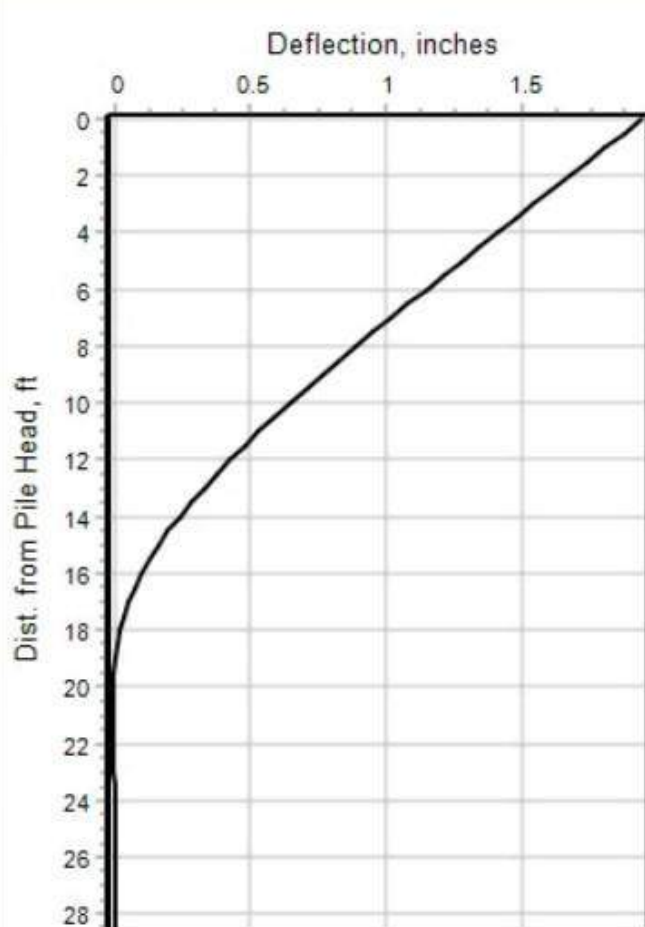
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	0.00	M, in-lb	0.00	0.00	1.2449	-0.00716	-90040.	3511005.

Maximum pile-head deflection = 1.2448523656 inches

Maximum pile-head rotation = -0.0071552506 radians = -0.409966 deg.

The analysis ended normally.



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Subjected to Lateral Loading Using the p-y Method
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This model was prepared by:
Zmankoci

Files Used for Analysis

Path to file locations:
\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\geotechnical\retaining
walls\Retaining Wall X (LocationDescription)\analysis\RW1\Lpile\Files\

Name of input data file:
RW 1_STA. 2+50_W24x62_Strength I_B-001-0-22.lp12d

Name of output report file:
RW 1_STA. 2+50_W24x62_Strength I_B-001-0-22.lp12o

Name of plot output file:

RW 1_STA. 2+50_W24x62_Strength I_B-001-0-22.lp12p

Name of runtime message file:

RW 1_STA. 2+50_W24x62_Strength I_B-001-0-22.lp12r

Date and Time of Analysis

Date: August 10, 2026

Time: 11:51:33

Problem Title

Project Name: SUM-77 SR-8

Job Number: RW1 Soldier Pile Lagging Wall

Client: ODOT / Arcadis

Engineer: ZM

Description: STA. 2+50

Program Options and Settings

Computational Options:

- Conventional Analysis

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 57

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- Analysis includes loading by one distributed lateral load acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 2
Total length of pile = 28.500 ft
Depth of ground surface below top of pile = 16.5000 ft

Pile diameters used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	7.0400
2	16.500	7.0400
3	16.500	42.0000
4	28.500	42.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile
Cross-sectional Shape = Strong H-Pile
Length of section = 16.500000 ft
Flange Width = 7.040000 in
Section Depth = 23.700000 in
Flange Thickness = 0.590000 in
Web Thickness = 0.430000 in
Section Area = 18.200000 sq. in

Moment of Inertia	=	1550. in^4
Elastic Modulus	=	29000000. psi

Pile Section No. 2:

Section 2 is an elastic pile	
Cross-sectional Shape	= Circular Pile
Length of section	= 12.000000 ft
Width of top of section	= 42.000000 in
Width of bottom of section	= 42.000000 in
Top Area	= 18.200000 sq. in
Bottom Area	= 20.100000 sq. in
Moment of Inertia at Top	= 1550. in^4
Moment of Inertia at Bottom	= 1830. in^4
Elastic Modulus	= 29000000. psi

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

Layer 1 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	16.500000 ft
Distance from top of pile to bottom of layer	=	45.700000 ft
Effective unit weight at top of layer	=	132.000000 pcf
Effective unit weight at bottom of layer	=	132.000000 pcf
Uniaxial compressive strength at top of layer	=	360.000000 psi
Uniaxial compressive strength at bottom of layer	=	360.000000 psi
Initial modulus of rock at top of layer	=	2880. psi
Initial modulus of rock at bottom of layer	=	2880. psi
RQD of rock at top of layer	=	42.000000 %
RQD of rock at bottom of layer	=	42.000000 %
k _{rm} of rock at top of layer	=	0.0005000
k _{rm} of rock at bottom of layer	=	0.0005000

(Depth of the lowest soil layer extends 17.200 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Uniaxial qu psi	RQD %	E50 or krm	Rock Mass Modulus psi
1	Weak Rock	16.5000 45.7000	132.0000 132.0000	360.0000 360.0000	42.0000 42.0000	5.00E-04 5.00E-04	2880. 2880.

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Distributed Lateral Loading Used For All Load Cases

Distributed lateral load intensity defined using 4 points

Point No.	Depth X ft	Dist. Load lb/in
1	0.000	0.000
2	7.410	620.468
3	7.410	206.823

4

16.500

206.823

 Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Analysis No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run
1	1	V = 0.0000 lbs	M = 0.0000 in-lbs	0.0000000	Yes	

Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

 Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Pile Section No. 2:

Moment-curvature properties were derived from elastic section properties

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head = 0.0 lbs
Applied moment at pile head = 0.0 in-lbs
Axial thrust load on pile head = 0.0 lbs

Depth	Deflect.	Bending	Shear	Slope	Total	Bending	Soil Res.	Soil Spr.	Distrib.
X	y	Moment	Force	S	Stress	Stiffness	p	Es*H	Lat.
Load									
feet	inches	in-lbs	lbs	radians	psi*	lb-in^2	lb/inch	lb/inch	lb/inch
-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
0.00	1.9419	-7.76E-06	0.00	-0.01106	1.76E-08	4.50E+10	0.00	0.00	
10.4667									
0.5000	1.8755	188.4012	157.0010	-0.01106	0.4279	4.50E+10	0.00	0.00	
41.8669									
1.0000	1.8092	1884.	533.8034	-0.01106	4.2785	4.50E+10	0.00	0.00	
83.7339									
1.5000	1.7429	6594.	1162.	-0.01105	14.9749	4.50E+10	0.00	0.00	

125.6008								
2.0000	1.6765	15826.	2041.	-0.01105	35.9397	4.50E+10	0.00	0.00
167.4677								
2.5000	1.6102	31086.	3171.	-0.01105	70.5958	4.50E+10	0.00	0.00
209.3347								
3.0000	1.5439	53883.	4553.	-0.01104	122.3660	4.50E+10	0.00	0.00
251.2016								
3.5000	1.4777	85723.	6186.	-0.01104	194.6732	4.50E+10	0.00	0.00
293.0686								
4.0000	1.4115	128113.	8070.	-0.01102	290.9401	4.50E+10	0.00	0.00
334.9355								
4.5000	1.3454	182561.	10205.	-0.01100	414.5896	4.50E+10	0.00	0.00
376.8024								
5.0000	1.2795	250574.	12591.	-0.01097	569.0446	4.50E+10	0.00	0.00
418.6694								
5.5000	1.2138	333659.	15229.	-0.01093	757.7278	4.50E+10	0.00	0.00
460.5363								
6.0000	1.1483	433323.	18118.	-0.01088	984.0621	4.50E+10	0.00	0.00
502.4032								
6.5000	1.0832	551074.	21258.	-0.01082	1251.	4.50E+10	0.00	0.00
544.2702								
7.0000	1.0185	688418.	24649.	-0.01073	1563.	4.50E+10	0.00	0.00
586.1371								
7.5000	0.9544	846863.	27419.	-0.01063	1923.	4.50E+10	0.00	0.00
337.0458								
8.0000	0.8910	1017443.	29050.	-0.01051	2311.	4.50E+10	0.00	0.00
206.8230								
8.5000	0.8284	1195467.	30291.	-0.01036	2715.	4.50E+10	0.00	0.00
206.8230								
9.0000	0.7667	1380938.	31532.	-0.01019	3136.	4.50E+10	0.00	0.00
206.8230								
9.5000	0.7061	1573854.	32773.	-0.00999	3574.	4.50E+10	0.00	0.00
206.8230								
10.0000	0.6468	1774215.	34014.	-0.00977	4029.	4.50E+10	0.00	0.00
206.8230								
10.5000	0.5889	1982022.	35255.	-0.00951	4501.	4.50E+10	0.00	0.00
206.8230								
11.0000	0.5326	2197275.	36496.	-0.00924	4990.	4.50E+10	0.00	0.00

206.8230								
11.5000	0.4781	2419974.	37737.	-0.00893	5496.	4.50E+10	0.00	0.00
206.8230								
12.0000	0.4255	2650118.	38978.	-0.00859	6018.	4.50E+10	0.00	0.00
206.8230								
12.5000	0.3750	2887707.	40219.	-0.00822	6558.	4.50E+10	0.00	0.00
206.8230								
13.0000	0.3269	3132743.	41460.	-0.00782	7114.	4.50E+10	0.00	0.00
206.8230								
13.5000	0.2812	3385224.	42701.	-0.00738	7688.	4.50E+10	0.00	0.00
206.8230								
14.0000	0.2383	3645150.	43942.	-0.00691	8278.	4.50E+10	0.00	0.00
206.8230								
14.5000	0.1982	3912523.	45183.	-0.00641	8885.	4.50E+10	0.00	0.00
206.8230								
15.0000	0.1614	4187340.	46423.	-0.00587	9509.	4.50E+10	0.00	0.00
206.8230								
15.5000	0.1278	4469604.	47664.	-0.00529	10150.	4.50E+10	0.00	0.00
206.8230								
16.0000	0.09786	4759313.	48905.	-0.00468	10808.	4.50E+10	0.00	0.00
206.8230								
16.5000	0.07171	5056468.	44362.	-0.00402	11483.	4.50E+10	-1825.	152673.
103.4115								
17.0000	0.04962	5291653.	14902.	-0.00333	71189.	4.53E+10	-8098.	979270.
0.00								
17.5000	0.03173	5235295.	-34740.	-0.00264	69936.	4.56E+10	-8449.	1597577.
0.00								
18.0000	0.01798	4874771.	-84499.	-0.00197	64664.	4.59E+10	-8137.	2715429.
0.00								
18.5000	0.00805	4221313.	-121163.	-0.00138	55604.	4.62E+10	-4084.	3044571.
0.00								
19.0000	0.00141	3420814.	-135789.	-8.87E-04	44745.	4.66E+10	-790.680	3373714.
0.00								
19.5000	-0.00259	2591850.	-133362.	-5.01E-04	33665.	4.69E+10	1600.	3702857.
0.00								
20.0000	-0.00460	1820473.	-119289.	-2.19E-04	23482.	4.72E+10	3091.	4032000.
0.00								
20.5000	-0.00522	1160382.	-98632.	-3.01E-05	14863.	4.75E+10	3794.	4361143.

0.00									
21.0000	-0.00496	636886.	-75614.	8.30E-05	8101.	4.79E+10	3879.	4690286.	
0.00									
21.5000	-0.00422	253017.	-53377.	1.39E-04	3196.	4.82E+10	3534.	5019429.	
0.00									
22.0000	-0.00330	-3635.	-33956.	1.54E-04	45.6013	4.85E+10	2940.	5348571.	
0.00									
22.5000	-0.00237	-154459.	-18398.	1.44E-04	1924.	4.89E+10	2246.	5677714.	
0.00									
23.0000	-0.00156	-224409.	-6961.	1.21E-04	2776.	4.92E+10	1566.	6006857.	
0.00									
23.5000	-9.18E-04	-237988.	645.7714	9.32E-05	2924.	4.96E+10	969.6585	6336000.	
0.00									
24.0000	-4.45E-04	-216660.	5039.	6.58E-05	2644.	4.99E+10	494.6318	6665143.	
0.00									
24.5000	-1.29E-04	-177524.	6972.	4.22E-05	2151.	5.03E+10	149.9013	6994286.	
0.00									
25.0000	6.09E-05	-132993.	7199.	2.37E-05	1601.	5.06E+10	-74.341	7323429.	
0.00									
25.5000	1.56E-04	-91137.	6380.	1.04E-05	1089.	5.09E+10	-198.685	7652571.	
0.00									
26.0000	1.86E-04	-56435.	5040.	1.78E-06	670.0280	5.13E+10	-247.765	7981714.	
0.00									
26.5000	1.77E-04	-30651.	3561.	-3.30E-06	361.4356	5.16E+10	-245.326	8310857.	
0.00									
27.0000	1.47E-04	-13700.	2192.	-5.87E-06	160.4485	5.20E+10	-211.118	8640000.	
0.00									
27.5000	1.07E-04	-4349.	1098.	-6.91E-06	50.5838	5.24E+10	-153.536	8640000.	
0.00									
28.0000	6.36E-05	-524.616	362.3807	-7.19E-06	6.0612	5.27E+10	-91.648	8640000.	
0.00									
28.5000	2.03E-05	0.00	0.00	-7.22E-06	0.00	5.31E+10	-29.245	4320000.	
0.00									

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 1.94187543 inches
 Computed slope at pile head = -0.0110556 radians
 Maximum bending moment = 5291653. inch-lbs
 Maximum shear force = -135789. lbs
 Depth of maximum bending moment = 17.00000000 feet below pile head
 Depth of maximum shear force = 19.00000000 feet below pile head
 Number of iterations = 11
 Number of zero deflection points = 3
 Pile deflection at ground = 0.07171399 inches

Pile-head Deflection vs. Pile Length for Load Case 1

Boundary Condition Type 1, Shear and Moment

Shear = 0. lbs
 Moment = 0. in-lbs
 Axial Load = 0. lbs

Pile Length feet	Pile Head Deflection inches	Maximum Moment ln-lbs	Maximum Shear lbs
28.50000	1.94187543	5291653.	-135789.
27.07500	1.87485052	5179836.	-132290.
25.65000	1.88595893	5202466.	-132756.
24.22500	1.86715692	5169983.	-131181.
22.80000	1.93853741	5298391.	-136558.
21.37500	1.99010969	5268359.	-159143.
19.95000	5.26012545	5237385.	-248571.

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	0.00	M, in-lb	0.00	0.00	1.9419	-0.01106	-135789.	5291653.

Maximum pile-head deflection = 1.9418754300 inches

Maximum pile-head rotation = -0.0110556488 radians = -0.633442 deg.

The analysis ended normally.

SUM-77/SR-8 Retaining Wall 1 STA. 2+50		
Depth (ft)	Shear Force from Lpile(lb)	Average Shear (lb) based on ODOT GDM 1501.7.9
0	0	0
0.5	157.001	230.2681333
1	533.8034	778.76088
1.5	1162	1659.686343
2	2041	2543.400629
2.5	3171	3673.829057
3	4553	5055.428571
3.5	6186	6688.142857
4	8070	8572.142857
4.5	10205	10707.42857
5	12591	13093.85714
5.5	15229	15731.42857
6	18118	18495.57143
6.5	21258	21187.71429
7	24649	23716.28571
7.5	27419	26045.28571
8	29050	28138.85714
8.5	30291	29961.14286
9	31532	31476.28571
9.5	32773	32773
10	34014	34014
10.5	35255	35255.0000
11	36496	36496.0000
11.5	37737	37737.0000
12	38978	38978.0000
12.5	40219	40219.0000
13	41460	41460.0000
13.5	42701	42700.8571
14	43942	43941.7143
14.5	45183	45182.5714
15	46423	45597.1429
15.5	47664	41625.8571
16	48905	30385.5714
16.5	44362	11859.5714
17	14902	-12081.2857
17.5	-34740	-38288.8571
18	-84499	-64327.0000
18.5	-121163	-87705.7143
19	-135789	-103924.8571
19.5	-133362	-109764.0000
20	-119289	-105318.0000
20.5	-98632	-92859.8571
21	-75614	-76089.7143
21.5	-53377	-58032.4286
22	-33956	-40898.8898
22.5	-18398	-26088.7469
23	-6961	-14290.7469
23.5	645.7714	-5637.0327
24	5039	125.2531
24.5	6972	3473.5388
25	7199	4976.6816
25.5	6380	5197.5714
26	5040	4634.5714
26.5	3561	3690.3401
27	2192	2661.9115
27.5	1098	1442.6761
28	362.3807	486.7936
28.5	0	0.0000

RW1 B-002-0-22 for STA. 5+80:

Active Earth Pressure Coefficient:

$$\phi'_f := 30 \text{ deg}$$

Effective angle of internal friction
(Average)

$$\delta := 0 \cdot \phi'_f$$

Friction angle between backfill and
wall taken as specified in LRFD
BDS C3.11.5.3 (degrees)

$$\theta := 90 \cdot \text{deg}$$

Angle of back face of wall to horizontal

$$\beta := 26.565 \text{ deg}$$

Inclination of ground slope behind
face of wall. Horizontal backfill
behind CIP wall, 0 degree

$$\gamma := 125 \text{ pcf}$$

Unit Weight of backfill

$$K := \left(1 + \sqrt{\frac{(\sin(\phi'_f + \delta) \cdot \sin(\phi'_f - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 1.4$$

Active earth pressure coefficient
(LRFD Sect. 3.11.5.3.)

$$K_{af} := \left(\frac{(\sin(\theta + \phi'_f))^2}{(K \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) = 0.536$$

Triangular Distribution Load:

$$H := 3.31 \text{ ft}$$

1088.91 ft - 1085.6 (top of bedrock) = 3.31ft

$$CTC := 10 \text{ ft}$$

Center-to-center spacing between shafts

$$\text{Surcharge} := 0 \text{ psf}$$

Live load surcharge

$$EH := K_{af} \cdot \gamma \cdot H \cdot CTC = 184.773 \frac{\text{lbf}}{\text{in}}$$

$$\text{FactEH} := 1.5 \cdot EH = 277.159 \frac{\text{lbf}}{\text{in}}$$

$$LL_H := K_{af} \cdot \text{Surcharge} \cdot CTC = 0 \frac{\text{lbf}}{\text{in}}$$

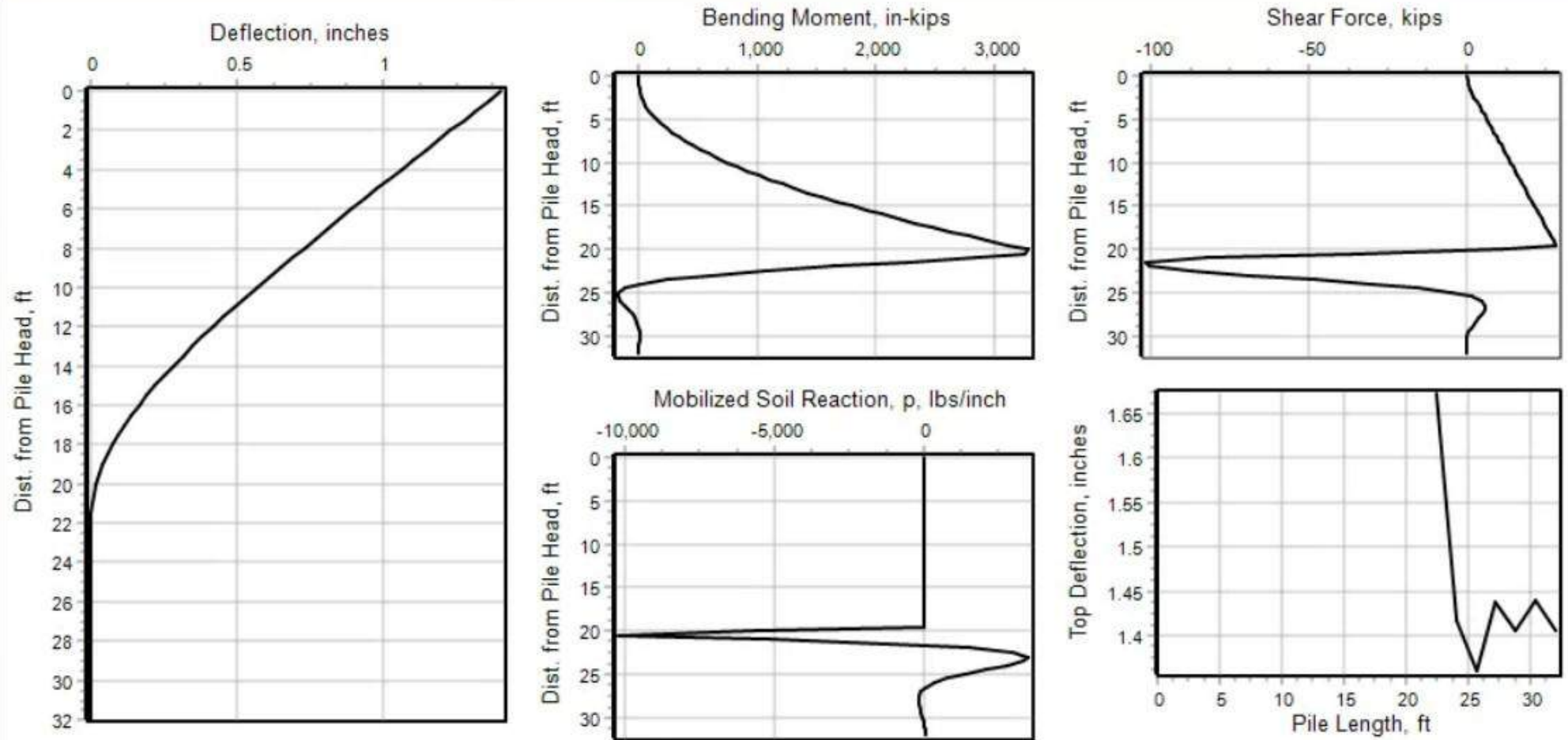
$$\text{FactLL}_H := 1.75 \cdot LL_H = 0 \frac{\text{lbf}}{\text{in}}$$

$$\text{ServiceI} := EH + LL_H = 184.773 \frac{\text{lbf}}{\text{in}}$$

$$\text{StrengthI} := \text{FactEH} + \text{FactLL}_H = 277.159 \frac{\text{lbf}}{\text{in}}$$



Results for RW 1_STA. 5+80_W24x62_Service I_B-002-0-22.lp12d



☐ Show All Legends

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Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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=====

This model was prepared by:
Zmankoci

Files Used for Analysis

Path to file locations:
\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\geotechnical\retaining
walls\Retaining Wall X (LocationDescription)\analysis\RW1\Lpile\Files\

Name of input data file:
RW 1_STA. 5+80_W24x62_Service I_B-002-0-22.lp12d

Name of output report file:
RW 1_STA. 5+80_W24x62_Service I_B-002-0-22.lp12o

Name of plot output file:

RW 1_STA. 5+80_W24x62_Service I_B-002-0-22.lp12p

Name of runtime message file:

RW 1_STA. 5+80_W24x62_Service I_B-002-0-22.lp12r

Date and Time of Analysis

Date: August 10, 2026

Time: 11:36:55

Problem Title

Project Name: SUM-77 SR-8

Job Number: RW1 Soldier Pile Lagging Wall

Client: ODOT / Arcadis

Engineer: ZM

Description: STA. 5+80

Program Options and Settings

Computational Options:

- Conventional Analysis

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 64

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- Analysis includes loading by one distributed lateral load acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 2
Total length of pile = 32.000 ft
Depth of ground surface below top of pile = 20.0000 ft

Pile diameters used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	7.0400
2	20.000	7.0400
3	20.000	42.0000
4	32.000	42.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile
Cross-sectional Shape = Strong H-Pile
Length of section = 20.000000 ft
Flange Width = 7.040000 in
Section Depth = 23.700000 in
Flange Thickness = 0.590000 in
Web Thickness = 0.430000 in
Section Area = 18.200000 sq. in

Moment of Inertia	=	1550. in^4
Elastic Modulus	=	29000000. psi

Pile Section No. 2:

Section 2 is an elastic pile	
Cross-sectional Shape	= Circular Pile
Length of section	= 12.000000 ft
Width of top of section	= 42.000000 in
Width of bottom of section	= 42.000000 in
Top Area	= 18.200000 sq. in
Bottom Area	= 47.700000 sq. in
Moment of Inertia at Top	= 1550. in^4
Moment of Inertia at Bottom	= 5170. in^4
Elastic Modulus	= 29000000. psi

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

Layer 1 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	20.000000 ft
Distance from top of pile to bottom of layer	=	33.910000 ft
Effective unit weight at top of layer	=	140.000000 pcf
Effective unit weight at bottom of layer	=	140.000000 pcf
Uniaxial compressive strength at top of layer	=	1125. psi
Uniaxial compressive strength at bottom of layer	=	1125. psi
Initial modulus of rock at top of layer	=	9000. psi
Initial modulus of rock at bottom of layer	=	9000. psi
RQD of rock at top of layer	=	41.000000 %
RQD of rock at bottom of layer	=	41.000000 %
k _{rm} of rock at top of layer	=	0.0001600
k _{rm} of rock at bottom of layer	=	0.0001600

(Depth of the lowest soil layer extends 1.910 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Uniaxial qu psi	RQD %	E50 or krm	Rock Mass Modulus psi
1	Weak Rock	20.0000 33.9100	140.0000 140.0000	1125. 1125.	41.0000 41.0000	1.60E-04 1.60E-04	9000. 9000.

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Distributed Lateral Loading Used For All Load Cases

Distributed lateral load intensity defined using 4 points

Point No.	Depth X ft	Dist. Load lb/in
1	0.000	0.000
2	3.310	184.773
3	3.310	125.000

4

20.000

125.000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Analysis No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run
1	1	V = 0.0000 lbs	M = 0.0000 in-lbs	0.0000000	Yes	

Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Pile Section No. 2:

Moment-curvature properties were derived from elastic section properties

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head = 0.0 lbs
Applied moment at pile head = 0.0 in-lbs
Axial thrust load on pile head = 0.0 lbs

Depth	Deflect.	Bending	Shear	Slope	Total	Bending	Soil Res.	Soil Spr.	Distrib.
X	y	Moment	Force	S	Stress	Stiffness	p	Es*H	Lat.
Load feet	inches	in-lbs	lbs	radians	psi*	lb-in^2	lb/inch	lb/inch	lb/inch
-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
0.00	1.4050	0.00	0.00	-0.00711	0.00	4.50E+10	0.00	0.00	
6.9778									
0.5000	1.3623	125.6010	104.6675	-0.00711	0.2852	4.50E+10	0.00	0.00	
27.9113									
1.0000	1.3197	1256.	355.8694	-0.00711	2.8524	4.50E+10	0.00	0.00	
55.8227									
1.5000	1.2770	4396.	774.5394	-0.00711	9.9833	4.50E+10	0.00	0.00	

83.7340								
2.0000	1.2343	10550.	1361.	-0.00711	23.9598	4.50E+10	0.00	0.00
111.6453								
2.5000	1.1917	20724.	2114.	-0.00711	47.0639	4.50E+10	0.00	0.00
139.5566								
3.0000	1.1490	35922.	3035.	-0.00711	81.5774	4.50E+10	0.00	0.00
167.4680								
3.5000	1.1064	57148.	3934.	-0.00710	129.7823	4.50E+10	0.00	0.00
131.9718								
4.0000	1.0638	83126.	4705.	-0.00709	188.7765	4.50E+10	0.00	0.00
125.0000								
4.5000	1.0213	113604.	5455.	-0.00708	257.9900	4.50E+10	0.00	0.00
125.0000								
5.0000	0.9789	148581.	6205.	-0.00706	337.4229	4.50E+10	0.00	0.00
125.0000								
5.5000	0.9366	188059.	6955.	-0.00704	427.0751	4.50E+10	0.00	0.00
125.0000								
6.0000	0.8945	232036.	7705.	-0.00701	526.9467	4.50E+10	0.00	0.00
125.0000								
6.5000	0.8525	280514.	8455.	-0.00697	637.0377	4.50E+10	0.00	0.00
125.0000								
7.0000	0.8108	333491.	9205.	-0.00693	757.3480	4.50E+10	0.00	0.00
125.0000								
7.5000	0.7693	390969.	9955.	-0.00688	887.8777	4.50E+10	0.00	0.00
125.0000								
8.0000	0.7282	452946.	10705.	-0.00683	1029.	4.50E+10	0.00	0.00
125.0000								
8.5000	0.6874	519424.	11455.	-0.00676	1180.	4.50E+10	0.00	0.00
125.0000								
9.0000	0.6470	590402.	12205.	-0.00669	1341.	4.50E+10	0.00	0.00
125.0000								
9.5000	0.6071	665879.	12955.	-0.00661	1512.	4.50E+10	0.00	0.00
125.0000								
10.0000	0.5677	745857.	13705.	-0.00651	1694.	4.50E+10	0.00	0.00
125.0000								
10.5000	0.5290	830334.	14455.	-0.00641	1886.	4.50E+10	0.00	0.00
125.0000								
11.0000	0.4909	919312.	15205.	-0.00629	2088.	4.50E+10	0.00	0.00

125.0000								
11.5000	0.4535	1012789.	15955.	-0.00616	2300.	4.50E+10	0.00	0.00
125.0000								
12.0000	0.4169	1110767.	16705.	-0.00602	2523.	4.50E+10	0.00	0.00
125.0000								
12.5000	0.3813	1213244.	17455.	-0.00586	2755.	4.50E+10	0.00	0.00
125.0000								
13.0000	0.3466	1320222.	18205.	-0.00569	2998.	4.50E+10	0.00	0.00
125.0000								
13.5000	0.3129	1431699.	18955.	-0.00551	3251.	4.50E+10	0.00	0.00
125.0000								
14.0000	0.2804	1547677.	19705.	-0.00531	3515.	4.50E+10	0.00	0.00
125.0000								
14.5000	0.2492	1668155.	20455.	-0.00510	3788.	4.50E+10	0.00	0.00
125.0000								
15.0000	0.2193	1793132.	21205.	-0.00487	4072.	4.50E+10	0.00	0.00
125.0000								
15.5000	0.1908	1922610.	21955.	-0.00462	4366.	4.50E+10	0.00	0.00
125.0000								
16.0000	0.1638	2056587.	22705.	-0.00435	4670.	4.50E+10	0.00	0.00
125.0000								
16.5000	0.1385	2195065.	23455.	-0.00407	4985.	4.50E+10	0.00	0.00
125.0000								
17.0000	0.1150	2338042.	24205.	-0.00377	5310.	4.50E+10	0.00	0.00
125.0000								
17.5000	0.09333	2485520.	24955.	-0.00344	5645.	4.50E+10	0.00	0.00
125.0000								
18.0000	0.07366	2637497.	25705.	-0.00310	5990.	4.50E+10	0.00	0.00
125.0000								
18.5000	0.05610	2793975.	26455.	-0.00274	6345.	4.50E+10	0.00	0.00
125.0000								
19.0000	0.04077	2954953.	27205.	-0.00236	6711.	4.50E+10	0.00	0.00
125.0000								
19.5000	0.02782	3120430.	27955.	-0.00195	7086.	4.50E+10	0.00	0.00
125.0000								
20.0000	0.01736	3290408.	11413.	-0.00152	7472.	4.50E+10	-5702.	1970728.
62.5000								
20.5000	0.00954	3257381.	-36156.	-0.00110	41639.	4.76E+10	-10217.	6428571.

0.00								
21.0000	0.00417	2856538.	-82373.	-7.24E-04	34481.	5.05E+10	-5189.	7457143.
0.00								
21.5000	8.52E-04	2268903.	-101554.	-4.26E-04	25883.	5.34E+10	-1205.	8485714.
0.00								
22.0000	-9.41E-04	1637892.	-100693.	-2.12E-04	17672.	5.64E+10	1492.	9514286.
0.00								
22.5000	-0.00169	1060592.	-87313.	-7.13E-05	10832.	5.96E+10	2968.	1.05E+07
0.00								
23.0000	-0.00180	590132.	-68014.	1.01E-05	5709.	6.30E+10	3465.	1.16E+07
0.00								
23.5000	-0.00157	244424.	-47745.	4.93E-05	2241.	6.64E+10	3291.	1.26E+07
0.00								
24.0000	-0.00121	17196.	-29660.	6.11E-05	149.5879	7.00E+10	2737.	1.36E+07
0.00								
24.5000	-8.34E-04	-111491.	-15335.	5.73E-05	920.6156	7.38E+10	2038.	1.47E+07
0.00								
25.0000	-5.18E-04	-166827.	-5164.	4.63E-05	1308.	7.76E+10	1353.	1.57E+07
0.00								
25.5000	-2.78E-04	-173455.	1221.	3.35E-05	1293.	8.17E+10	775.3433	1.67E+07
0.00								
26.0000	-1.16E-04	-152171.	4573.	2.18E-05	1079.	8.59E+10	341.7416	1.77E+07
0.00								
26.5000	-1.66E-05	-118585.	5753.	1.26E-05	800.2311	9.02E+10	51.8855	1.88E+07
0.00								
27.0000	-3.51E-05	-83130.	5562.	5.98E-06	534.2032	9.48E+10	-115.802	1.98E+07
0.00								
27.5000	-5.52E-05	-51844.	4640.	1.79E-06	317.4446	9.95E+10	-191.585	2.08E+07
0.00								
28.0000	5.65E-05	-27456.	3447.	-5.67E-07	160.2757	1.04E+11	-205.900	2.19E+07
0.00								
28.5000	4.84E-05	-10479.	2276.	-1.64E-06	58.3563	1.09E+11	-184.533	2.29E+07
0.00								
29.0000	3.68E-05	-146.431	1282.	-1.94E-06	0.7783	1.15E+11	-146.626	2.39E+07
0.00								
29.5000	2.52E-05	4908.	528.7655	-1.82E-06	24.9114	1.20E+11	-104.552	2.49E+07
0.00								
30.0000	1.50E-05	6199.	20.5184	-1.55E-06	30.0609	1.26E+11	-64.863	2.60E+07

0.00									
30.5000	6.60E-06	5154.	-263.129	-1.28E-06	23.8949	1.31E+11	-29.686	2.70E+07	
0.00									
31.0000	-3.79E-07	3041.	-347.072	-1.10E-06	13.4845	1.37E+11	1.7046	2.70E+07	
0.00									
31.5000	-6.56E-06	989.4642	-253.434	-1.01E-06	4.1981	1.44E+11	29.5079	2.70E+07	
0.00									
32.0000	-1.25E-05	0.00	0.00	-9.88E-07	0.00	1.50E+11	56.1944	1.35E+07	
0.00									

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection	=	1.40501994 inches
Computed slope at pile head	=	-0.0071125 radians
Maximum bending moment	=	3290408. inch-lbs
Maximum shear force	=	-101554. lbs
Depth of maximum bending moment	=	20.00000000 feet below pile head
Depth of maximum shear force	=	21.50000000 feet below pile head
Number of iterations	=	7
Number of zero deflection points	=	3
Pile deflection at ground	=	0.01735858 inches

Pile-head Deflection vs. Pile Length for Load Case 1

Boundary Condition Type 1, Shear and Moment

Shear	=	0. lbs
Moment	=	0. in-lbs
Axial Load	=	0. lbs

Pile Length feet	Pile Head Deflection inches	Maximum Moment In-lbs	Maximum Shear lbs
32.00000	1.40501994	3290408.	-101554.
30.40000	1.43970319	3436702.	-107343.
28.80000	1.40556405	3376901.	-104928.
27.20000	1.43879827	3428154.	-106763.
25.60000	1.36048764	3290415.	-101006.
24.00000	1.41732646	3376650.	-113122.
22.40000	1.67232260	3393871.	-186339.

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	0.00	M, in-lb	0.00	0.00	1.4050	-0.00711	-101554.	3290408.

Maximum pile-head deflection = 1.4050199448 inches

Maximum pile-head rotation = -0.0071125466 radians = -0.407519 deg.

Summary of Warning Messages

The following warning was reported 926 times

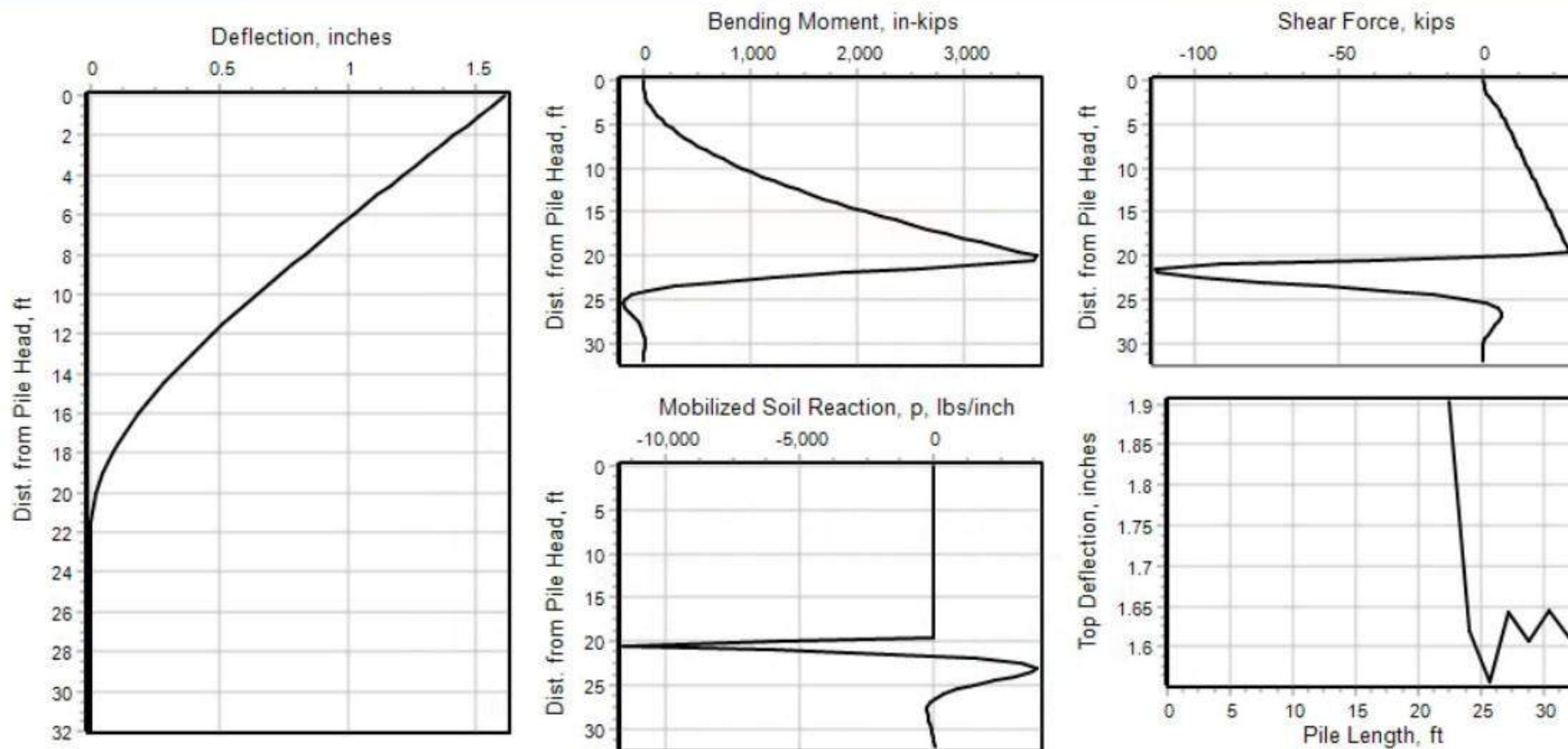
**** Warning ****

An unreasonable input value for unconfined compressive strength has been specified for a soil defined using the weak rock criteria. The input value is greater than 500 psi. Please check your input data for correctness.

The analysis ended normally.



Results for RW 1_STA. 5+80_W24x62_Strength I_B-002-0-22.lp12d



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Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Files Used for Analysis

Path to file locations:
\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\geotechnical\retaining
walls\Retaining Wall X (LocationDescription)\analysis\RW1\Lpile\Files\

Name of input data file:
RW 1_STA. 5+80_W24x62_Strength I_B-002-0-22.lp12d

Name of output report file:
RW 1_STA. 5+80_W24x62_Strength I_B-002-0-22.lp12o

Name of plot output file:

RW 1_STA. 5+80_W24x62_Strength I_B-002-0-22.lp12p

Name of runtime message file:

RW 1_STA. 5+80_W24x62_Strength I_B-002-0-22.lp12r

Date and Time of Analysis

Date: August 10, 2026

Time: 11:45:45

Problem Title

Project Name: SUM-77 SR-8

Job Number: RW1 Soldier Pile Lagging Wall

Client: ODOT / Arcadis

Engineer: ZM

Description: STA. 5+80

Program Options and Settings

Computational Options:

- Conventional Analysis

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 64

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- Analysis includes loading by one distributed lateral load acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 2
Total length of pile = 32.000 ft
Depth of ground surface below top of pile = 20.0000 ft

Pile diameters used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	7.0400
2	20.000	7.0400
3	20.000	42.0000
4	32.000	42.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile
Cross-sectional Shape = Strong H-Pile
Length of section = 20.000000 ft
Flange Width = 7.040000 in
Section Depth = 23.700000 in
Flange Thickness = 0.590000 in
Web Thickness = 0.430000 in
Section Area = 18.200000 sq. in

Moment of Inertia	=	1550. in^4
Elastic Modulus	=	29000000. psi

Pile Section No. 2:

Section 2 is an elastic pile	
Cross-sectional Shape	= Circular Pile
Length of section	= 12.000000 ft
Width of top of section	= 42.000000 in
Width of bottom of section	= 42.000000 in
Top Area	= 18.200000 sq. in
Bottom Area	= 47.700000 sq. in
Moment of Inertia at Top	= 1550. in^4
Moment of Inertia at Bottom	= 5170. in^4
Elastic Modulus	= 29000000. psi

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

Layer 1 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	20.000000 ft
Distance from top of pile to bottom of layer	=	33.910000 ft
Effective unit weight at top of layer	=	140.000000 pcf
Effective unit weight at bottom of layer	=	140.000000 pcf
Uniaxial compressive strength at top of layer	=	1125. psi
Uniaxial compressive strength at bottom of layer	=	1125. psi
Initial modulus of rock at top of layer	=	9000. psi
Initial modulus of rock at bottom of layer	=	9000. psi
RQD of rock at top of layer	=	41.000000 %
RQD of rock at bottom of layer	=	41.000000 %
k _{rm} of rock at top of layer	=	0.0001600
k _{rm} of rock at bottom of layer	=	0.0001600

(Depth of the lowest soil layer extends 1.910 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Uniaxial qu psi	RQD %	E50 or krm	Rock Mass Modulus psi
1	Weak Rock	20.0000 33.9100	140.0000 140.0000	1125. 1125.	41.0000 41.0000	1.60E-04 1.60E-04	9000. 9000.

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Distributed Lateral Loading Used For All Load Cases

Distributed lateral load intensity defined using 4 points

Point No.	Depth X ft	Dist. Load lb/in
1	0.000	0.000
2	3.310	277.159
3	3.310	125.000

4

20.000

125.000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Analysis No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run
1	1	V = 0.0000 lbs	M = 0.0000 in-lbs	0.0000000	Yes	

Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Pile Section No. 2:

Moment-curvature properties were derived from elastic section properties

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head = 0.0 lbs
Applied moment at pile head = 0.0 in-lbs
Axial thrust load on pile head = 0.0 lbs

Depth	Deflect.	Bending	Shear	Slope	Total	Bending	Soil Res.	Soil Spr.	Distrib.
X	y	Moment	Force	S	Stress	Stiffness	p	Es*H	Lat.
Load									
feet	inches	in-lbs	lbs	radians	psi*	lb-in^2	lb/inch	lb/inch	lb/inch
-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
0.00	1.6110	-3.33E-06	0.00	-0.00824	7.56E-09	4.50E+10	0.00	0.00	
10.4667									
0.5000	1.5615	188.4011	157.0009	-0.00824	0.4279	4.50E+10	0.00	0.00	
41.8669									
1.0000	1.5121	1884.	533.8032	-0.00824	4.2785	4.50E+10	0.00	0.00	
83.7338									
1.5000	1.4627	6594.	1162.	-0.00824	14.9749	4.50E+10	0.00	0.00	

125.6008								
2.0000	1.4132	15826.	2041.	-0.00824	35.9396	4.50E+10	0.00	0.00
167.4677								
2.5000	1.3638	31086.	3171.	-0.00823	70.5957	4.50E+10	0.00	0.00
209.3346								
3.0000	1.3144	53883.	4553.	-0.00823	122.3659	4.50E+10	0.00	0.00
251.2015								
3.5000	1.2651	85723.	5736.	-0.00822	194.6731	4.50E+10	0.00	0.00
142.9576								
4.0000	1.2158	122709.	6539.	-0.00820	278.6677	4.50E+10	0.00	0.00
125.0000								
4.5000	1.1666	164195.	7289.	-0.00819	372.8817	4.50E+10	0.00	0.00
125.0000								
5.0000	1.1176	210181.	8039.	-0.00816	477.3150	4.50E+10	0.00	0.00
125.0000								
5.5000	1.0687	260668.	8789.	-0.00813	591.9677	4.50E+10	0.00	0.00
125.0000								
6.0000	1.0201	315654.	9539.	-0.00809	716.8397	4.50E+10	0.00	0.00
125.0000								
6.5000	0.9716	375140.	10289.	-0.00804	851.9311	4.50E+10	0.00	0.00
125.0000								
7.0000	0.9235	439126.	11039.	-0.00799	997.2418	4.50E+10	0.00	0.00
125.0000								
7.5000	0.8758	507613.	11789.	-0.00793	1153.	4.50E+10	0.00	0.00
125.0000								
8.0000	0.8284	580599.	12539.	-0.00785	1319.	4.50E+10	0.00	0.00
125.0000								
8.5000	0.7815	658085.	13289.	-0.00777	1494.	4.50E+10	0.00	0.00
125.0000								
9.0000	0.7352	740071.	14039.	-0.00768	1681.	4.50E+10	0.00	0.00
125.0000								
9.5000	0.6894	826558.	14789.	-0.00757	1877.	4.50E+10	0.00	0.00
125.0000								
10.0000	0.6443	917544.	15539.	-0.00746	2084.	4.50E+10	0.00	0.00
125.0000								
10.5000	0.5999	1013030.	16289.	-0.00733	2301.	4.50E+10	0.00	0.00
125.0000								
11.0000	0.5563	1113017.	17039.	-0.00719	2528.	4.50E+10	0.00	0.00

125.0000								
11.5000	0.5137	1217503.	17789.	-0.00703	2765.	4.50E+10	0.00	0.00
125.0000								
12.0000	0.4720	1326489.	18539.	-0.00686	3012.	4.50E+10	0.00	0.00
125.0000								
12.5000	0.4313	1439975.	19289.	-0.00668	3270.	4.50E+10	0.00	0.00
125.0000								
13.0000	0.3918	1557962.	20039.	-0.00648	3538.	4.50E+10	0.00	0.00
125.0000								
13.5000	0.3536	1680448.	20789.	-0.00626	3816.	4.50E+10	0.00	0.00
125.0000								
14.0000	0.3167	1807434.	21539.	-0.00603	4105.	4.50E+10	0.00	0.00
125.0000								
14.5000	0.2813	1938920.	22289.	-0.00578	4403.	4.50E+10	0.00	0.00
125.0000								
15.0000	0.2474	2074907.	23039.	-0.00551	4712.	4.50E+10	0.00	0.00
125.0000								
15.5000	0.2152	2215393.	23789.	-0.00522	5031.	4.50E+10	0.00	0.00
125.0000								
16.0000	0.1847	2360379.	24539.	-0.00492	5360.	4.50E+10	0.00	0.00
125.0000								
16.5000	0.1562	2509865.	25289.	-0.00459	5700.	4.50E+10	0.00	0.00
125.0000								
17.0000	0.1296	2663852.	26039.	-0.00425	6050.	4.50E+10	0.00	0.00
125.0000								
17.5000	0.1052	2822338.	26789.	-0.00388	6409.	4.50E+10	0.00	0.00
125.0000								
18.0000	0.08303	2985324.	27539.	-0.00349	6780.	4.50E+10	0.00	0.00
125.0000								
18.5000	0.06326	3152811.	28289.	-0.00308	7160.	4.50E+10	0.00	0.00
125.0000								
19.0000	0.04602	3324797.	29039.	-0.00265	7551.	4.50E+10	0.00	0.00
125.0000								
19.5000	0.03144	3501283.	29789.	-0.00220	7951.	4.50E+10	0.00	0.00
125.0000								
20.0000	0.01967	3682269.	13086.	-0.00172	8362.	4.50E+10	-5755.	1755680.
62.5000								
20.5000	0.01084	3658313.	-38847.	-0.00124	46765.	4.76E+10	-11618.	6428571.

0.00								
21.0000	0.00478	3216104.	-91534.	-8.19E-04	38822.	5.05E+10	-5944.	7457143.
0.00								
21.5000	0.00102	2559904.	-113681.	-4.84E-04	29203.	5.34E+10	-1438.	8485714.
0.00								
22.0000	-0.00102	1851938.	-113128.	-2.42E-04	19982.	5.64E+10	1622.	9514286.
0.00								
22.5000	-0.00188	1202368.	-98344.	-8.26E-05	12280.	5.96E+10	3306.	1.05E+07
0.00								
23.0000	-0.00201	671812.	-76773.	9.92E-06	6499.	6.30E+10	3884.	1.16E+07
0.00								
23.5000	-0.00176	281086.	-54018.	5.46E-05	2578.	6.64E+10	3701.	1.26E+07
0.00								
24.0000	-0.00136	23601.	-33657.	6.83E-05	205.3034	7.00E+10	3086.	1.36E+07
0.00								
24.5000	-9.42E-04	-122798.	-17494.	6.44E-05	1014.	7.38E+10	2302.	1.47E+07
0.00								
25.0000	-5.86E-04	-186321.	-5989.	5.22E-05	1461.	7.76E+10	1533.	1.57E+07
0.00								
25.5000	-3.16E-04	-194670.	1253.	3.78E-05	1451.	8.17E+10	881.6094	1.67E+07
0.00								
26.0000	-1.33E-04	-171281.	5074.	2.47E-05	1215.	8.59E+10	391.8254	1.77E+07
0.00								
26.5000	-2.03E-05	-133787.	6440.	1.42E-05	902.8158	9.02E+10	63.5745	1.88E+07
0.00								
27.0000	3.85E-05	-94003.	6249.	6.83E-06	604.0757	9.48E+10	-127.023	1.98E+07
0.00								
27.5000	6.16E-05	-58793.	5227.	2.08E-06	359.9906	9.95E+10	-213.823	2.08E+07
0.00								
28.0000	6.34E-05	-31280.	3892.	-5.96E-07	182.6010	1.04E+11	-231.024	2.19E+07
0.00								
28.5000	5.44E-05	-12084.	2576.	-1.83E-06	67.2923	1.09E+11	-207.677	2.29E+07
0.00								
29.0000	4.15E-05	-364.644	1457.	-2.17E-06	1.9381	1.15E+11	-165.401	2.39E+07
0.00								
29.5000	2.84E-05	5400.	606.2285	-2.04E-06	27.4105	1.20E+11	-118.208	2.49E+07
0.00								
30.0000	1.70E-05	6910.	30.9520	-1.74E-06	33.5105	1.26E+11	-73.551	2.60E+07

0.00	30.5000	7.53E-06	5772.	-291.351	-1.45E-06	26.7579	1.31E+11	-33.884	2.70E+07
0.00	31.0000	-3.51E-07	3414.	-388.267	-1.24E-06	15.1369	1.37E+11	1.5783	2.70E+07
0.00	31.5000	-7.34E-06	1113.	-284.491	-1.14E-06	4.7210	1.44E+11	33.0137	2.70E+07
0.00	32.0000	-1.40E-05	0.00	0.00	-1.12E-06	0.00	1.50E+11	63.1932	1.35E+07
0.00									

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection	=	1.61096082 inches
Computed slope at pile head	=	-0.0082386 radians
Maximum bending moment	=	3682269. inch-lbs
Maximum shear force	=	-113681. lbs
Depth of maximum bending moment	=	20.00000000 feet below pile head
Depth of maximum shear force	=	21.50000000 feet below pile head
Number of iterations	=	7
Number of zero deflection points	=	3
Pile deflection at ground	=	0.01966882 inches

Pile-head Deflection vs. Pile Length for Load Case 1

Boundary Condition Type 1, Shear and Moment

Shear	=	0. lbs
Moment	=	0. in-lbs
Axial Load	=	0. lbs

Pile Length feet	Pile Head Deflection inches	Maximum Moment In-lbs	Maximum Shear lbs
32.00000	1.61096082	3682269.	-113681.
30.40000	1.64440685	3837699.	-119874.
28.80000	1.60661807	3774230.	-117325.
27.20000	1.64345187	3828676.	-119321.
25.60000	1.55650143	3682172.	-112958.
24.00000	1.61959635	3773867.	-126281.
22.40000	1.90345872	3792342.	-207964.

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	0.00	M, in-lb	0.00	0.00	1.6110	-0.00824	-113681.	3682269.

Maximum pile-head deflection = 1.6109608212 inches

Maximum pile-head rotation = -0.0082385729 radians = -0.472035 deg.

Summary of Warning Messages

The following warning was reported 926 times

**** Warning ****

An unreasonable input value for unconfined compressive strength has been specified for a soil defined using the weak rock criteria. The input value is greater than 500 psi. Please check your input data for correctness.

The analysis ended normally.

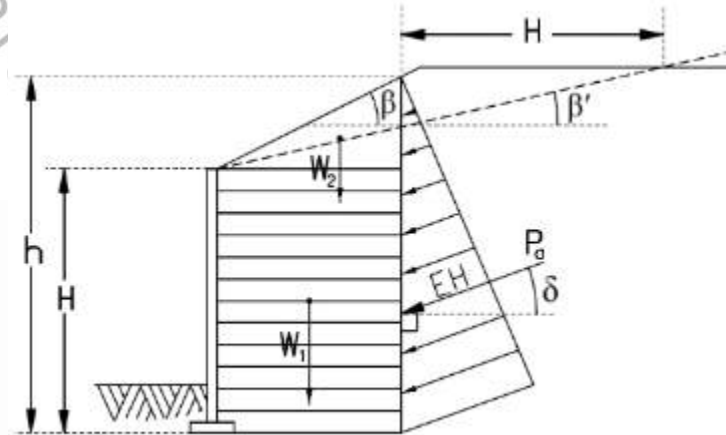
SUM-77/SR-8 Retaining Wall 1 STA. 5+80		
Depth (ft)	Shear Force from Lpile(lb)	Average Shear (lb) based on ODOT GDM 1501.7.9
0	0	0
0.5	157.0009	230.2680333
1	533.8032	778.76082
1.5	1162	1659.6863
2	2041	2479.114871
2.5	3171	3390.829029
3	4553	4355.857143
3.5	5736	5338.285714
4	6539	6302.285714
4.5	7289	7212
5	8039	8031.428571
5.5	8789	8789
6	9539	9539
6.5	10289	10289
7	11039	11039
7.5	11789	11789
8	12539	12539
8.5	13289	13289
9	14039	14039
9.5	14789	14789
10	15539	15539
10.5	16289	16289
11	17039	17039
11.5	17789	17789
12	18539	18539
12.5	19289	19289
13	20039	20039
13.5	20789	20789
14	21539	21539
14.5	22289	22289
15	23039	23039
15.5	23789	23789
16	24539	24539
16.5	25289	25289
17	26039	26039
17.5	26789	26789
18	27539	27539
18.5	28289	25795.71429
19	29039	16526.28571
19.5	29789	-377
20	13086	-20551.28571
20.5	-38847	-40753.71429
21	-91534	-58951.28571
21.5	-113681	-74174.42857
22	-113128	-83760.71429
22.5	-98344	-83019.28571
23	-76773	-72442.14286
23.5	-54018	-57057.57143
24	-33657	-40717.42857
24.5	-17494	-25943.42857
25	-5989	-14055.85714
25.5	1253	-5446.285714
26	5074	108.5714286
26.5	6440	3163.714286
27	6249	4387.285714
27.5	5227	4416.428571
28	3892	3778.1755
28.5	2576	2862.597214
29	1457	1928.261357
29.5	606.2285	1126.080357
30	30.952	529.4387857
30.5	-291.351	161.4387857
31	-388.267	-186.6314
31.5	-284.491	-224.2526667
32	0	0

RETAINING WALL 2

Objective: To evaluate the external stability of MSE wall design with vertical wall face and broken backslope case.
Method: In accordance with ODOT Bridge Design Manual, 2026 [Sect. 307.4] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 3.11.5.8 and Sect. 11.10.5].

Assumptions:

- Horizontal backfill, Infinite backslope or Broken backslope behind MSE wall, see LRFD Sect. 3.11.5.8.1.
- For battered or vertical walls with a back face of wall angle of θ to horizontal.
- Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ .
- MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall



Givens:

Wall Geometry:

$$H_e := 18.00 \text{ ft}$$

$$\theta := 90 \cdot \text{deg}$$

$$\beta_{\text{BackSlope}} := 0 \text{ deg}$$

$$\lambda := 0 \text{ ft}$$

Exposed wall height (Proposed profile grade to bottom of leveling pad)

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Inclination of ground slope behind face of wall. If it is horizontal backfill behind MSE wall, $\beta_{\text{BackSlope}} = 0$ deg

Horizontal distance from the back of MSE wall to the top of slope. If it is infinite slope behind MSE Wall, input λ larger than 2H; If it is horizontal backfill behind MSE wall, $\lambda = 0$ ft

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \text{ deg}$$

$$\gamma_r := 120 \frac{\text{lb}}{\text{ft}^3}$$

$$c'_r := 0 \frac{\text{lb}}{\text{ft}^2}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \text{ deg}$$

$$\gamma_b := 120 \frac{\text{lb}}{\text{ft}^3}$$

$$c'_b := 0 \frac{\text{lb}}{\text{ft}^2}$$

$$\delta := \frac{2}{3} \cdot \phi'_b$$

$$\delta = 20 \text{ deg}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Soil Design Parameters for Bearing Resistance (Average 1.5 B Below Leveling Pad)

Drained Conditions (Effective Stress):

$$\phi'_{fd} := 24 \text{ deg} \quad \text{Effective angle of internal friction}$$

$$\gamma_{fd} := 112 \frac{\text{lb}_f}{\text{ft}^3} \quad \text{Unit weight}$$

$$c'_{fd} := 200 \frac{\text{lb}_f}{\text{ft}^2} \quad \text{Effective Cohesion}$$

Undrained Conditions (Total Stress):

$$\phi_{fdu} := 0 \text{ deg} \quad \text{Angle of internal friction (Same as Drained Conditions if Sand)}$$

$$Su_{fdu} := 2000 \frac{\text{lb}_f}{\text{ft}^2} \quad \text{Undrained Shear Strength}$$

Undercut & Replacement Design Parameters:

$$\phi_{Re} := 34 \text{ deg} \quad \text{Angle of internal friction for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.}$$

$$c_{Re} := 0 \frac{\text{lb}_f}{\text{ft}^2} \quad \text{Cohesion for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.}$$

$$\gamma_{Re} := 130 \frac{\text{lb}_f}{\text{ft}^3} \quad \text{Unit Weight for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.}$$

$$\delta_{Re} := \frac{2}{3} \cdot \phi_{Re} \quad \delta_{Re} = 22.7 \text{ deg} \quad \text{Friction angle between Replacement soils and footing taken as specified in LRFD BDS C3.11.5.3 (degrees)}$$

$$D_{undercut} := 1 \text{ ft} \quad \text{Depth of Undercut below bottom of footing}$$

Foundation Surcharge Soil Parameters

$$\gamma_q := 120 \frac{\text{lb}_f}{\text{ft}^3} \quad \text{Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)}$$

Depth of Embedment Check:

$$d_{frost} := 3.25 \text{ ft} \quad d_{user} := 5.4 \text{ ft} \quad \text{Local Frost Depth / User Input}$$

$$Slope_{fw} := 0 \text{ deg}$$

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user}) \quad d_{est} = 5.4 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e \quad H_{est} = 23.4 \text{ ft}$$

Inclination of ground slope in front of wall :

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1.2 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope}) \quad d_e = 5.4 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e \quad H = 23.4 \text{ ft}$$

Initial design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 0.7 \cdot H$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max \left(8 \cdot \text{ft}, 0.7 \cdot H, L_{user} \right)$$

$$L = 16.4 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L < \lambda, 0 \frac{\text{lbf}}{\text{ft}^2}, 250 \frac{\text{lbf}}{\text{ft}^2} \right) = 250 \frac{\text{lbf}}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

$$SUR_{@MSE} := \text{if} \left(L < \lambda, 0 \frac{\text{lbf}}{\text{ft}^2}, 250 \frac{\text{lbf}}{\text{ft}^2} \right) = 250 \frac{\text{lbf}}{\text{ft}^2}$$

Live load surcharge above the MSE Wall soil reinforcement (per LRFD BDS [3.11.6.4])

Calculations:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan \left(\beta_{BackSlope} \right)$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h := \text{if} \left(L < \lambda, H + L \cdot \tan \left(\beta_{BackSlope} \right), H + H_{BackSlope} \right)$$

Height of retained fill at the back of the reinforced soil

$$h = 23.4 \text{ ft}$$

$$I := \text{atan} \left(\frac{H_{BackSlope}}{2 \cdot H} \right)$$

$$\beta := \text{if} \left(\lambda < 2 \cdot H, I, \beta_{BackSlope} \right)$$

Angle of friction between retained backfill and reinforced soil

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 2.6845$$

$$k_{ab} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right)$$

$$k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot h^2 \cdot k_{ab}$$

$$F_T = 9767.8 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{ab}$$

$$F_{SUR} = 1739.3 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 45995 \frac{\text{lbf}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_2 = 0 \frac{\text{lbf}}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_3 := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lbf}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_3 = 0 \frac{\text{lbf}}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_4 := F_T \cdot \sin(\beta)$$

$$V_4 = 0 \frac{\text{lb}f}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_5 := F_{SUR} \cdot \sin(\beta)$$

$$V_5 = 0 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_6 := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_6 = 4095 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1} := \frac{L}{2}$$

$$d_{v1} = 8.2 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 376699.4 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := \text{if}\left(L < \lambda, \frac{2L}{3}, \frac{2 \cdot \lambda}{3}\right)$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$d_{v3} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v3} = 8.2 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3}$$

$$MV_3 = 0 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$d_{v4} := L$$

$$d_{v4} = 16.4 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4}$$

$$MV_4 = 0 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$d_{v5} := L$$

$$d_{v5} = 16.4 \text{ ft}$$

$$MV_5 := V_5 \cdot d_{v5}$$

$$MV_5 = 0 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$d_{v6} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v6} = 8.2 \text{ ft}$$

$$MV_6 := V_6 \cdot d_{v6}$$

$$MV_6 = 33538.1 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T \cdot \cos(\beta) = 9767.8 \frac{\text{lb}f}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} \cdot \cos(\beta) = 1739.3 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

Moment:

$$d_{h1} := \frac{h}{3}$$

$$d_{h1} = 7.8 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 76189.1 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{2}$$

$$d_{h2} = 11.7 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 20349.6 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1 + V_2 + V_3$$

$$V_{EV} = 45995 \frac{\text{lb}f}{\text{ft}}$$

$$V_{EH} := V_4$$

$$V_{EH} = 0 \frac{\text{lb}f}{\text{ft}}$$

$$V_{LS} := V_5$$

$$V_{LS} = 0 \frac{\text{lb}f}{\text{ft}}$$

$$V_{LS@MSE} := V_6$$

$$V_{LS@MSE} = 4095 \frac{\text{lb}f}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 9767.8 \frac{\text{lb}f}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 1739.3 \frac{\text{lb}f}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1 + MV_2 + MV_3$$

$$M_{EV} = 376699.4 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$M_{EHI} := MV_4$$

$$M_{EHI} = 0 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1} := MV_5$$

$$M_{LS1@MSE} := MV_6$$

$$M_{EH2} := MH_1$$

$$M_{LS2} := MH_2$$

$$M_{LS1} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE} = 33538.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} = 76189.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} = 20349.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1 \quad \text{LRFD Load Modifier}$$

$$\begin{aligned} \text{Strength Limit State I:} \quad & \text{EV}(\min) = 1.00 \quad \text{EV}(\max) = 1.35 \\ & \text{EH}(\min) = 0.90 \quad \text{EH}(\max) = 1.50 \\ & \text{LS} = 1.75 \end{aligned}$$

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot \left((Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE}) \right)$$

$$V_{Ia} = 53161.3 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot \left((Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE}) \right)$$

$$V_{Ib} = 69259.6 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot \left((Ia_{EH} \cdot H_{EH}) + (Ia_{LS} \cdot H_{LS}) \right)$$

$$H_{Ia} = 17695.5 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot \left((Ib_{EH} \cdot H_{EH}) + (Ib_{LS} \cdot H_{LS}) \right)$$

$$H_{Ib} = 17695.5 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot \left((Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + Ia_{LS} \cdot (M_{LS1} + M_{LS1@MSE}) \right)$$

$$MV_{Ia} = 435391 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot \left((Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + Ib_{LS} \cdot (M_{LS1} + M_{LS1@MSE}) \right)$$

$$MV_{Ib} = 567235.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot \left((Ia_{EH} \cdot M_{EH2}) + (Ia_{LS} \cdot M_{LS2}) \right)$$

$$MH_{Ia} = 149895.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot \left((Ib_{EH} \cdot M_{EH2}) + (Ib_{LS} \cdot M_{LS2}) \right)$$

$$MH_{Ib} = 149895.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Calculations at Bottom of Foundation Preparation Layer:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan(\beta_{BackSlope})$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h_{FPL} := \text{if}(L < \lambda, H + L \cdot \tan(\beta_{BackSlope}) + 1 \text{ ft}, H + H_{BackSlope} + 1 \text{ ft})$$

$$h_{FPL} = 24.4 \text{ ft}$$

Height of retained fill at the back of the reinforced soil

$$I := \text{atan}\left(\frac{H_{BackSlope}}{2 \cdot H}\right)$$

$$\beta := \text{if}(\lambda < 2 \cdot H, I, \beta_{BackSlope})$$

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

Angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 2.6845$$

$$k_{ab} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_{T_FPL} := \frac{1}{2} \cdot \gamma_b \cdot h_{FPL}^2 \cdot k_{ab} \quad F_T = 9767.8 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR_FPL} := SUR \cdot h_{FPL} \cdot k_{ab} \quad F_{SUR} = 1739.3 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_{1_FPL} := \gamma_r \cdot H \cdot L \quad V_{1_FPL} = 45995 \frac{\text{lbf}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_{2_FPL} := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_{2_FPL} = 0 \frac{\text{lbf}}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_{3_FPL} := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lbf}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_{3_FPL} = 0 \frac{\text{lbf}}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_{4_FPL} := F_{T_FPL} \cdot \sin(\beta)$$

$$V_{4_FPL} = 0 \frac{\text{lbf}}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_{5_FPL} := F_{SUR_FPL} \cdot \sin(\beta)$$

$$V_{5_FPL} = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_{6_FPL} := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_{6_FPL} = 4095 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

$$V_{7_FPL} := \gamma_{Re} \cdot (L + 1 \text{ ft}) \cdot D_{undercut}$$

$$V_{7_FPL} = 2259.4 \frac{\text{lbf}}{\text{ft}}$$

Foundation Preparation Layer Weight

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1_FPL} := \frac{L}{2} + 1 \text{ ft}$$

$$d_{v1_FPL} = 9.2 \text{ ft}$$

$$MV_{1_FPL} := V_{1_FPL} \cdot d_{v1_FPL} \quad MV_{1_FPL} = 422694.4 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2_FPL} := \text{if} \left(L < \lambda, \frac{2L}{3}, \frac{2\lambda}{3} \right) + 1 \text{ ft}$$

$$d_{v2_FPL} = 1 \text{ ft}$$

$$MV_{2_FPL} := V_{2_FPL} \cdot d_{v2_FPL} \quad MV_{2_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v3_FPL} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right) + 1 \text{ ft} \quad d_{v3_FPL} = 9.2 \text{ ft}$$

$$MV_{3_FPL} := V_{3_FPL} \cdot d_{v3_FPL} \quad MV_{3_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v4_FPL} := L + 1 \text{ ft}$$

$$d_{v4_FPL} = 17.4 \text{ ft}$$

$$MV_{4_FPL} := V_{4_FPL} \cdot d_{v4_FPL} \quad MV_{4_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v5_FPL} := L + 1 \text{ ft}$$

$$d_{v5_FPL} = 17.4 \text{ ft}$$

$$MV_{5_FPL} := V_{5_FPL} \cdot d_{v5_FPL} \quad MV_{5_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v6_FPL} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right) + 1 \text{ ft} \quad d_{v6_FPL} = 9.2 \text{ ft}$$

$$MV_{6_FPL} := V_{6_FPL} \cdot d_{v6_FPL} \quad MV_{6_FPL} = 37633.1 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v7_FPL} := \frac{(L + 1 \text{ ft})}{2}$$

$$d_{v7_FPL} = 8.7 \text{ ft}$$

$$MV_{7_FPL} := V_{7_FPL} \cdot d_{v7_FPL} \quad MV_{7_FPL} = 19634.2 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_{1_FPL} := F_{T_FPL} \cdot \cos(\beta) = 10620.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$H_{2_FPL} := F_{SUR_FPL} \cdot \cos(\beta) = 1813.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1_FPL} := \frac{h_{FPL}}{3} \quad d_{h1_FPL} = 8.1 \text{ ft}$$

$$d_{h2_FPL} := \frac{h_{FPL}}{2} \quad d_{h2_FPL} = 12.2 \text{ ft}$$

Moment:

$$MH_{1_FPL} := H_{1_FPL} \cdot d_{h1_FPL} \quad MH_{1_FPL} = 86380.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{2_FPL} := H_{2_FPL} \cdot d_{h2_FPL} \quad MH_{2_FPL} = 22126.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV_FPL} := V_{1_FPL} + V_{2_FPL} + V_{3_FPL} + V_{7_FPL}$$

$$V_{EV_FPL} = 48254.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{EH_FPL} := V_{4_FPL}$$

$$V_{EH_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{LS_FPL} := V_{5_FPL}$$

$$V_{LS_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{LS@MSE_FPL} := V_{6_FPL}$$

$$V_{LS@MSE_FPL} = 4095 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$H_{EH_FPL} := H_{1_FPL}$$

$$H_{EH_FPL} = 10620.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$H_{LS_FPL} := H_{2_FPL}$$

$$H_{LS_FPL} = 1813.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV_FPL} := MV_{1_FPL} + MV_{2_FPL} + MV_{3_FPL} + MV_{7_FPL}$$

$$M_{EV_FPL} = 442328.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1_FPL} := MV_{4_FPL}$$

$$M_{EH1_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1_FPL} := MV_{5_FPL}$$

$$M_{LS1_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE_FPL} := MV_{6_FPL}$$

$$M_{LS1@MSE_FPL} = 37633.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2_FPL} := MH_{1_FPL}$$

$$M_{EH2_FPL} = 86380.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2_FPL} := MH_{2_FPL}$$

$$M_{LS2_FPL} = 22126.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1$$

LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia_FPL} := \eta \cdot ((Ia_{EV} \cdot V_{EV_FPL}) + (Ia_{EH} \cdot V_{EH_FPL}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE_FPL})) \quad V_{Ia_FPL} = 55420.7 \frac{lb_f}{ft}$$

$$V_{Ib_FPL} := \eta \cdot ((Ib_{EV} \cdot V_{EV_FPL}) + (Ib_{EH} \cdot V_{EH_FPL}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE_FPL})) \quad V_{Ib_FPL} = 72309.7 \frac{lb_f}{ft}$$

Compute Bearing Resistance:

$$H_{Ia_FPL} := \eta \cdot ((Ia_{EH} \cdot H_{EH_FPL}) + (Ia_{LS} \cdot H_{LS_FPL})) \quad H_{Ia_FPL} = 19104.6 \frac{lb_f}{ft}$$

$$H_{Ib_FPL} := \eta \cdot ((Ib_{EH} \cdot H_{EH_FPL}) + (Ib_{LS} \cdot H_{LS_FPL})) \quad H_{Ib_FPL} = 19104.6 \frac{lb_f}{ft}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia_FPL} := \eta \cdot ((Ia_{EV} \cdot M_{EV_FPL}) + (Ia_{EH} \cdot M_{EH1_FPL}) + Ia_{LS} \cdot (M_{LS1_FPL} + M_{LS1@MSE_FPL})) \quad MV_{Ia_FPL} = 508186.4 \frac{lb_f \cdot ft}{ft}$$

$$MV_{Ib_FPL} := \eta \cdot ((Ib_{EV} \cdot M_{EV_FPL}) + (Ib_{EH} \cdot M_{EH1_FPL}) + Ib_{LS} \cdot (M_{LS1_FPL} + M_{LS1@MSE_FPL})) \quad MV_{Ib_FPL} = 663001.5 \frac{lb_f \cdot ft}{ft}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia_FPL} := \eta \cdot ((Ia_{EH} \cdot M_{EH2_FPL}) + (Ia_{LS} \cdot M_{LS2_FPL})) \quad MH_{Ia_FPL} = 168291.1 \frac{lb_f \cdot ft}{ft}$$

$e = 2.72$

$$MH_{Ib_FPL} := \eta \cdot ((Ib_{EH} \cdot M_{EH2_FPL}) + (Ib_{LS} \cdot M_{LS2_FPL})) \quad MH_{Ib_FPL} = 168291.1 \frac{lb_f \cdot ft}{ft}$$

Factored Horizontal Loads by Limit State:

Compute the Effective Bearing Length (Strength Ib) about the Toe "O" of base length (the pivot):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 567235.7 \frac{lb_f \cdot ft}{ft} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 149895.5 \frac{lb_f \cdot ft}{ft} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 69259.6 \frac{lb_f}{ft} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V} \quad x = 6 \text{ ft} \quad \text{Distance from Point "O" the resultant intersects the base}$$

$$e := \left| \frac{L}{2} - x \right| \quad e = 2.2 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B'_{top} := L - 2 \cdot e \quad B'_{top} = 12.1 \text{ ft} \quad \text{Effective Footing Width, at the top of Foundation Preparation Layer}$$

$$\Sigma M_{R_FPL} := MV_{lb_FPL}$$

$$\Sigma M_{R_FPL} = 663001.5 \frac{lb \cdot ft}{ft}$$

Sum of Resisting Moments (Strength lb)

$$\Sigma M_{O_FPL} := MH_{lb_FPL}$$

$$\Sigma M_{O_FPL} = 168291.1 \frac{lb \cdot ft}{ft}$$

Sum of Overturning Moments (Strength lb)

$$\Sigma V_{FPL} := V_{lb_FPL}$$

$$\Sigma V_{FPL} = 72309.7 \frac{lb}{ft}$$

Sum of Vertical Loads (Strength lb)

$$x_{FPL} := \frac{(\Sigma M_{R_FPL} - \Sigma M_{O_FPL})}{\Sigma V_{FPL}}$$

$$x_{FPL} = 6.8 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e_{FPL} := \left| \frac{(L + 1 \text{ ft})}{2} - x_{FPL} \right|$$

$$e_{FPL} = 1.8 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B'_{bottom} := (L + 1 \text{ ft}) - 2 \cdot e_{FPL}$$

$$B'_{bottom} = 13.7 \text{ ft}$$

Effective Footing Width, at the bottom of Foundation Preparation Layer (FPL)

Foundation Layout:

$$L_{Wall} := 1423.79 \text{ ft}$$

Assumed Footing Length (Wall Section Length)

$$D_f := d_e$$

$$D_f = 5.4 \text{ ft}$$

Footing embedment

$$D_F := D_f + D_{undercut}$$

Embenment Depth at bottom of Undercut

$$d_w := D_f - 1 \text{ ft}$$

Depth of Groundwater below ground surface in front of wall

Compute Bearing Resistance at the top of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 9.6$$

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

$$N_c = 19.32$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 9.4$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.004$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \cdot \tan(\phi'_{fd}) \right), 1 \right)$$

$$s_q = 1.004$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.997$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.15$$

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

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

$$N_{cm} = 19.405$$

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

$$N_{qm} = 11.114$$

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

$$N_{\gamma m} = 9.41$$

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

$$q_{nd_top} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad \text{at top of foundation preparation layer}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_top} := \phi_b \cdot q_{nd_top} \quad q_{Rd_top} = 6.9 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

$$s_c := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.002$$

$$s_q := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \tan(\phi_{fdu}), 1\right)$$

$$s_q = 1$$

$$s_\gamma := \text{if}\left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}}\right), 1\right)$$

$$s_\gamma = 1$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \tan\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.15$$

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

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

$$N_{cm} = 5.149$$

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

$$N_{qm} = 1.153$$

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

$$N_{\gamma m} = 0$$

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

$$q_{nu_top} := Su_{fdu} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nu_top} = 10671 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_top} := \phi_b \cdot q_{nu_top} \quad q_{Ru_top} = 6.9 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_top} = 6.9 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_top} = 6.9 \text{ ksf}$$

Compute Bearing Resistance at the bottom of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 9.6$$

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

$$N_c = 19.32$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 9.4$$

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

$$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.005$$

$$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \tan(\phi'_{fd}), 1\right)$$

$$s_q = 1.004$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.996$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right)$$

$$d_q = 1.14$$

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

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

$$N_{cm} = 19.416$$

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

$$N_{qm} = 10.967$$

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

$$N_{\gamma m} = 9.406$$

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

$$q_{nd_bottom} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

at bottom of foundation preparation layer

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_bottom} := \phi_b \cdot q_{nd_bottom} \quad q_{Rd_bottom} = 7.6 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.002$$

$$s_q := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \cdot \tan(\phi_{fdu}) \right), 1 \right) \quad s_q = 1$$

$$s_\gamma := \text{if} \left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right) \quad s_\gamma = 1$$

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

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

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

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

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right)$$

$$d_q = 1.14$$

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

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

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 1.137$$

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

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

$$q_{nu_bottom} := Su_{fdu} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu_top} = 10671 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_bottom} := \phi_b \cdot q_{nu_top} \quad q_{Ru_bottom} = 6.9 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_bottom} = 7.6 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_bottom} = 6.9 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location about the toe "O" of base length (the pivot):

$$e := \left| \frac{L}{2} - x \right| \quad e = 2.16 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2]. The effective bearing width is equal to B-2e.

$$\sigma_v := \frac{\Sigma V}{B'_{top}} \quad \sigma_v = 5747 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Bearing Stress}$$

$$\sigma_{v_FPL} := \frac{\Sigma V_{FPL}}{B'_{bottom}} \quad \sigma_{v_FPL} = 5284.6 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Bearing Stress}$$

Bearing Resistance Capacity:Demand Ratio (CDR)

At the top of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_top} := \frac{q_{Rd_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_top} = 1.21$

Undrained Conditions: $CDR_{Bearing_u_top} := \frac{q_{Ru_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_top} = 1.21$

At the bottom of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_bottom} := \frac{q_{Rd_bottom}}{\sigma_{v_FPL}}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_bottom} = 1.44$

Undrained Conditions: $CDR_{Bearing_u_bottom} := \frac{q_{Ru_bottom}}{\sigma_{v_FPL}}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_bottom} = 1.31$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$e_{max} := \frac{L}{3}$

$e_{max} = 5.5 \text{ ft}$

Maximum Eccentricity LRFD [C11.6.3.3.]

$\Sigma M_R := MV_{Ia}$

$\Sigma M_R = 435391 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$

Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$

$\Sigma M_O = 149895.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$

Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$

$\Sigma V = 53161.3 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$

$x = 5.4 \text{ ft}$

Distance from Point "O" the resultant intersects the base

$e := \left| \frac{L}{2} - x \right|$

$e = 2.82 \text{ ft}$

Wall eccentricity

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e}$

Is the CDR > or = to 1.0?

$CDR_{Eccentricity} = 1.94$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$

$R_u = 17695.5 \frac{\text{lb}}{\text{ft}}$

At the top of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$\Sigma V := V_{Ia}$

$\Sigma V = 53161.3 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$R_{td_top} := \Sigma V \cdot \tan(\phi_{Re})$

$R_{td_top} = 35857.7 \frac{\text{lb}}{\text{ft}}$

Nominal sliding resistance Drained Conditions

Compute factored resistance against failure by sliding LRFD [10.6.3.4]:

$\phi_\tau := 1.0$

Resistance factor for sliding resistance specified in LRFD Table 11.5.7-1.

Drained Conditions:

$\phi R_{n_d_top} := \phi_\tau \cdot R_{td_top}$

$R_{R_d_top} := \phi R_{n_d_top}$

$R_{R_d_top} = 35.9 \frac{\text{kip}}{\text{ft}}$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_top} := \frac{R_{R_d_top}}{R_u} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_d_top} = 2.03$$

At the bottom of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V_{FPL} := V_{Ia_FPL} \quad \Sigma V_{FPL} = 55420.7 \frac{lb}{ft} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$R_{td_bottom} := \Sigma V_{FPL} \cdot \tan(\phi'_{fd}) \quad R_{td_bottom} = 24674.9 \frac{lb}{ft} \quad \text{Nominal sliding resistance Drained Conditions}$$

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_r := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\text{Drained Conditions:} \quad \phi R_{n_d_bottom} := \phi_r \cdot R_{td_bottom} \quad R_{R_d_bottom} := \phi R_{n_d_bottom} \quad R_{R_d_bottom} = 24.7 \frac{kip}{ft}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_bottom} := \frac{R_{R_d_bottom}}{H_{Ia_FPL}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_d_bottom} = 1.29$$

Undrained Conditions (Total Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V_{FPL} := V_{Ia_FPL} \quad \Sigma V_{FPL} = 55420.7 \frac{lb}{ft} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$B := L + 1 \text{ ft} \quad \text{Footing base width}$$

$$\sigma_{v_FPL} := \frac{\Sigma V_{FPL}}{B} \quad \sigma_{v_FPL} = 3188.8 \frac{lb}{ft^2} \quad \text{Vertical stress ODOT GDM [1504]}$$

$$q_{FPL} := \frac{1}{2} \cdot \sigma_{v_FPL} \quad q_{FPL} = 1594.4 \frac{lb}{ft^2} \quad \text{Uniform unit shear resistance as 1/2 vertical stress ODOT GDM [1504]}$$

Determine which Cohesive Soil Resistance Case is Present:

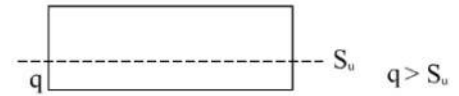
$$Case_1 := \text{if}(q_{FPL} > Su_{fdu}, 1, 0) \quad Case_1 = 0$$

$$Case_2 := \text{if}(Su_{fdu} \geq q_{FPL}, 1, 0) \quad Case_2 = 1$$

Unit Shear Resistance for Case 1:

$$R_{\tau_case1} := Su_{fdu} \cdot B = 34760 \frac{lb}{ft}$$

Case 1:



Unit Shear Resistance for Case 2:

$$R_{\tau_case2} := q_{FPL} \cdot B = 27710.3 \frac{lb}{ft}$$

Case 2:



Define the Applicable Case:

$$R_{\tau} := R_{\tau_case2}$$

$$R_{\tau} = 27710.3 \frac{lb}{ft}$$

Nominal sliding resistance Cohesive Soils

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]**:

$$\phi_{ep} := 0.5$$

Resistance factor for passive resistance specified in
LRFD Table 10.5.5.2.2-1

$$\phi_{\tau} := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

Factored Sliding Resistance to be used in CDR Calculations:

$$R_R = 27710.345 \frac{lb}{ft}$$

Sliding Capacity:Demand Ratio (CDR)

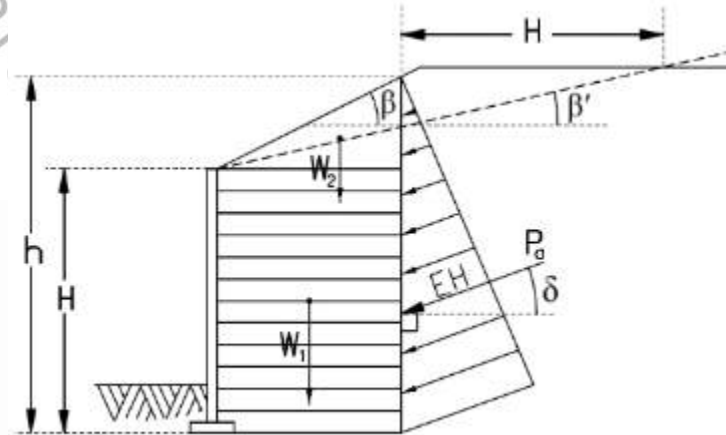
$$CDR_{Sliding_u_bottom} := \frac{R_R}{H_{Ia_FPL}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_u_bottom} = 1.45$$

Objective: To evaluate the external stability of MSE wall design with vertical wall face and broken backslope case.
Method: In accordance with ODOT Bridge Design Manual, 2026 [Sect. 307.4] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 3.11.5.8 and Sect. 11.10.5].

Assumptions:

- Horizontal backfill, Infinite backslope or Broken backslope behind MSE wall, see LRFD Sect. 3.11.5.8.1.
- For battered or vertical walls with a back face of wall angle of θ to horizontal.
- Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ .
- MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall



Givens:

Wall Geometry:

$$H_e := 28.9 \text{ ft}$$

$$\theta := 90 \cdot \text{deg}$$

$$\beta_{\text{BackSlope}} := 0 \text{ deg}$$

$$\lambda := 0 \text{ ft}$$

Exposed wall height (Proposed profile grade to bottom of leveling pad)

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Inclination of ground slope behind face of wall. If it is horizontal backfill behind MSE wall, $\beta_{\text{BackSlope}} = 0$ deg

Horizontal distance from the back of MSE wall to the top of slope. If it is infinite slope behind MSE Wall, input λ larger than 2H; If it is horizontal backfill behind MSE wall, $\lambda = 0$ ft

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \text{ deg}$$

$$\gamma_r := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_r := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \text{ deg}$$

$$\gamma_b := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_b := 0 \frac{\text{lbf}}{\text{ft}^2}$$

$$\delta := \frac{2}{3} \cdot \phi'_b$$

$$\delta = 20 \text{ deg}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Soil Design Parameters for Bearing Resistance (Average 1.5 B Below Leveling Pad)

Drained Conditions (Effective Stress):

$\phi'_{fd} := 25 \text{ deg}$ Effective angle of internal friction

$\gamma_{fd} := 116 \frac{\text{lb}}{\text{ft}^3}$ Unit weight

$c'_{fd} := 250 \frac{\text{lb}}{\text{ft}^2}$ Effective Cohesion

Undrained Conditions (Total Stress):

$\phi_{fdu} := 0 \text{ deg}$ Angle of internal friction (Same as Drained Conditions if Sand)

$Su_{fdu} := 2600 \frac{\text{lb}}{\text{ft}^2}$ Undrained Shear Strength

Undercut & Replacement Design Parameters:

$\phi_{Re} := 34 \text{ deg}$ Angle of internal friction for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$c_{Re} := 0 \frac{\text{lb}}{\text{ft}^2}$ Cohesion for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$\gamma_{Re} := 130 \frac{\text{lb}}{\text{ft}^3}$ Unit Weight for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$\delta_{Re} := \frac{2}{3} \cdot \phi_{Re}$ Friction angle between Replacement soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

$\delta_{Re} = 22.7 \text{ deg}$

$D_{undercut} := 1 \text{ ft}$ Depth of Undercut below bottom of footing

Foundation Surcharge Soil Parameters

$\gamma_q := 120 \frac{\text{lb}}{\text{ft}^3}$ Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Depth of Embedment Check:

$d_{frost} := 3.25 \text{ ft}$ $d_{user} := 4.7 \text{ ft}$ Local Frost Depth / User Input

$Slope_{fw} := 0 \text{ deg}$

$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$ $d_{est} = 4.7 \text{ ft}$

$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$ $H_{est} = 33.6 \text{ ft}$

Inclination of ground slope in front of wall :

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$d_{eSlope} := \text{if}\left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if}\left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if}\left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5}\right)\right)\right)$

$d_{eSlope} = 1.7 \text{ ft}$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$d_e := \max(d_{est}, d_{eSlope})$ $d_e = 4.7 \text{ ft}$

Minimum Required Embedment Depth used in analysis.

$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$ $H = 33.6 \text{ ft}$

Initial design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 0.7 \cdot H$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max \left(8 \cdot \text{ft}, 0.7 \cdot H, L_{user} \right)$$

$$L = 23.5 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L < \lambda, 0 \frac{\text{lb}}{\text{ft}^2}, 250 \frac{\text{lb}}{\text{ft}^2} \right) = 250 \frac{\text{lb}}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

$$SUR_{@MSE} := \text{if} \left(L < \lambda, 0 \frac{\text{lb}}{\text{ft}^2}, 250 \frac{\text{lb}}{\text{ft}^2} \right) = 250 \frac{\text{lb}}{\text{ft}^2}$$

Live load surcharge above the MSE Wall soil reinforcement (per LRFD BDS [3.11.6.4])

Calculations:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan \left(\beta_{BackSlope} \right)$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h := \text{if} \left(L < \lambda, H + L \cdot \tan \left(\beta_{BackSlope} \right), H + H_{BackSlope} \right)$$

Height of retained fill at the back of the reinforced soil

$$h = 33.6 \text{ ft}$$

$$I := \text{atan} \left(\frac{H_{BackSlope}}{2 \cdot H} \right)$$

$$\beta := \text{if} \left(\lambda < 2 \cdot H, I, \beta_{BackSlope} \right)$$

Angle of friction between retained backfill and reinforced soil

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

$$\Gamma := \left(1 + \sqrt{\frac{\left(\sin \left(\phi'_b + \delta \right) \cdot \sin \left(\phi'_b - \beta \right) \right)}{\left(\sin \left(\theta - \delta \right) \cdot \sin \left(\theta + \beta \right) \right)}} \right)^2 = 2.6845$$

$$k_{ab} := \left(\frac{\left(\sin \left(\theta + \phi'_b \right) \right)^2}{\left(\Gamma \cdot \left(\sin \left(\theta \right) \right)^2 \cdot \sin \left(\theta - \delta \right) \right)} \right)$$

$$k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot h^2 \cdot k_{ab}$$

$$F_T = 20139.3 \frac{\text{lb}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{ab}$$

$$F_{SUR} = 2497.4 \frac{\text{lb}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 94832.6 \frac{\text{lb}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_2 = 0 \frac{\text{lb}}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_3 := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lb}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_3 = 0 \frac{\text{lb}}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_4 := F_T \cdot \sin(\beta)$$

$$V_4 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_5 := F_{SUR} \cdot \sin(\beta)$$

$$V_5 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_6 := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_6 = 5880 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1} := \frac{L}{2}$$

$$d_{v1} = 11.8 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 1115231.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := \text{if}\left(L < \lambda, \frac{2L}{3}, \frac{2 \cdot \lambda}{3}\right)$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v3} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v3} = 11.8 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3}$$

$$MV_3 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v4} := L$$

$$d_{v4} = 23.5 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4}$$

$$MV_4 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v5} := L$$

$$d_{v5} = 23.5 \text{ ft}$$

$$MV_5 := V_5 \cdot d_{v5}$$

$$MV_5 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v6} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v6} = 11.8 \text{ ft}$$

$$MV_6 := V_6 \cdot d_{v6}$$

$$MV_6 = 69148.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T \cdot \cos(\beta) = 20139.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} \cdot \cos(\beta) = 2497.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

Moment:

$$d_{h1} := \frac{h}{3}$$

$$d_{h1} = 11.2 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 225560.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{2}$$

$$d_{h2} = 16.8 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 41956.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1 + V_2 + V_3$$

$$V_{EV} = 94832.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{EH} := V_4$$

$$V_{EH} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{LS} := V_5$$

$$V_{LS} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{LS@MSE} := V_6$$

$$V_{LS@MSE} = 5880 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 20139.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 2497.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1 + MV_2 + MV_3$$

$$M_{EV} = 1115231.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_4$$

$$M_{EH1} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1} := MV_5$$

$$M_{LS1@MSE} := MV_6$$

$$M_{EH2} := MH_1$$

$$M_{LS2} := MH_2$$

$$M_{LS1} = 0 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$M_{LS1@MSE} = 69148.8 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$M_{EH2} = 225560.5 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$M_{LS2} = 41956.9 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1 \quad \text{LRFD Load Modifier}$$

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot \left((Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE}) \right)$$

$$V_{Ia} = 105122.6 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot \left((Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE}) \right)$$

$$V_{Ib} = 138314.1 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot \left((Ia_{EH} \cdot H_{EH}) + (Ia_{LS} \cdot H_{LS}) \right)$$

$$H_{Ia} = 34579.5 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot \left((Ib_{EH} \cdot H_{EH}) + (Ib_{LS} \cdot H_{LS}) \right)$$

$$H_{Ib} = 34579.5 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot \left((Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + Ia_{LS} \cdot (M_{LS1} + M_{LS1@MSE}) \right)$$

$$MV_{Ia} = 1236242.2 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot \left((Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + Ib_{LS} \cdot (M_{LS1} + M_{LS1@MSE}) \right)$$

$$MV_{Ib} = 1626573.4 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot \left((Ia_{EH} \cdot M_{EH2}) + (Ia_{LS} \cdot M_{LS2}) \right)$$

$$MH_{Ia} = 411765.3 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot \left((Ib_{EH} \cdot M_{EH2}) + (Ib_{LS} \cdot M_{LS2}) \right)$$

$$MH_{Ib} = 411765.3 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

Calculations at Bottom of Foundation Preparation Layer:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan(\beta_{BackSlope})$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h_{FPL} := \text{if}(L < \lambda, H + L \cdot \tan(\beta_{BackSlope}) + 1 \text{ ft}, H + H_{BackSlope} + 1 \text{ ft})$$

$$h_{FPL} = 34.6 \text{ ft}$$

Height of retained fill at the back of the reinforced soil

$$I := \text{atan}\left(\frac{H_{BackSlope}}{2 \cdot H}\right)$$

$$\beta := \text{if}(\lambda < 2 \cdot H, I, \beta_{BackSlope})$$

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

Angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 2.6845$$

$$k_{ab} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_{T_FPL} := \frac{1}{2} \cdot \gamma_b \cdot h_{FPL}^2 \cdot k_{ab} \quad F_T = 20139.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR_FPL} := SUR \cdot h_{FPL} \cdot k_{ab} \quad F_{SUR} = 2497.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_{1_FPL} := \gamma_r \cdot H \cdot L \quad V_{1_FPL} = 94832.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_{2_FPL} := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_{2_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_{3_FPL} := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_{3_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_{4_FPL} := F_{T_FPL} \cdot \sin(\beta)$$

$$V_{4_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_{5_FPL} := F_{SUR_FPL} \cdot \sin(\beta)$$

$$V_{5_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_{6_FPL} := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_{6_FPL} = 5880 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

$$V_{7_FPL} := \gamma_{Re} \cdot (L + 1 \text{ ft}) \cdot D_{undercut}$$

$$V_{7_FPL} = 3187.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Foundation Preparation Layer Weight

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1_FPL} := \frac{L}{2} + 1 \text{ ft}$$

$$d_{v1_FPL} = 12.8 \text{ ft}$$

$$MV_{1_FPL} := V_{1_FPL} \cdot d_{v1_FPL} \quad MV_{1_FPL} = 1210064.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2_FPL} := \text{if} \left(L < \lambda, \frac{2L}{3}, \frac{2\lambda}{3} \right) + 1 \text{ ft}$$

$$d_{v2_FPL} = 1 \text{ ft}$$

$$MV_{2_FPL} := V_{2_FPL} \cdot d_{v2_FPL} \quad MV_{2_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v3_FPL} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right) + 1 \text{ ft} \quad d_{v3_FPL} = 12.8 \text{ ft}$$

$$MV_{3_FPL} := V_{3_FPL} \cdot d_{v3_FPL} \quad MV_{3_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v4_FPL} := L + 1 \text{ ft}$$

$$d_{v4_FPL} = 24.5 \text{ ft}$$

$$MV_{4_FPL} := V_{4_FPL} \cdot d_{v4_FPL} \quad MV_{4_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v5_FPL} := L + 1 \text{ ft}$$

$$d_{v5_FPL} = 24.5 \text{ ft}$$

$$MV_{5_FPL} := V_{5_FPL} \cdot d_{v5_FPL} \quad MV_{5_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v6_FPL} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right) + 1 \text{ ft} \quad d_{v6_FPL} = 12.8 \text{ ft}$$

$$MV_{6_FPL} := V_{6_FPL} \cdot d_{v6_FPL} \quad MV_{6_FPL} = 75028.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v7_FPL} := \frac{(L + 1 \text{ ft})}{2}$$

$$d_{v7_FPL} = 12.3 \text{ ft}$$

$$MV_{7_FPL} := V_{7_FPL} \cdot d_{v7_FPL} \quad MV_{7_FPL} = 39080 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_{1_FPL} := F_{T_FPL} \cdot \cos(\beta) = 21355.9 \frac{\text{lb}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_{2_FPL} := F_{SUR_FPL} \cdot \cos(\beta) = 2571.8 \frac{\text{lb}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1_FPL} := \frac{h_{FPL}}{3} \quad d_{h1_FPL} = 11.5 \text{ ft}$$

Moment:

$$MH_{1_FPL} := H_{1_FPL} \cdot d_{h1_FPL} \quad MH_{1_FPL} = 246305.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2_FPL} := \frac{h_{FPL}}{2} \quad d_{h2_FPL} = 17.3 \text{ ft}$$

$$MH_{2_FPL} := H_{2_FPL} \cdot d_{h2_FPL} \quad MH_{2_FPL} = 44491.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV_FPL} := V_{1_FPL} + V_{2_FPL} + V_{3_FPL} + V_{7_FPL}$$

$$V_{EV_FPL} = 98020.2 \frac{\text{lb}}{\text{ft}}$$

$$V_{EH_FPL} := V_{4_FPL}$$

$$V_{EH_FPL} = 0 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS_FPL} := V_{5_FPL}$$

$$V_{LS_FPL} = 0 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS@MSE_FPL} := V_{6_FPL}$$

$$V_{LS@MSE_FPL} = 5880 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH_FPL} := H_{1_FPL}$$

$$H_{EH_FPL} = 21355.9 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS_FPL} := H_{2_FPL}$$

$$H_{LS_FPL} = 2571.8 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV_FPL} := MV_{1_FPL} + MV_{2_FPL} + MV_{3_FPL} + MV_{7_FPL}$$

$$M_{EV_FPL} = 1249144.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1_FPL} := MV_{4_FPL}$$

$$M_{EH1_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1_FPL} := MV_{5_FPL}$$

$$M_{LS1_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE_FPL} := MV_{6_FPL}$$

$$M_{LS1@MSE_FPL} = 75028.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2_FPL} := MH_{1_FPL}$$

$$M_{EH2_FPL} = 246305.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2_FPL} := MH_{2_FPL}$$

$$M_{LS2_FPL} = 44491.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1$$

LRFD Load Modifier

Strength Limit State I:
EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia_FPL} := \eta \cdot \left((Ia_{EV} \cdot V_{EV_FPL}) + (Ia_{EH} \cdot V_{EH_FPL}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE_FPL}) \right) \quad V_{Ia_FPL} = 108310.2 \frac{lb_f}{ft}$$

$$V_{Ib_FPL} := \eta \cdot \left((Ib_{EV} \cdot V_{EV_FPL}) + (Ib_{EH} \cdot V_{EH_FPL}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE_FPL}) \right) \quad V_{Ib_FPL} = 142617.3 \frac{lb_f}{ft}$$

Compute Bearing Resistance:

$$H_{Ia_FPL} := \eta \cdot \left((Ia_{EH} \cdot H_{EH_FPL}) + (Ia_{LS} \cdot H_{LS_FPL}) \right) \quad H_{Ia_FPL} = 36534.5 \frac{lb_f}{ft}$$

$$H_{Ib_FPL} := \eta \cdot \left((Ib_{EH} \cdot H_{EH_FPL}) + (Ib_{LS} \cdot H_{LS_FPL}) \right) \quad H_{Ib_FPL} = 36534.5 \frac{lb_f}{ft}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia_FPL} := \eta \cdot \left((Ia_{EV} \cdot M_{EV_FPL}) + (Ia_{EH} \cdot M_{EH1_FPL}) + Ia_{LS} \cdot (M_{LS1_FPL} + M_{LS1@MSE_FPL}) \right) \quad MV_{Ia_FPL} = 1380444.9 \frac{lb_f \cdot ft}{ft}$$

$$MV_{Ib_FPL} := \eta \cdot \left((Ib_{EV} \cdot M_{EV_FPL}) + (Ib_{EH} \cdot M_{EH1_FPL}) + Ib_{LS} \cdot (M_{LS1_FPL} + M_{LS1@MSE_FPL}) \right) \quad MV_{Ib_FPL} = 1817645.4 \frac{lb_f \cdot ft}{ft}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia_FPL} := \eta \cdot \left((Ia_{EH} \cdot M_{EH2_FPL}) + (Ia_{LS} \cdot M_{LS2_FPL}) \right) \quad MH_{Ia_FPL} = 447317.9 \frac{lb_f \cdot ft}{ft}$$

$e = 2.72$

$$MH_{Ib_FPL} := \eta \cdot \left((Ib_{EH} \cdot M_{EH2_FPL}) + (Ib_{LS} \cdot M_{LS2_FPL}) \right) \quad MH_{Ib_FPL} = 447317.9 \frac{lb_f \cdot ft}{ft}$$

Factored Horizontal Loads by Limit State:

Compute the Effective Bearing Length (Strength Ib) about the Toe "O" of base length (the pivot):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 1626573.4 \frac{lb_f \cdot ft}{ft} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 411765.3 \frac{lb_f \cdot ft}{ft} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 138314.1 \frac{lb_f}{ft} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V} \quad x = 8.8 \text{ ft} \quad \text{Distance from Point "O" the resultant intersects the base}$$

$$e := \left| \frac{L}{2} - x \right| \quad e = 3 \text{ ft} \quad \text{Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.}$$

$$B'_{top} := L - 2 \cdot e \quad B'_{top} = 17.6 \text{ ft} \quad \text{Effective Footing Width, at the top of Foundation Preparation Layer}$$

$$\Sigma M_{R_FPL} := MV_{lb_FPL}$$

$$\Sigma M_{R_FPL} = 1817645.4 \frac{lb \cdot ft}{ft}$$

Sum of Resisting Moments (Strength lb)

$$\Sigma M_{O_FPL} := MH_{lb_FPL}$$

$$\Sigma M_{O_FPL} = 447317.9 \frac{lb \cdot ft}{ft}$$

Sum of Overturning Moments (Strength lb)

$$\Sigma V_{FPL} := V_{lb_FPL}$$

$$\Sigma V_{FPL} = 142617.3 \frac{lb}{ft}$$

Sum of Vertical Loads (Strength lb)

$$x_{FPL} := \frac{(\Sigma M_{R_FPL} - \Sigma M_{O_FPL})}{\Sigma V_{FPL}}$$

$$x_{FPL} = 9.6 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e_{FPL} := \left| \frac{(L + 1 \text{ ft})}{2} - x_{FPL} \right|$$

$$e_{FPL} = 2.7 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B'_{bottom} := (L + 1 \text{ ft}) - 2 \cdot e_{FPL}$$

$$B'_{bottom} = 19.2 \text{ ft}$$

Effective Footing Width, at the bottom of Foundation Preparation Layer (FPL)

Foundation Layout:

$$L_{Wall} := 1423.79 \text{ ft}$$

Assumed Footing Length (Wall Section Length)

$$D_f := d_e$$

$$D_f = 4.7 \text{ ft}$$

Footing embedment

$$D_F := D_f + D_{undercut}$$

Embenment Depth at bottom of Undercut

$$d_w := D_f - 1 \text{ ft}$$

Depth of Groundwater below ground surface in front of wall

Compute Bearing Resistance at the top of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 10.66$$

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

$$N_c = 20.72$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 10.9$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.006$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \cdot \tan(\phi'_{fd}) \right), 1 \right)$$

$$s_q = 1.006$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.995$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 20.852$$

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

$$N_{qm} = 11.77$$

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

$$N_{\gamma m} = 10.823$$

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

$$q_{nd_top} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad \text{at top of foundation preparation layer}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_top} := \phi_b \cdot q_{nd_top} \quad q_{Rd_top} = 9.1 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

$$s_c := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.002$$

$$s_q := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \tan(\phi_{fdu}), 1\right)$$

$$s_q = 1$$

$$s_\gamma := \text{if}\left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}}\right), 1\right)$$

$$s_\gamma = 1$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \tan\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 5.153$$

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

$$N_{qm} = 1.098$$

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

$$N_{\gamma m} = 0$$

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

$$q_{nu_top} := Su_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nu_top} = 13706.5 \frac{lbf}{ft^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_top} := \phi_b \cdot q_{nu_top} \quad q_{Ru_top} = 8.9 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_top} = 9.1 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_top} = 8.9 \text{ ksf}$$

Compute Bearing Resistance at the bottom of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 10.66$$

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

$$N_c = 20.72$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 10.9$$

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

$$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.007$$

$$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \tan(\phi'_{fd}), 1\right)$$

$$s_q = 1.006$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.995$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right)$$

$$d_q = 1.09$$

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

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

$$N_{cm} = 20.864$$

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

$$N_{qm} = 11.691$$

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

$$N_{\gamma m} = 10.818$$

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

$$q_{nd_bottom} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

at bottom of foundation preparation layer

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_bottom} := \phi_b \cdot q_{nd_bottom} \quad q_{Rd_bottom} = 9.9 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.003$$

$$s_q := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \cdot \tan(\phi_{fdu}) \right), 1 \right) \quad s_q = 1$$

$$s_\gamma := \text{if} \left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right) \quad s_\gamma = 1$$

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

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

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

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

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right)$$

$$d_q = 1.09$$

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

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

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 1.09$$

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

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

$$q_{nu_bottom} := Su_{fdu} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu_top} = 13706.5 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_bottom} := \phi_b \cdot q_{nu_top} \quad q_{Ru_bottom} = 8.9 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_bottom} = 9.9 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_bottom} = 8.9 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location about the toe "O" of base length (the pivot):

$$e := \left| \frac{L}{2} - x \right| \quad e = 2.98 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2]. The effective bearing width is equal to B-2e.

$$\sigma_v := \frac{\Sigma V}{B'_{top}} \quad \sigma_v = 7874 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Bearing Stress}$$

$$\sigma_{v_FPL} := \frac{\Sigma V_{FPL}}{B'_{bottom}} \quad \sigma_{v_FPL} = 7421.5 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Bearing Stress}$$

Bearing Resistance Capacity:Demand Ratio (CDR)

At the top of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_top} := \frac{q_{Rd_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_top} = 1.16$

Undrained Conditions: $CDR_{Bearing_u_top} := \frac{q_{Ru_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_top} = 1.13$

At the bottom of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_bottom} := \frac{q_{Rd_bottom}}{\sigma_{v_FPL}}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_bottom} = 1.34$

Undrained Conditions: $CDR_{Bearing_u_bottom} := \frac{q_{Ru_bottom}}{\sigma_{v_FPL}}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_bottom} = 1.20$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$e_{max} := \frac{L}{3}$

$e_{max} = 7.8 \text{ ft}$

Maximum Eccentricity **LRFD [C11.6.3.3.]**

$\Sigma M_R := MV_{Ia}$

$\Sigma M_R = 1236242.2 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$

Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$

$\Sigma M_O = 411765.3 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$

Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$

$\Sigma V = 105122.6 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$

$x = 7.8 \text{ ft}$

Distance from Point "O" the resultant intersects the base

$e := \left| \frac{L}{2} - x \right|$

$e = 3.92 \text{ ft}$

Wall eccentricity

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e}$

Is the CDR > or = to 1.0?

$CDR_{Eccentricity} = 2.00$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$

$R_u = 34579.5 \frac{\text{lb}}{\text{ft}}$

At the top of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$\Sigma V := V_{Ia}$

$\Sigma V = 105122.6 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$R_{td_top} := \Sigma V \cdot \tan(\phi_{Re})$

$R_{td_top} = 70906.1 \frac{\text{lb}}{\text{ft}}$

Nominal sliding resistance Drained Conditions

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$\phi_\tau := 1.0$

Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

Drained Conditions:

$\phi R_{n_d_top} := \phi_\tau \cdot R_{td_top}$

$R_{R_d_top} := \phi R_{n_d_top}$

$R_{R_d_top} = 70.9 \frac{\text{kip}}{\text{ft}}$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_top} := \frac{R_{R_d_top}}{R_u} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_d_top} = 2.05$$

At the bottom of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V_{FPL} := V_{Ia_FPL} \quad \Sigma V_{FPL} = 108310.2 \frac{lbf}{ft} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$R_{td_bottom} := \Sigma V_{FPL} \cdot \tan(\phi'_{fd}) \quad R_{td_bottom} = 50505.9 \frac{lbf}{ft} \quad \text{Nominal sliding resistance Drained Conditions}$$

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_r := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\text{Drained Conditions:} \quad \phi R_{n_d_bottom} := \phi_r \cdot R_{td_bottom} \quad R_{R_d_bottom} := \phi R_{n_d_bottom} \quad R_{R_d_bottom} = 50.5 \frac{kip}{ft}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_bottom} := \frac{R_{R_d_bottom}}{H_{Ia_FPL}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_d_bottom} = 1.38$$

Undrained Conditions (Total Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V_{FPL} := V_{Ia_FPL} \quad \Sigma V_{FPL} = 108310.2 \frac{lbf}{ft} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$B := L + 1 \text{ ft} \quad \text{Footing base width}$$

$$\sigma_{v_FPL} := \frac{\Sigma V_{FPL}}{B} \quad \sigma_{v_FPL} = 4417.2 \frac{lbf}{ft^2} \quad \text{Vertical stress ODOT GDM [1504]}$$

$$q_{FPL} := \frac{1}{2} \cdot \sigma_{v_FPL} \quad q_{FPL} = 2208.6 \frac{lbf}{ft^2} \quad \text{Uniform unit shear resistance as 1/2 vertical stress ODOT GDM [1504]}$$

Determine which Cohesive Soil Resistance Case is Present:

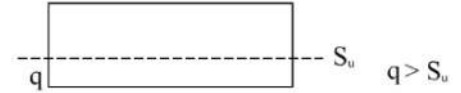
$$Case_1 := \text{if}(q_{FPL} > Su_{fdu}, 1, 0) \quad Case_1 = 0$$

$$Case_2 := \text{if}(Su_{fdu} \geq q_{FPL}, 1, 0) \quad Case_2 = 1$$

Unit Shear Resistance for Case 1:

$$R_{\tau_case1} := Su_{fdu} \cdot B = 63752 \frac{lb}{ft}$$

Case 1:



Unit Shear Resistance for Case 2:

$$R_{\tau_case2} := q_{FPL} \cdot B = 54155.1 \frac{lb}{ft}$$

Case 2:



Define the Applicable Case:

$$R_{\tau} := R_{\tau_case2}$$

$$R_{\tau} = 54155.1 \frac{lb}{ft}$$

Nominal sliding resistance Cohesive Soils

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_{ep} := 0.5$$

Resistance factor for passive resistance specified in
LRFD Table 10.5.5.2.2-1

$$\phi_{\tau} := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

Factored Sliding Resistance to be used in CDR Calculations:

$$R_R = 54155.12 \frac{lb}{ft}$$

Sliding Capacity:Demand Ratio (CDR)

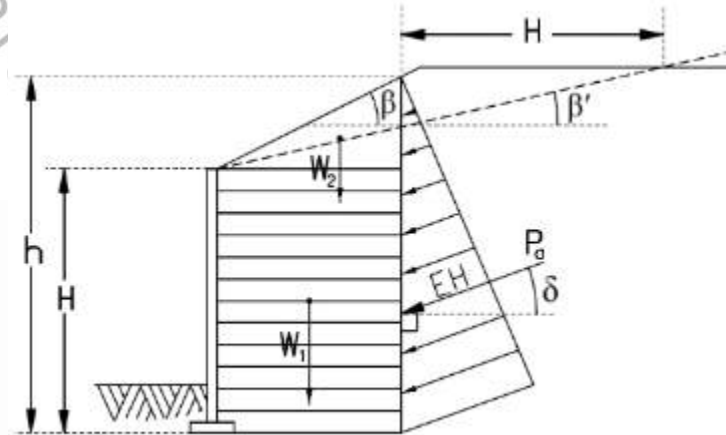
$$CDR_{Sliding_u_bottom} := \frac{R_R}{H_{Ia_FPL}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_u_bottom} = 1.48$$

Objective: To evaluate the external stability of MSE wall design with vertical wall face and broken backslope case.
Method: In accordance with ODOT Bridge Design Manual, 2026 [Sect. 307.4] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 3.11.5.8 and Sect. 11.10.5].

Assumptions:

- Horizontal backfill, Infinite backslope or Broken backslope behind MSE wall, see LRFD Sect. 3.11.5.8.1.
- For battered or vertical walls with a back face of wall angle of θ to horizontal.
- Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ .
- MSE wall not acting as abutment, if so must meet minimum embedment depth of $H/10$ if no slope in front of wall



Givens:

Wall Geometry:

$$H_e := 19.8 \text{ ft}$$

$$\theta := 90 \cdot \text{deg}$$

$$\beta_{\text{BackSlope}} := 0 \text{ deg}$$

$$\lambda := 0 \text{ ft}$$

Exposed wall height (Proposed profile grade to bottom of leveling pad)

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Inclination of ground slope behind face of wall. If it is horizontal backfill behind MSE wall, $\beta_{\text{BackSlope}} = 0$ deg

Horizontal distance from the back of MSE wall to the top of slope. If it is infinite slope behind MSE Wall, input λ larger than 2H; If it is horizontal backfill behind MSE wall, $\lambda = 0$ ft

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \text{ deg}$$

$$\gamma_r := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_r := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \text{ deg}$$

$$\gamma_b := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_b := 0 \frac{\text{lbf}}{\text{ft}^2}$$

$$\delta := \frac{2}{3} \cdot \phi'_b$$

$$\delta = 20 \text{ deg}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Soil Design Parameters for Bearing Resistance (Average 1.5 B Below Leveling Pad)

Drained Conditions (Effective Stress):

$$\phi'_{fd} := 25 \text{ deg}$$

Effective angle of internal friction

$$\gamma_{fd} := 112 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight

$$c'_{fd} := 200 \frac{\text{lb}_f}{\text{ft}^2}$$

Effective Cohesion

Undrained Conditions (Total Stress):

$$\phi_{fdu} := 0 \text{ deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$Su_{fdu} := 1750 \frac{\text{lb}_f}{\text{ft}^2}$$

Undrained Shear Strength

Undercut & Replacement Design Parameters:

$$\phi_{Re} := 34 \text{ deg}$$

Angle of internal friction for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$c_{Re} := 0 \frac{\text{lb}_f}{\text{ft}^2}$$

Cohesion for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$\gamma_{Re} := 130 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit Weight for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$\delta_{Re} := \frac{2}{3} \cdot \phi_{Re} \quad \delta_{Re} = 22.7 \text{ deg}$$

Friction angle between Replacement soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

$$D_{undercut} := 1 \text{ ft}$$

Depth of Undercut below bottom of footing

Foundation Surcharge Soil Parameters

$$\gamma_q := 120 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Depth of Embedment Check:

$$d_{frost} := 3.25 \text{ ft}$$

$$d_{user} := 3.5 \text{ ft}$$

Local Frost Depth / User Input

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 3.5 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 23.3 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1.2 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 3.5 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 23.3 \text{ ft}$$

Initial design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 0.7 \cdot H$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max \left(8 \cdot \text{ft}, 0.7 \cdot H, L_{user} \right)$$

$$L = 16.3 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L < \lambda, 0 \frac{\text{lb}}{\text{ft}^2}, 250 \frac{\text{lb}}{\text{ft}^2} \right) = 250 \frac{\text{lb}}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

$$SUR_{@MSE} := \text{if} \left(L < \lambda, 0 \frac{\text{lb}}{\text{ft}^2}, 250 \frac{\text{lb}}{\text{ft}^2} \right) = 250 \frac{\text{lb}}{\text{ft}^2}$$

Live load surcharge above the MSE Wall soil reinforcement (per LRFD BDS [3.11.6.4])

Calculations:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan \left(\beta_{BackSlope} \right)$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h := \text{if} \left(L < \lambda, H + L \cdot \tan \left(\beta_{BackSlope} \right), H + H_{BackSlope} \right)$$

Height of retained fill at the back of the reinforced soil

$$h = 23.3 \text{ ft}$$

$$I := \text{atan} \left(\frac{H_{BackSlope}}{2 \cdot H} \right)$$

$$\beta := \text{if} \left(\lambda < 2 \cdot H, I, \beta_{BackSlope} \right)$$

Angle of friction between retained backfill and reinforced soil

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 2.6845$$

$$k_{ab} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right)$$

$$k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot h^2 \cdot k_{ab}$$

$$F_T = 9684.5 \frac{\text{lb}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{ab}$$

$$F_{SUR} = 1731.9 \frac{\text{lb}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 45602.8 \frac{\text{lb}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_2 = 0 \frac{\text{lb}}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_3 := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lb}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_3 = 0 \frac{\text{lb}}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_4 := F_T \cdot \sin(\beta)$$

$$V_4 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_5 := F_{SUR} \cdot \sin(\beta)$$

$$V_5 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_6 := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_6 = 4077.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1} := \frac{L}{2}$$

$$d_{v1} = 8.2 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 371890.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := \text{if}\left(L < \lambda, \frac{2L}{3}, \frac{2 \cdot \lambda}{3}\right)$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v3} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v3} = 8.2 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3}$$

$$MV_3 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v4} := L$$

$$d_{v4} = 16.3 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4}$$

$$MV_4 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v5} := L$$

$$d_{v5} = 16.3 \text{ ft}$$

$$MV_5 := V_5 \cdot d_{v5}$$

$$MV_5 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v6} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v6} = 8.2 \text{ ft}$$

$$MV_6 := V_6 \cdot d_{v6}$$

$$MV_6 = 33252 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T \cdot \cos(\beta) = 9684.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} \cdot \cos(\beta) = 1731.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

Moment:

$$d_{h1} := \frac{h}{3}$$

$$d_{h1} = 7.8 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 75216.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{2}$$

$$d_{h2} = 11.7 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 20176.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1 + V_2 + V_3$$

$$V_{EV} = 45602.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{EH} := V_4$$

$$V_{EH} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{LS} := V_5$$

$$V_{LS} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{LS@MSE} := V_6$$

$$V_{LS@MSE} = 4077.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 9684.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 1731.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1 + MV_2 + MV_3$$

$$M_{EV} = 371890.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_4$$

$$M_{EH1} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1} := MV_5$$

$$M_{LS1@MSE} := MV_6$$

$$M_{EH2} := MH_1$$

$$M_{LS2} := MH_2$$

$$M_{LS1} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE} = 33252 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} = 75216.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} = 20176.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1 \quad \text{LRFD Load Modifier}$$

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot ((Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE}))$$

$$V_{Ia} = 52738.4 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE}))$$

$$V_{Ib} = 68699.4 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{EH} \cdot H_{EH}) + (Ia_{LS} \cdot H_{LS}))$$

$$H_{Ia} = 17557.5 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{EH} \cdot H_{EH}) + (Ib_{LS} \cdot H_{LS}))$$

$$H_{Ib} = 17557.5 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot ((Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + Ia_{LS} \cdot (M_{LS1} + M_{LS1@MSE}))$$

$$MV_{Ia} = 430081.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + Ib_{LS} \cdot (M_{LS1} + M_{LS1@MSE}))$$

$$MV_{Ib} = 560243.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{EH} \cdot M_{EH2}) + (Ia_{LS} \cdot M_{LS2}))$$

$$MH_{Ia} = 148132.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{EH} \cdot M_{EH2}) + (Ib_{LS} \cdot M_{LS2}))$$

$$MH_{Ib} = 148132.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Calculations at Bottom of Foundation Preparation Layer:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan(\beta_{BackSlope})$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h_{FPL} := \text{if}(L < \lambda, H + L \cdot \tan(\beta_{BackSlope}) + 1 \text{ ft}, H + H_{BackSlope} + 1 \text{ ft})$$

$$h_{FPL} = 24.3 \text{ ft}$$

Height of retained fill at the back of the reinforced soil

$$I := \text{atan}\left(\frac{H_{BackSlope}}{2 \cdot H}\right)$$

$$\beta := \text{if}(\lambda < 2 \cdot H, I, \beta_{BackSlope})$$

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

Angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 2.6845$$

$$k_{ab} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{ab} = 0.2973 \quad \text{Active Earth Pressure Coefficient}$$

$$F_{T_FPL} := \frac{1}{2} \cdot \gamma_b \cdot h_{FPL}^2 \cdot k_{ab} \quad F_T = 9684.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Active Earth Force Resultant (EH)}$$

$$F_{SUR_FPL} := SUR \cdot h_{FPL} \cdot k_{ab} \quad F_{SUR} = 1731.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Live Load Surcharge (LS)}$$

Vertical Loads:

$$V_{1_FPL} := \gamma_r \cdot H \cdot L \quad V_{1_FPL} = 45602.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Soil backfill - reinforced soil (EV)}$$

$$V_{2_FPL} := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right) \quad V_{2_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Triangular Soil backfill - backslope soil (EV)}$$

$$V_{3_FPL} := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right) \quad V_{3_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Rectangular Soil backfill - backslope soil (EV)}$$

$$V_{4_FPL} := F_{T_FPL} \cdot \sin(\beta) \quad V_{4_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Active earth force resultant (vertical component - EH)}$$

$$V_{5_FPL} := F_{SUR_FPL} \cdot \sin(\beta) \quad V_{5_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Live Load Surcharge (vertical component - LS)}$$

$$V_{6_FPL} := SUR_{@MSE} \cdot (L - \lambda) \quad V_{6_FPL} = 4077.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Live Load Surcharge above MSE Wall soil reinforcement}$$

$$V_{7_FPL} := \gamma_{Re} \cdot (L + 1 \text{ ft}) \cdot D_{undercut} \quad V_{7_FPL} = 2250.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Foundation Preparation Layer Weight}$$

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1_FPL} := \frac{L}{2} + 1 \text{ ft} \quad d_{v1_FPL} = 9.2 \text{ ft} \quad MV_{1_FPL} := V_{1_FPL} \cdot d_{v1_FPL} \quad MV_{1_FPL} = 417493.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2_FPL} := \text{if} \left(L < \lambda, \frac{2L}{3}, \frac{2\lambda}{3} \right) + 1 \text{ ft} \quad d_{v2_FPL} = 1 \text{ ft} \quad MV_{2_FPL} := V_{2_FPL} \cdot d_{v2_FPL} \quad MV_{2_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v3_FPL} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right) + 1 \text{ ft} \quad d_{v3_FPL} = 9.2 \text{ ft} \quad MV_{3_FPL} := V_{3_FPL} \cdot d_{v3_FPL} \quad MV_{3_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v4_FPL} := L + 1 \text{ ft} \quad d_{v4_FPL} = 17.3 \text{ ft} \quad MV_{4_FPL} := V_{4_FPL} \cdot d_{v4_FPL} \quad MV_{4_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v5_FPL} := L + 1 \text{ ft} \quad d_{v5_FPL} = 17.3 \text{ ft} \quad MV_{5_FPL} := V_{5_FPL} \cdot d_{v5_FPL} \quad MV_{5_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v6_FPL} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right) + 1 \text{ ft} \quad d_{v6_FPL} = 9.2 \text{ ft} \quad MV_{6_FPL} := V_{6_FPL} \cdot d_{v6_FPL} \quad MV_{6_FPL} = 37329.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v7_FPL} := \frac{(L + 1 \text{ ft})}{2} \quad d_{v7_FPL} = 8.7 \text{ ft} \quad MV_{7_FPL} := V_{7_FPL} \cdot d_{v7_FPL} \quad MV_{7_FPL} = 19476.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_{1_FPL} := F_{T_FPL} \cdot \cos(\beta) = 10533.7 \frac{\text{lb}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_{2_FPL} := F_{SUR_FPL} \cdot \cos(\beta) = 1806.2 \frac{\text{lb}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1_FPL} := \frac{h_{FPL}}{3} \quad d_{h1_FPL} = 8.1 \text{ ft}$$

Moment:

$$MH_{1_FPL} := H_{1_FPL} \cdot d_{h1_FPL} \quad MH_{1_FPL} = 85322.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2_FPL} := \frac{h_{FPL}}{2} \quad d_{h2_FPL} = 12.2 \text{ ft}$$

$$MH_{2_FPL} := H_{2_FPL} \cdot d_{h2_FPL} \quad MH_{2_FPL} = 21945.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV_FPL} := V_{1_FPL} + V_{2_FPL} + V_{3_FPL} + V_{7_FPL}$$

$$V_{EV_FPL} = 47853.1 \frac{\text{lb}}{\text{ft}}$$

$$V_{EH_FPL} := V_{4_FPL}$$

$$V_{EH_FPL} = 0 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS_FPL} := V_{5_FPL}$$

$$V_{LS_FPL} = 0 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS@MSE_FPL} := V_{6_FPL}$$

$$V_{LS@MSE_FPL} = 4077.5 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH_FPL} := H_{1_FPL}$$

$$H_{EH_FPL} = 10533.7 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS_FPL} := H_{2_FPL}$$

$$H_{LS_FPL} = 1806.2 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV_FPL} := MV_{1_FPL} + MV_{2_FPL} + MV_{3_FPL} + MV_{7_FPL}$$

$$M_{EV_FPL} = 436969.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1_FPL} := MV_{4_FPL}$$

$$M_{EH1_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1_FPL} := MV_{5_FPL}$$

$$M_{LS1_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE_FPL} := MV_{6_FPL}$$

$$M_{LS1@MSE_FPL} = 37329.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2_FPL} := MH_{1_FPL}$$

$$M_{EH2_FPL} = 85322.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2_FPL} := MH_{2_FPL}$$

$$M_{LS2_FPL} = 21945.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1$$

LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia_FPL} := \eta \cdot ((Ia_{EV} \cdot V_{EV_FPL}) + (Ia_{EH} \cdot V_{EH_FPL}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE_FPL})) \quad V_{Ia_FPL} = 54988.7 \frac{lb_f}{ft}$$

$$V_{Ib_FPL} := \eta \cdot ((Ib_{EV} \cdot V_{EV_FPL}) + (Ib_{EH} \cdot V_{EH_FPL}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE_FPL})) \quad V_{Ib_FPL} = 71737.3 \frac{lb_f}{ft}$$

Compute Bearing Resistance:

$$H_{Ia_FPL} := \eta \cdot ((Ia_{EH} \cdot H_{EH_FPL}) + (Ia_{LS} \cdot H_{LS_FPL})) \quad H_{Ia_FPL} = 18961.3 \frac{lb_f}{ft}$$

$$H_{Ib_FPL} := \eta \cdot ((Ib_{EH} \cdot H_{EH_FPL}) + (Ib_{LS} \cdot H_{LS_FPL})) \quad H_{Ib_FPL} = 18961.3 \frac{lb_f}{ft}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia_FPL} := \eta \cdot ((Ia_{EV} \cdot M_{EV_FPL}) + (Ia_{EH} \cdot M_{EH1_FPL}) + Ia_{LS} \cdot (M_{LS1_FPL} + M_{LS1@MSE_FPL})) \quad MV_{Ia_FPL} = 502296.3 \frac{lb_f \cdot ft}{ft}$$

$$MV_{Ib_FPL} := \eta \cdot ((Ib_{EV} \cdot M_{EV_FPL}) + (Ib_{EH} \cdot M_{EH1_FPL}) + Ib_{LS} \cdot (M_{LS1_FPL} + M_{LS1@MSE_FPL})) \quad MV_{Ib_FPL} = 655235.6 \frac{lb_f \cdot ft}{ft}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia_FPL} := \eta \cdot ((Ia_{EH} \cdot M_{EH2_FPL}) + (Ia_{LS} \cdot M_{LS2_FPL})) \quad MH_{Ia_FPL} = 166387.8 \frac{lb_f \cdot ft}{ft}$$

$e = 2.72$

$$MH_{Ib_FPL} := \eta \cdot ((Ib_{EH} \cdot M_{EH2_FPL}) + (Ib_{LS} \cdot M_{LS2_FPL})) \quad MH_{Ib_FPL} = 166387.8 \frac{lb_f \cdot ft}{ft}$$

Factored Horizontal Loads by Limit State:

Compute the Effective Bearing Length (Strength Ib) about the Toe "O" of base length (the pivot):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 560243.2 \frac{lb_f \cdot ft}{ft} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 148132.9 \frac{lb_f \cdot ft}{ft} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 68699.4 \frac{lb_f}{ft} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V} \quad x = 6 \text{ ft} \quad \text{Distance from Point "O" the resultant intersects the base}$$

$$e := \left| \frac{L}{2} - x \right| \quad e = 2.2 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B'_{top} := L - 2 \cdot e \quad B'_{top} = 12 \text{ ft} \quad \text{Effective Footing Width, at the top of Foundation Preparation Layer}$$

$$\Sigma M_{R_FPL} := MV_{lb_FPL}$$

$$\Sigma M_{R_FPL} = 655235.6 \frac{lb \cdot ft}{ft}$$

Sum of Resisting Moments (Strength lb)

$$\Sigma M_{O_FPL} := MH_{lb_FPL}$$

$$\Sigma M_{O_FPL} = 166387.8 \frac{lb \cdot ft}{ft}$$

Sum of Overturning Moments (Strength lb)

$$\Sigma V_{FPL} := V_{lb_FPL}$$

$$\Sigma V_{FPL} = 71737.3 \frac{lb}{ft}$$

Sum of Vertical Loads (Strength lb)

$$x_{FPL} := \frac{(\Sigma M_{R_FPL} - \Sigma M_{O_FPL})}{\Sigma V_{FPL}}$$

$$x_{FPL} = 6.8 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e_{FPL} := \left| \frac{(L + 1 \text{ ft})}{2} - x_{FPL} \right|$$

$$e_{FPL} = 1.8 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B'_{bottom} := (L + 1 \text{ ft}) - 2 \cdot e_{FPL}$$

$$B'_{bottom} = 13.6 \text{ ft}$$

Effective Footing Width, at the bottom of Foundation Preparation Layer (FPL)

Foundation Layout:

$$L_{Wall} := 1423.79 \text{ ft}$$

Assumed Footing Length (Wall Section Length)

$$D_f := d_e$$

$$D_f = 3.5 \text{ ft}$$

Footing embedment

$$D_F := D_f + D_{undercut}$$

Embenment Depth at bottom of Undercut

$$d_w := D_f - 1 \text{ ft}$$

Depth of Groundwater below ground surface in front of wall

Compute Bearing Resistance at the top of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 10.66$$

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

$$N_c = 20.72$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 10.9$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.004$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \cdot \tan(\phi'_{fd}) \right), 1 \right)$$

$$s_q = 1.004$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.997$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.11$$

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

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

$$N_{cm} = 20.81$$

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

$$N_{qm} = 11.898$$

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

$$N_{\gamma m} = 10.84$$

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

$$q_{nd_top} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad \text{at top of foundation preparation layer}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_top} := \phi_b \cdot q_{nd_top} \quad q_{Rd_top} = 6.7 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

$$s_c := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.002$$

$$s_q := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \tan(\phi_{fdu}), 1\right)$$

$$s_q = 1$$

$$s_\gamma := \text{if}\left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}}\right), 1\right)$$

$$s_\gamma = 1$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \tan\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.11$$

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

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

$$N_{cm} = 5.149$$

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

$$N_{qm} = 1.112$$

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

$$N_{\gamma m} = 0$$

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

$$q_{nu_top} := Su_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nu_top} = 9243.6 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_top} := \phi_b \cdot q_{nu_top} \quad q_{Ru_top} = 6 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_top} = 6.7 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_top} = 6 \text{ ksf}$$

Compute Bearing Resistance at the bottom of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 10.66$$

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

$$N_c = 20.72$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 10.9$$

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

$$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.005$$

$$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \tan(\phi'_{fd}), 1\right)$$

$$s_q = 1.004$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.996$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 20.823$$

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

$$N_{qm} = 11.772$$

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

$$N_{\gamma m} = 10.835$$

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

$$q_{nd_bottom} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

at bottom of foundation preparation layer

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_bottom} := \phi_b \cdot q_{nd_bottom} \quad q_{Rd_bottom} = 7.5 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.002$$

$$s_q := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \cdot \tan(\phi_{fdu}) \right), 1 \right) \quad s_q = 1$$

$$s_\gamma := \text{if} \left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right) \quad s_\gamma = 1$$

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

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

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

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

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right) \quad e$$

$$d_q = 1.1$$

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

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

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 1.099$$

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

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

$$q_{nu_bottom} := Su_{fdu} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu_top} = 9243.6 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_bottom} := \phi_b \cdot q_{nu_top} \quad q_{Ru_bottom} = 6 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_bottom} = 7.5 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_bottom} = 6 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location about the toe "O" of base length (the pivot):

$$e := \left| \frac{L}{2} - x \right| \quad e = 2.16 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2]. The effective bearing width is equal to B-2e.

$$\sigma_v := \frac{\Sigma V}{B'_{top}} \quad \sigma_v = 5726.1 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Bearing Stress}$$

$$\sigma_{v_FPL} := \frac{\Sigma V_{FPL}}{B'_{bottom}} \quad \sigma_{v_FPL} = 5263.6 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Bearing Stress}$$

Bearing Resistance Capacity:Demand Ratio (CDR)

At the top of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_top} := \frac{q_{Rd_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_top} = 1.17$

Undrained Conditions: $CDR_{Bearing_u_top} := \frac{q_{Ru_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_top} = 1.05$

At the bottom of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_bottom} := \frac{q_{Rd_bottom}}{\sigma_{v_FPL}}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_bottom} = 1.42$

Undrained Conditions: $CDR_{Bearing_u_bottom} := \frac{q_{Ru_bottom}}{\sigma_{v_FPL}}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_bottom} = 1.14$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$e_{max} := \frac{L}{3}$

$e_{max} = 5.4 \text{ ft}$

Maximum Eccentricity **LRFD [C11.6.3.3.]**

$\Sigma M_R := MV_{Ia}$

$\Sigma M_R = 430081.5 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$

Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$

$\Sigma M_O = 148132.9 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$

Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$

$\Sigma V = 52738.4 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$

$x = 5.3 \text{ ft}$

Distance from Point "O" the resultant intersects the base

$e := \left| \frac{L}{2} - x \right|$

$e = 2.81 \text{ ft}$

Wall eccentricity

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e}$

Is the CDR > or = to 1.0?

$CDR_{Eccentricity} = 1.94$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$

$R_u = 17557.5 \frac{\text{lb}}{\text{ft}}$

At the top of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$\Sigma V := V_{Ia}$

$\Sigma V = 52738.4 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$R_{td_top} := \Sigma V \cdot \tan(\phi_{Re})$

$R_{td_top} = 35572.5 \frac{\text{lb}}{\text{ft}}$

Nominal sliding resistance Drained Conditions

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$\phi_\tau := 1.0$

Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

Drained Conditions:

$\phi R_{n_d_top} := \phi_\tau \cdot R_{td_top}$

$R_{R_d_top} := \phi R_{n_d_top}$

$R_{R_d_top} = 35.6 \frac{\text{kip}}{\text{ft}}$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_top} := \frac{R_{R_d_top}}{R_u} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_d_top} = 2.03$$

At the bottom of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V_{FPL} := V_{Ia_FPL} \quad \Sigma V_{FPL} = 54988.7 \frac{lb}{ft} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$R_{td_bottom} := \Sigma V_{FPL} \cdot \tan(\phi'_{fd}) \quad R_{td_bottom} = 25641.6 \frac{lb}{ft} \quad \text{Nominal sliding resistance Drained Conditions}$$

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_r := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\text{Drained Conditions:} \quad \phi R_{n_d_bottom} := \phi_r \cdot R_{td_bottom} \quad R_{R_d_bottom} := \phi R_{n_d_bottom} \quad R_{R_d_bottom} = 25.6 \frac{kip}{ft}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_bottom} := \frac{R_{R_d_bottom}}{H_{Ia_FPL}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_d_bottom} = 1.35$$

Undrained Conditions (Total Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V_{FPL} := V_{Ia_FPL} \quad \Sigma V_{FPL} = 54988.7 \frac{lb}{ft} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$B := L + 1 \text{ ft} \quad \text{Footing base width}$$

$$\sigma_{v_FPL} := \frac{\Sigma V_{FPL}}{B} \quad \sigma_{v_FPL} = 3176.7 \frac{lb}{ft^2} \quad \text{Vertical stress ODOT GDM [1504]}$$

$$q_{FPL} := \frac{1}{2} \cdot \sigma_{v_FPL} \quad q_{FPL} = 1588.4 \frac{lb}{ft^2} \quad \text{Uniform unit shear resistance as 1/2 vertical stress ODOT GDM [1504]}$$

Determine which Cohesive Soil Resistance Case is Present:

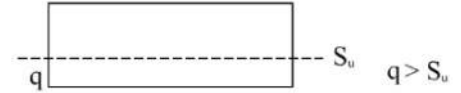
$$Case_1 := \text{if}(q_{FPL} > Su_{fdu}, 1, 0) \quad Case_1 = 0$$

$$Case_2 := \text{if}(Su_{fdu} \geq q_{FPL}, 1, 0) \quad Case_2 = 1$$

Unit Shear Resistance for Case 1:

$$R_{\tau_case1} := Su_{fdu} \cdot B = 30292.5 \frac{lb}{ft}$$

Case 1:



Unit Shear Resistance for Case 2:

$$R_{\tau_case2} := q_{FPL} \cdot B = 27494.3 \frac{lb}{ft}$$

Case 2:



Define the Applicable Case:

$$R_{\tau} := R_{\tau_case2}$$

$$R_{\tau} = 27494.3 \frac{lb}{ft}$$

Nominal sliding resistance Cohesive Soils

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_{ep} := 0.5$$

Resistance factor for passive resistance specified in
LRFD Table 10.5.5.2.2-1

$$\phi_{\tau} := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

Factored Sliding Resistance to be used in CDR Calculations:

$$R_R = 27494.343 \frac{lb}{ft}$$

Sliding Capacity:Demand Ratio (CDR)

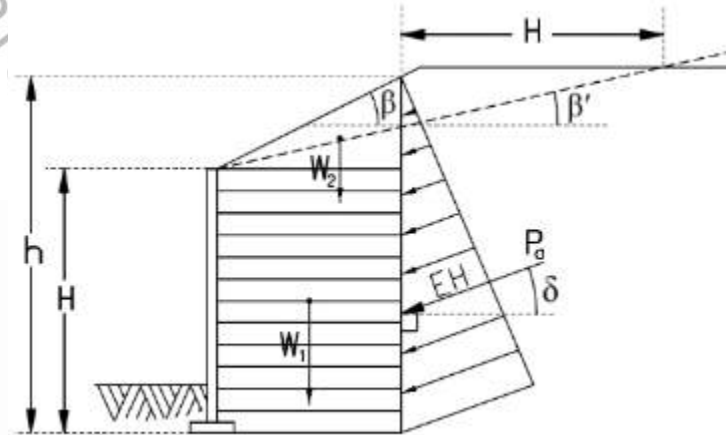
$$CDR_{Sliding_u_bottom} := \frac{R_R}{H_{Ia_FPL}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_u_bottom} = 1.45$$

Objective: To evaluate the external stability of MSE wall design with vertical wall face and broken backslope case.
Method: In accordance with ODOT Bridge Design Manual, 2026 [Sect. 307.4] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 3.11.5.8 and Sect. 11.10.5].

Assumptions:

- Horizontal backfill, Infinite backslope or Broken backslope behind MSE wall, see LRFD Sect. 3.11.5.8.1.
- For battered or vertical walls with a back face of wall angle of θ to horizontal.
- Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ .
- MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall



Givens:

Wall Geometry:

$$H_e := 21.9 \text{ ft}$$

$$\theta := 90 \cdot \text{deg}$$

$$\beta_{\text{BackSlope}} := 0 \text{ deg}$$

$$\lambda := 0 \text{ ft}$$

Exposed wall height (Proposed profile grade to bottom of leveling pad)

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Inclination of ground slope behind face of wall. If it is horizontal backfill behind MSE wall, $\beta_{\text{BackSlope}} = 0$ deg

Horizontal distance from the back of MSE wall to the top of slope. If it is infinite slope behind MSE Wall, input λ larger than 2H; If it is horizontal backfill behind MSE wall, $\lambda = 0$ ft

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \text{ deg}$$

$$\gamma_r := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_r := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \text{ deg}$$

$$\gamma_b := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_b := 0 \frac{\text{lbf}}{\text{ft}^2}$$

$$\delta := \frac{2}{3} \cdot \phi'_b$$

$$\delta = 20 \text{ deg}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Soil Design Parameters for Bearing Resistance (Average 1.5 B Below Leveling Pad)

Drained Conditions (Effective Stress):

$$\phi'_{fd} := 25 \text{ deg}$$

Effective angle of internal friction

$$\gamma_{fd} := 115 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} := 250 \frac{\text{lb}}{\text{ft}^2}$$

Effective Cohesion

Undrained Conditions (Total Stress):

$$\phi_{fdu} := 0 \text{ deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$Su_{fdu} := 2550 \frac{\text{lb}}{\text{ft}^2}$$

Undrained Shear Strength

Undercut & Replacement Design Parameters:

$$\phi_{Re} := 34 \text{ deg}$$

Angle of internal friction for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$c_{Re} := 0 \frac{\text{lb}}{\text{ft}^2}$$

Cohesion for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$\gamma_{Re} := 130 \frac{\text{lb}}{\text{ft}^3}$$

Unit Weight for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$\delta_{Re} := \frac{2}{3} \cdot \phi_{Re} \quad \delta_{Re} = 22.7 \text{ deg}$$

Friction angle between Replacement soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

$$D_{undercut} := 1 \text{ ft}$$

Depth of Undercut below bottom of footing

Foundation Surcharge Soil Parameters

$$\gamma_q := 120 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Depth of Embedment Check:

$$d_{frost} := 3.25 \text{ ft}$$

$$d_{user} := 5.4 \text{ ft}$$

Local Frost Depth / User Input

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 5.4 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 27.3 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1.4 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 5.4 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 27.3 \text{ ft}$$

Initial design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 0.7 \cdot H$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max \left(8 \cdot \text{ft}, 0.7 \cdot H, L_{user} \right)$$

$$L = 19.1 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L < \lambda, 0 \frac{\text{lbf}}{\text{ft}^2}, 250 \frac{\text{lbf}}{\text{ft}^2} \right) = 250 \frac{\text{lbf}}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

$$SUR_{@MSE} := \text{if} \left(L < \lambda, 0 \frac{\text{lbf}}{\text{ft}^2}, 250 \frac{\text{lbf}}{\text{ft}^2} \right) = 250 \frac{\text{lbf}}{\text{ft}^2}$$

Live load surcharge above the MSE Wall soil reinforcement (per LRFD BDS [3.11.6.4])

Calculations:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan \left(\beta_{BackSlope} \right)$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h := \text{if} \left(L < \lambda, H + L \cdot \tan \left(\beta_{BackSlope} \right), H + H_{BackSlope} \right)$$

Height of retained fill at the back of the reinforced soil

$$h = 27.3 \text{ ft}$$

$$I := \text{atan} \left(\frac{H_{BackSlope}}{2 \cdot H} \right)$$

$$\beta := \text{if} \left(\lambda < 2 \cdot H, I, \beta_{BackSlope} \right)$$

Angle of friction between retained backfill and reinforced soil

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

$$\Gamma := \left(1 + \sqrt{\frac{\left(\sin \left(\phi'_b + \delta \right) \cdot \sin \left(\phi'_b - \beta \right) \right)}{\left(\sin \left(\theta - \delta \right) \cdot \sin \left(\theta + \beta \right) \right)}} \right)^2 = 2.6845$$

$$k_{ab} := \left(\frac{\left(\sin \left(\theta + \phi'_b \right) \right)^2}{\left(\Gamma \cdot \left(\sin \left(\theta \right) \right)^2 \cdot \sin \left(\theta - \delta \right) \right)} \right)$$

$$k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot h^2 \cdot k_{ab}$$

$$F_T = 13295.1 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{ab}$$

$$F_{SUR} = 2029.2 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 62604.4 \frac{\text{lbf}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_2 = 0 \frac{\text{lbf}}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_3 := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lbf}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_3 = 0 \frac{\text{lbf}}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_4 := F_T \cdot \sin(\beta)$$

$$V_4 = 0 \frac{\text{lbf}}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_5 := F_{SUR} \cdot \sin(\beta)$$

$$V_5 = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_6 := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_6 = 4777.5 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1} := \frac{L}{2}$$

$$d_{v1} = 9.6 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 598184.7 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := \text{if}\left(L < \lambda, \frac{2L}{3}, \frac{2 \cdot \lambda}{3}\right)$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v3} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v3} = 9.6 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3}$$

$$MV_3 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v4} := L$$

$$d_{v4} = 19.1 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4}$$

$$MV_4 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v5} := L$$

$$d_{v5} = 19.1 \text{ ft}$$

$$MV_5 := V_5 \cdot d_{v5}$$

$$MV_5 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v6} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v6} = 9.6 \text{ ft}$$

$$MV_6 := V_6 \cdot d_{v6}$$

$$MV_6 = 45649 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T \cdot \cos(\beta) = 13295.1 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} \cdot \cos(\beta) = 2029.2 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

Moment:

$$d_{h1} := \frac{h}{3}$$

$$d_{h1} = 9.1 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 120985.4 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{2}$$

$$d_{h2} = 13.7 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 27698.1 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1 + V_2 + V_3$$

$$V_{EV} = 62604.4 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_4$$

$$V_{EH} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS} := V_5$$

$$V_{LS} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS@MSE} := V_6$$

$$V_{LS@MSE} = 4777.5 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 13295.1 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 2029.2 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1 + MV_2 + MV_3$$

$$M_{EV} = 598184.7 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_4$$

$$M_{EH1} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1} := MV_5$$

$$M_{LS1@MSE} := MV_6$$

$$M_{EH2} := MH_1$$

$$M_{LS2} := MH_2$$

$$M_{LS1} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE} = 45649 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} = 120985.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} = 27698.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1 \quad \text{LRFD Load Modifier}$$

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot \left((Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE}) \right)$$

$$V_{Ia} = 70965 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot \left((Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE}) \right)$$

$$V_{Ib} = 92876.5 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot \left((Ia_{EH} \cdot H_{EH}) + (Ia_{LS} \cdot H_{LS}) \right)$$

$$H_{Ia} = 23493.7 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot \left((Ib_{EH} \cdot H_{EH}) + (Ib_{LS} \cdot H_{LS}) \right)$$

$$H_{Ib} = 23493.7 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot \left((Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + Ia_{LS} \cdot (M_{LS1} + M_{LS1@MSE}) \right)$$

$$MV_{Ia} = 678070.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot \left((Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + Ib_{LS} \cdot (M_{LS1} + M_{LS1@MSE}) \right)$$

$$MV_{Ib} = 887435.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot \left((Ia_{EH} \cdot M_{EH2}) + (Ia_{LS} \cdot M_{LS2}) \right)$$

$$MH_{Ia} = 229949.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot \left((Ib_{EH} \cdot M_{EH2}) + (Ib_{LS} \cdot M_{LS2}) \right)$$

$$MH_{Ib} = 229949.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Calculations at Bottom of Foundation Preparation Layer:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan(\beta_{BackSlope})$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h_{FPL} := \text{if}(L < \lambda, H + L \cdot \tan(\beta_{BackSlope}) + 1 \text{ ft}, H + H_{BackSlope} + 1 \text{ ft})$$

$$h_{FPL} = 28.3 \text{ ft}$$

Height of retained fill at the back of the reinforced soil

$$I := \text{atan}\left(\frac{H_{BackSlope}}{2 \cdot H}\right)$$

$$\beta := \text{if}(\lambda < 2 \cdot H, I, \beta_{BackSlope})$$

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

Angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 2.6845$$

$$k_{ab} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_{T_FPL} := \frac{1}{2} \cdot \gamma_b \cdot h_{FPL}^2 \cdot k_{ab} \quad F_T = 13295.1 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR_FPL} := SUR \cdot h_{FPL} \cdot k_{ab} \quad F_{SUR} = 2029.2 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_{1_FPL} := \gamma_r \cdot H \cdot L \quad V_{1_FPL} = 62604.4 \frac{\text{lbf}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_{2_FPL} := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_{2_FPL} = 0 \frac{\text{lbf}}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_{3_FPL} := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lbf}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_{3_FPL} = 0 \frac{\text{lbf}}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_{4_FPL} := F_{T_FPL} \cdot \sin(\beta)$$

$$V_{4_FPL} = 0 \frac{\text{lbf}}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_{5_FPL} := F_{SUR_FPL} \cdot \sin(\beta)$$

$$V_{5_FPL} = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_{6_FPL} := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_{6_FPL} = 4777.5 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

$$V_{7_FPL} := \gamma_{Re} \cdot (L + 1 \text{ ft}) \cdot D_{undercut}$$

$$V_{7_FPL} = 2614.3 \frac{\text{lbf}}{\text{ft}}$$

Foundation Preparation Layer Weight

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1_FPL} := \frac{L}{2} + 1 \text{ ft}$$

$$d_{v1_FPL} = 10.6 \text{ ft}$$

$$MV_{1_FPL} := V_{1_FPL} \cdot d_{v1_FPL} \quad MV_{1_FPL} = 660789 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2_FPL} := \text{if} \left(L < \lambda, \frac{2L}{3}, \frac{2\lambda}{3} \right) + 1 \text{ ft}$$

$$d_{v2_FPL} = 1 \text{ ft}$$

$$MV_{2_FPL} := V_{2_FPL} \cdot d_{v2_FPL} \quad MV_{2_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v3_FPL} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right) + 1 \text{ ft} \quad d_{v3_FPL} = 10.6 \text{ ft}$$

$$MV_{3_FPL} := V_{3_FPL} \cdot d_{v3_FPL} \quad MV_{3_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v4_FPL} := L + 1 \text{ ft}$$

$$d_{v4_FPL} = 20.1 \text{ ft}$$

$$MV_{4_FPL} := V_{4_FPL} \cdot d_{v4_FPL} \quad MV_{4_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v5_FPL} := L + 1 \text{ ft}$$

$$d_{v5_FPL} = 20.1 \text{ ft}$$

$$MV_{5_FPL} := V_{5_FPL} \cdot d_{v5_FPL} \quad MV_{5_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v6_FPL} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right) + 1 \text{ ft} \quad d_{v6_FPL} = 10.6 \text{ ft}$$

$$MV_{6_FPL} := V_{6_FPL} \cdot d_{v6_FPL} \quad MV_{6_FPL} = 50426.5 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v7_FPL} := \frac{(L + 1 \text{ ft})}{2}$$

$$d_{v7_FPL} = 10.1 \text{ ft}$$

$$MV_{7_FPL} := V_{7_FPL} \cdot d_{v7_FPL} \quad MV_{7_FPL} = 26286.8 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_{1_FPL} := F_{T_FPL} \cdot \cos(\beta) = 14286.9 \frac{\text{lb}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_{2_FPL} := F_{SUR_FPL} \cdot \cos(\beta) = 2103.5 \frac{\text{lb}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1_FPL} := \frac{h_{FPL}}{3} \quad d_{h1_FPL} = 9.4 \text{ ft}$$

Moment:

$$MH_{1_FPL} := H_{1_FPL} \cdot d_{h1_FPL} \quad MH_{1_FPL} = 134773.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2_FPL} := \frac{h_{FPL}}{2} \quad d_{h2_FPL} = 14.2 \text{ ft}$$

$$MH_{2_FPL} := H_{2_FPL} \cdot d_{h2_FPL} \quad MH_{2_FPL} = 29764.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV_FPL} := V_{1_FPL} + V_{2_FPL} + V_{3_FPL} + V_{7_FPL}$$

$$V_{EV_FPL} = 65218.7 \frac{\text{lb}}{\text{ft}}$$

$$V_{EH_FPL} := V_{4_FPL}$$

$$V_{EH_FPL} = 0 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS_FPL} := V_{5_FPL}$$

$$V_{LS_FPL} = 0 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS@MSE_FPL} := V_{6_FPL}$$

$$V_{LS@MSE_FPL} = 4777.5 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH_FPL} := H_{1_FPL}$$

$$H_{EH_FPL} = 14286.9 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS_FPL} := H_{2_FPL}$$

$$H_{LS_FPL} = 2103.5 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV_FPL} := MV_{1_FPL} + MV_{2_FPL} + MV_{3_FPL} + MV_{7_FPL}$$

$$M_{EV_FPL} = 687075.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1_FPL} := MV_{4_FPL}$$

$$M_{EH1_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1_FPL} := MV_{5_FPL}$$

$$M_{LS1_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE_FPL} := MV_{6_FPL}$$

$$M_{LS1@MSE_FPL} = 50426.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2_FPL} := MH_{1_FPL}$$

$$M_{EH2_FPL} = 134773.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2_FPL} := MH_{2_FPL}$$

$$M_{LS2_FPL} = 29764.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1$$

LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia_FPL} := \eta \cdot \left((Ia_{EV} \cdot V_{EV_FPL}) + (Ia_{EH} \cdot V_{EH_FPL}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE_FPL}) \right) \quad V_{Ia_FPL} = 73579.3 \frac{lb}{ft}$$

$$V_{Ib_FPL} := \eta \cdot \left((Ib_{EV} \cdot V_{EV_FPL}) + (Ib_{EH} \cdot V_{EH_FPL}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE_FPL}) \right) \quad V_{Ib_FPL} = 96405.8 \frac{lb}{ft}$$

Compute Bearing Resistance:

$$H_{Ia_FPL} := \eta \cdot \left((Ia_{EH} \cdot H_{EH_FPL}) + (Ia_{LS} \cdot H_{LS_FPL}) \right) \quad H_{Ia_FPL} = 25111.5 \frac{lb}{ft}$$

$$H_{Ib_FPL} := \eta \cdot \left((Ib_{EH} \cdot H_{EH_FPL}) + (Ib_{LS} \cdot H_{LS_FPL}) \right) \quad H_{Ib_FPL} = 25111.5 \frac{lb}{ft}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia_FPL} := \eta \cdot \left((Ia_{EV} \cdot M_{EV_FPL}) + (Ia_{EH} \cdot M_{EH1_FPL}) + Ia_{LS} \cdot (M_{LS1_FPL} + M_{LS1@MSE_FPL}) \right) \quad MV_{Ia_FPL} = 775322.2 \frac{lb \cdot ft}{ft}$$

$$MV_{Ib_FPL} := \eta \cdot \left((Ib_{EV} \cdot M_{EV_FPL}) + (Ib_{EH} \cdot M_{EH1_FPL}) + Ib_{LS} \cdot (M_{LS1_FPL} + M_{LS1@MSE_FPL}) \right) \quad MV_{Ib_FPL} = 1015798.7 \frac{lb \cdot ft}{ft}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia_FPL} := \eta \cdot \left((Ia_{EH} \cdot M_{EH2_FPL}) + (Ia_{LS} \cdot M_{LS2_FPL}) \right) \quad MH_{Ia_FPL} = 254248 \frac{lb \cdot ft}{ft}$$

$e = 2.72$

$$MH_{Ib_FPL} := \eta \cdot \left((Ib_{EH} \cdot M_{EH2_FPL}) + (Ib_{LS} \cdot M_{LS2_FPL}) \right) \quad MH_{Ib_FPL} = 254248 \frac{lb \cdot ft}{ft}$$

Factored Horizontal Loads by Limit State:

Compute the Effective Bearing Length (Strength Ib) about the Toe "O" of base length (the pivot):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 887435.1 \frac{lb \cdot ft}{ft} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 229949.9 \frac{lb \cdot ft}{ft} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 92876.5 \frac{lb}{ft} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V} \quad x = 7.1 \text{ ft} \quad \text{Distance from Point "O" the resultant intersects the base}$$

$$e := \left| \frac{L}{2} - x \right| \quad e = 2.5 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B'_{top} := L - 2 \cdot e \quad B'_{top} = 14.2 \text{ ft} \quad \text{Effective Footing Width, at the top of Foundation Preparation Layer}$$

$$\Sigma M_{R_FPL} := MV_{lb_FPL}$$

$$\Sigma M_{R_FPL} = 1015798.7 \frac{lb \cdot ft}{ft}$$

Sum of Resisting Moments (Strength Ib)

$$\Sigma M_{O_FPL} := MH_{lb_FPL}$$

$$\Sigma M_{O_FPL} = 254248 \frac{lb \cdot ft}{ft}$$

Sum of Overturning Moments (Strength Ib)

$$\Sigma V_{FPL} := V_{lb_FPL}$$

$$\Sigma V_{FPL} = 96405.8 \frac{lb}{ft}$$

Sum of Vertical Loads (Strength Ib)

$$x_{FPL} := \frac{(\Sigma M_{R_FPL} - \Sigma M_{O_FPL})}{\Sigma V_{FPL}}$$

$$x_{FPL} = 7.9 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e_{FPL} := \left| \frac{(L + 1 \text{ ft})}{2} - x_{FPL} \right|$$

$$e_{FPL} = 2.2 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B'_{bottom} := (L + 1 \text{ ft}) - 2 \cdot e_{FPL}$$

$$B'_{bottom} = 15.8 \text{ ft}$$

Effective Footing Width, at the bottom of Foundation Preparation Layer (FPL)

Foundation Layout:

$$L_{Wall} := 1423.79 \text{ ft}$$

Assumed Footing Length (Wall Section Length)

$$D_f := d_e$$

$$D_f = 5.4 \text{ ft}$$

Footing embedment

$$D_F := D_f + D_{undercut}$$

Embenment Depth at bottom of Undercut

$$d_w := D_f - 1 \text{ ft}$$

Depth of Groundwater below ground surface in front of wall

Compute Bearing Resistance at the top of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 10.66$$

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

$$N_c = 20.72$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 10.9$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.005$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \cdot \tan(\phi'_{fd}) \right), 1 \right)$$

$$s_q = 1.005$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.996$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.13$$

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

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

$$N_{cm} = 20.827$$

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

$$N_{qm} = 12.125$$

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

$$N_{\gamma m} = 10.833$$

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

$$q_{nd_top} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad \text{at top of foundation preparation layer}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_top} := \phi_b \cdot q_{nd_top} \quad q_{Rd_top} = 8.8 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

$$s_c := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.002$$

$$s_q := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \tan(\phi_{fdu}), 1\right)$$

$$s_q = 1$$

$$s_\gamma := \text{if}\left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}}\right), 1\right)$$

$$s_\gamma = 1$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \tan\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.13$$

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

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

$$N_{cm} = 5.15$$

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

$$N_{qm} = 1.132$$

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

$$N_{\gamma m} = 0$$

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

$$q_{nu_top} := Su_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nu_top} = 13499.8 \frac{lbf}{ft^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_top} := \phi_b \cdot q_{nu_top} \quad q_{Ru_top} = 8.8 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_top} = 8.8 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_top} = 8.8 \text{ ksf}$$

Compute Bearing Resistance at the bottom of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 10.66$$

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

$$N_c = 20.72$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 10.9$$

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

$$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.006$$

$$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \tan(\phi'_{fd}), 1\right)$$

$$s_q = 1.005$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.996$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right)$$

$$d_q = 1.12$$

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

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

$$N_{cm} = 20.839$$

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

$$N_{qm} = 12$$

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

$$N_{\gamma m} = 10.828$$

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

$$q_{nd_bottom} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

at bottom of foundation preparation layer

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_bottom} := \phi_b \cdot q_{nd_bottom} \quad q_{Rd_bottom} = 9.6 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.002$$

$$s_q := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \cdot \tan(\phi_{fdu}) \right), 1 \right) \quad s_q = 1$$

$$s_\gamma := \text{if} \left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right) \quad s_\gamma = 1$$

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

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

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

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

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right) \quad e$$

$$d_q = 1.12$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 5.151$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 1.12$$

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

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

$$q_{nu_bottom} := Su_{fdu} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu_top} = 13499.8 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_bottom} := \phi_b \cdot q_{nu_top} \quad q_{Ru_bottom} = 8.8 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_bottom} = 9.6 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_bottom} = 8.8 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location about the toe "O" of base length (the pivot):

$$e := \left| \frac{L}{2} - x \right| \quad e = 2.48 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2]. The effective bearing width is equal to B-2e.

$$\sigma_v := \frac{\Sigma V}{B'_{top}} \quad \sigma_v = 6559.9 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Bearing Stress}$$

$$\sigma_{v_FPL} := \frac{\Sigma V_{FPL}}{B'_{bottom}} \quad \sigma_{v_FPL} = 6102.1 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Bearing Stress}$$

Bearing Resistance Capacity:Demand Ratio (CDR)

At the top of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_top} := \frac{q_{Rd_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_top} = 1.34$

Undrained Conditions: $CDR_{Bearing_u_top} := \frac{q_{Ru_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_top} = 1.34$

At the bottom of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_bottom} := \frac{q_{Rd_bottom}}{\sigma_{v_FPL}}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_bottom} = 1.57$

Undrained Conditions: $CDR_{Bearing_u_bottom} := \frac{q_{Ru_bottom}}{\sigma_{v_FPL}}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_bottom} = 1.44$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$e_{max} := \frac{L}{3}$

$e_{max} = 6.4 \text{ ft}$

Maximum Eccentricity **LRFD [C11.6.3.3.]**

$\Sigma M_R := MV_{Ia}$

$\Sigma M_R = 678070.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$

Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$

$\Sigma M_O = 229949.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$

Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$

$\Sigma V = 70965 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$

$x = 6.3 \text{ ft}$

Distance from Point "O" the resultant intersects the base

$e := \left| \frac{L}{2} - x \right|$

$e = 3.24 \text{ ft}$

Wall eccentricity

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e}$

Is the CDR > or = to 1.0?

$CDR_{Eccentricity} = 1.97$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$

$R_u = 23493.7 \frac{\text{lb}}{\text{ft}}$

At the top of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$\Sigma V := V_{Ia}$

$\Sigma V = 70965 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$R_{td_top} := \Sigma V \cdot \tan(\phi_{Re})$

$R_{td_top} = 47866.5 \frac{\text{lb}}{\text{ft}}$

Nominal sliding resistance Drained Conditions

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$\phi_\tau := 1.0$

Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

Drained Conditions:

$\phi R_{n_d_top} := \phi_\tau \cdot R_{td_top}$

$R_{R_d_top} := \phi R_{n_d_top}$

$R_{R_d_top} = 47.9 \frac{\text{kip}}{\text{ft}}$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_top} := \frac{R_{R_d_top}}{R_u} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_d_top} = 2.04$$

At the bottom of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V_{FPL} := V_{Ia_FPL} \quad \Sigma V_{FPL} = 73579.3 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$R_{td_bottom} := \Sigma V_{FPL} \cdot \tan(\phi'_{fd}) \quad R_{td_bottom} = 34310.6 \frac{\text{lb}}{\text{ft}} \quad \text{Nominal sliding resistance Drained Conditions}$$

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_r := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\text{Drained Conditions:} \quad \phi R_{n_d_bottom} := \phi_r \cdot R_{td_bottom} \quad R_{R_d_bottom} := \phi R_{n_d_bottom} \quad R_{R_d_bottom} = 34.3 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_bottom} := \frac{R_{R_d_bottom}}{H_{Ia_FPL}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_d_bottom} = 1.37$$

Undrained Conditions (Total Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V_{FPL} := V_{Ia_FPL} \quad \Sigma V_{FPL} = 73579.3 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$B := L + 1 \text{ ft} \quad \text{Footing base width}$$

$$\sigma_{v_FPL} := \frac{\Sigma V_{FPL}}{B} \quad \sigma_{v_FPL} = 3658.8 \frac{\text{lb}}{\text{ft}^2} \quad \text{Vertical stress **ODOT GDM [1504]**}$$

$$q_{FPL} := \frac{1}{2} \cdot \sigma_{v_FPL} \quad q_{FPL} = 1829.4 \frac{\text{lb}}{\text{ft}^2} \quad \text{Uniform unit shear resistance as 1/2 vertical stress **ODOT GDM [1504]**}$$

Determine which Cohesive Soil Resistance Case is Present:

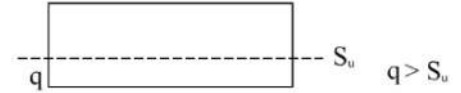
$$Case_1 := \text{if}(q_{FPL} > Su_{fdu}, 1, 0) \quad Case_1 = 0$$

$$Case_2 := \text{if}(Su_{fdu} \geq q_{FPL}, 1, 0) \quad Case_2 = 1$$

Unit Shear Resistance for Case 1:

$$R_{\tau_case1} := Su_{fdu} \cdot B = 51280.5 \frac{lb}{ft}$$

Case 1:



Unit Shear Resistance for Case 2:

$$R_{\tau_case2} := q_{FPL} \cdot B = 36789.6 \frac{lb}{ft}$$

Case 2:



Define the Applicable Case:

$$R_{\tau} := R_{\tau_case2}$$

$$R_{\tau} = 36789.6 \frac{lb}{ft}$$

Nominal sliding resistance Cohesive Soils

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_{ep} := 0.5$$

Resistance factor for passive resistance specified in
LRFD Table 10.5.5.2.2-1

$$\phi_{\tau} := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

Factored Sliding Resistance to be used in CDR Calculations:

$$R_R = 36789.643 \frac{lb}{ft}$$

Sliding Capacity:Demand Ratio (CDR)

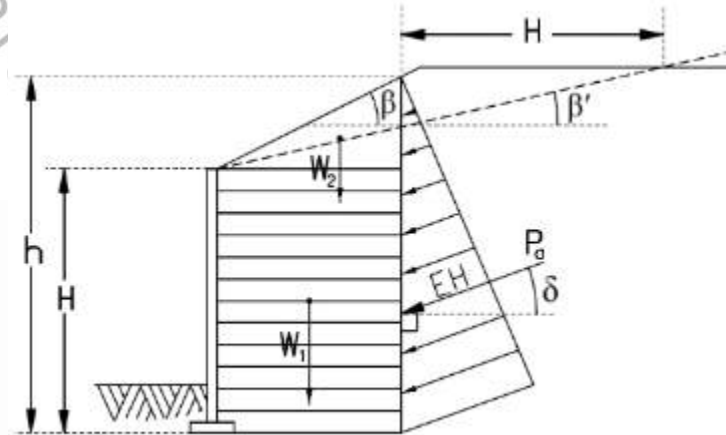
$$CDR_{Sliding_u_bottom} := \frac{R_R}{H_{Ia_FPL}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_u_bottom} = 1.47$$

Objective: To evaluate the external stability of MSE wall design with vertical wall face and broken backslope case.
Method: In accordance with ODOT Bridge Design Manual, 2026 [Sect. 307.4] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 3.11.5.8 and Sect. 11.10.5].

Assumptions:

- Horizontal backfill, Infinite backslope or Broken backslope behind MSE wall, see LRFD Sect. 3.11.5.8.1.
- For battered or vertical walls with a back face of wall angle of θ to horizontal.
- Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ .
- MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall



Givens:

Wall Geometry:

$$H_e := 25.2 \text{ ft}$$

$$\theta := 90 \cdot \text{deg}$$

$$\beta_{\text{BackSlope}} := 0 \text{ deg}$$

$$\lambda := 0 \text{ ft}$$

Exposed wall height (Proposed profile grade to bottom of leveling pad)

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Inclination of ground slope behind face of wall. If it is horizontal backfill behind MSE wall, $\beta_{\text{BackSlope}} = 0$ deg

Horizontal distance from the back of MSE wall to the top of slope. If it is infinite slope behind MSE Wall, input λ larger than 2H; If it is horizontal backfill behind MSE wall, $\lambda = 0$ ft

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \text{ deg}$$

$$\gamma_r := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_r := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \text{ deg}$$

$$\gamma_b := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_b := 0 \frac{\text{lbf}}{\text{ft}^2}$$

$$\delta := \frac{2}{3} \cdot \phi'_b$$

$$\delta = 20 \text{ deg}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Soil Design Parameters for Bearing Resistance (Average 1.5 B Below Leveling Pad)

Drained Conditions (Effective Stress):

$$\phi'_{fd} := 24 \text{ deg}$$

Effective angle of internal friction

$$\gamma_{fd} := 115 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} := 250 \frac{\text{lbf}}{\text{ft}^2}$$

Effective Cohesion

Undrained Conditions (Total Stress):

$$\phi_{fdu} := 0 \text{ deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$Su_{fdu} := 2700 \frac{\text{lbf}}{\text{ft}^2}$$

Undrained Shear Strength

Undercut & Replacement Design Parameters:

$$\phi_{Re} := 34 \text{ deg}$$

Angle of internal friction for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$c_{Re} := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$\gamma_{Re} := 130 \frac{\text{lbf}}{\text{ft}^3}$$

Unit Weight for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$\delta_{Re} := \frac{2}{3} \cdot \phi_{Re} \quad \delta_{Re} = 22.7 \text{ deg}$$

Friction angle between Replacement soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

$$D_{undercut} := 1 \text{ ft}$$

Depth of Undercut below bottom of footing

Foundation Surcharge Soil Parameters

$$\gamma_q := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Depth of Embedment Check:

$$d_{frost} := 3.25 \text{ ft}$$

$$d_{user} := 4.9 \text{ ft}$$

Local Frost Depth / User Input

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 4.9 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 30.1 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1.5 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 4.9 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 30.1 \text{ ft}$$

Initial design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 0.7 \cdot H$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max \left(8 \cdot \text{ft}, 0.7 \cdot H, L_{user} \right)$$

$$L = 21.1 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L < \lambda, 0 \frac{\text{lbf}}{\text{ft}^2}, 250 \frac{\text{lbf}}{\text{ft}^2} \right) = 250 \frac{\text{lbf}}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

$$SUR_{@MSE} := \text{if} \left(L < \lambda, 0 \frac{\text{lbf}}{\text{ft}^2}, 250 \frac{\text{lbf}}{\text{ft}^2} \right) = 250 \frac{\text{lbf}}{\text{ft}^2}$$

Live load surcharge above the MSE Wall soil reinforcement (per LRFD BDS [3.11.6.4])

Calculations:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan \left(\beta_{BackSlope} \right)$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h := \text{if} \left(L < \lambda, H + L \cdot \tan \left(\beta_{BackSlope} \right), H + H_{BackSlope} \right)$$

Height of retained fill at the back of the reinforced soil

$$h = 30.1 \text{ ft}$$

$$I := \text{atan} \left(\frac{H_{BackSlope}}{2 \cdot H} \right)$$

$$\beta := \text{if} \left(\lambda < 2 \cdot H, I, \beta_{BackSlope} \right)$$

Angle of friction between retained backfill and reinforced soil

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

$$\Gamma := \left(1 + \sqrt{\frac{\left(\sin \left(\phi'_b + \delta \right) \cdot \sin \left(\phi'_b - \beta \right) \right)}{\left(\sin \left(\theta - \delta \right) \cdot \sin \left(\theta + \beta \right) \right)}} \right)^2 = 2.6845$$

$$k_{ab} := \left(\frac{\left(\sin \left(\theta + \phi'_b \right) \right)^2}{\left(\Gamma \cdot \left(\sin \left(\theta \right) \right)^2 \cdot \sin \left(\theta - \delta \right) \right)} \right)$$

$$k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot h^2 \cdot k_{ab}$$

$$F_T = 16162.2 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{ab}$$

$$F_{SUR} = 2237.3 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 76104.8 \frac{\text{lbf}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_2 = 0 \frac{\text{lbf}}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_3 := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lbf}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_3 = 0 \frac{\text{lbf}}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_4 := F_T \cdot \sin(\beta)$$

$$V_4 = 0 \frac{\text{lbf}}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_5 := F_{SUR} \cdot \sin(\beta)$$

$$V_5 = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_6 := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_6 = 5267.5 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1} := \frac{L}{2}$$

$$d_{v1} = 10.5 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 801764.5 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := \text{if}\left(L < \lambda, \frac{2L}{3}, \frac{2 \cdot \lambda}{3}\right)$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v3} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v3} = 10.5 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3}$$

$$MV_3 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v4} := L$$

$$d_{v4} = 21.1 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4}$$

$$MV_4 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v5} := L$$

$$d_{v5} = 21.1 \text{ ft}$$

$$MV_5 := V_5 \cdot d_{v5}$$

$$MV_5 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v6} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v6} = 10.5 \text{ ft}$$

$$MV_6 := V_6 \cdot d_{v6}$$

$$MV_6 = 55493.1 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T \cdot \cos(\beta) = 16162.2 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} \cdot \cos(\beta) = 2237.3 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

Moment:

$$d_{h1} := \frac{h}{3}$$

$$d_{h1} = 10 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 162160.3 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{2}$$

$$d_{h2} = 15.1 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 33671.2 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1 + V_2 + V_3$$

$$V_{EV} = 76104.8 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_4$$

$$V_{EH} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS} := V_5$$

$$V_{LS} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS@MSE} := V_6$$

$$V_{LS@MSE} = 5267.5 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 16162.2 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 2237.3 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1 + MV_2 + MV_3$$

$$M_{EV} = 801764.5 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_4$$

$$M_{EH1} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1} := MV_5$$

$$M_{LS1@MSE} := MV_6$$

$$M_{EH2} := MH_1$$

$$M_{LS2} := MH_2$$

$$M_{LS1} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE} = 55493.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} = 162160.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} = 33671.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1 \quad \text{LRFD Load Modifier}$$

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot \left((Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE}) \right)$$

$$V_{Ia} = 85323 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot \left((Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE}) \right)$$

$$V_{Ib} = 111959.7 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot \left((Ia_{EH} \cdot H_{EH}) + (Ia_{LS} \cdot H_{LS}) \right)$$

$$H_{Ia} = 28158.5 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot \left((Ib_{EH} \cdot H_{EH}) + (Ib_{LS} \cdot H_{LS}) \right)$$

$$H_{Ib} = 28158.5 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot \left((Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + Ia_{LS} \cdot (M_{LS1} + M_{LS1@MSE}) \right)$$

$$MV_{Ia} = 898877.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot \left((Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + Ib_{LS} \cdot (M_{LS1} + M_{LS1@MSE}) \right)$$

$$MV_{Ib} = 1179495 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot \left((Ia_{EH} \cdot M_{EH2}) + (Ia_{LS} \cdot M_{LS2}) \right)$$

$$MH_{Ia} = 302165 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot \left((Ib_{EH} \cdot M_{EH2}) + (Ib_{LS} \cdot M_{LS2}) \right)$$

$$MH_{Ib} = 302165 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Calculations at Bottom of Foundation Preparation Layer:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan(\beta_{BackSlope})$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h_{FPL} := \text{if}(L < \lambda, H + L \cdot \tan(\beta_{BackSlope}) + 1 \text{ ft}, H + H_{BackSlope} + 1 \text{ ft})$$

$$h_{FPL} = 31.1 \text{ ft}$$

Height of retained fill at the back of the reinforced soil

$$I := \text{atan}\left(\frac{H_{BackSlope}}{2 \cdot H}\right)$$

$$\beta := \text{if}(\lambda < 2 \cdot H, I, \beta_{BackSlope})$$

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

Angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 2.6845$$

$$k_{ab} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{ab} = 0.2973 \quad \text{Active Earth Pressure Coefficient}$$

$$F_{T_FPL} := \frac{1}{2} \cdot \gamma_b \cdot h_{FPL}^2 \cdot k_{ab} \quad F_T = 16162.2 \frac{\text{lbf}}{\text{ft}} \quad \text{Active Earth Force Resultant (EH)}$$

$$F_{SUR_FPL} := SUR \cdot h_{FPL} \cdot k_{ab} \quad F_{SUR} = 2237.3 \frac{\text{lbf}}{\text{ft}} \quad \text{Live Load Surcharge (LS)}$$

Vertical Loads:

$$V_{1_FPL} := \gamma_r \cdot H \cdot L \quad V_{1_FPL} = 76104.8 \frac{\text{lbf}}{\text{ft}} \quad \text{Soil backfill - reinforced soil (EV)}$$

$$V_{2_FPL} := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right) \quad V_{2_FPL} = 0 \frac{\text{lbf}}{\text{ft}} \quad \text{Triangular Soil backfill - backslope soil (EV)}$$

$$V_{3_FPL} := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lbf}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right) \quad V_{3_FPL} = 0 \frac{\text{lbf}}{\text{ft}} \quad \text{Rectangular Soil backfill - backslope soil (EV)}$$

$$V_{4_FPL} := F_{T_FPL} \cdot \sin(\beta) \quad V_{4_FPL} = 0 \frac{\text{lbf}}{\text{ft}} \quad \text{Active earth force resultant (vertical component - EH)}$$

$$V_{5_FPL} := F_{SUR_FPL} \cdot \sin(\beta) \quad V_{5_FPL} = 0 \frac{\text{lbf}}{\text{ft}} \quad \text{Live Load Surcharge (vertical component - LS)}$$

$$V_{6_FPL} := SUR_{@MSE} \cdot (L - \lambda) \quad V_{6_FPL} = 5267.5 \frac{\text{lbf}}{\text{ft}} \quad \text{Live Load Surcharge above MSE Wall soil reinforcement}$$

$$V_{7_FPL} := \gamma_{Re} \cdot (L + 1 \text{ ft}) \cdot D_{undercut} \quad V_{7_FPL} = 2869.1 \frac{\text{lbf}}{\text{ft}} \quad \text{Foundation Preparation Layer Weight}$$

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1_FPL} := \frac{L}{2} + 1 \text{ ft} \quad d_{v1_FPL} = 11.5 \text{ ft} \quad MV_{1_FPL} := V_{1_FPL} \cdot d_{v1_FPL} \quad MV_{1_FPL} = 877869.3 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2_FPL} := \text{if} \left(L < \lambda, \frac{2L}{3}, \frac{2\lambda}{3} \right) + 1 \text{ ft} \quad d_{v2_FPL} = 1 \text{ ft} \quad MV_{2_FPL} := V_{2_FPL} \cdot d_{v2_FPL} \quad MV_{2_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v3_FPL} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right) + 1 \text{ ft} \quad d_{v3_FPL} = 11.5 \text{ ft} \quad MV_{3_FPL} := V_{3_FPL} \cdot d_{v3_FPL} \quad MV_{3_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v4_FPL} := L + 1 \text{ ft} \quad d_{v4_FPL} = 22.1 \text{ ft} \quad MV_{4_FPL} := V_{4_FPL} \cdot d_{v4_FPL} \quad MV_{4_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v5_FPL} := L + 1 \text{ ft} \quad d_{v5_FPL} = 22.1 \text{ ft} \quad MV_{5_FPL} := V_{5_FPL} \cdot d_{v5_FPL} \quad MV_{5_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v6_FPL} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right) + 1 \text{ ft} \quad d_{v6_FPL} = 11.5 \text{ ft} \quad MV_{6_FPL} := V_{6_FPL} \cdot d_{v6_FPL} \quad MV_{6_FPL} = 60760.6 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v7_FPL} := \frac{(L + 1 \text{ ft})}{2} \quad d_{v7_FPL} = 11 \text{ ft} \quad MV_{7_FPL} := V_{7_FPL} \cdot d_{v7_FPL} \quad MV_{7_FPL} = 31660.5 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_{1_FPL} := F_{T_FPL} \cdot \cos(\beta) = 17253.9 \frac{\text{lb}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_{2_FPL} := F_{SUR_FPL} \cdot \cos(\beta) = 2311.6 \frac{\text{lb}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1_FPL} := \frac{h_{FPL}}{3} \quad d_{h1_FPL} = 10.4 \text{ ft}$$

Moment:

$$MH_{1_FPL} := H_{1_FPL} \cdot d_{h1_FPL} \quad MH_{1_FPL} = 178865.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2_FPL} := \frac{h_{FPL}}{2} \quad d_{h2_FPL} = 15.6 \text{ ft}$$

$$MH_{2_FPL} := H_{2_FPL} \cdot d_{h2_FPL} \quad MH_{2_FPL} = 35945.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV_FPL} := V_{1_FPL} + V_{2_FPL} + V_{3_FPL} + V_{7_FPL}$$

$$V_{EV_FPL} = 78973.9 \frac{\text{lb}}{\text{ft}}$$

$$V_{EH_FPL} := V_{4_FPL}$$

$$V_{EH_FPL} = 0 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS_FPL} := V_{5_FPL}$$

$$V_{LS_FPL} = 0 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS@MSE_FPL} := V_{6_FPL}$$

$$V_{LS@MSE_FPL} = 5267.5 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH_FPL} := H_{1_FPL}$$

$$H_{EH_FPL} = 17253.9 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS_FPL} := H_{2_FPL}$$

$$H_{LS_FPL} = 2311.6 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV_FPL} := MV_{1_FPL} + MV_{2_FPL} + MV_{3_FPL} + MV_{7_FPL}$$

$$M_{EV_FPL} = 909529.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1_FPL} := MV_{4_FPL}$$

$$M_{EH1_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1_FPL} := MV_{5_FPL}$$

$$M_{LS1_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE_FPL} := MV_{6_FPL}$$

$$M_{LS1@MSE_FPL} = 60760.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2_FPL} := MH_{1_FPL}$$

$$M_{EH2_FPL} = 178865.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2_FPL} := MH_{2_FPL}$$

$$M_{LS2_FPL} = 35945.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1$$

LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia_FPL} := \eta \cdot \left((Ia_{EV} \cdot V_{EV_FPL}) + (Ia_{EH} \cdot V_{EH_FPL}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE_FPL}) \right) \quad V_{Ia_FPL} = 88192.1 \frac{lb}{ft}$$

$$V_{Ib_FPL} := \eta \cdot \left((Ib_{EV} \cdot V_{EV_FPL}) + (Ib_{EH} \cdot V_{EH_FPL}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE_FPL}) \right) \quad V_{Ib_FPL} = 115832.9 \frac{lb}{ft}$$

Compute Bearing Resistance:

$$H_{Ia_FPL} := \eta \cdot \left((Ia_{EH} \cdot H_{EH_FPL}) + (Ia_{LS} \cdot H_{LS_FPL}) \right) \quad H_{Ia_FPL} = 29926.2 \frac{lb}{ft}$$

$$H_{Ib_FPL} := \eta \cdot \left((Ib_{EH} \cdot H_{EH_FPL}) + (Ib_{LS} \cdot H_{LS_FPL}) \right) \quad H_{Ib_FPL} = 29926.2 \frac{lb}{ft}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia_FPL} := \eta \cdot \left((Ia_{EV} \cdot M_{EV_FPL}) + (Ia_{EH} \cdot M_{EH1_FPL}) + Ia_{LS} \cdot (M_{LS1_FPL} + M_{LS1@MSE_FPL}) \right) \quad MV_{Ia_FPL} = 1015860.9 \frac{lb \cdot ft}{ft}$$

$$MV_{Ib_FPL} := \eta \cdot \left((Ib_{EV} \cdot M_{EV_FPL}) + (Ib_{EH} \cdot M_{EH1_FPL}) + Ib_{LS} \cdot (M_{LS1_FPL} + M_{LS1@MSE_FPL}) \right) \quad MV_{Ib_FPL} = 1334196.4 \frac{lb \cdot ft}{ft}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia_FPL} := \eta \cdot \left((Ia_{EH} \cdot M_{EH2_FPL}) + (Ia_{LS} \cdot M_{LS2_FPL}) \right) \quad MH_{Ia_FPL} = 331202.9 \frac{lb \cdot ft}{ft}$$

$e = 2.72$

$$MH_{Ib_FPL} := \eta \cdot \left((Ib_{EH} \cdot M_{EH2_FPL}) + (Ib_{LS} \cdot M_{LS2_FPL}) \right) \quad MH_{Ib_FPL} = 331202.9 \frac{lb \cdot ft}{ft}$$

Factored Horizontal Loads by Limit State:

Compute the Effective Bearing Length (Strength Ib) about the Toe "O" of base length (the pivot):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 1179495 \frac{lb \cdot ft}{ft} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 302165 \frac{lb \cdot ft}{ft} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 111959.7 \frac{lb}{ft} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V} \quad x = 7.8 \text{ ft} \quad \text{Distance from Point "O" the resultant intersects the base}$$

$$e := \left| \frac{L}{2} - x \right| \quad e = 2.7 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B'_{top} := L - 2 \cdot e \quad B'_{top} = 15.7 \text{ ft} \quad \text{Effective Footing Width, at the top of Foundation Preparation Layer}$$

$$\Sigma M_{R_FPL} := MV_{lb_FPL}$$

$$\Sigma M_{R_FPL} = 1334196.4 \frac{lb \cdot ft}{ft}$$

Sum of Resisting Moments (Strength lb)

$$\Sigma M_{O_FPL} := MH_{lb_FPL}$$

$$\Sigma M_{O_FPL} = 331202.9 \frac{lb \cdot ft}{ft}$$

Sum of Overturning Moments (Strength lb)

$$\Sigma V_{FPL} := V_{lb_FPL}$$

$$\Sigma V_{FPL} = 115832.9 \frac{lb}{ft}$$

Sum of Vertical Loads (Strength lb)

$$x_{FPL} := \frac{(\Sigma M_{R_FPL} - \Sigma M_{O_FPL})}{\Sigma V_{FPL}}$$

$$x_{FPL} = 8.7 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e_{FPL} := \left| \frac{(L + 1 \text{ ft})}{2} - x_{FPL} \right|$$

$$e_{FPL} = 2.4 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B'_{bottom} := (L + 1 \text{ ft}) - 2 \cdot e_{FPL}$$

$$B'_{bottom} = 17.3 \text{ ft}$$

Effective Footing Width, at the bottom of Foundation Preparation Layer (FPL)

Foundation Layout:

$$L_{Wall} := 1423.79 \text{ ft}$$

Assumed Footing Length (Wall Section Length)

$$D_f := d_e$$

$$D_f = 4.9 \text{ ft}$$

Footing embedment

$$D_F := D_f + D_{undercut}$$

Embenment Depth at bottom of Undercut

$$d_w := D_f - 1 \text{ ft}$$

Depth of Groundwater below ground surface in front of wall

Compute Bearing Resistance at the top of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 9.6$$

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

$$N_c = 19.32$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 9.4$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.005$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \cdot \tan(\phi'_{fd}) \right), 1 \right)$$

$$s_q = 1.005$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.996$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.11$$

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

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

$$N_{cm} = 19.429$$

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

$$N_{qm} = 10.739$$

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

$$N_{\gamma m} = 9.4$$

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

$$q_{nd_top} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad \text{at top of foundation preparation layer}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_top} := \phi_b \cdot q_{nd_top} \quad q_{Rd_top} = 8 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

$$s_c := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.002$$

$$s_q := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \tan(\phi_{fdu}), 1\right)$$

$$s_q = 1$$

$$s_\gamma := \text{if}\left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}}\right), 1\right)$$

$$s_\gamma = 1$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \tan\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.11$$

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

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

$$N_{cm} = 5.151$$

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

$$N_{qm} = 1.113$$

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

$$N_{\gamma m} = 0$$

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

$$q_{nu_top} := Su_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nu_top} = 14235.7 \frac{lbf}{ft^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_top} := \phi_b \cdot q_{nu_top} \quad q_{Ru_top} = 9.3 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_top} = 8 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_top} = 9.3 \text{ ksf}$$

Compute Bearing Resistance at the bottom of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 9.6$$

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

$$N_c = 19.32$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 9.4$$

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

$$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.006$$

$$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \tan(\phi'_{fd}), 1\right)$$

$$s_q = 1.005$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.995$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 19.44$$

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

$$N_{qm} = 10.649$$

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

$$N_{\gamma m} = 9.396$$

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

$$q_{nd_bottom} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

at bottom of foundation preparation layer

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_bottom} := \phi_b \cdot q_{nd_bottom} \quad q_{Rd_bottom} = 8.7 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.002$$

$$s_q := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \cdot \tan(\phi_{fdu}) \right), 1 \right) \quad s_q = 1$$

$$s_\gamma := \text{if} \left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right) \quad s_\gamma = 1$$

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

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

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

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

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right) \quad e$$

$$d_q = 1.1$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 5.153$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 1.103$$

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

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

$$q_{nu_bottom} := Su_{fdu} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu_top} = 14235.7 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_bottom} := \phi_b \cdot q_{nu_top} \quad q_{Ru_bottom} = 9.3 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_bottom} = 8.7 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_bottom} = 9.3 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location about the toe "O" of base length (the pivot):

$$e := \left| \frac{L}{2} - x \right| \quad e = 2.7 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$\sigma_v := \frac{\Sigma V}{B'_{top}} \quad \sigma_v = 7143.8 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Bearing Stress}$$

$$\sigma_{v_FPL} := \frac{\Sigma V_{FPL}}{B'_{bottom}} \quad \sigma_{v_FPL} = 6688.6 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Bearing Stress}$$

Bearing Resistance Capacity:Demand Ratio (CDR)

At the top of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_top} := \frac{q_{Rd_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_top} = 1.11$

Undrained Conditions: $CDR_{Bearing_u_top} := \frac{q_{Ru_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_top} = 1.30$

At the bottom of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_bottom} := \frac{q_{Rd_bottom}}{\sigma_{v_FPL}}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_bottom} = 1.29$

Undrained Conditions: $CDR_{Bearing_u_bottom} := \frac{q_{Ru_bottom}}{\sigma_{v_FPL}}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_bottom} = 1.38$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$e_{max} := \frac{L}{3}$

$e_{max} = 7 \text{ ft}$

Maximum Eccentricity LRFD [C11.6.3.3.]

$\Sigma M_R := MV_{Ia}$

$\Sigma M_R = 898877.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$

Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$

$\Sigma M_O = 302165 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$

Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$

$\Sigma V = 85323 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$

$x = 7 \text{ ft}$

Distance from Point "O" the resultant intersects the base

$e := \left| \frac{L}{2} - x \right|$

$e = 3.54 \text{ ft}$

Wall eccentricity

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e}$

Is the CDR > or = to 1.0?

$CDR_{Eccentricity} = 1.98$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$

$R_u = 28158.5 \frac{\text{lb}}{\text{ft}}$

At the top of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$\Sigma V := V_{Ia}$

$\Sigma V = 85323 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$R_{td_top} := \Sigma V \cdot \tan(\phi_{Re})$

$R_{td_top} = 57551.1 \frac{\text{lb}}{\text{ft}}$

Nominal sliding resistance Drained Conditions

Compute factored resistance against failure by sliding LRFD [10.6.3.4]:

$\phi_\tau := 1.0$

Resistance factor for sliding resistance specified in LRFD Table 11.5.7-1.

Drained Conditions:

$\phi R_{n_d_top} := \phi_\tau \cdot R_{td_top}$

$R_{R_d_top} := \phi R_{n_d_top}$

$R_{R_d_top} = 57.6 \frac{\text{kip}}{\text{ft}}$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_top} := \frac{R_{R_d_top}}{R_u} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_d_top} = 2.04$$

At the bottom of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V_{FPL} := V_{Ia_FPL} \quad \Sigma V_{FPL} = 88192.1 \frac{lb}{ft} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$R_{td_bottom} := \Sigma V_{FPL} \cdot \tan(\phi'_{fd}) \quad R_{td_bottom} = 39265.6 \frac{lb}{ft} \quad \text{Nominal sliding resistance Drained Conditions}$$

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_r := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\text{Drained Conditions:} \quad \phi R_{n_d_bottom} := \phi_r \cdot R_{td_bottom} \quad R_{R_d_bottom} := \phi R_{n_d_bottom} \quad R_{R_d_bottom} = 39.3 \frac{kip}{ft}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_bottom} := \frac{R_{R_d_bottom}}{H_{Ia_FPL}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_d_bottom} = 1.31$$

Undrained Conditions (Total Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V_{FPL} := V_{Ia_FPL} \quad \Sigma V_{FPL} = 88192.1 \frac{lb}{ft} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$B := L + 1 \text{ ft} \quad \text{Footing base width}$$

$$\sigma_{v_FPL} := \frac{\Sigma V_{FPL}}{B} \quad \sigma_{v_FPL} = 3996 \frac{lb}{ft^2} \quad \text{Vertical stress ODOT GDM [1504]}$$

$$q_{FPL} := \frac{1}{2} \cdot \sigma_{v_FPL} \quad q_{FPL} = 1998 \frac{lb}{ft^2} \quad \text{Uniform unit shear resistance as 1/2 vertical stress ODOT GDM [1504]}$$

Determine which Cohesive Soil Resistance Case is Present:

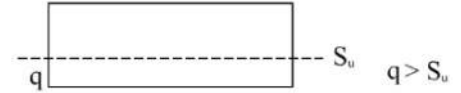
$$Case_1 := \text{if}(q_{FPL} > Su_{fdu}, 1, 0) \quad Case_1 = 0$$

$$Case_2 := \text{if}(Su_{fdu} \geq q_{FPL}, 1, 0) \quad Case_2 = 1$$

Unit Shear Resistance for Case 1:

$$R_{\tau_case1} := Su_{fdu} \cdot B = 59589 \frac{lb}{ft}$$

Case 1:



Unit Shear Resistance for Case 2:

$$R_{\tau_case2} := q_{FPL} \cdot B = 44096 \frac{lb}{ft}$$

Case 2:



Define the Applicable Case:

$$R_{\tau} := R_{\tau_case2}$$

$$R_{\tau} = 44096 \frac{lb}{ft}$$

Nominal sliding resistance Cohesive Soils

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_{ep} := 0.5$$

Resistance factor for passive resistance specified in
LRFD Table 10.5.5.2.2-1

$$\phi_{\tau} := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

Factored Sliding Resistance to be used in CDR Calculations:

$$R_R = 44096.033 \frac{lb}{ft}$$

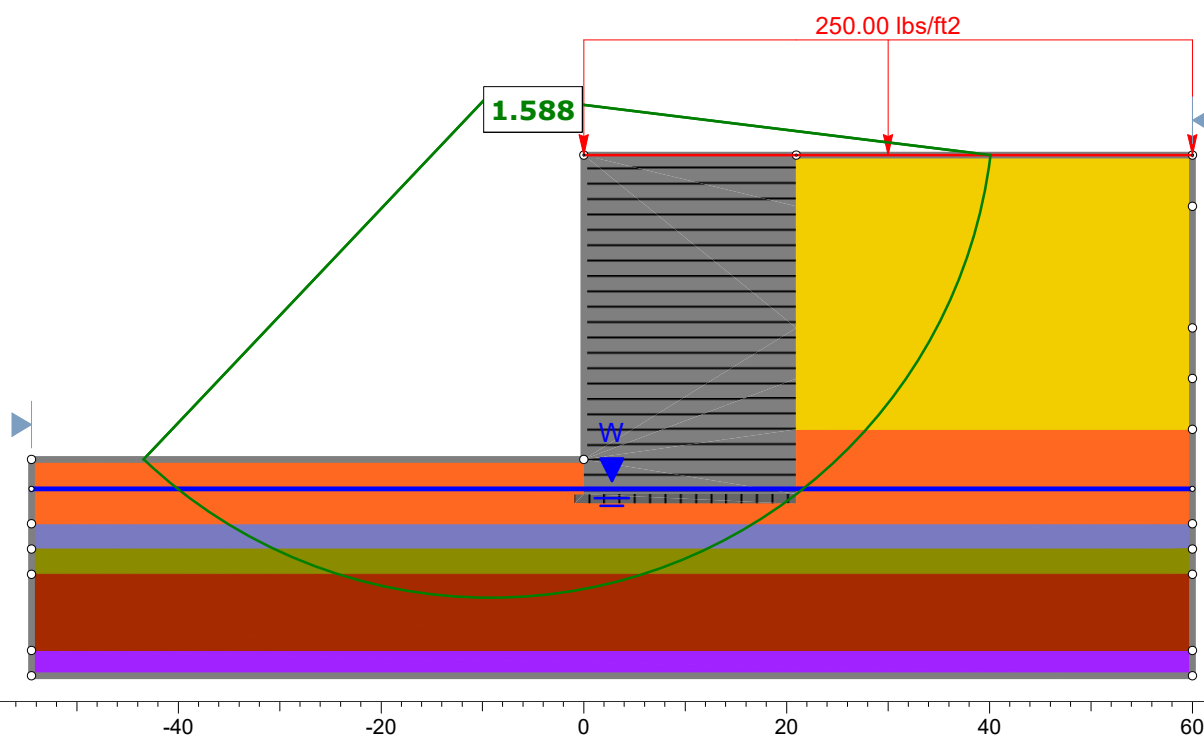
Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_u_bottom} := \frac{R_R}{H_{Ia_FPL}} \quad \text{Is the CDR > or = to 1.0?}$$

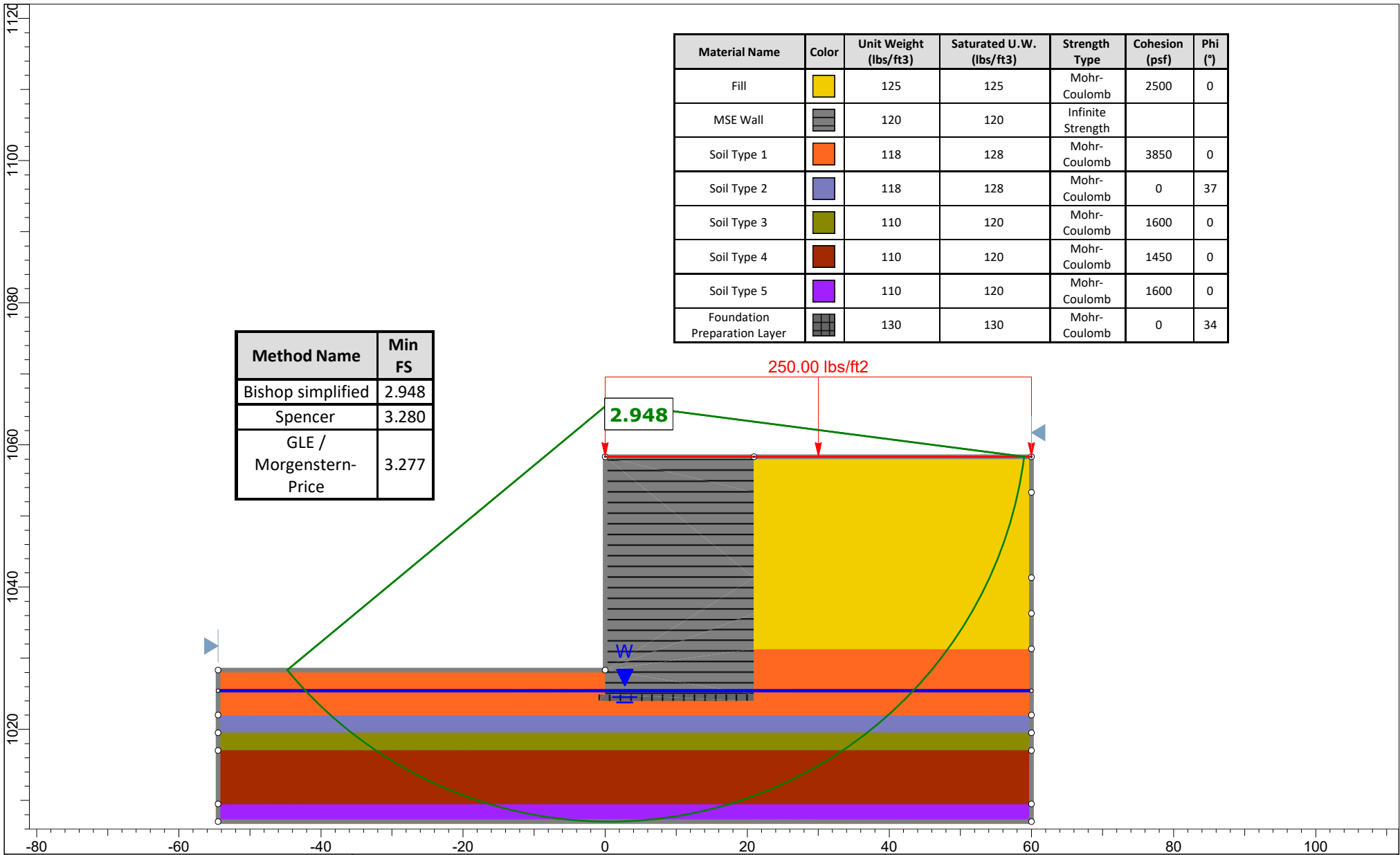
$$CDR_{Sliding_u_bottom} = 1.47$$

Method Name	Min FS
Bishop simplified	1.588
Spencer	1.599
GLE / Morgenstern-Price	1.595

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Fill		125	Mohr-Coulomb	250	26
MSE Wall		120	Infinite Strength		
Soil Type 1		118	Mohr-Coulomb	300	26
Soil Type 2		118	Mohr-Coulomb	0	37
Soil Type 3		110	Mohr-Coulomb	150	24
Soil Type 4		110	Mohr-Coulomb	150	24
Soil Type 5		110	Mohr-Coulomb	150	24
Foundation Preparation Layer		130	Mohr-Coulomb	0	34



Project			SUM 77 SR8-Retaining Wall 2	
Group	SUM-77 SR8-RW2-STA.3+25-Effective-B-032.slim		Scenario	SUM-77 SR8-RW2-STA.3+25-Effective-B-032.slim
Drawn By	ZM		Company	NEAS, Inc
Date	1/8/2025		File Name	SUM-77 SR8-RW2-STA.3+25-Effective-B-032.slim

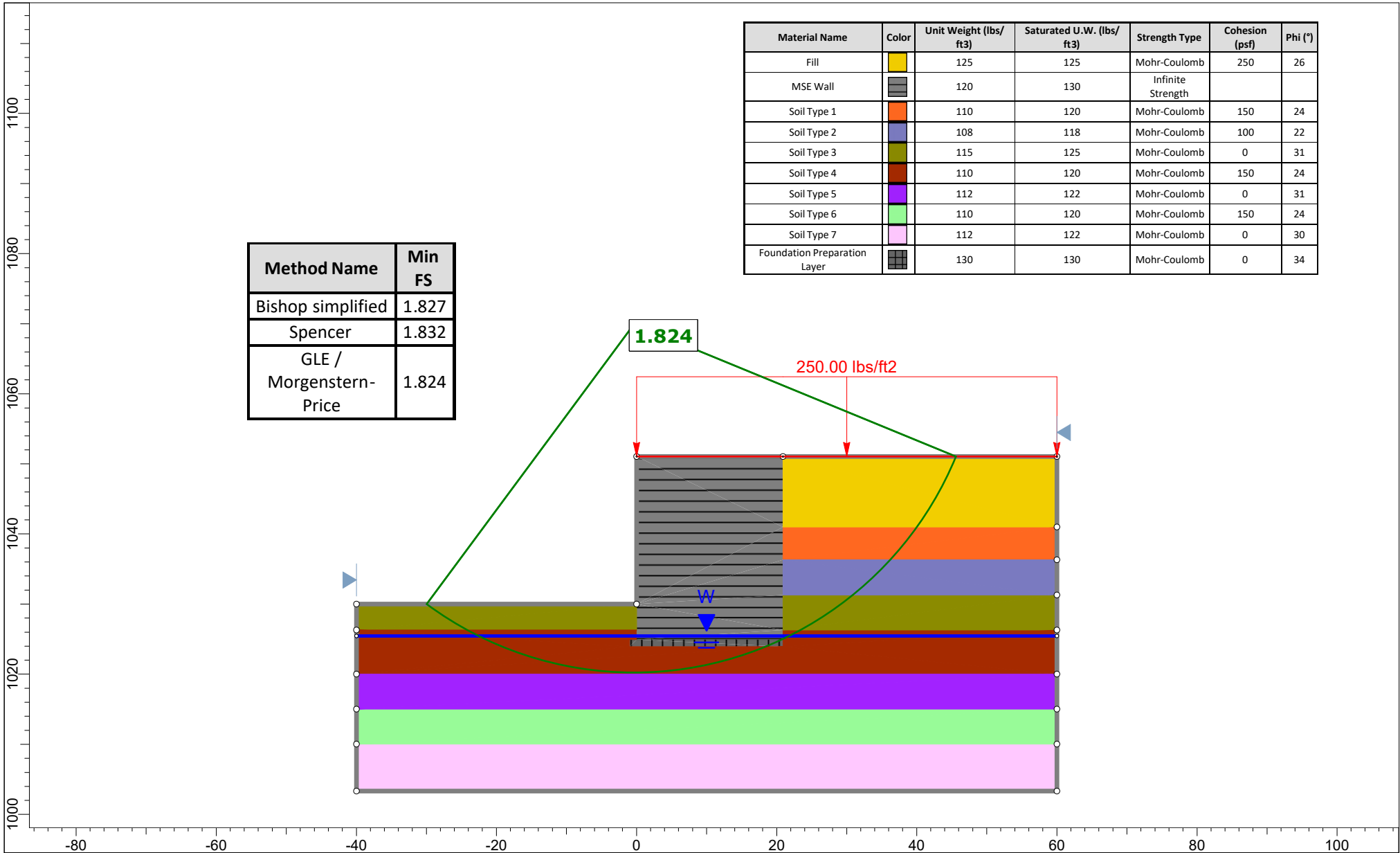


Method Name	Min FS
Bishop simplified	2.948
Spencer	3.280
GLE / Morgenstern-Price	3.277

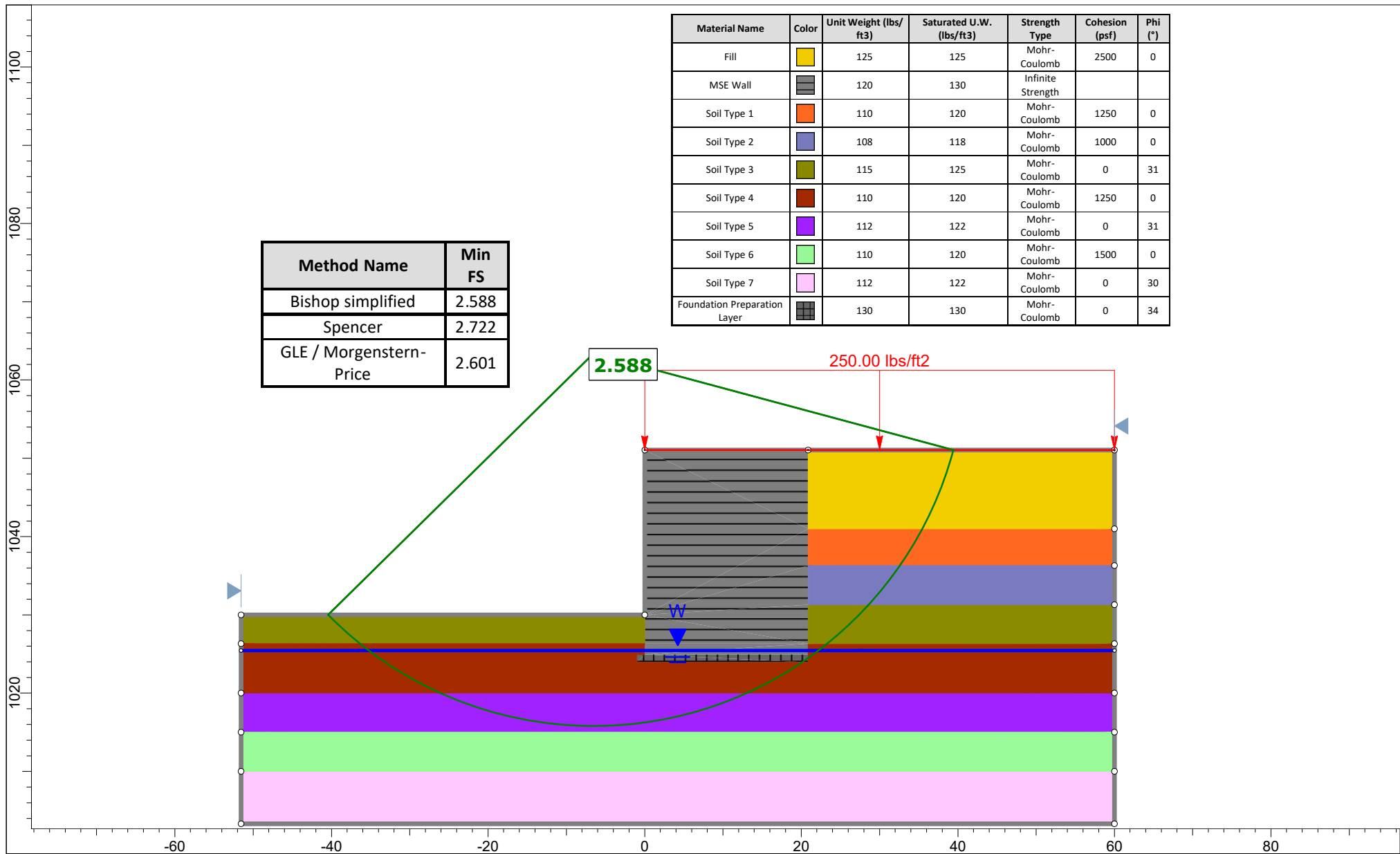
Material Name	Color	Unit Weight (lbs/ft ³)	Saturated U.W. (lbs/ft ³)	Strength Type	Cohesion (psf)	Phi (°)
Fill		125	125	Mohr-Coulomb	2500	0
MSE Wall		120	120	Infinite Strength		
Soil Type 1		118	128	Mohr-Coulomb	3850	0
Soil Type 2		118	128	Mohr-Coulomb	0	37
Soil Type 3		110	120	Mohr-Coulomb	1600	0
Soil Type 4		110	120	Mohr-Coulomb	1450	0
Soil Type 5		110	120	Mohr-Coulomb	1600	0
Foundation Preparation Layer		130	130	Mohr-Coulomb	0	34



Project		SUM 77 SR8-Retaining Wall 2	
Group	SUM-77 SR8-RW2-STA.3+25-Total-B-032.slim	Scenario	SUM-77 SR8-RW2-STA.3+25-Total-B-032.slim
Drawn By	ZM	Company	NEAS, Inc
Date	1/8/2025	File Name	SUM-77 SR8-RW2-STA.3+25-Total-B-032.slim



Project			SUM 77 SR8-Retaining Wall 2	
Group	SUM-77 SR8-RW2-STA.6+00-Effective-B-033.slim		Scenario	SUM-77 SR8-RW2-STA.6+00-Effective-B-033.slim
Drawn By	ZM		Company	NEAS, Inc
Date	1/8/2025		File Name	SUM-77 SR8-RW2-STA.6+00-Effective-B-033.slim



Project			SUM 77 SR8-Retaining Wall 2		
Group		SUM-77 SR8-RW2-STA.6+00-Total-B-033.slim		Scenario	
Drawn By		ZM		Company	
Date		1/8/2025		File Name	
				SUM-77 SR8-RW2-STA.6+00-Total-B-033.slim	

INPUT DATA – FOUNDATION LAYERS – 4 layers

	Wet Unit Weight, γ [lb/ft ³]	Poisson's Ratio μ	Description of Soil
1	112.00	0.35	A-4b
2	112.00	0.35	A-4a
3	120.00	0.30	A-1-b
4	140.00	0.30	Bedrock

INPUT DATA – EMBANKMENT LAYERS – 1 layers

	Wet Unit Weight, γ [lb/ft ³]	Description of Soil
1	125.00	Embankment Fills

INPUT DATA OF WATER

Point #	Coordinates (X, Z): (X) (Z) [ft.] [ft.]
1	0.00 1018.30
2	10.00 1018.30
3	25.00 1018.30
4	40.00 1018.30

License number FoSSA-200410

IMMEDIATE SETTLEMENT. Si

Node #	Settlement along section:		Layer	Young's Modulus, E	Poisson's Ratio, μ	Settlement of each layer, Si(k)	Initial Z	Final Z *	Total Settlement Sum of Si(k),
	X	Y	(k)	[lb/ft ²]		[ft.]	[ft.]	[ft.]	[ft.]
	[ft.]	[ft.]							
1	-60.00	0.00	1	650000	0.3500	-0.0000	1029.10	1029.05	0.05
			2	500000	0.3500	-0.0019			
			3	200000	0.3000	-0.0052			
			4	300000	0.3000	0.0558			
2	-48.00	0.00	1	650000	0.3500	-0.0002	1029.10	1028.99	0.11
			2	500000	0.3500	-0.0034			
			3	200000	0.3000	-0.0037			
			4	300000	0.3000	0.1173			
3	-36.00	0.00	1	650000	0.3500	-0.0005	1029.10	1028.85	0.25
			2	500000	0.3500	0.0064			
			3	200000	0.3000	0.0256			
			4	300000	0.3000	0.2200			
4	-24.00	0.00	1	650000	0.3500	0.0068	1029.10	1028.68	0.42
			2	500000	0.3500	0.0254			
			3	200000	0.3000	0.0627			
			4	300000	0.3000	0.3256			
5	-12.00	0.00	1	650000	0.3500	0.0066	1029.10	1028.62	0.48
			2	500000	0.3500	0.0241			
			3	200000	0.3000	0.0666			
			4	300000	0.3000	0.3842			
6	0.00	0.00	1	650000	0.3500	0.0066	1029.10	1028.60	0.50
			2	500000	0.3500	0.0236			
			3	200000	0.3000	0.0664			
			4	300000	0.3000	0.4017			
7	12.00	0.00	1	650000	0.3500	0.0066	1029.10	1028.62	0.48
			2	500000	0.3500	0.0241			
			3	200000	0.3000	0.0666			
			4	300000	0.3000	0.3842			
8	24.00	0.00	1	650000	0.3500	0.0068	1029.10	1028.68	0.42
			2	500000	0.3500	0.0254			
			3	200000	0.3000	0.0627			
			4	300000	0.3000	0.3256			
9	36.00	0.00	1	650000	0.3500	-0.0005	1029.10	1028.85	0.25
			2	500000	0.3500	0.0064			
			3	200000	0.3000	0.0256			
			4	300000	0.3000	0.2199			
10	48.00	0.00	1	650000	0.3500	-0.0002	1029.10	1028.99	0.11
			2	500000	0.3500	-0.0034			
			3	200000	0.3000	-0.0037			
			4	300000	0.3000	0.1173			
11	60.00	0.00	1	650000	0.3500	-0.0000	1029.10	1029.05	0.05
			2	500000	0.3500	-0.0019			
			3	200000	0.3000	-0.0052			
			4	300000	0.3000	0.0558			

*Note: Final Z is calculated assuming only 'Immediate Settlement' exists.

[illegible]

ULTIMATE SETTLEMENT, Sc

Node #	X [ft.]	Y [ft.]	Original Z [ft.]	Settlement Sc [ft.]	Final Z * [ft.]
1	-60.00	0.00	1029.10	0.00	1029.10
2	-48.00	0.00	1029.10	0.00	1029.10
3	-36.00	0.00	1029.10	0.07	1029.03
4	-24.00	0.00	1029.10	0.45	1028.65
5	-12.00	0.00	1029.10	0.46	1028.64
6	0.00	0.00	1029.10	0.46	1028.64
7	12.00	0.00	1029.10	0.46	1028.64
8	24.00	0.00	1029.10	0.45	1028.65
9	36.00	0.00	1029.10	0.07	1029.03
10	48.00	0.00	1029.10	0.00	1029.10
11	60.00	0.00	1029.10	0.00	1029.10

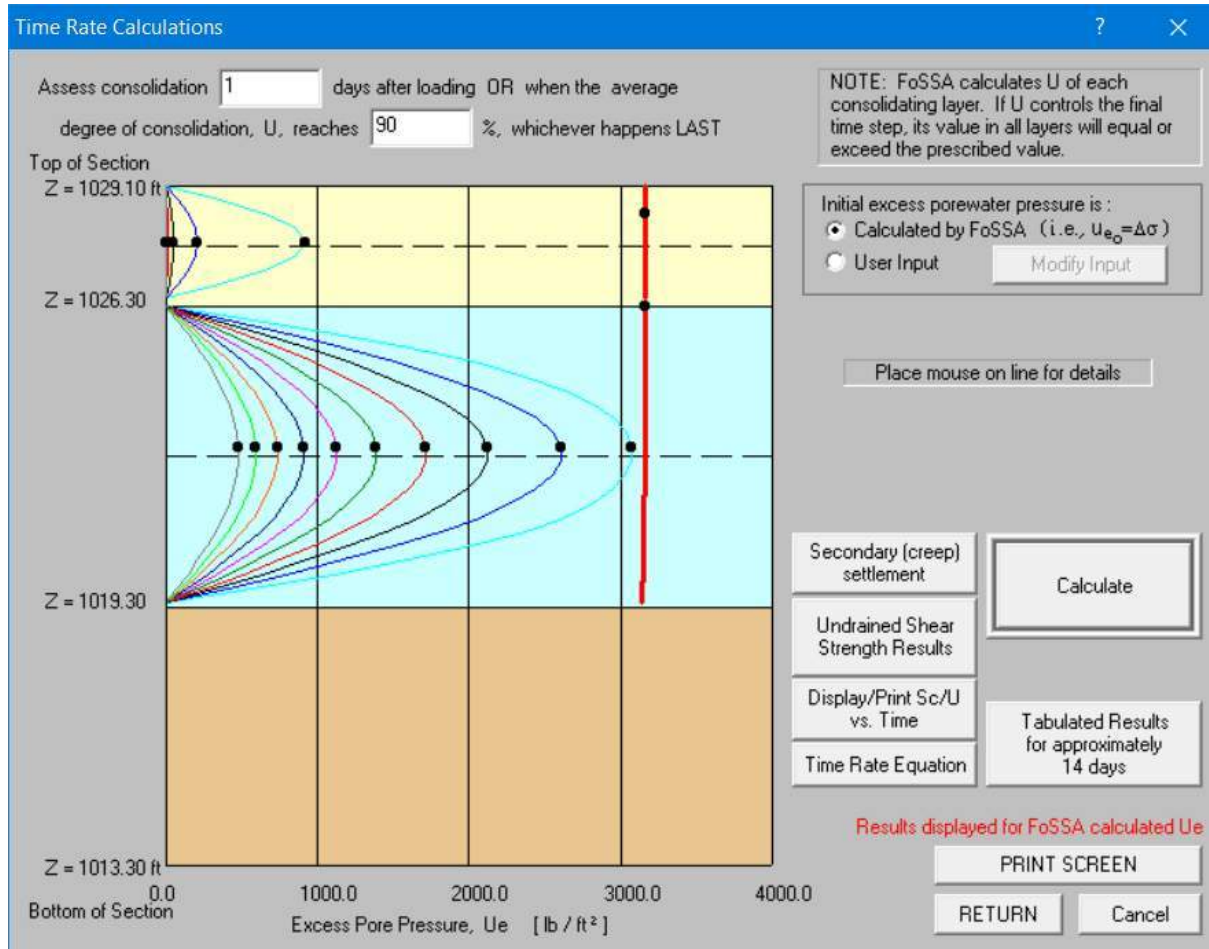
*Note: Final Z is calculated assuming only 'Ultimate Settlement' exists.

TABULATED GEOMETRY: INPUT OF FOUNDATION SOILS

Found. Soil #	Point #	Coordinates (X, Z) :		DESCRIPTION
		(X) [ft.]	(Z) [ft.]	
1	1	-62.50	1029.10	A-4b
	2	0.00	1029.10	
	3	62.50	1029.10	
2	1	-62.50	1026.30	A-4a
	2	0.00	1026.30	
	3	62.50	1026.30	
3	1	-62.50	1019.30	A-1-b
	2	0.00	1019.30	
	3	62.50	1019.30	
4	1	-62.50	1013.30	Bedrock
	2	0.00	1013.30	
	3	62.50	1013.30	

TABULATED GEOMETRY: INPUT OF EMBANKMENT SOILS

Embank. Soil #		Point #	Coordinates (X, Z) : (X) [ft.] (Z) [ft.]		DESCRIPTION
1	X1 = -35.00 [ft]	1	-35.00	1054.47	Embankment Fills
	X2 = 35.00 [ft]	2	35.00	1054.47	



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INPUT DATA – FOUNDATION LAYERS – 6 layers

	Wet Unit Weight, γ [lb/ft ³]	Poisson's Ratio μ	Description of Soil
1	112.00	0.35	A-4a
2	108.00	0.35	A-4b
3	122.00	0.30	A-6a
4	125.00	0.35	A-4a
5	130.00	0.35	A-6a
6	140.00	0.25	Bedrock

INPUT DATA – EMBANKMENT LAYERS – 1 layers

	Wet Unit Weight, γ [lb/ft ³]	Description of Soil
1	125.00	Embankment Fills

INPUT DATA OF WATER

Point #	Coordinates (X, Z) :	
	(X) [ft.]	(Z) [ft.]
1	0.00	1018.30
2	10.00	1018.30
3	25.00	1018.30
4	40.00	1018.30

License number FoSSA-200410

*Note: Final Z is calculated assuming only 'Immediate Settlement' exists.

[illegible]

IMMEDIATE SETTLEMENT, Si

Node #	Settlement along section:		Layer	Young's Modulus, E	Poisson's Ratio, μ	Settlement of each layer, Si(k)	Initial Z	Final Z *	Total Settlement Sum of Si(k),
	[ft.]	[ft.]	(k)	[lb/ft ²]		[ft.]	[ft.]	[ft.]	[ft.]
9	36.00	0.00	1	300000	0.3500	0.0101	1034.52	1034.48	0.04
			2	200000	0.3500	0.0118			
			3	600000	0.3000	0.0100			
			4	700000	0.3500	0.0042			
			5	900000	0.3500	0.0051			
			6	10000000	0.2500	0.0030			
10	48.00	0.00	1	300000	0.3500	-0.0050	1034.52	1034.52	0.00
			2	200000	0.3500	-0.0021			
			3	600000	0.3000	0.0020			
			4	700000	0.3500	0.0013			
			5	900000	0.3500	0.0022			
			6	10000000	0.2500	0.0021			
11	60.00	0.00	1	300000	0.3500	-0.0032	1034.52	1034.53	-0.01
			2	200000	0.3500	-0.0035			
			3	600000	0.3000	-0.0008			
			4	700000	0.3500	-0.0001			
			5	900000	0.3500	0.0004			
			6	10000000	0.2500	0.0014			

*Note: Final Z is calculated assuming only 'Immediate Settlement' exists.

[illegible]

ULTIMATE SETTLEMENT, S_c

Node #	X [ft.]	Y [ft.]	Original Z [ft.]	Settlement Sc [ft.]	Final Z * [ft.]
1	-60.00	0.00	1034.52	0.01	1034.51
2	-48.00	0.00	1034.52	0.03	1034.49
3	-36.00	0.00	1034.52	0.09	1034.43
4	-24.00	0.00	1034.52	0.25	1034.27
5	-12.00	0.00	1034.52	0.26	1034.26
6	0.00	0.00	1034.52	0.27	1034.25
7	12.00	0.00	1034.52	0.26	1034.26
8	24.00	0.00	1034.52	0.25	1034.27
9	36.00	0.00	1034.52	0.09	1034.43
10	48.00	0.00	1034.52	0.03	1034.49
11	60.00	0.00	1034.52	0.01	1034.51

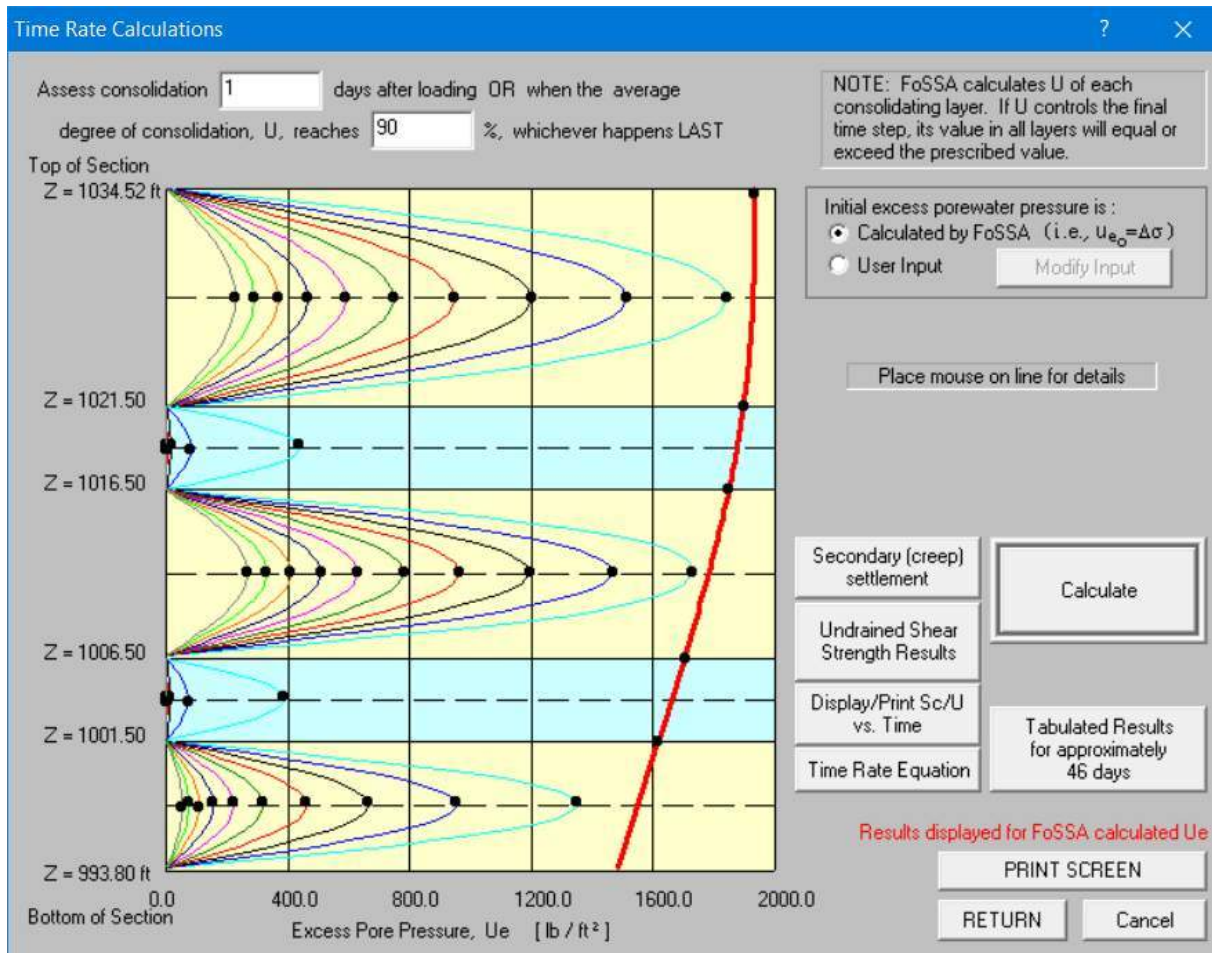
*Note: Final Z is calculated assuming only 'Ultimate Settlement' exists.

TABULATED GEOMETRY: INPUT OF FOUNDATION SOILS

Found. Soil #	Point #	Coordinates (X, Z) :		DESCRIPTION
		(X) [ft.]	(Z) [ft.]	
1	1	-62.50	1034.52	A-4a
	2	0.00	1034.52	
	3	62.50	1034.52	
2	1	-62.50	1021.50	A-4b
	2	0.00	1021.50	
	3	62.50	1021.50	
3	1	-62.50	1016.50	A-6a
	2	0.00	1016.50	
	3	62.50	1016.50	
4	1	-62.50	1006.50	A-4a
	2	0.00	1006.50	
	3	62.50	1006.50	
5	1	0.00	1001.50	A-6a
6	1	0.00	993.80	Bedrock

TABULATED GEOMETRY: INPUT OF EMBANKMENT SOILS

Embank. Soil #	Point #	Coordinates (X, Z) :		DESCRIPTION
		(X) [ft.]	(Z) [ft.]	
1	X1 = -35.00 [ft]	-35.00	1050.00	Embankment Fills
	X2 = 35.00 [ft]	35.00	1050.00	

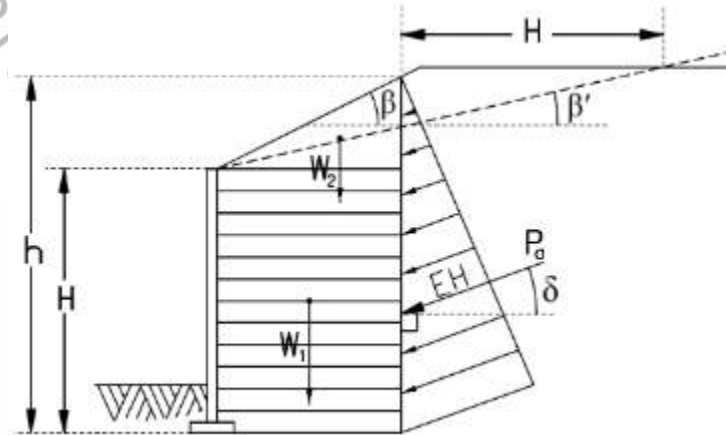


RETAINING WALL 3

Objective: To evaluate the external stability of MSE wall design with vertical wall face and broken backslope case.
Method: In accordance with ODOT Bridge Design Manual, 2026 [Sect. 307.4] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 3.11.5.8 and Sect. 11.10.5].

Assumptions:

- Horizontal backfill, Infinite backslope or Broken backslope behind MSE wall, see LRFD Sect. 3.11.5.8.1.
- For battered or vertical walls with a back face of wall angle of θ to horizontal.
- Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ .
- MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall



Givens:

Wall Geometry:

$$H_e := 25.4 \text{ ft}$$

$$\theta := 90 \cdot \text{deg}$$

$$\beta_{\text{BackSlope}} := 0 \text{ deg}$$

$$\lambda := 0 \text{ ft}$$

Exposed wall height (Proposed profile grade to bottom of leveling pad)

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Inclination of ground slope behind face of wall. If it is horizontal backfill behind MSE wall, $\beta_{\text{BackSlope}} = 0 \text{ deg}$

Horizontal distance from the back of MSE wall to the top of slope. If it is infinite slope behind MSE Wall, input λ larger than 2H; If it is horizontal backfill behind MSE wall, $\lambda = 0 \text{ ft}$

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \text{ deg}$$

$$\gamma_r := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_r := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \text{ deg}$$

$$\gamma_b := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_b := 0 \frac{\text{lbf}}{\text{ft}^2}$$

$$\delta := \frac{2}{3} \cdot \phi'_b$$

$$\delta = 20 \text{ deg}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Soil Design Parameters for Bearing Resistance (Average 1.5 B Below Leveling Pad)

Drained Conditions (Effective Stress):

$$\phi'_{fd} := 24 \text{ deg}$$

Effective angle of internal friction

$$\gamma_{fd} := 112 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} := 200 \frac{\text{lbf}}{\text{ft}^2}$$

Effective Cohesion

Undrained Conditions (Total Stress):

$$\phi_{fdu} := 0 \text{ deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$Su_{fdu} := 1850 \frac{\text{lbf}}{\text{ft}^2}$$

Undrained Shear Strength

Undercut & Replacement Design Parameters:

$$\phi_{Re} := 34 \text{ deg}$$

Angle of internal friction for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$c_{Re} := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$\gamma_{Re} := 130 \frac{\text{lbf}}{\text{ft}^3}$$

Unit Weight for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$\delta_{Re} := \frac{2}{3} \cdot \phi_{Re} \quad \delta_{Re} = 22.7 \text{ deg}$$

Friction angle between Replacement soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

$$D_{undercut} := 1 \text{ ft}$$

Depth of Undercut below bottom of footing

Foundation Surcharge Soil Parameters

$$\gamma_q := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Depth of Embedment Check:

$$d_{frost} := 3.25 \text{ ft}$$

$$d_{user} := 3.2 \text{ ft}$$

Local Frost Depth / User Input

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 3.3 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 28.7 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if}\left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if}\left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if}\left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5}\right)\right)\right)$$

$$d_{eSlope} = 1.4 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 3.3 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 28.7 \text{ ft}$$

Initial design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 0.8 \cdot H$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot ft, 0.7 \cdot H, L_{user})$$

$$L = 22.9 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := \text{if}\left(L < \lambda, 0 \frac{lbf}{ft^2}, 250 \frac{lbf}{ft^2}\right) = 250 \frac{lbf}{ft^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

$$SUR_{@MSE} := \text{if}\left(L < \lambda, 0 \frac{lbf}{ft^2}, 250 \frac{lbf}{ft^2}\right) = 250 \frac{lbf}{ft^2}$$

Live load surcharge above the MSE Wall soil reinforcement (per LRFD BDS [3.11.6.4])

Calculations:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan(\beta_{BackSlope})$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h := \text{if}\left(L < \lambda, H + L \cdot \tan(\beta_{BackSlope}), H + H_{BackSlope}\right)$$

Height of retained fill at the back of the reinforced soil

$$h = 28.7 \text{ ft}$$

$$I := \text{atan}\left(\frac{H_{BackSlope}}{2 \cdot H}\right)$$

$$\beta := \text{if}\left(\lambda < 2 \cdot H, I, \beta_{BackSlope}\right)$$

Angle of friction between retained backfill and reinforced soil

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}}\right)^2 = 2.6845$$

$$k_{ab} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))}\right)$$

$$k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot h^2 \cdot k_{ab}$$

$$F_T = 14642.5 \frac{lbf}{ft}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{ab}$$

$$F_{SUR} = 2129.5 \frac{lbf}{ft}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 78799 \frac{lbf}{ft}$$

Soil backfill - reinforced soil (EV)

$$V_2 := \text{if}\left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H)\right)$$

$$V_2 = 0 \frac{lbf}{ft}$$

Triangular Soil backfill - backslope soil (EV)

$$V_3 := \text{if}\left(L < \lambda, 0 \cdot \frac{lbf}{ft}, \gamma_b \cdot (L - \lambda) \cdot (h - H)\right)$$

$$V_3 = 0 \frac{lbf}{ft}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_4 := F_T \cdot \sin(\beta)$$

$$V_4 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_5 := F_{SUR} \cdot \sin(\beta)$$

$$V_5 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_6 := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_6 = 5730 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1} := \frac{L}{2}$$

$$d_{v1} = 11.5 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 903036.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := \text{if}\left(L < \lambda, \frac{2L}{3}, \frac{2 \cdot \lambda}{3}\right)$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v3} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v3} = 11.5 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3}$$

$$MV_3 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v4} := L$$

$$d_{v4} = 22.9 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4}$$

$$MV_4 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v5} := L$$

$$d_{v5} = 22.9 \text{ ft}$$

$$MV_5 := V_5 \cdot d_{v5}$$

$$MV_5 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{v6} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v6} = 11.5 \text{ ft}$$

$$MV_6 := V_6 \cdot d_{v6}$$

$$MV_6 = 65665.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T \cdot \cos(\beta) = 14642.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} \cdot \cos(\beta) = 2129.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

Moment:

$$d_{h1} := \frac{h}{3}$$

$$d_{h1} = 9.6 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 139836 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{2}$$

$$d_{h2} = 14.3 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 30505.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1 + V_2 + V_3$$

$$V_{EV} = 78799 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{EH} := V_4$$

$$V_{EH} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{LS} := V_5$$

$$V_{LS} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{LS@MSE} := V_6$$

$$V_{LS@MSE} = 5730 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 14642.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 2129.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1 + MV_2 + MV_3$$

$$M_{EV} = 903036.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_4$$

$$M_{EH1} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1} := MV_5$$

$$M_{LS1@MSE} := MV_6$$

$$M_{EH2} := MH_1$$

$$M_{LS2} := MH_2$$

$$M_{LS1} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE} = 65665.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} = 139836 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} = 30505.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1 \quad \text{LRFD Load Modifier}$$

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot ((Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE}))$$

$$V_{Ia} = 88826.5 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE}))$$

$$V_{Ib} = 116406.1 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{EH} \cdot H_{EH}) + (Ia_{LS} \cdot H_{LS}))$$

$$H_{Ia} = 25690.4 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{EH} \cdot H_{EH}) + (Ib_{LS} \cdot H_{LS}))$$

$$H_{Ib} = 25690.4 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot ((Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + Ia_{LS} \cdot (M_{LS1} + M_{LS1@MSE}))$$

$$MV_{Ia} = 1017951.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + Ib_{LS} \cdot (M_{LS1} + M_{LS1@MSE}))$$

$$MV_{Ib} = 1334013.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{EH} \cdot M_{EH2}) + (Ia_{LS} \cdot M_{LS2}))$$

$$MH_{Ia} = 263138.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{EH} \cdot M_{EH2}) + (Ib_{LS} \cdot M_{LS2}))$$

$$MH_{Ib} = 263138.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Calculations at Bottom of Foundation Preparation Layer:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan(\beta_{BackSlope})$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h_{FPL} := \text{if}(L < \lambda, H + L \cdot \tan(\beta_{BackSlope}) + 1 \text{ ft}, H + H_{BackSlope} + 1 \text{ ft})$$

$$h_{FPL} = 29.7 \text{ ft}$$

Height of retained fill at the back of the reinforced soil

$$I := \text{atan}\left(\frac{H_{BackSlope}}{2 \cdot H}\right)$$

$$\beta := \text{if}(\lambda < 2 \cdot H, I, \beta_{BackSlope})$$

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

Angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 2.6845$$

$$k_{ab} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_{T_FPL} := \frac{1}{2} \cdot \gamma_b \cdot h_{FPL}^2 \cdot k_{ab} \quad F_T = 14642.5 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR_FPL} := SUR \cdot h_{FPL} \cdot k_{ab} \quad F_{SUR} = 2129.5 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_{1_FPL} := \gamma_r \cdot H \cdot L \quad V_{1_FPL} = 78799 \frac{\text{lbf}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_{2_FPL} := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_{2_FPL} = 0 \frac{\text{lbf}}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_{3_FPL} := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lbf}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_{3_FPL} = 0 \frac{\text{lbf}}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_{4_FPL} := F_{T_FPL} \cdot \sin(\beta)$$

$$V_{4_FPL} = 0 \frac{\text{lbf}}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_{5_FPL} := F_{SUR_FPL} \cdot \sin(\beta)$$

$$V_{5_FPL} = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_{6_FPL} := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_{6_FPL} = 5730 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

$$V_{7_FPL} := \gamma_{Re} \cdot (L + 1 \text{ ft}) \cdot D_{undercut}$$

$$V_{7_FPL} = 3109.6 \frac{\text{lbf}}{\text{ft}}$$

Foundation Preparation Layer Weight

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1_FPL} := \frac{L}{2} + 1 \text{ ft}$$

$$d_{v1_FPL} = 12.5 \text{ ft}$$

$$MV_{1_FPL} := V_{1_FPL} \cdot d_{v1_FPL} \quad MV_{1_FPL} = 981835 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2_FPL} := \text{if} \left(L < \lambda, \frac{2L}{3}, \frac{2\lambda}{3} \right) + 1 \text{ ft}$$

$$d_{v2_FPL} = 1 \text{ ft}$$

$$MV_{2_FPL} := V_{2_FPL} \cdot d_{v2_FPL} \quad MV_{2_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v3_FPL} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right) + 1 \text{ ft} \quad d_{v3_FPL} = 12.5 \text{ ft}$$

$$MV_{3_FPL} := V_{3_FPL} \cdot d_{v3_FPL} \quad MV_{3_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v4_FPL} := L + 1 \text{ ft}$$

$$d_{v4_FPL} = 23.9 \text{ ft}$$

$$MV_{4_FPL} := V_{4_FPL} \cdot d_{v4_FPL} \quad MV_{4_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v5_FPL} := L + 1 \text{ ft}$$

$$d_{v5_FPL} = 23.9 \text{ ft}$$

$$MV_{5_FPL} := V_{5_FPL} \cdot d_{v5_FPL} \quad MV_{5_FPL} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v6_FPL} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right) + 1 \text{ ft} \quad d_{v6_FPL} = 12.5 \text{ ft}$$

$$MV_{6_FPL} := V_{6_FPL} \cdot d_{v6_FPL} \quad MV_{6_FPL} = 71395.8 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v7_FPL} := \frac{(L + 1 \text{ ft})}{2}$$

$$d_{v7_FPL} = 12 \text{ ft}$$

$$MV_{7_FPL} := V_{7_FPL} \cdot d_{v7_FPL} \quad MV_{7_FPL} = 37190.8 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_{1_FPL} := F_{T_FPL} \cdot \cos(\beta) = 15682.5 \frac{\text{lb}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_{2_FPL} := F_{SUR_FPL} \cdot \cos(\beta) = 2203.8 \frac{\text{lb}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1_FPL} := \frac{h_{FPL}}{3} \quad d_{h1_FPL} = 9.9 \text{ ft}$$

Moment:

$$MH_{1_FPL} := H_{1_FPL} \cdot d_{h1_FPL} \quad MH_{1_FPL} = 154995.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2_FPL} := \frac{h_{FPL}}{2} \quad d_{h2_FPL} = 14.8 \text{ ft}$$

$$MH_{2_FPL} := H_{2_FPL} \cdot d_{h2_FPL} \quad MH_{2_FPL} = 32671.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV_FPL} := V_{1_FPL} + V_{2_FPL} + V_{3_FPL} + V_{7_FPL}$$

$$V_{EV_FPL} = 81908.6 \frac{\text{lb}}{\text{ft}}$$

$$V_{EH_FPL} := V_{4_FPL}$$

$$V_{EH_FPL} = 0 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS_FPL} := V_{5_FPL}$$

$$V_{LS_FPL} = 0 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS@MSE_FPL} := V_{6_FPL}$$

$$V_{LS@MSE_FPL} = 5730 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH_FPL} := H_{1_FPL}$$

$$H_{EH_FPL} = 15682.5 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS_FPL} := H_{2_FPL}$$

$$H_{LS_FPL} = 2203.8 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV_FPL} := MV_{1_FPL} + MV_{2_FPL} + MV_{3_FPL} + MV_{7_FPL}$$

$$M_{EV_FPL} = 1019025.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1_FPL} := MV_{4_FPL}$$

$$M_{EH1_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1_FPL} := MV_{5_FPL}$$

$$M_{LS1_FPL} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE_FPL} := MV_{6_FPL}$$

$$M_{LS1@MSE_FPL} = 71395.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2_FPL} := MH_{1_FPL}$$

$$M_{EH2_FPL} = 154995.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2_FPL} := MH_{2_FPL}$$

$$M_{LS2_FPL} = 32671.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1$$

LRFD Load Modifier

Strength Limit State I:
EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia_FPL} := \eta \cdot \left((Ia_{EV} \cdot V_{EV_FPL}) + (Ia_{EH} \cdot V_{EH_FPL}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE_FPL}) \right) \quad V_{Ia_FPL} = 91936.1 \frac{lb}{ft}$$

$$V_{Ib_FPL} := \eta \cdot \left((Ib_{EV} \cdot V_{EV_FPL}) + (Ib_{EH} \cdot V_{EH_FPL}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE_FPL}) \right) \quad V_{Ib_FPL} = 120604.1 \frac{lb}{ft}$$

Compute Bearing Resistance:

$$H_{Ia_FPL} := \eta \cdot \left((Ia_{EH} \cdot H_{EH_FPL}) + (Ia_{LS} \cdot H_{LS_FPL}) \right) \quad H_{Ia_FPL} = 27380.5 \frac{lb}{ft}$$

$$H_{Ib_FPL} := \eta \cdot \left((Ib_{EH} \cdot H_{EH_FPL}) + (Ib_{LS} \cdot H_{LS_FPL}) \right) \quad H_{Ib_FPL} = 27380.5 \frac{lb}{ft}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia_FPL} := \eta \cdot \left((Ia_{EV} \cdot M_{EV_FPL}) + (Ia_{EH} \cdot M_{EH1_FPL}) + Ia_{LS} \cdot (M_{LS1_FPL} + M_{LS1@MSE_FPL}) \right) \quad MV_{Ia_FPL} = 1143968.5 \frac{lb \cdot ft}{ft}$$

$$MV_{Ib_FPL} := \eta \cdot \left((Ib_{EV} \cdot M_{EV_FPL}) + (Ib_{EH} \cdot M_{EH1_FPL}) + Ib_{LS} \cdot (M_{LS1_FPL} + M_{LS1@MSE_FPL}) \right) \quad MV_{Ib_FPL} = 1500627.6 \frac{lb \cdot ft}{ft}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia_FPL} := \eta \cdot \left((Ia_{EH} \cdot M_{EH2_FPL}) + (Ia_{LS} \cdot M_{LS2_FPL}) \right) \quad MH_{Ia_FPL} = 289669.2 \frac{lb \cdot ft}{ft}$$

$e = 2.72$

$$MH_{Ib_FPL} := \eta \cdot \left((Ib_{EH} \cdot M_{EH2_FPL}) + (Ib_{LS} \cdot M_{LS2_FPL}) \right) \quad MH_{Ib_FPL} = 289669.2 \frac{lb \cdot ft}{ft}$$

Factored Horizontal Loads by Limit State:

Compute the Effective Bearing Length (Strength Ib) about the Toe "O" of base length (the pivot):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 1334013.9 \frac{lb \cdot ft}{ft} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 263138.2 \frac{lb \cdot ft}{ft} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 116406.1 \frac{lb}{ft} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V} \quad x = 9.2 \text{ ft} \quad \text{Distance from Point "O" the resultant intersects the base}$$

$$e := \left| \frac{L}{2} - x \right| \quad e = 2.3 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B'_{top} := L - 2 \cdot e \quad B'_{top} = 18.4 \text{ ft} \quad \text{Effective Footing Width, at the top of Foundation Preparation Layer}$$

$$\Sigma M_{R_FPL} := MV_{lb_FPL}$$

$$\Sigma M_{R_FPL} = 1500627.6 \frac{lb \cdot ft}{ft}$$

Sum of Resisting Moments (Strength lb)

$$\Sigma M_{O_FPL} := MH_{lb_FPL}$$

$$\Sigma M_{O_FPL} = 289669.2 \frac{lb \cdot ft}{ft}$$

Sum of Overturning Moments (Strength lb)

$$\Sigma V_{FPL} := V_{lb_FPL}$$

$$\Sigma V_{FPL} = 120604.1 \frac{lb}{ft}$$

Sum of Vertical Loads (Strength lb)

$$x_{FPL} := \frac{(\Sigma M_{R_FPL} - \Sigma M_{O_FPL})}{\Sigma V_{FPL}}$$

$$x_{FPL} = 10 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e_{FPL} := \left| \frac{(L + 1 \text{ ft})}{2} - x_{FPL} \right|$$

$$e_{FPL} = 1.9 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B'_{bottom} := (L + 1 \text{ ft}) - 2 \cdot e_{FPL}$$

$$B'_{bottom} = 20.1 \text{ ft}$$

Effective Footing Width, at the bottom of Foundation Preparation Layer (FPL)

Foundation Layout:

$$L_{Wall} := 125 \text{ ft}$$

Assumed Footing Length (Wall Section Length)

$$D_f := d_e$$

$$D_f = 3.3 \text{ ft}$$

Footing embedment

$$D_F := D_f + D_{undercut}$$

Embenment Depth at bottom of Undercut

$$d_w := D_f - 1 \text{ ft}$$

Depth of Groundwater below ground surface in front of wall

Compute Bearing Resistance at the top of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 9.6$$

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

$$N_c = 19.32$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 9.4$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.073$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \cdot \tan(\phi'_{fd}) \right), 1 \right)$$

$$s_q = 1.066$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.941$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.07$$

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

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

$$N_{cm} = 20.737$$

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

$$N_{qm} = 10.961$$

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

$$N_{\gamma m} = 8.886$$

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

$$q_{nd_top} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad \text{at top of foundation preparation layer}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_top} := \phi_b \cdot q_{nd_top} \quad q_{Rd_top} = 7.1 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

$$s_c := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.029$$

$$s_q := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \tan(\phi_{fdu}), 1\right)$$

$$s_q = 1$$

$$s_\gamma := \text{if}\left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}}\right), 1\right)$$

$$s_\gamma = 1$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \tan\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.07$$

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

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

$$N_{cm} = 5.291$$

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

$$N_{qm} = 1.071$$

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

$$N_{\gamma m} = 0$$

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

$$q_{nu_top} := Su_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nu_top} = 9997.8 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_top} := \phi_b \cdot q_{nu_top} \quad q_{Ru_top} = 6.5 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_top} = 7.1 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_top} = 6.5 \text{ ksf}$$

Compute Bearing Resistance at the bottom of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 9.6$$

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

$$N_c = 19.32$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 9.4$$

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

$$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.08$$

$$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \tan(\phi'_{fd}), 1\right)$$

$$s_q = 1.072$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.936$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right)$$

$$d_q = 1.07$$

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

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

$$N_{cm} = 20.866$$

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

$$N_{qm} = 10.963$$

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

$$N_{\gamma m} = 8.835$$

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

$$q_{nd_bottom} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

at bottom of foundation preparation layer

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_bottom} := \phi_b \cdot q_{nd_bottom} \quad q_{Rd_bottom} = 7.8 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.032$$

$$s_q := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \cdot \tan(\phi_{fdu}) \right), 1 \right) \quad s_q = 1$$

$$s_\gamma := \text{if} \left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right) \quad s_\gamma = 1$$

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

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

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

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

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right) \quad e$$

$$d_q = 1.07$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 5.305$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 1.065$$

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

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

$$q_{nu_bottom} := Su_{fdu} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu_top} = 9997.8 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_bottom} := \phi_b \cdot q_{nu_top} \quad q_{Ru_bottom} = 6.5 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_bottom} = 7.8 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_bottom} = 6.5 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location about the toe "O" of base length (the pivot):

$$e := \left| \frac{L}{2} - x \right| \quad e = 2.26 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2]. The effective bearing width is equal to B-2e.

$$\sigma_v := \frac{\Sigma V}{B'_{top}} \quad \sigma_v = 6326.8 \frac{\text{lb}}{\text{ft}^2} \quad \text{Bearing Stress}$$

$$\sigma_{v_FPL} := \frac{\Sigma V_{FPL}}{B'_{bottom}} \quad \sigma_{v_FPL} = 6005.7 \frac{\text{lb}}{\text{ft}^2} \quad \text{Bearing Stress}$$

Bearing Resistance Capacity:Demand Ratio (CDR)

At the top of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_top} := \frac{q_{Rd_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_top} = 1.12$

Undrained Conditions: $CDR_{Bearing_u_top} := \frac{q_{Ru_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_top} = 1.03$

At the bottom of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_bottom} := \frac{q_{Rd_bottom}}{\sigma_{v_FPL}}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_bottom} = 1.29$

Undrained Conditions: $CDR_{Bearing_u_bottom} := \frac{q_{Ru_bottom}}{\sigma_{v_FPL}}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_bottom} = 1.08$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$e_{max} := \frac{L}{3}$

$e_{max} = 7.6 \text{ ft}$

Maximum Eccentricity **LRFD [C11.6.3.3.]**

$\Sigma M_R := MV_{Ia}$

$\Sigma M_R = 1017951.2 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$

Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$

$\Sigma M_O = 263138.2 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$

Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$

$\Sigma V = 88826.5 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$

$x = 8.5 \text{ ft}$

Distance from Point "O" the resultant intersects the base

$e := \left| \frac{L}{2} - x \right|$

$e = 2.96 \text{ ft}$

Wall eccentricity

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e}$

Is the CDR > or = to 1.0?

$CDR_{Eccentricity} = 2.58$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$

$R_u = 25690.4 \frac{\text{lb}}{\text{ft}}$

At the top of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$\Sigma V := V_{Ia}$

$\Sigma V = 88826.5 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$R_{td_top} := \Sigma V \cdot \tan(\phi_{Re})$

$R_{td_top} = 59914.2 \frac{\text{lb}}{\text{ft}}$

Nominal sliding resistance Drained Conditions

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$\phi_\tau := 1.0$

Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

Drained Conditions:

$\phi R_{n_d_top} := \phi_\tau \cdot R_{td_top}$

$R_{R_d_top} := \phi R_{n_d_top}$

$R_{R_d_top} = 59.9 \frac{\text{kip}}{\text{ft}}$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_top} := \frac{R_{R_d_top}}{R_u} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_d_top} = 2.33$$

At the bottom of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V_{FPL} := V_{Ia_FPL} \quad \Sigma V_{FPL} = 91936.1 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$R_{td_bottom} := \Sigma V_{FPL} \cdot \tan(\phi'_{fd}) \quad R_{td_bottom} = 40932.6 \frac{\text{lb}}{\text{ft}} \quad \text{Nominal sliding resistance Drained Conditions}$$

Compute factored resistance against failure by sliding LRFD [10.6.3.4]:

$$\phi_r := 1.0$$

Resistance factor for sliding resistance specified in LRFD Table 11.5.7-1.

$$\text{Drained Conditions:} \quad \phi R_{n_d_bottom} := \phi_r \cdot R_{td_bottom} \quad R_{R_d_bottom} := \phi R_{n_d_bottom} \quad R_{R_d_bottom} = 40.9 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_bottom} := \frac{R_{R_d_bottom}}{H_{Ia_FPL}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_d_bottom} = 1.49$$

Undrained Conditions (Total Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V_{FPL} := V_{Ia_FPL} \quad \Sigma V_{FPL} = 91936.1 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$B := L + 1 \text{ ft} \quad \text{Footing base width}$$

$$\sigma_{v_FPL} := \frac{\Sigma V_{FPL}}{B} \quad \sigma_{v_FPL} = 3843.5 \frac{\text{lb}}{\text{ft}^2} \quad \text{Vertical stress ODOT GDM [1504]}$$

$$q_{FPL} := \frac{1}{2} \cdot \sigma_{v_FPL} \quad q_{FPL} = 1921.7 \frac{\text{lb}}{\text{ft}^2} \quad \text{Uniform unit shear resistance as 1/2 vertical stress ODOT GDM [1504]}$$

Determine which Cohesive Soil Resistance Case is Present:

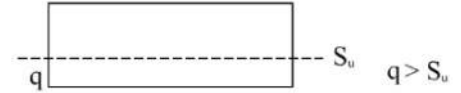
$$Case_1 := \text{if}(q_{FPL} > Su_{fdu}, 1, 0) \quad Case_1 = 1$$

$$Case_2 := \text{if}(Su_{fdu} \geq q_{FPL}, 1, 0) \quad Case_2 = 0$$

Unit Shear Resistance for Case 1:

$$R_{\tau_case1} := Su_{fdu} \cdot B = 44252 \frac{lb}{ft}$$

Case 1:



Unit Shear Resistance for Case 2:

$$R_{\tau_case2} := q_{FPL} \cdot B = 45968 \frac{lb}{ft}$$

Case 2:



Define the Applicable Case:

$$R_{\tau} := R_{\tau_case1}$$

$$R_{\tau} = 44252 \frac{lb}{ft}$$

Nominal sliding resistance Cohesive Soils

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_{ep} := 0.5$$

Resistance factor for passive resistance specified in
LRFD Table 10.5.5.2.2-1

$$\phi_{\tau} := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

Factored Sliding Resistance to be used in CDR Calculations:

$$R_R = 44252 \frac{lb}{ft}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_u_bottom} := \frac{R_R}{H_{Ia_FPL}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Sliding_u_bottom} = 1.62$$

RW3 R-003-0-99:

Active Earth Pressure Coefficient:

$$\phi'_f := 30 \text{ deg}$$

Effective angle of internal friction

$$\delta := 0 \cdot \phi'_f$$

$\delta = 0$ (degrees) based on ODOT
GDM 1501.1.2 for nongravity
cantilever walls

$$\theta := 90 \cdot \text{deg}$$

Angle of back face of wall to horizontal

$$\beta := 26.565 \text{ deg}$$

Inclination of ground slope behind
face of wall. Horizontal backfill
behind CIP wall, 0 degree

$$\gamma := 120 \text{ pcf}$$

Unit Weight of backfill

$$K := \left(1 + \sqrt{\frac{(\sin(\phi'_f + \delta) \cdot \sin(\phi'_f - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 1.4$$

Active earth pressure coefficient
(LRFD Sect. 3.11.5.3.)

$$K_{af} := \left(\frac{(\sin(\theta + \phi'_f))^2}{(K \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) = 0.536$$

Triangular Distribution Load:

$$H := 14 \text{ ft}$$

Exposed height of soil 1046.41ft
1032.41ft

$$CTC := 10 \text{ ft}$$

Center-to-center spacing between shafts

$$\text{Surcharge} := 0 \text{ psf}$$

Live load surcharge

$$EH := K_{af} \cdot \gamma \cdot H \cdot CTC = 750.256 \frac{\text{lbf}}{\text{in}}$$

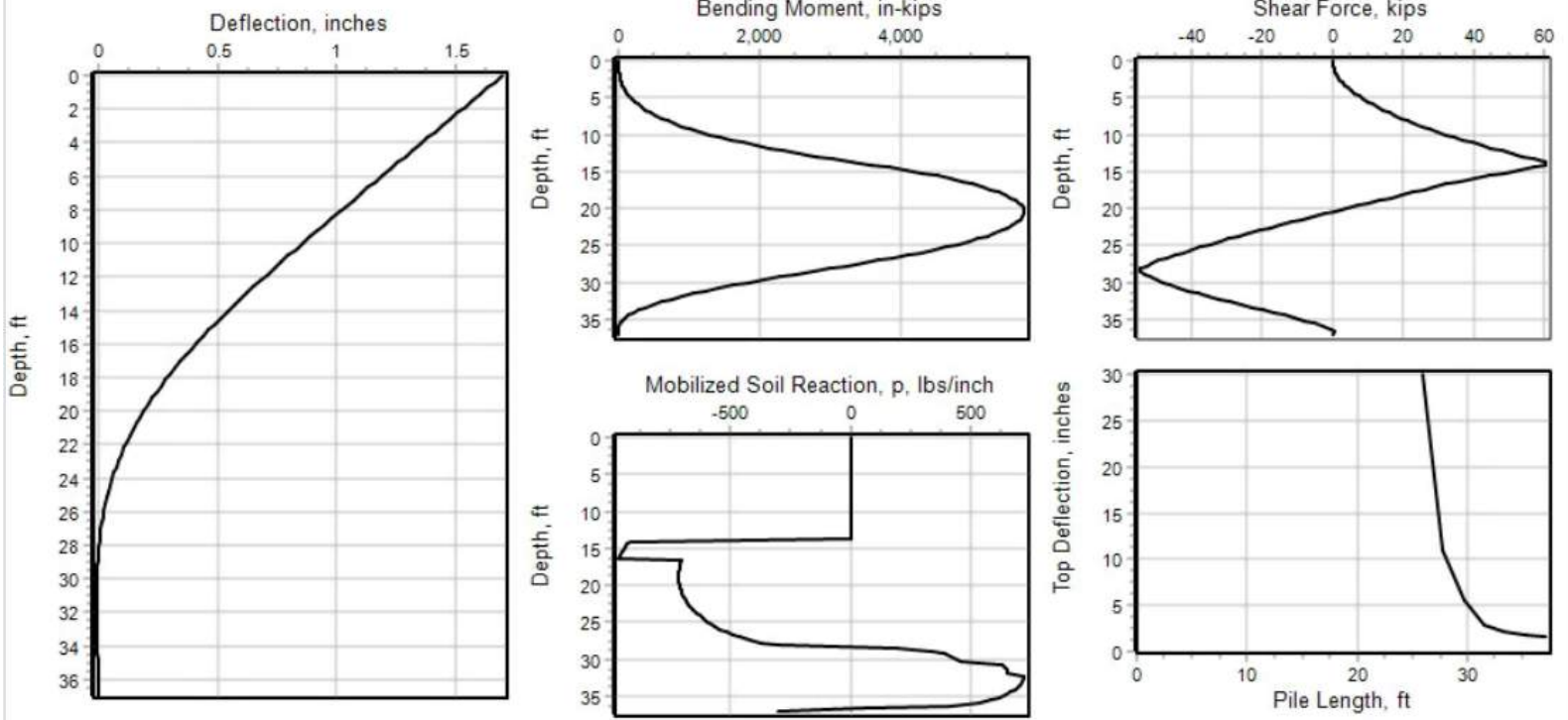
$$\text{FactEH} := 1.5 \cdot EH = (1.125 \cdot 10^3) \frac{\text{lbf}}{\text{in}}$$

$$LL_H := K_{af} \cdot \text{Surcharge} \cdot CTC = 0 \frac{\text{lbf}}{\text{in}}$$

$$\text{FactLL}_H := 1.75 \cdot LL_H = 0 \frac{\text{lbf}}{\text{in}}$$

$$\text{ServiceI} := EH + LL_H = 750.256 \frac{\text{lbf}}{\text{in}}$$

$$\text{StrengthI} := \text{FactEH} + \text{FactLL}_H = (1.125 \cdot 10^3) \frac{\text{lbf}}{\text{in}}$$



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LPIle for Windows, Version 2016-09.003

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Files Used for Analysis

Path to file locations:

\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\geotechnical\retaining
walls\Retaining Wall X (LocationDescription)\analysis\RW3\Lpile\Files\

Name of input data file:

RW 3_STA. 1+25_D=3.5ft_W24x162_Service I_R-003-0-99.lp9d

Name of output report file:

RW 3_STA. 1+25_D=3.5ft_W24x162_Service I_R-003-0-99.lp9o

Name of plot output file:

RW 3_STA. 1+25_D=3.5ft_W24x162_Service I_R-003-0-99.lp9p

Name of runtime message file:

RW 3_STA. 1+25_D=3.5ft_W24x162_Service I_R-003-0-99.lp9r

Date and Time of Analysis

Date: April 23, 2026

Time: 12:52:05

Problem Title

Project Name: SUM-77 SR-8

Job Number: RW3 Soldier Pile Lagging Wall

Client: ODOT / Arcadis

Engineer: ZM

Description: STA. 1+25

Program Options and Settings

Computational Options:

- Use unfactored loads in computations (conventional analysis)

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- | | | |
|--|---|---------------|
| - Maximum number of iterations allowed | = | 100 |
| - Deflection tolerance for convergence | = | 1.0000E-05 in |
| - Maximum allowable deflection | = | 100.0000 in |
| - Number of pile increments | = | 100 |

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Analysis uses p-y modification factors for p-y curves
- Analysis includes loading by one distributed lateral load acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 2
Total length of pile = 37.000 ft
Depth of ground surface below top of pile = 14.0000 ft

Pile diameters used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	13.0000
2	14.000	13.0000
3	14.000	42.0000
4	37.000	42.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile
Cross-sectional Shape = Strong H-Pile
Length of section = 14.000000 ft
Flange Width = 13.000000 in
Section Depth = 25.000000 in
Flange Thickness = 0.0000 in
Web Thickness = 0.0000 in
Section Area = 47.700000 sq. in
Moment of Inertia = 5170. in^4
Elastic Modulus = 29000000. psi

Pile Section No. 2:

Section 2 is an elastic pile

Cross-sectional Shape	=	Circular Pile
Length of section	=	23.000000 ft
Width of top of section	=	42.000000 in
Width of bottom of section	=	42.000000 in
Top Area	=	47.700000 sq. in
Bottom Area	=	47.700000 sq. in
Moment of Inertia at Top	=	5170. in^4
Moment of Inertia at Bottom	=	5170. in^4
Elastic Modulus	=	29000000. psi

Ground Slope and Pile Batter Angles

Ground Slope Angle	=	0.000 degrees
	=	0.000 radians
Pile Batter Angle	=	0.000 degrees
	=	0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is stiff clay without free water

Distance from top of pile to top of layer	=	14.000000 ft
Distance from top of pile to bottom of layer	=	16.410000 ft

Effective unit weight at top of layer	=	112.000000 pcf
Effective unit weight at bottom of layer	=	112.000000 pcf
Undrained cohesion at top of layer	=	2350. psf
Undrained cohesion at bottom of layer	=	2350. psf
Epsilon-50 at top of layer	=	0.005700
Epsilon-50 at bottom of layer	=	0.005700

Layer 2 is stiff clay without free water

Distance from top of pile to top of layer	=	16.410000 ft
Distance from top of pile to bottom of layer	=	30.410000 ft
Effective unit weight at top of layer	=	112.000000 pcf
Effective unit weight at bottom of layer	=	112.000000 pcf
Undrained cohesion at top of layer	=	1750. psf
Undrained cohesion at bottom of layer	=	1750. psf
Epsilon-50 at top of layer	=	0.006700
Epsilon-50 at bottom of layer	=	0.006700

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer	=	30.410000 ft
Distance from top of pile to bottom of layer	=	46.410000 ft
Effective unit weight at top of layer	=	112.000000 pcf
Effective unit weight at bottom of layer	=	112.000000 pcf
Undrained cohesion at top of layer	=	2350. psf
Undrained cohesion at bottom of layer	=	2350. psf
Epsilon-50 at top of layer	=	0.005700
Epsilon-50 at bottom of layer	=	0.005700

Layer 4 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	46.410000 ft
Distance from top of pile to bottom of layer	=	50.000000 ft
Effective unit weight at top of layer	=	146.600000 pcf
Effective unit weight at bottom of layer	=	146.600000 pcf

Uniaxial compressive strength at top of layer	=	390.000000	psi
Uniaxial compressive strength at bottom of layer	=	390.000000	psi
Initial modulus of rock at top of layer	=	35000.	psi
Initial modulus of rock at bottom of layer	=	35000.	psi
RQD of rock at top of layer	=	0.0000	%
RQD of rock at bottom of layer	=	0.0000	%
k rm of rock at top of layer	=	0.0005000	
k rm of rock at bottom of layer	=	0.0005000	

(Depth of the lowest soil layer extends 13.000 ft below the pile tip)

**** Warning - Possible Input Data Error ****

Values entered for effective unit weights of soil were outside the limits of 20 pcf to 140 pcf.

The maximum input value for effective unit weight = 146.60 pcf

This data may be erroneous. Please check your data.

Summary of Input Soil Properties							
Layer	Soil Type	Layer	Effective	Undrained	Uniaxial		E50
Rock Mass							
Layer	Name	Depth	Unit Wt.	Cohesion	qu	RQD %	or
Modulus							
Num.	(p-y Curve Type)	ft	pcf	psf	psi		krm
psi							
1	Stiff Clay	14.0000	112.0000	2350.	--	--	0.00570
--							

--	w/o Free Water	16.4100	112.0000	2350.	--	--	0.00570
2	Stiff Clay	16.4100	112.0000	1750.	--	--	0.00670
--	w/o Free Water	30.4100	112.0000	1750.	--	--	0.00670
3	Stiff Clay	30.4100	112.0000	2350.	--	--	0.00570
--	w/o Free Water	46.4100	112.0000	2350.	--	--	0.00570
4	Weak	46.4100	146.6000	--	390.0000	0.00	5.00E-04
35000.	Rock	50.0000	146.6000	--	390.0000	0.00	5.00E-04
35000.							

p-y Modification Factors for Group Action

Distribution of p-y modifiers with depth defined using 2 points

Point No.	Depth X ft	p-mult	y-mult
1	14.000	0.9100	1.0000
2	32.110	0.9100	1.0000

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Distributed Lateral Loading Used For All Load Cases

Distributed lateral load intensity defined using 2 points

Point No.	Depth X in	Dist. Load lb/in
-----	-----	-----
1	0.000	0.000
2	168.000	750.000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length
-----	-----	-----	-----	-----	-----
1	1	V = 0.0000 lbs	M = 0.0000 in-lbs	0.0000000	Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

 Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Pile Section No. 2:

Moment-curvature properties were derived from elastic section properties

 Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	14.0000	0.00	N.A.	No	0.00	63852.
2	16.4100	2.4100	Yes	No	63852.	424590.
3	30.4100	16.4100	Yes	No	488443.	365969.
4	46.4100	32.4100	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays,

2.5900	1.4775	22224.	2167.	-0.00704	27.9412	1.50E+11	0.00	0.00
138.7500								
2.9600	1.4463	33214.	2827.	-0.00704	41.7583	1.50E+11	0.00	0.00
158.5714								
3.3300	1.4150	47330.	3575.	-0.00704	59.5056	1.50E+11	0.00	0.00
178.3929								
3.7000	1.3838	64962.	4411.	-0.00704	81.6743	1.50E+11	0.00	0.00
198.2143								
4.0700	1.3525	86503.	5335.	-0.00704	108.7558	1.50E+11	0.00	0.00
218.0357								
4.4400	1.3213	112341.	6348.	-0.00703	141.2413	1.50E+11	0.00	0.00
237.8571								
4.8100	1.2901	142869.	7448.	-0.00703	179.6220	1.50E+11	0.00	0.00
257.6786								
5.1800	1.2589	178476.	8636.	-0.00702	224.3894	1.50E+11	0.00	0.00
277.5000								
5.5500	1.2277	219554.	9912.	-0.00702	276.0345	1.50E+11	0.00	0.00
297.3214								
5.9200	1.1965	266493.	11276.	-0.00701	335.0488	1.50E+11	0.00	0.00
317.1429								
6.2900	1.1654	319684.	12728.	-0.00700	401.9235	1.50E+11	0.00	0.00
336.9643								
6.6600	1.1344	379518.	14268.	-0.00699	477.1498	1.50E+11	0.00	0.00
356.7857								
7.0300	1.1033	446385.	15896.	-0.00698	561.2190	1.50E+11	0.00	0.00
376.6071								
7.4000	1.0724	520677.	17612.	-0.00697	654.6225	1.50E+11	0.00	0.00
396.4286								
7.7700	1.0415	602783.	19417.	-0.00695	757.8514	1.50E+11	0.00	0.00
416.2500								
8.1400	1.0107	693096.	21309.	-0.00693	871.3971	1.50E+11	0.00	0.00
436.0714								
8.5100	0.9800	792005.	23289.	-0.00691	995.7508	1.50E+11	0.00	0.00
455.8929								
8.8800	0.9493	899901.	25357.	-0.00688	1131.	1.50E+11	0.00	0.00
475.7143								
9.2500	0.9189	1017176.	27513.	-0.00685	1279.	1.50E+11	0.00	0.00
495.5357								

9.6200	0.8885	1144219.	29757.	-0.00682	1439.	1.50E+11	0.00	0.00
515.3571								
9.9900	0.8583	1281421.	32090.	-0.00679	1611.	1.50E+11	0.00	0.00
535.1786								
10.3600	0.8282	1429174.	34510.	-0.00675	1797.	1.50E+11	0.00	0.00
555.0000								
10.7300	0.7984	1587868.	37018.	-0.00670	1996.	1.50E+11	0.00	0.00
574.8214								
11.1000	0.7687	1757894.	39614.	-0.00665	2210.	1.50E+11	0.00	0.00
594.6429								
11.4700	0.7393	1939643.	42298.	-0.00660	2439.	1.50E+11	0.00	0.00
614.4643								
11.8400	0.7101	2133504.	45071.	-0.00654	2682.	1.50E+11	0.00	0.00
634.2857								
12.2100	0.6813	2339870.	47931.	-0.00647	2942.	1.50E+11	0.00	0.00
654.1071								
12.5800	0.6527	2559131.	50879.	-0.00640	3217.	1.50E+11	0.00	0.00
673.9286								
12.9500	0.6244	2791677.	53915.	-0.00632	3510.	1.50E+11	0.00	0.00
693.7500								
13.3200	0.5966	3037899.	57040.	-0.00623	3819.	1.50E+11	0.00	0.00
713.5714								
13.6900	0.5691	3298189.	60252.	-0.00614	4147.	1.50E+11	0.00	0.00
733.3929								
14.0600	0.5421	3572936.	60406.	-0.00604	14513.	1.50E+11	-916.1980	7504.
252.2472								
14.4300	0.5155	3834594.	56876.	-0.00593	15576.	1.50E+11	-925.9381	7975.
0.00								
14.8000	0.4894	4077999.	52745.	-0.00581	16564.	1.50E+11	-934.9112	8481.
0.00								
15.1700	0.4639	4302974.	48576.	-0.00569	17478.	1.50E+11	-943.1024	9026.
0.00								
15.5400	0.4389	4509356.	44372.	-0.00556	18317.	1.50E+11	-950.4961	9614.
0.00								
15.9100	0.4146	4697001.	40138.	-0.00542	19079.	1.50E+11	-957.0761	10250.
0.00								
16.2800	0.3908	4865778.	35875.	-0.00528	19764.	1.50E+11	-962.8260	10938.
0.00								

16.6500	0.3677	5015575.	32183.	-0.00513	20373.	1.50E+11	-700.6587	8460.
0.00								
17.0200	0.3453	5151559.	29063.	-0.00498	20925.	1.50E+11	-704.5873	9061.
0.00								
17.3900	0.3235	5273653.	25927.	-0.00483	21421.	1.50E+11	-707.8430	9715.
0.00								
17.7600	0.3024	5381793.	22779.	-0.00467	21860.	1.50E+11	-710.4107	10430.
0.00								
18.1300	0.2820	5475928.	19620.	-0.00451	22243.	1.50E+11	-712.2749	11213.
0.00								
18.5000	0.2624	5556022.	16455.	-0.00434	22568.	1.50E+11	-713.4192	12073.
0.00								
18.8700	0.2435	5622052.	13287.	-0.00418	22836.	1.50E+11	-713.8265	13018.
0.00								
19.2400	0.2253	5674009.	10118.	-0.00401	23047.	1.50E+11	-713.4788	14062.
0.00								
19.6100	0.2078	5711902.	6953.	-0.00384	23201.	1.50E+11	-712.3569	15218.
0.00								
19.9800	0.1911	5735751.	3794.	-0.00367	23298.	1.50E+11	-710.4403	16502.
0.00								
20.3500	0.1752	5745595.	645.9809	-0.00350	23338.	1.50E+11	-707.7073	17933.
0.00								
20.7200	0.1600	5741487.	-2488.	-0.00333	23321.	1.50E+11	-704.1338	19535.
0.00								
21.0900	0.1456	5723498.	-5605.	-0.00316	23248.	1.50E+11	-699.6942	21334.
0.00								
21.4600	0.1319	5691716.	-8700.	-0.00299	23119.	1.50E+11	-694.3598	23365.
0.00								
21.8300	0.1190	5646246.	-11769.	-0.00283	22934.	1.50E+11	-688.0988	25668.
0.00								
22.2000	0.1068	5587211.	-14808.	-0.00266	22695.	1.50E+11	-680.8757	28294.
0.00								
22.5700	0.09540	5514753.	-17813.	-0.00250	22400.	1.50E+11	-672.6496	31305.
0.00								
22.9400	0.08468	5429035.	-20779.	-0.00233	22052.	1.50E+11	-663.3739	34781.
0.00								
23.3100	0.07468	5330239.	-23701.	-0.00217	21651.	1.50E+11	-652.9936	38824.
0.00								

23.6800 0.00	0.06537	5218571.	-26575.	-0.00202	21197.	1.50E+11	-641.4434	43565.
24.0500 0.00	0.05676	5094257.	-29394.	-0.00187	20692.	1.50E+11	-628.6442	49179.
24.4200 0.00	0.04881	4957550.	-32154.	-0.00172	20137.	1.50E+11	-614.4981	55902.
24.7900 0.00	0.04151	4808730.	-34848.	-0.00157	19533.	1.50E+11	-598.8814	64058.
25.1600 0.00	0.03485	4648103.	-37468.	-0.00143	18880.	1.50E+11	-581.6330	74111.
25.5300 0.00	0.02879	4476011.	-40008.	-0.00130	18181.	1.50E+11	-562.5370	86747.
25.9000 0.00	0.02333	4292828.	-42459.	-0.00117	17437.	1.50E+11	-541.2931	103025.
26.2700 0.00	0.01843	4098975.	-44809.	-0.00104	16650.	1.50E+11	-517.4636	124679.
26.6400 0.00	0.01407	3894921.	-47047.	-9.25E-04	15821.	1.50E+11	-490.3719	154784.
27.0100 0.00	0.01022	3681200.	-49154.	-8.12E-04	14953.	1.50E+11	-458.8854	199412.
27.3800 0.00	0.00685	3458432.	-51107.	-7.07E-04	14048.	1.50E+11	-420.8695	272710.
27.7500 0.00	0.00394	3227368.	-52866.	-6.08E-04	13109.	1.50E+11	-371.4324	418375.
28.1200 0.00	0.00146	2988981.	-54342.	-5.16E-04	12141.	1.50E+11	-293.4554	894994.
28.4900 0.00	-6.37E-04	2744810.	-54558.	-4.31E-04	11149.	1.50E+11	196.4005	1368528.
28.8600 0.00	-0.00237	2504510.	-53367.	-3.53E-04	10173.	1.50E+11	339.7040	636596.
29.2300 0.00	-0.00377	2270907.	-51755.	-2.82E-04	9224.	1.50E+11	386.4455	454872.
29.6000 0.00	-0.00488	2044922.	-49971.	-2.18E-04	8306.	1.50E+11	417.2146	379885.
29.9700 0.00	-0.00571	1827162.	-48070.	-1.61E-04	7422.	1.50E+11	439.3844	341560.
30.3400 0.00	-0.00631	1618063.	-46082.	-1.10E-04	6572.	1.50E+11	455.8840	320948.

30.7100	-0.00669	1417952.	-43678.	-6.51E-05	5760.	1.50E+11	626.9637	416161.
0.00								
31.0800	-0.00688	1230201.	-40868.	-2.59E-05	4997.	1.50E+11	638.7086	411894.
0.00								
31.4500	-0.00692	1055041.	-38015.	7.93E-06	4285.	1.50E+11	646.7107	414997.
0.00								
31.8200	-0.00681	892630.	-35133.	3.68E-05	3626.	1.50E+11	651.4365	424445.
0.00								
32.1900	-0.00659	743061.	-32093.	6.10E-05	3018.	1.50E+11	717.7909	483424.
0.00								
32.5600	-0.00627	607642.	-28909.	8.10E-05	2468.	1.50E+11	716.6583	507257.
0.00								
32.9300	-0.00587	486351.	-25736.	9.72E-05	1976.	1.50E+11	712.5679	538672.
0.00								
33.3000	-0.00541	379107.	-22588.	1.10E-04	1540.	1.50E+11	705.5231	579042.
0.00								
33.6700	-0.00490	285772.	-19478.	1.20E-04	1161.	1.50E+11	695.4213	630588.
0.00								
34.0400	-0.00435	206146.	-16420.	1.27E-04	837.3445	1.50E+11	682.0286	696848.
0.00								
34.4100	-0.00377	139966.	-13429.	1.32E-04	568.5255	1.50E+11	664.9258	783605.
0.00								
34.7800	-0.00317	86893.	-10525.	1.36E-04	352.9501	1.50E+11	643.4041	900852.
0.00								
35.1500	-0.00256	46504.	-7728.	1.38E-04	188.8951	1.50E+11	616.2498	1067444.
0.00								
35.5200	-0.00195	18264.	-5070.	1.39E-04	74.1860	1.50E+11	581.2491	1323932.
0.00								
35.8900	-0.00133	1482.	-2595.	1.39E-04	6.0202	1.50E+11	533.8101	1778120.
0.00								
36.2600	-7.16E-04	-4776.	-500.2086	1.39E-04	19.4010	1.50E+11	409.6185	2538778.
0.00								
36.6300	-1.00E-04	-2960.	537.8766	1.39E-04	12.0222	1.50E+11	57.9874	2563494.
0.00								
37.0000	5.15E-04	0.00	0.00	1.39E-04	0.00	1.50E+11	-300.2741	1294105.
0.00								

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 1.69638945 inches
Computed slope at pile head = -0.00704237 radians
Maximum bending moment = 5745595. inch-lbs
Maximum shear force = 60406. lbs
Depth of maximum bending moment = 20.35000000 feet below pile head
Depth of maximum shear force = 14.06000000 feet below pile head
Number of iterations = 29
Number of zero deflection points = 2

Pile-head Deflection vs. Pile Length for Load Case 1

Boundary Condition Type 1, Shear and Moment

Shear = 0. lbs
Moment = 0. in-lbs
Axial Load = 0. lbs

Pile Length feet	Pile Head Deflection inches	Maximum Moment ln-lbs	Maximum Shear lbs
37.00000	1.69638945	5745595.	60406.
35.15000	1.73262369	5746438.	60533.
33.30000	2.10554366	5758786.	-63379.
31.45000	2.88004341	5399939.	-65065.
29.60000	5.55463115	5126391.	-68755.
27.75000	10.98944942	4829736.	-71216.
25.90000	30.04984155	4634853.	-76877.

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

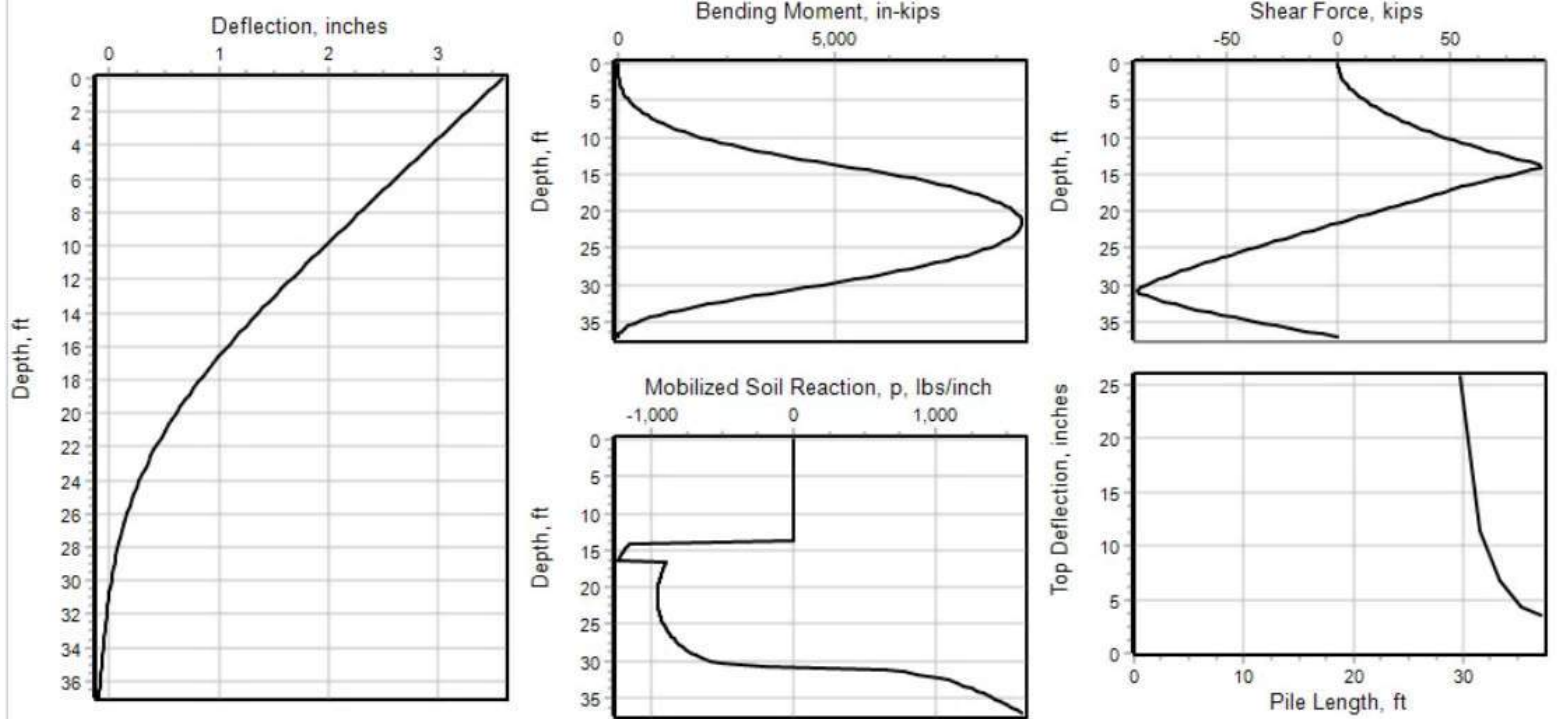
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	0.00	M, in-lb	0.00	0.00	1.6964	-0.00704	60406.	5745595.

Maximum pile-head deflection = 1.6963894497 inches

Maximum pile-head rotation = -0.0070423680 radians = -0.403498 deg.

The analysis ended normally.



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Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Files Used for Analysis

Path to file locations:

\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\geotechnical\retaining
walls\Retaining Wall X (LocationDescription)\analysis\RW3\Lfile\Files\

Name of input data file:

RW 3_STA. 1+25_D=3.5ft_W24x162_Strength I_R-003-0-99.lp9d

Name of output report file:

RW 3_STA. 1+25_D=3.5ft_W24x162_Strength I_R-003-0-99.lp9o

Name of plot output file:

RW 3_STA. 1+25_D=3.5ft_W24x162_Strength I_R-003-0-99.lp9p

Name of runtime message file:

RW 3_STA. 1+25_D=3.5ft_W24x162_Strength I_R-003-0-99.lp9r

Date and Time of Analysis

Date: April 23, 2026

Time: 12:56:47

Problem Title

Project Name: SUM-77 SR-8

Job Number: RW3 Soldier Pile Lagging Wall

Client: ODOT / Arcadis

Engineer: ZM

Description: STA. 1+25

Program Options and Settings

Computational Options:

- Use unfactored loads in computations (conventional analysis)

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- | | | |
|--|---|---------------|
| - Maximum number of iterations allowed | = | 100 |
| - Deflection tolerance for convergence | = | 1.0000E-05 in |
| - Maximum allowable deflection | = | 100.0000 in |
| - Number of pile increments | = | 100 |

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Analysis uses p-y modification factors for p-y curves
- Analysis includes loading by one distributed lateral load acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 2
Total length of pile = 37.000 ft
Depth of ground surface below top of pile = 14.0000 ft

Pile diameters used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	13.0000
2	14.000	13.0000
3	14.000	42.0000
4	37.000	42.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile
Cross-sectional Shape = Strong H-Pile
Length of section = 14.000000 ft
Flange Width = 13.000000 in
Section Depth = 25.000000 in
Flange Thickness = 0.0000 in
Web Thickness = 0.0000 in
Section Area = 47.700000 sq. in
Moment of Inertia = 5170. in^4
Elastic Modulus = 29000000. psi

Pile Section No. 2:

Section 2 is an elastic pile

Cross-sectional Shape	=	Circular Pile
Length of section	=	23.000000 ft
Width of top of section	=	42.000000 in
Width of bottom of section	=	42.000000 in
Top Area	=	47.700000 sq. in
Bottom Area	=	47.700000 sq. in
Moment of Inertia at Top	=	5170. in^4
Moment of Inertia at Bottom	=	5170. in^4
Elastic Modulus	=	29000000. psi

Ground Slope and Pile Batter Angles

Ground Slope Angle	=	0.000 degrees
	=	0.000 radians
Pile Batter Angle	=	0.000 degrees
	=	0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is stiff clay without free water

Distance from top of pile to top of layer	=	14.000000 ft
Distance from top of pile to bottom of layer	=	16.410000 ft

Effective unit weight at top of layer	=	112.000000 pcf
Effective unit weight at bottom of layer	=	112.000000 pcf
Undrained cohesion at top of layer	=	2350. psf
Undrained cohesion at bottom of layer	=	2350. psf
Epsilon-50 at top of layer	=	0.005700
Epsilon-50 at bottom of layer	=	0.005700

Layer 2 is stiff clay without free water

Distance from top of pile to top of layer	=	16.410000 ft
Distance from top of pile to bottom of layer	=	30.410000 ft
Effective unit weight at top of layer	=	112.000000 pcf
Effective unit weight at bottom of layer	=	112.000000 pcf
Undrained cohesion at top of layer	=	1750. psf
Undrained cohesion at bottom of layer	=	1750. psf
Epsilon-50 at top of layer	=	0.006700
Epsilon-50 at bottom of layer	=	0.006700

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer	=	30.410000 ft
Distance from top of pile to bottom of layer	=	46.410000 ft
Effective unit weight at top of layer	=	112.000000 pcf
Effective unit weight at bottom of layer	=	112.000000 pcf
Undrained cohesion at top of layer	=	2350. psf
Undrained cohesion at bottom of layer	=	2350. psf
Epsilon-50 at top of layer	=	0.005700
Epsilon-50 at bottom of layer	=	0.005700

Layer 4 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	46.410000 ft
Distance from top of pile to bottom of layer	=	50.000000 ft
Effective unit weight at top of layer	=	146.600000 pcf
Effective unit weight at bottom of layer	=	146.600000 pcf

Uniaxial compressive strength at top of layer	=	390.000000	psi
Uniaxial compressive strength at bottom of layer	=	390.000000	psi
Initial modulus of rock at top of layer	=	35000.	psi
Initial modulus of rock at bottom of layer	=	35000.	psi
RQD of rock at top of layer	=	0.0000	%
RQD of rock at bottom of layer	=	0.0000	%
k rm of rock at top of layer	=	0.0005000	
k rm of rock at bottom of layer	=	0.0005000	

(Depth of the lowest soil layer extends 13.000 ft below the pile tip)

**** Warning - Possible Input Data Error ****

Values entered for effective unit weights of soil were outside the limits of 20 pcf to 140 pcf.

The maximum input value for effective unit weight = 146.60 pcf

This data may be erroneous. Please check your data.

Summary of Input Soil Properties							
Layer Rock Mass Layer Modulus Num. psi	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Uniaxial qu psi	RQD %	E50 or krm
1	Stiff Clay	14.0000	112.0000	2350.	--	--	0.00570
--							

--	w/o Free Water	16.4100	112.0000	2350.	--	--	0.00570
2	Stiff Clay	16.4100	112.0000	1750.	--	--	0.00670
--	w/o Free Water	30.4100	112.0000	1750.	--	--	0.00670
3	Stiff Clay	30.4100	112.0000	2350.	--	--	0.00570
--	w/o Free Water	46.4100	112.0000	2350.	--	--	0.00570
4	Weak	46.4100	146.6000	--	390.0000	0.00	5.00E-04
35000.	Rock	50.0000	146.6000	--	390.0000	0.00	5.00E-04
35000.							

p-y Modification Factors for Group Action

Distribution of p-y modifiers with depth defined using 2 points

Point No.	Depth X ft	p-mult	y-mult
1	14.000	0.9100	1.0000
2	32.110	0.9100	1.0000

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Distributed Lateral Loading Used For All Load Cases

Distributed lateral load intensity defined using 2 points

Point No.	Depth X in	Dist. Load lb/in
-----	-----	-----
1	0.000	0.000
2	168.000	1125.000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length
-----	-----	-----	-----	-----	-----
1	1	V = 0.0000 lbs	M = 0.0000 in-lbs	0.0000000	Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

 Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Pile Section No. 2:

Moment-curvature properties were derived from elastic section properties

 Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	14.0000	0.00	N.A.	No	0.00	63852.
2	16.4100	2.4100	Yes	No	63852.	424590.
3	30.4100	16.4100	Yes	No	488443.	365969.
4	46.4100	32.4100	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays,

2.5900	3.1701	33336.	3251.	-0.01358	41.9118	1.50E+11	0.00	0.00
208.1250								
2.9600	3.1097	49821.	4241.	-0.01358	62.6374	1.50E+11	0.00	0.00
237.8571								
3.3300	3.0494	70995.	5363.	-0.01358	89.2583	1.50E+11	0.00	0.00
267.5893								
3.7000	2.9891	97444.	6617.	-0.01358	122.5114	1.50E+11	0.00	0.00
297.3214								
4.0700	2.9289	129754.	8003.	-0.01358	163.1336	1.50E+11	0.00	0.00
327.0536								
4.4400	2.8686	168512.	9521.	-0.01357	211.8619	1.50E+11	0.00	0.00
356.7857								
4.8100	2.8083	214303.	11171.	-0.01357	269.4330	1.50E+11	0.00	0.00
386.5179								
5.1800	2.7481	267714.	12954.	-0.01356	336.5840	1.50E+11	0.00	0.00
416.2500								
5.5500	2.6879	329330.	14868.	-0.01355	414.0518	1.50E+11	0.00	0.00
445.9821								
5.9200	2.6278	399739.	16914.	-0.01354	502.5732	1.50E+11	0.00	0.00
475.7143								
6.2900	2.5677	479526.	19092.	-0.01353	602.8852	1.50E+11	0.00	0.00
505.4464								
6.6600	2.5077	569276.	21402.	-0.01351	715.7247	1.50E+11	0.00	0.00
535.1786								
7.0300	2.4478	669577.	23844.	-0.01349	841.8286	1.50E+11	0.00	0.00
564.9107								
7.4000	2.3879	781015.	26419.	-0.01347	981.9337	1.50E+11	0.00	0.00
594.6429								
7.7700	2.3281	904175.	29125.	-0.01345	1137.	1.50E+11	0.00	0.00
624.3750								
8.1400	2.2685	1039644.	31963.	-0.01342	1307.	1.50E+11	0.00	0.00
654.1071								
8.5100	2.2090	1188007.	34933.	-0.01338	1494.	1.50E+11	0.00	0.00
683.8393								
8.8800	2.1497	1349852.	38036.	-0.01335	1697.	1.50E+11	0.00	0.00
713.5714								
9.2500	2.0905	1525763.	41270.	-0.01330	1918.	1.50E+11	0.00	0.00
743.3036								

9.6200	2.0315	1716328.	44636.	-0.01326	2158.	1.50E+11	0.00	0.00
773.0357								
9.9900	1.9728	1922132.	48134.	-0.01320	2417.	1.50E+11	0.00	0.00
802.7679								
10.3600	1.9143	2143762.	51765.	-0.01314	2695.	1.50E+11	0.00	0.00
832.5000								
10.7300	1.8561	2381803.	55527.	-0.01307	2995.	1.50E+11	0.00	0.00
862.2321								
11.1000	1.7982	2636841.	59421.	-0.01300	3315.	1.50E+11	0.00	0.00
891.9643								
11.4700	1.7406	2909464.	63448.	-0.01292	3658.	1.50E+11	0.00	0.00
921.6964								
11.8400	1.6835	3200257.	67606.	-0.01283	4024.	1.50E+11	0.00	0.00
951.4286								
12.2100	1.6267	3509805.	71896.	-0.01273	4413.	1.50E+11	0.00	0.00
981.1607								
12.5800	1.5705	3838696.	76319.	-0.01262	4826.	1.50E+11	0.00	0.00
1011.								
12.9500	1.5147	4187515.	80873.	-0.01250	5265.	1.50E+11	0.00	0.00
1041.								
13.3200	1.4594	4556849.	85559.	-0.01237	5729.	1.50E+11	0.00	0.00
1070.								
13.6900	1.4048	4947283.	90378.	-0.01223	6220.	1.50E+11	0.00	0.00
1100.								
14.0600	1.3508	5359404.	91105.	-0.01208	21769.	1.50E+11	-1151.	3784.
378.3708								
14.4300	1.2976	5756291.	86800.	-0.01191	23381.	1.50E+11	-1166.	3991.
0.00								
14.8000	1.2451	6130186.	81589.	-0.01174	24900.	1.50E+11	-1181.	4211.
0.00								
15.1700	1.1933	6480806.	76317.	-0.01155	26324.	1.50E+11	-1194.	4444.
0.00								
15.5400	1.1425	6807879.	70985.	-0.01135	27653.	1.50E+11	-1207.	4692.
0.00								
15.9100	1.0925	7111153.	65598.	-0.01115	28885.	1.50E+11	-1219.	4956.
0.00								
16.2800	1.0435	7390388.	60158.	-0.01093	30019.	1.50E+11	-1231.	5237.
0.00								

16.6500	0.9954	7645361.	55431.	-0.01071	31055.	1.50E+11	-898.7331	4009.
0.00								
17.0200	0.9484	7882616.	51422.	-0.01048	32018.	1.50E+11	-907.0731	4247.
0.00								
17.3900	0.9024	8101989.	47378.	-0.01024	32909.	1.50E+11	-914.7815	4501.
0.00								
17.7600	0.8574	8303329.	43300.	-0.01000	33727.	1.50E+11	-921.8473	4774.
0.00								
18.1300	0.8136	8486496.	39193.	-0.00975	34471.	1.50E+11	-928.2594	5066.
0.00								
18.5000	0.7708	8651363.	35059.	-0.00950	35141.	1.50E+11	-934.0062	5380.
0.00								
18.8700	0.7292	8797818.	30901.	-0.00924	35736.	1.50E+11	-939.0759	5718.
0.00								
19.2400	0.6888	8925760.	26721.	-0.00898	36256.	1.50E+11	-943.4561	6082.
0.00								
19.6100	0.6495	9035104.	22524.	-0.00871	36700.	1.50E+11	-947.1341	6474.
0.00								
19.9800	0.6114	9125776.	18312.	-0.00844	37068.	1.50E+11	-950.0966	6899.
0.00								
20.3500	0.5745	9197718.	14089.	-0.00817	37360.	1.50E+11	-952.3294	7359.
0.00								
20.7200	0.5389	9250886.	9857.	-0.00790	37576.	1.50E+11	-953.8179	7859.
0.00								
21.0900	0.5044	9285251.	5621.	-0.00762	37716.	1.50E+11	-954.5461	8402.
0.00								
21.4600	0.4712	9300799.	1383.	-0.00735	37779.	1.50E+11	-954.4970	8995.
0.00								
21.8300	0.4392	9297530.	-2853.	-0.00707	37766.	1.50E+11	-953.6524	9642.
0.00								
22.2000	0.4084	9275461.	-7084.	-0.00680	37676.	1.50E+11	-951.9921	10351.
0.00								
22.5700	0.3788	9234624.	-11305.	-0.00652	37510.	1.50E+11	-949.4941	11130.
0.00								
22.9400	0.3504	9175070.	-15513.	-0.00625	37268.	1.50E+11	-946.1337	11988.
0.00								
23.3100	0.3233	9096865.	-19705.	-0.00598	36951.	1.50E+11	-941.8833	12936.
0.00								

23.6800 0.00	0.2973	9000091.	-23875.	-0.00571	36557.	1.50E+11	-936.7114	13988.
24.0500 0.00	0.2725	8884851.	-28021.	-0.00545	36089.	1.50E+11	-930.5820	15160.
24.4200 0.00	0.2489	8751266.	-32137.	-0.00519	35547.	1.50E+11	-923.4530	16470.
24.7900 0.00	0.2265	8599477.	-36219.	-0.00493	34930.	1.50E+11	-915.2750	17943.
25.1600 0.00	0.2052	8429644.	-40262.	-0.00468	34240.	1.50E+11	-905.9894	19607.
25.5300 0.00	0.1850	8241951.	-44261.	-0.00443	33478.	1.50E+11	-895.5250	21498.
25.9000 0.00	0.1658	8036604.	-48211.	-0.00419	32644.	1.50E+11	-883.7950	23665.
26.2700 0.00	0.1477	7813834.	-52106.	-0.00396	31739.	1.50E+11	-870.6918	26166.
26.6400 0.00	0.1307	7573900.	-55940.	-0.00373	30764.	1.50E+11	-856.0793	29083.
27.0100 0.00	0.1146	7317089.	-59705.	-0.00351	29721.	1.50E+11	-839.7827	32523.
27.3800 0.00	0.09956	7043724.	-63393.	-0.00329	28611.	1.50E+11	-821.5730	36641.
27.7500 0.00	0.08539	6754162.	-66995.	-0.00309	27435.	1.50E+11	-801.1420	41656.
28.1200 0.00	0.07212	6448806.	-70501.	-0.00289	26194.	1.50E+11	-778.0633	47903.
28.4900 0.00	0.05969	6128112.	-73897.	-0.00271	24892.	1.50E+11	-751.7238	55917.
28.8600 0.00	0.04807	5792599.	-77167.	-0.00253	23529.	1.50E+11	-721.1987	66617.
29.2300 0.00	0.03721	5442869.	-80289.	-0.00237	22108.	1.50E+11	-684.9965	81741.
29.6000 0.00	0.02706	5079635.	-83231.	-0.00221	20633.	1.50E+11	-640.4672	105075.
29.9700 0.00	0.01759	4703775.	-85945.	-0.00206	19106.	1.50E+11	-582.1095	146959.
30.3400 0.00	0.00873	4316439.	-88336.	-0.00193	17533.	1.50E+11	-494.5269	251540.

30.7100 0.00	4.39E-04	3919355.	-89866.	-0.00181	15920.	1.50E+11	-194.9106	1972924.
31.0800 0.00	-0.00734	3518428.	-88858.	-0.00170	14291.	1.50E+11	648.9868	392767.
31.4500 0.00	-0.01465	3130295.	-85685.	-0.00160	12715.	1.50E+11	780.1596	236463.
31.8200 0.00	-0.02155	2757541.	-82025.	-0.00151	11201.	1.50E+11	868.7738	178998.
32.1900 0.00	-0.02809	2401915.	-77807.	-0.00144	9756.	1.50E+11	1031.	163027.
32.5600 0.00	-0.03431	2066619.	-73084.	-0.00137	8394.	1.50E+11	1096.	141837.
32.9300 0.00	-0.04026	1752931.	-68091.	-0.00131	7120.	1.50E+11	1153.	127161.
33.3000 0.00	-0.04598	1461974.	-62856.	-0.00127	5938.	1.50E+11	1205.	116330.
33.6700 0.00	-0.05151	1194767.	-57401.	-0.00123	4853.	1.50E+11	1252.	107963.
34.0400 0.00	-0.05688	952250.	-51741.	-0.00120	3868.	1.50E+11	1297.	101270.
34.4100 0.00	-0.06213	735310.	-45886.	-0.00117	2987.	1.50E+11	1340.	95767.
34.7800 0.00	-0.06727	544785.	-39845.	-0.00115	2213.	1.50E+11	1381.	91138.
35.1500 0.00	-0.07235	381484.	-33626.	-0.00114	1550.	1.50E+11	1421.	87172.
35.5200 0.00	-0.07738	246186.	-27233.	-0.00113	999.9809	1.50E+11	1459.	83720.
35.8900 0.00	-0.08237	139651.	-20671.	-0.00112	567.2495	1.50E+11	1497.	80677.
36.2600 0.00	-0.08735	62624.	-13943.	-0.00112	254.3719	1.50E+11	1534.	77963.
36.6300 0.00	-0.09232	15834.	-7052.	-0.00112	64.3146	1.50E+11	1570.	75523.
37.0000 0.00	-0.09729	0.00	0.00	-0.00112	0.00	1.50E+11	1606.	36656.

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 3.59230259 inches
Computed slope at pile head = -0.01358616 radians
Maximum bending moment = 9300799. inch-lbs
Maximum shear force = 91105. lbs
Depth of maximum bending moment = 21.46000000 feet below pile head
Depth of maximum shear force = 14.06000000 feet below pile head
Number of iterations = 44
Number of zero deflection points = 1

Pile-head Deflection vs. Pile Length for Load Case 1

Boundary Condition Type 1, Shear and Moment

Shear = 0. lbs
Moment = 0. in-lbs
Axial Load = 0. lbs

Pile Length feet	Pile Head Deflection inches	Maximum Moment ln-lbs	Maximum Shear lbs
37.00000	3.59230259	9300799.	91105.
35.15000	4.33593852	9037526.	-95128.
33.30000	6.80785602	8806063.	-99507.
31.45000	11.45481259	8160513.	-99225.
29.60000	25.78648619	7717332.	-103906.

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	0.00	M, in-lb	0.00	0.00	3.5923	-0.01359	91105.	9300799.

Maximum pile-head deflection = 3.5923025905 inches

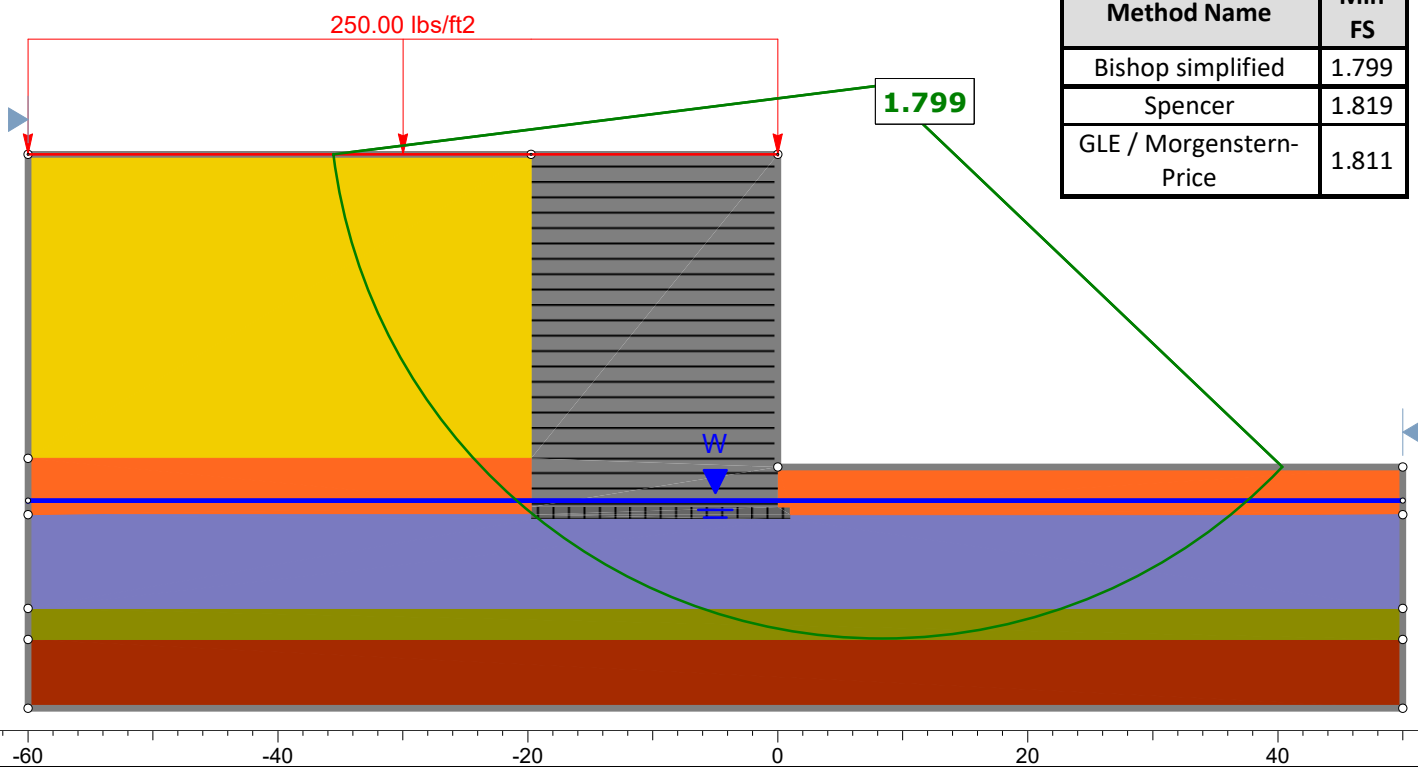
Maximum pile-head rotation = -0.0135861582 radians = -0.778430 deg.

The analysis ended normally.

SUM-77/SR-8 Retaining Wall 3 STA. 1+25		
Depth (ft)	Shear Force from Lpile(lb)	Average Shear (lb) based on ODOT GDM 1501.7.9
0	0.00000038	0.00000038
0.37	82.5067	121.0098335
0.74	280.5228	409.3158201
1.11	610.5496	872.3684429
1.48	1073	1336.797014
1.85	1667	1930.867486
2.22	2393	2656.935657
2.59	3251	3515
2.96	4241	4505
3.33	5363	5627
3.7	6617	6881
4.07	8003	8267.142857
4.44	9521	9785.285714
4.81	11171	11435.42857
5.18	12954	13217.57143
5.55	14868	15131.71429
5.92	16914	17177.85714
6.29	19092	19356.14286
6.66	21402	21666.28571
7.03	23844	24108.42857
7.4	26419	26682.57143
7.77	29125	29388.85714
8.14	31963	32227.14286
8.51	34933	35197.42857
8.88	38036	38299.57143
9.25	41270	41533.85714
9.62	44636	44900.14286
9.99	48134	48398.42857
10.36	51765	52028.71429
10.73	55527	55791
11.1	59421	59685.28571
11.47	63448	63711.71429
11.84	67606	67870
12.21	71896	72160.28571
12.58	76319	76582.71429
12.95	80873	80533.71429
13.32	85559	83275.71429
13.69	90378	84660.42857
14.06	91105	84660.14286
14.43	86800	83247.57143
14.8	81589	80396
15.17	76317	76078.85714
15.54	70985	70982.57143
15.91	65598	65928.57143
16.28	60158	61041.28571
16.65	55431	56324.57143
17.02	51422	51782.85714
17.39	47378	47420.14286
17.76	43300	43240.57143
18.13	39193	39139.14286
18.5	35059	35010.85714
18.87	30901	30858.57143
19.24	26721	26685.57143
19.61	22524	22494.71429
19.98	18312	18289.28571
20.35	14089	14072.42857
20.72	9857	9847.571429
21.09	5621	5617.857143
21.46	1383	1386.857143
21.83	-2853	-2842
22.2	-7084	-7065.142857
22.57	-11305	-11278.85714
22.94	-15513	-15496.4
23.31	-19705	-19697.66667
23.68	-23875	-23867
24.05	-28021	-28011
24.42	-32137	-32125.66667
24.79	-36219	-36206
25.16	-40262	-40247.33333
25.53	-44261	-44244.66667
25.9	-48211	-48192.66667
26.27	-52106	-52085.66667
26.64	-55940	-55917
27.01	-59705	-59679.33333
27.38	-63393	-63364.33333
27.75	-66995	-66963
28.12	-70501	-70464.33333
28.49	-73897	-73855
28.86	-77167	-77117.66667
29.23	-80289	-80229
29.6	-83231	-83155
29.97	-85945	-85837.33333
30.34	-88336	-88049
30.71	-89666	-89020
31.08	-88858	-88136.33333
31.45	-85685	-85522.66667
31.82	-82025	-81839
32.19	-77807	-77638.66667
32.56	-73084	-72994
32.93	-68091	-68010.33333
33.3	-62856	-62782.66667
33.67	-57401	-57332.66667
34.04	-51741	-51676
34.41	-45896	-45824
34.78	-39845	-39785.66667
35.15	-33626	-33568
35.52	-27233	-27176.66667
35.89	-20671	-20615.66667
36.26	-13943	-13888.66667
36.63	-7052	-6998.333333
37	0	-3526

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Fill		125	125	Mohr-Coulomb	250	26
MSE Wall		120	130	Infinite Strength		
Soil Type 1		112	122	Mohr-Coulomb	200	26
Soil Type 2		115	125	Mohr-Coulomb	0	35
Soil Type 3		112	122	Mohr-Coulomb	200	25
Soil Type 4		120	130	Mohr-Coulomb	0	40
Foundation Preparation Layer		130	130	Mohr-Coulomb	0	34

Method Name	Min FS
Bishop simplified	1.799
Spencer	1.819
GLE / Morgenstern-Price	1.811



Project

SUM 77 SR8-Retaining Wall 3

Group

SUM-77 SR8-RW3-STA.0+00-Effective-B-022.slim

Scenario

SUM-77 SR8-RW3-STA.0+00-Effective-B-022.slim

Drawn By

ZM

Company

NEAS, Inc

Date

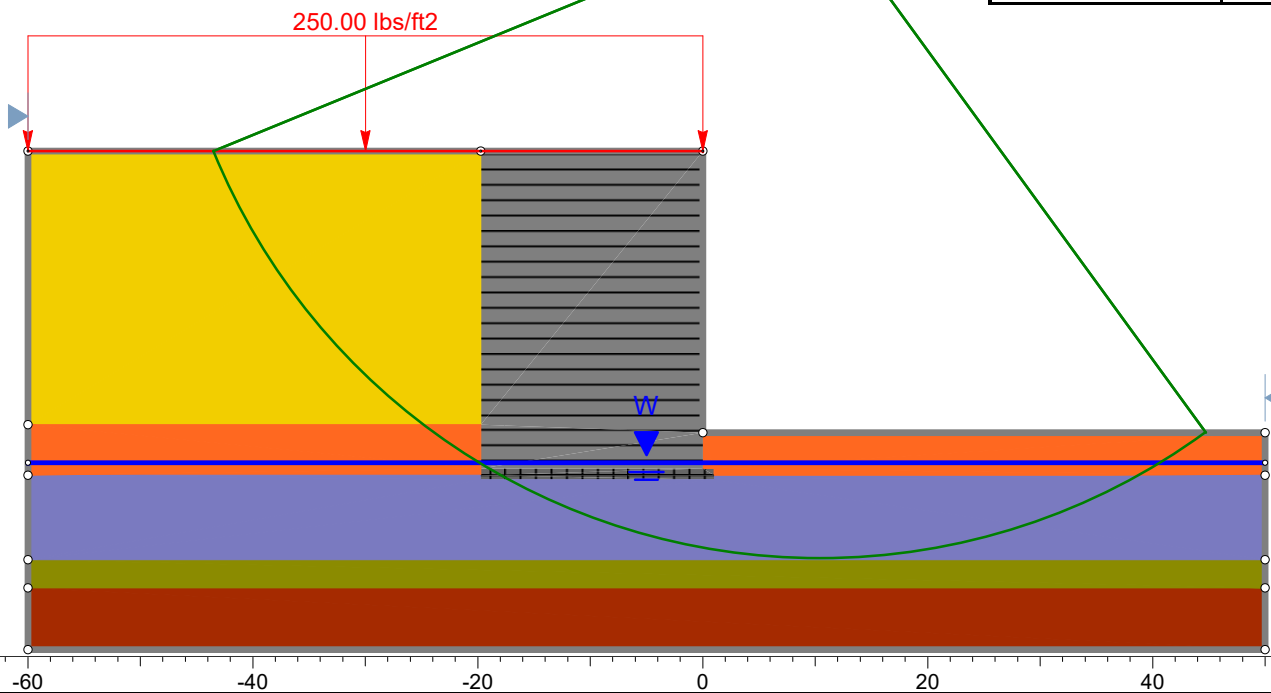
1/8/2025

File Name

SUM-77 SR8-RW3-STA.0+00-Effective-B-022.slim

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Fill		125	125	Mohr-Coulomb	2500	0
MSE Wall		120	130	Infinite Strength		
Soil Type 1		112	122	Mohr-Coulomb	2250	0
Soil Type 2		115	125	Mohr-Coulomb	0	35
Soil Type 3		112	122	Mohr-Coulomb	2000	0
Soil Type 4		120	130	Mohr-Coulomb	0	40
Foundation Preparation Layer		130	130	Mohr-Coulomb	0	34

Method Name	Min FS
Bishop simplified	3.006
Spencer	3.239
GLE / Morgenstern-Price	3.184



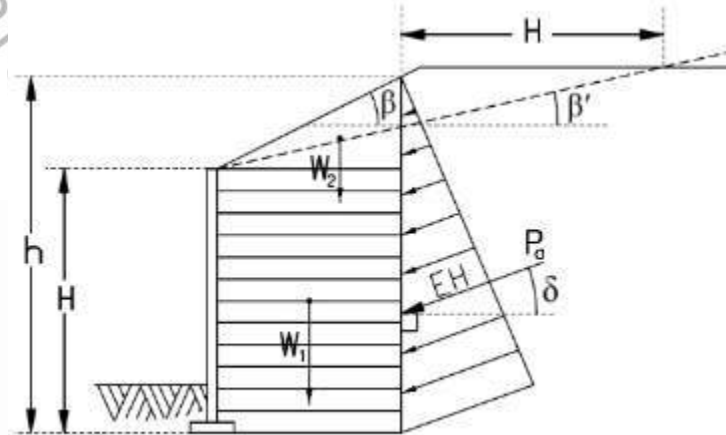
Project	SUM 77 SR8-Retaining Wall 3	
Group	SUM-77 SR8-RW3-STA.0+00-Total-B-022.slim	Scenario SUM-77 SR8-RW3-STA.0+00-Total-B-022.slim
Drawn By	ZM	Company NEAS, Inc
Date	1/8/2025	File Name SUM-77 SR8-RW3-STA.0+00-Total-B-022.slim

RETAINING WALL 4

Objective: To evaluate the external stability of MSE wall design with vertical wall face and broken backslope case.
Method: In accordance with ODOT Bridge Design Manual, 2026 [Sect. 307.4] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 3.11.5.8 and Sect. 11.10.5].

Assumptions:

- Horizontal backfill, Infinite backslope or Broken backslope behind MSE wall, see LRFD Sect. 3.11.5.8.1.
- For battered or vertical walls with a back face of wall angle of θ to horizontal.
- Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ .
- MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall



Givens:

Wall Geometry:

$$H_e := 16.5 \text{ ft}$$

$$\theta := 90 \cdot \text{deg}$$

$$\beta_{BackSlope} := 0 \text{ deg}$$

$$\lambda := 0 \text{ ft}$$

Exposed wall height (Proposed profile grade to bottom of leveling pad)

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Inclination of ground slope behind face of wall. If it is horizontal backfill behind MSE wall, $\beta_{BackSlope} = 0$ deg

Horizontal distance from the back of MSE wall to the top of slope. If it is infinite slope behind MSE Wall, input λ larger than 2H; If it is horizontal backfill behind MSE wall, $\lambda = 0$ ft

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \text{ deg}$$

$$\gamma_r := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_r := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \text{ deg}$$

$$\gamma_b := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_b := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

$$\delta := 0.67 \cdot \phi'_b$$

$$\delta = 20.1 \text{ deg}$$

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Soil Design Parameters for Bearing Resistance (Average 1.5 B Below Leveling Pad)

Drained Conditions (Effective Stress):

$$\phi'_{fd} := 31 \text{ deg} \quad \text{Effective angle of internal friction}$$

$$\gamma_{fd} := 112 \frac{\text{lb}_f}{\text{ft}^3} \quad \text{Unit weight}$$

$$c'_{fd} := 0 \frac{\text{lb}_f}{\text{ft}^2} \quad \text{Effective Cohesion}$$

Undrained Conditions (Total Stress):

$$\phi_{fdu} := 31 \text{ deg} \quad \text{Angle of internal friction (Same as Drained Conditions if Sand)}$$

$$Su_{fdu} := 0 \frac{\text{lb}_f}{\text{ft}^2} \quad \text{Undrained Shear Strength}$$

Undercut & Replacement Design Parameters:

$$\phi_{Re} := 34 \text{ deg} \quad \text{Angle of internal friction for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.}$$

$$c_{Re} := 0 \frac{\text{lb}_f}{\text{ft}^2} \quad \text{Cohesion for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.}$$

$$\gamma_{Re} := 130 \frac{\text{lb}_f}{\text{ft}^3} \quad \text{Unit Weight for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.}$$

$$\delta_{Re} := 0.67 \cdot \phi_{Re} \quad \delta_{Re} = 22.8 \text{ deg} \quad \text{Friction angle between Replacement soils and footing taken as specified in LRFD BDS C3.11.5.3 (degrees)}$$

$$D_{undercut} := 1 \text{ ft} \quad \text{Depth of Undercut below bottom of footing}$$

Foundation Surcharge Soil Parameters

$$\gamma_q := 120 \frac{\text{lb}_f}{\text{ft}^3} \quad \text{Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)}$$

Depth of Embedment Check:

$$d_{frost} := 3.25 \text{ ft} \quad d_{user} := 6.5 \text{ ft} \quad \text{Local Frost Depth / User Input}$$

$$Slope_{fw} := 0 \text{ deg}$$

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user}) \quad d_{est} = 6.5 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e \quad H_{est} = 23 \text{ ft}$$

Inclination of ground slope in front of wall :

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1.2 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope}) \quad d_e = 6.5 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e \quad H = 23 \text{ ft}$$

Initial design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 0.7 \cdot H$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max \left(8 \cdot \text{ft}, 0.7 \cdot H, L_{user} \right)$$

$$L = 16.1 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L < \lambda, 0 \frac{\text{lb}}{\text{ft}^2}, 250 \frac{\text{lb}}{\text{ft}^2} \right) = 250 \frac{\text{lb}}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

$$SUR_{@MSE} := \text{if} \left(L < \lambda, 0 \frac{\text{lb}}{\text{ft}^2}, 250 \frac{\text{lb}}{\text{ft}^2} \right) = 250 \frac{\text{lb}}{\text{ft}^2}$$

Live load surcharge above the MSE Wall soil reinforcement (per LRFD BDS [3.11.6.4])

Calculations:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan \left(\beta_{BackSlope} \right)$$

$$H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h := \text{if} \left(L < \lambda, H + L \cdot \tan \left(\beta_{BackSlope} \right), H + H_{BackSlope} \right)$$

Height of retained fill at the back of the reinforced soil

$$h = 23 \text{ ft}$$

$$I := \text{atan} \left(\frac{H_{BackSlope}}{2 \cdot H} \right)$$

$$\beta := \text{if} \left(\lambda < 2 \cdot H, I, \beta_{BackSlope} \right)$$

Angle of friction between retained backfill and reinforced soil

$$I = 0 \text{ deg}$$

$$\beta = 0 \text{ deg}$$

$$\Gamma := \left(1 + \sqrt{\frac{\left(\sin \left(\phi'_b + \delta \right) \cdot \sin \left(\phi'_b - \beta \right) \right)}{\left(\sin \left(\theta - \delta \right) \cdot \sin \left(\theta + \beta \right) \right)}} \right)^2 = 2.6867$$

$$k_{ab} := \left(\frac{\left(\sin \left(\theta + \phi'_b \right) \right)^2}{\left(\Gamma \cdot \left(\sin \left(\theta \right) \right)^2 \cdot \sin \left(\theta - \delta \right) \right)} \right)$$

$$k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot h^2 \cdot k_{ab}$$

$$F_T = 9435 \frac{\text{lb}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{ab}$$

$$F_{SUR} = 1709.2 \frac{\text{lb}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 44436 \frac{\text{lb}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_2 = 0 \frac{\text{lb}}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_3 := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lb}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_3 = 0 \frac{\text{lb}}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_4 := F_T \cdot \sin(\beta)$$

$$V_4 = 0 \frac{\text{lbf}}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_5 := F_{SUR} \cdot \sin(\beta)$$

$$V_5 = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_6 := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_6 = 4025 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1} := \frac{L}{2}$$

$$d_{v1} = 8.1 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 357709.8 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := \text{if}\left(L < \lambda, \frac{2L}{3}, \frac{2 \cdot \lambda}{3}\right)$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v3} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v3} = 8.1 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3}$$

$$MV_3 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v4} := L$$

$$d_{v4} = 16.1 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4}$$

$$MV_4 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v5} := L$$

$$d_{v5} = 16.1 \text{ ft}$$

$$MV_5 := V_5 \cdot d_{v5}$$

$$MV_5 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{v6} := \text{if}\left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2}\right)$$

$$d_{v6} = 8.1 \text{ ft}$$

$$MV_6 := V_6 \cdot d_{v6}$$

$$MV_6 = 32401.3 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T \cdot \cos(\beta) = 9435 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} \cdot \cos(\beta) = 1709.2 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

Moment:

$$d_{h1} := \frac{h}{3}$$

$$d_{h1} = 7.7 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 72335.2 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{2}$$

$$d_{h2} = 11.5 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 19656.3 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1 + V_2 + V_3$$

$$V_{EV} = 44436 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_4$$

$$V_{EH} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS} := V_5$$

$$V_{LS} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS@MSE} := V_6$$

$$V_{LS@MSE} = 4025 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 9435 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 1709.2 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1 + MV_2 + MV_3$$

$$M_{EV} = 357709.8 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_4$$

$$M_{EH1} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1} := MV_5$$

$$M_{LS1@MSE} := MV_6$$

$$M_{EH2} := MH_1$$

$$M_{LS2} := MH_2$$

$$M_{LS1} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE} = 32401.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} = 72335.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} = 19656.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1 \quad \text{LRFD Load Modifier}$$

$$\begin{aligned} \text{Strength Limit State I:} \quad & \text{EV(min)} = 1.00 \quad \text{EV(max)} = 1.35 \\ & \text{EH(min)} = 0.90 \quad \text{EH(max)} = 1.50 \\ & \text{LS} = 1.75 \end{aligned}$$

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot \left((Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE}) \right)$$

$$V_{Ia} = 51479.8 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot \left((Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE}) \right)$$

$$V_{Ib} = 67032.4 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot \left((Ia_{EH} \cdot H_{EH}) + (Ia_{LS} \cdot H_{LS}) \right)$$

$$H_{Ia} = 17143.7 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot \left((Ib_{EH} \cdot H_{EH}) + (Ib_{LS} \cdot H_{LS}) \right)$$

$$H_{Ib} = 17143.7 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot \left((Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + Ia_{LS} \cdot (M_{LS1} + M_{LS1@MSE}) \right)$$

$$MV_{Ia} = 414412 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot \left((Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + Ib_{LS} \cdot (M_{LS1} + M_{LS1@MSE}) \right)$$

$$MV_{Ib} = 539610.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot \left((Ia_{EH} \cdot M_{EH2}) + (Ia_{LS} \cdot M_{LS2}) \right)$$

$$MH_{Ia} = 142901.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot \left((Ib_{EH} \cdot M_{EH2}) + (Ib_{LS} \cdot M_{LS2}) \right)$$

$$MH_{Ib} = 142901.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb) about the Toe "O" of base length (the pivot):

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 539610.4 \frac{lb \cdot ft}{ft}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 142901.4 \frac{lb \cdot ft}{ft}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 67032.4 \frac{lb \cdot ft}{ft}$	Sum of Vertical Loads (Strength lb)
$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$	$x = 5.9 \text{ ft}$	Distance from Point "O" the resultant intersects the base
$e := \left \frac{L}{2} - x \right $	$e = 2.13 \text{ ft}$	Wall eccentricity, Note: The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2] . The effective bearing width is equal to B-2e.

$B'_{top} := L - 2 \cdot e$	$B'_{top} = 11.8 \text{ ft}$	Effective Footing Width, at the top of Foundation Preparation Layer
$B'_{bottom} := L - 2 \cdot e + 4.25 \text{ ft}$	$B'_{bottom} = 16.1 \text{ ft}$	Effective Footing Width, at the top of Foundation Preparation Layer (FPL), 4.25 ft is the extra length of FPL

Foundation Layout:

$L_{Wall} := 376.54 \text{ ft}$		Assumed Footing Length (Wall Section Length)
$D_f := d_e$	$D_f = 6.5 \text{ ft}$	Footing embedment
$D_F := D_f + D_{undercut}$		Embenment Depth at bottom of Undercut
$d_w := D_f$		Depth of Groundwater below ground surface in front of wall

Compute Bearing Resistance at the top of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right)$	$N_q = 20.63$
$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right)$	$N_c = 32.67$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$	$N_\gamma = 26$

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

$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}} \right) \right)$	$s_c = 1.02$
$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \right) \cdot \tan(\phi'_{fd}), 1 \right)$	$s_q = 1.019$
$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}} \right), 1 \right)$	$s_\gamma = 0.987$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 1$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.16$$

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

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

$$N_{cm} = 33.32$$

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

$$N_{qm} = 24.376$$

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

$$N_{\gamma m} = 25.667$$

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

$$q_{nd_top} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

at top of foundation preparation layer

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_top} := \phi_b \cdot q_{nd_top}$$

$$q_{Rd_top} = 17.9 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

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

$$N_q = 20.63$$

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

$$N_c = 32.67$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 26$$

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

$$s_c := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}}\right)\right)$$

$$s_c = 1.02$$

$$s_q := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}}\right) \cdot \tan(\phi_{fdu}), 1\right)$$

$$s_q = 1.019$$

$$s_\gamma := \text{if} \left(\phi_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.987$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot \left(1 - \sin(\phi'_{fd}) \right)^2 \cdot \text{atan} \left(\frac{D_F}{B'_{top}} \right)$$

$$d_q = 1.16$$

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

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

$$N_{cm} = 33.32$$

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

$$N_{qm} = 24.376$$

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

$$N_{\gamma m} = 25.667$$

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

$$q_{nu_top} := Su_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nu_top} = 27519.8 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_top} := \phi_b \cdot q_{nu_top} \quad q_{Ru_top} = 17.9 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_top} = 17.9 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_top} = 17.9 \text{ ksf}$$

Compute Bearing Resistance at the bottom of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 20.63$$

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

$$N_c = 32.67$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 26$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.027$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \cdot \tan(\phi'_{fd}) \right), 1 \right) \quad s_q = 1.026$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right) \quad s_\gamma = 0.983$$

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]:

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5) \quad C_{wq} = 1$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5) \quad C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{top}} \right)$$

$$d_q = 1.16$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 33.553$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 24.538$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 25.55$$

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

$$q_{nd_bottom} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad \text{at bottom of foundation preparation layer}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_bottom} := \phi_b \cdot q_{nd_bottom} \quad q_{Rd_top} = 17.9 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Total Stress):

$$N_q := \text{if} \left(\phi_{fdu} > 0, e^{\pi \cdot \tan(\phi_{fdu})} \cdot \tan \left(45 \text{ deg} + \frac{\phi_{fdu}}{2} \right), 1.0 \right) \quad N_q = 20.63$$

$$N_c := \text{if} \left(\phi_{fdu} > 0, \frac{N_q - 1}{\tan(\phi_{fdu})}, 5.14 \right) \quad N_c = 32.67$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu}) \quad N_\gamma = 26$$

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

$$s_c := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.027$$

$$s_q := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}} \cdot \tan(\phi_{fdu}) \right), 1 \right) \quad s_q = 1.026$$

$$s_\gamma := \text{if} \left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}} \right), 1 \right) \quad s_\gamma = 0.983$$

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

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

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

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

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'_{bottom}} \right)$$

$$d_q = 1.12$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 33.553$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 23.77$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 25.55$$

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

$$q_{nu_bottom} := Su_{fdu} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu_top} = 27519.8 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru_bottom} := \phi_b \cdot q_{nu_top} \quad q_{Ru_bottom} = 17.9 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_bottom} = 21.8 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru_bottom} = 17.9 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location about the toe "O" of base length (the pivot):

$$e := \left| \frac{L}{2} - x \right| \quad e = 2.13 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B' , since the wall is supported by a soil foundation LRFD [11.6.3.2]. The effective bearing width is equal to $B - 2e$.

$$\sigma_v := \frac{\sum V}{B'_{top}} \quad \sigma_v = 5663.3 \frac{\text{lb}}{\text{ft}^2}$$

Bearing Stress

Bearing Resistance Capacity:Demand Ratio (CDR)

At the top of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_top} := \frac{q_{Rd_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_top} = 3.16$

Undrained Conditions: $CDR_{Bearing_u_top} := \frac{q_{Ru_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_top} = 3.16$

At the bottom of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_bottom} := \frac{q_{Rd_bottom}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_bottom} = 3.86$

Undrained Conditions: $CDR_{Bearing_u_bottom} := \frac{q_{Ru_bottom}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_u_bottom} = 3.16$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$e_{max} := \frac{L}{3}$

$e_{max} = 5.4 \text{ ft}$

Maximum Eccentricity **LRFD [C11.6.3.3.]**

$\Sigma M_R := MV_{Ia}$

$\Sigma M_R = 414412 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$

Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$

$\Sigma M_O = 142901.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$

Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$

$\Sigma V = 51479.8 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$

$x = 5.3 \text{ ft}$

Distance from Point "O" the resultant intersects the base

$e := \left| \frac{L}{2} - x \right|$

$e = 2.78 \text{ ft}$

Wall eccentricity

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e}$

Is the CDR > or = to 1.0?

$CDR_{Eccentricity} = 1.93$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$

$R_u = 17143.7 \frac{\text{lb}}{\text{ft}}$

At the top of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$\Sigma V := V_{Ia}$

$\Sigma V = 51479.8 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$R_{td_top} := \Sigma V \cdot \tan(\phi_{Re})$

$R_{td_top} = 34723.5 \frac{\text{lb}}{\text{ft}}$

Nominal sliding resistance Drained Conditions

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$\phi_\tau := 1.0$

Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

Drained Conditions:

$\phi R_{n_d_top} := \phi_\tau \cdot R_{td_top}$

$R_{R_d_top} := \phi R_{n_d_top}$

$R_{R_d_top} = 34.7 \frac{\text{kip}}{\text{ft}}$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_top} := \frac{R_{R_d_top}}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding_d_top} = 2.03$$

At the bottom of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 51479.8 \frac{lb}{ft}$$

Sum of Vertical Loads (Strength Ia)

$$R_{\tau d_bottom} := \Sigma V \cdot \tan(\phi'_{fd})$$

$$R_{\tau d_bottom} = 30932.2 \frac{lb}{ft}$$

Nominal sliding resistance Drained Conditions

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_{\tau} := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

Drained Conditions:

$$\phi R_{n_d_bottom} := \phi_{\tau} \cdot R_{\tau d_bottom}$$

$$R_{R_d_bottom} := \phi R_{n_d_bottom}$$

$$R_{R_d_bottom} = 30.9 \frac{kip}{ft}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_bottom} := \frac{R_{R_d_bottom}}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding_d_bottom} = 1.80$$

Objective: To evaluate the external stability of MSE wall design with vertical wall face and broken backslope case.
Method: In accordance with ODOT Bridge Design Manual, 2026 [Sect. 307.4] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 3.11.5.8 and Sect. 11.10.5].

Assumptions:

- Horizontal backfill, Infinite backslope or Broken backslope behind MSE wall, see LRFD Sect. 3.11.5.8.1.
- For battered or vertical walls with a back face of wall angle of θ to horizontal.
- Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ .
- MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	OR 0		OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5		
		General Conditions	OR Completely Dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 0 + 3 + 20 + 12 + 7 = 42$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 10$$

Rock strength constant using **GDM Section 1303.3.3**.

Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 360 \text{ psi}$$

Unconfined compressive strength of rock core, based on the boring description and **GDM Table 400-6**.

$$\gamma_{rock} := 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (Hoek and Brown, 1988):

$$s := e^{\left(\frac{RMR - 100}{28}\right)}$$

$$s = 0.126005645$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)}$$

$$m = 0.0158932728$$

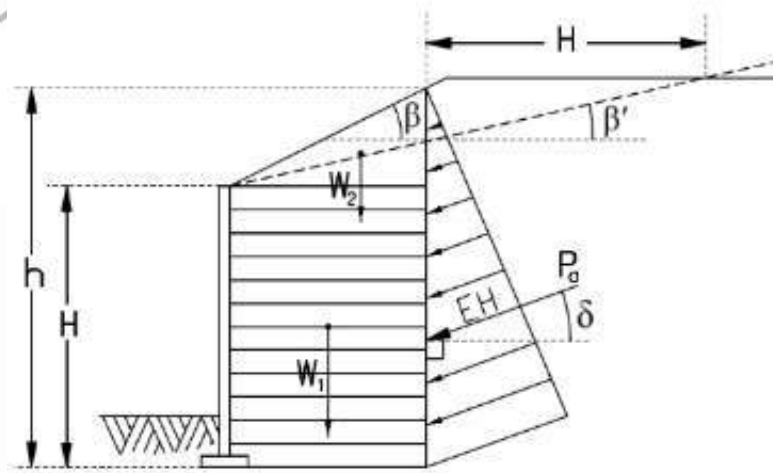
Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$\phi'_{fd} = 26 \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$c'_{fd} = 4368 \text{ psf}$$



Givens:

Wall Geometry:

$$H_e := 8.33 \text{ ft}$$

$$\theta := 90 \cdot \text{deg}$$

$$\beta_{BackSlope} := 0 \text{ deg}$$

$$\lambda := 0 \text{ ft}$$

Exposed wall height

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Inclination of ground slope behind face of wall. If it is horizontal backfill behind MSE wall, $\beta_{BackSlope} = 0$ deg

Horizontal distance from the back of MSE wall to the top of slope. If it is infinite slope behind MSE Wall, input λ larger than 2H; If it is horizontal backfill behind MSE wall, $\lambda = 0$ ft

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \text{ deg}$$

$$\gamma_r := 120 \frac{\text{lb}}{\text{ft}^3}$$

$$c'_r := 0 \frac{\text{lb}}{\text{ft}^2}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \text{ deg}$$

$$\gamma_b := 120 \frac{\text{lb}}{\text{ft}^3}$$

$$c'_b := 0 \frac{\text{lb}}{\text{ft}^2}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

$$\delta := 0.67 \cdot \phi'_b$$

$$\delta = 20.1 \text{ deg}$$

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Soil Design Parameters for Bearing Resistance (Average 1.5 B Below Leveling Pad)

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 26 \text{ deg}$$

Effective angle of internal friction

$$\gamma_{fd} := \gamma_{rock}$$

Unit weight

$$c'_{fd} = 4368 \frac{\text{lbf}}{\text{ft}^2}$$

Effective Cohesion

Undercut & Replacement Design Parameters:

$$\phi_{Re} := 34 \text{ deg}$$

Angle of internal friction for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$c_{Re} := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$\gamma_{Re} := 130 \frac{\text{lbf}}{\text{ft}^3}$$

Unit Weight for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$\delta_{Re} := \frac{2}{3} \cdot \phi_{Re} \quad \delta_{Re} = 22.7 \text{ deg}$$

Friction angle between Replacement soils and footing taken as specified in LRFD BDS C3.11.5.3 (degrees)

$$D_{undercut} := 1 \text{ ft}$$

Depth of Undercut below bottom of footing

Foundation Surcharge Soil Parameters

$$\gamma_q := 120 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Depth of Embedment Check:

$$d_{frost} := 3.25 \text{ ft}$$

$$d_{user} := 7 \text{ ft}$$

Local Frost Depth / User Input

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 7 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 15.3 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if}\left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if}\left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if}\left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5}\right)\right)\right)$$

$$d_{eSlope} = 0.8 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 7 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 15.3 \text{ ft}$$

Initial design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 0.7 \cdot H$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot \text{ft}, 0.7 \cdot H, L_{user})$$

$$L = 10.7 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L < \lambda, 0 \frac{\text{lb}f}{\text{ft}^2}, 250 \frac{\text{lb}f}{\text{ft}^2} \right) = 250 \frac{\text{lb}f}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

$$SUR_{@MSE} := \text{if} \left(L < \lambda, 0 \frac{\text{lb}f}{\text{ft}^2}, 250 \frac{\text{lb}f}{\text{ft}^2} \right) = 250 \frac{\text{lb}f}{\text{ft}^2}$$

Live load surcharge above the MSE Wall soil reinforcement (per LRFD BDS [3.11.6.4])

Calculations:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan(\beta_{BackSlope}) \quad H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h := \text{if} \left(L < \lambda, H + L \cdot \tan(\beta_{BackSlope}), H + H_{BackSlope} \right)$$

Height of retained fill at the back of the reinforced soil

$$h = 15.3 \text{ ft}$$

$$I := \text{atan} \left(\frac{H_{BackSlope}}{2 \cdot H} \right) \quad \beta := \text{if} \left(\lambda < 2 \cdot H, I, \beta_{BackSlope} \right)$$

$$I = 0 \text{ deg} \quad \beta = 0 \text{ deg}$$

Angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 2.6867$$

$$k_{ab} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right)$$

$$k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot h^2 \cdot k_{ab}$$

$$F_T = 4191.5 \frac{\text{lb}f}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{ab}$$

$$F_{SUR} = 1139.2 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 19740.7 \frac{\text{lb}f}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_2 = 0 \frac{\text{lb}f}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_3 := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lb}f}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_3 = 0 \frac{\text{lb}f}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_4 := F_T \cdot \sin(\beta)$$

$$V_4 = 0 \frac{\text{lb}f}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_5 := F_{SUR} \cdot \sin(\beta)$$

$$V_5 = 0 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_6 := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_6 = 2682.8 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1} := \frac{L}{2}$$

$$d_{v1} = 5.4 \text{ ft}$$

$$d_{v2} := \text{if} \left(L < \lambda, \frac{2L}{3}, \frac{2 \cdot \lambda}{3} \right)$$

$$d_{v2} = 0 \text{ ft}$$

$$d_{v3} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right)$$

$$d_{v3} = 5.4 \text{ ft}$$

$$d_{v4} := L$$

$$d_{v4} = 10.7 \text{ ft}$$

$$d_{v5} := L$$

$$d_{v5} = 10.7 \text{ ft}$$

$$d_{v6} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right)$$

$$d_{v6} = 5.4 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 105919 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_3 := V_3 \cdot d_{v3}$$

$$MV_3 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_4 := V_4 \cdot d_{v4}$$

$$MV_4 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_5 := V_5 \cdot d_{v5}$$

$$MV_5 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_6 := V_6 \cdot d_{v6}$$

$$MV_6 = 14394.3 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T \cdot \cos(\beta) = 4191.5 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} \cdot \cos(\beta) = 1139.2 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

Moment:

$$d_{h1} := \frac{h}{3}$$

$$d_{h1} = 5.1 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 21418.7 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{2}$$

$$d_{h2} = 7.7 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 8732.3 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1 + V_2 + V_3$$

$$V_{EV} = 19740.7 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_4$$

$$V_{EH} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS} := V_5$$

$$V_{LS} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS@MSE} := V_6$$

$$V_{LS@MSE} = 2682.8 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 4191.5 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 1139.2 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1 + MV_2 + MV_3$$

$$M_{EV} = 105919 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_4$$

$$M_{EH1} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1} := MV_5$$

$$M_{LS1} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE} := MV_6$$

$$M_{LS1@MSE} = 14394.3 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 21418.7 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 8732.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1 \quad \text{LRFD Load Modifier}$$

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot \left(\langle Ia_{EV} \cdot V_{EV} \rangle + \langle Ia_{EH} \cdot V_{EH} \rangle + Ia_{LS} \cdot \langle V_{LS} + V_{LS@MSE} \rangle \right)$$

$$V_{Ia} = 24435.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot \left(\langle Ib_{EV} \cdot V_{EV} \rangle + \langle Ib_{EH} \cdot V_{EH} \rangle + Ib_{LS} \cdot \langle V_{LS} + V_{LS@MSE} \rangle \right)$$

$$V_{Ib} = 31344.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot \left(\langle Ia_{EH} \cdot H_{EH} \rangle + \langle Ia_{LS} \cdot H_{LS} \rangle \right)$$

$$H_{Ia} = 8281 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot \left(\langle Ib_{EH} \cdot H_{EH} \rangle + \langle Ib_{LS} \cdot H_{LS} \rangle \right)$$

$$H_{Ib} = 8281 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot \left(\langle Ia_{EV} \cdot M_{EV} \rangle + \langle Ia_{EH} \cdot M_{EH1} \rangle + Ia_{LS} \cdot \langle M_{LS1} + M_{LS1@MSE} \rangle \right)$$

$$MV_{Ia} = 131109 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot \left(\langle Ib_{EV} \cdot M_{EV} \rangle + \langle Ib_{EH} \cdot M_{EH1} \rangle + Ib_{LS} \cdot \langle M_{LS1} + M_{LS1@MSE} \rangle \right)$$

$$MV_{Ib} = 168180.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot \left(\langle Ia_{EH} \cdot M_{EH2} \rangle + \langle Ia_{LS} \cdot M_{LS2} \rangle \right)$$

$$MH_{Ia} = 47409.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot \left(\langle Ib_{EH} \cdot M_{EH2} \rangle + \langle Ib_{LS} \cdot M_{LS2} \rangle \right)$$

$$MH_{Ib} = 47409.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength Ib) about the Toe "O" of base length (the pivot):

$$\Sigma M_R := MV_{Ib}$$

$$\Sigma M_R = 168180.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ib)

$$\Sigma M_O := MH_{Ib}$$

$$\Sigma M_O = 47409.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ib)

$$\Sigma V := V_{Ib}$$

$$\Sigma V = 31344.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ib)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 3.9 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \left| \frac{L}{2} - x \right|$$

$$e = 1.51 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B'_{top} := L - 2 \cdot e$$

$$B'_{top} = 7.7 \text{ ft}$$

Effective Footing Width, at the top of Foundation Preparation Layer

$$B'_{bottom} := L - 2 \cdot e + 4.25 \text{ ft}$$

$$B'_{bottom} = 12 \text{ ft}$$

Effective Footing Width, at the top of Foundation Preparation Layer (FPL), 4.25 ft is the extra length of FPL

Foundation Layout:

$$L_{Wall} := 376.54 \text{ ft}$$

Assumed Footing Length (Wall Section Length)

$$D_f := d_e$$

$$D_f = 7 \text{ ft}$$

Footing embedment

$$D_F := D_f + D_{undercut}$$

Embenment Depth at bottom of Undercut

$$d_w := D_f - 1 \text{ ft}$$

Depth of Groundwater below ground surface in front of wall

Compute Bearing Resistance at the top of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

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

$$N_q = 11.85$$

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

$$N_c = 22.25$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 12.5$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'_{top}}{5 \cdot L_{Wall}} \right) \right)$$

$$s_c = 1.011$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{top}}{L_{Wall}} \cdot \tan(\phi'_{fd}) \right), 1 \right)$$

$$s_q = 1.01$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{top}}{L_{Wall}} \right), 1 \right)$$

$$s_\gamma = 0.992$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \operatorname{atan}\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.25$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 22.497$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 14.935$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 12.436$$

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

$$q_{nd_top} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{top} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

at top of foundation preparation layer

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd_top} := \phi_b \cdot q_{nd_top} \quad q_{Rd_top} = 70.1 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd_top} = 70.1 \text{ ksf}$

Compute Bearing Resistance at the bottom of Foundation Preparation Layer:

Drained Conditions (Effective Stress):

$$N_q := \text{if}\left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan\left(45 \text{ deg} + \frac{\phi'_{fd}}{2}\right)^2, 1.0\right) \quad N_q = 11.85$$

$$N_c := \text{if}\left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14\right) \quad N_c = 22.25$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd}) \quad N_\gamma = 12.5$$

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

$$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'_{bottom}}{5 \cdot L_{Wall}}\right)\right) \quad s_c = 1.017$$

$$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'_{bottom}}{L_{Wall}}\right) \cdot \tan(\phi'_{fd}), 1\right) \quad s_q = 1.015$$

$$s_\gamma := \text{if}\left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'_{bottom}}{L_{Wall}}\right), 1\right) \quad s_\gamma = 0.987$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_F}{B'_{top}}\right)$$

$$d_q = 1.25$$

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

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

$$N_{cm} = 22.631$$

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

$$N_{qm} = 15.016$$

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

$$N_{\gamma m} = 12.38$$

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

$$q_{nd_bottom} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B'_{bottom} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

at bottom of foundation preparation layer

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

$$\phi_b := 0.65$$

Bearing resistance factor **LRFD Table 11.5.7-1.**

$$q_{Rd_bottom} := \phi_b \cdot q_{nd_bottom} \quad q_{Rd_top} = 70.1 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd_bottom} = 72.3 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location about the toe "O" of base length (the pivot):

$$e := \left| \frac{L}{2} - x \right|$$

$$e = 1.51 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$\sigma_v := \frac{\Sigma V}{B'_{top}}$$

$$\sigma_v = 4067.6 \frac{\text{lb}}{\text{ft}^2}$$

Bearing Stress

Bearing Resistance Capacity:Demand Ratio (CDR)

At the top of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_top} := \frac{q_{Rd_top}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_top} = 17.24$

At the bottom of Foundation Preparation Layer

Drained Conditions: $CDR_{Bearing_d_bottom} := \frac{q_{Rd_bottom}}{\sigma_v}$

Is the CDR > or = to 1.0?

$CDR_{Bearing_d_bottom} = 17.78$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$e_{max} := \frac{L}{3}$

$e_{max} = 3.6 \text{ ft}$

Maximum Eccentricity **LRFD [C11.6.3.3.]**

$\Sigma M_R := MV_{Ia}$

$\Sigma M_R = 131109 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$

Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$

$\Sigma M_O = 47409.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$

Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$

$\Sigma V = 24435.6 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$

$x = 3.4 \text{ ft}$

Distance from Point "O" the resultant intersects the base

$e := \left| \frac{L}{2} - x \right|$

$e = 1.94 \text{ ft}$

Wall eccentricity

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e}$

Is the CDR > or = to 1.0?

$CDR_{Eccentricity} = 1.84$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$

$R_u = 8281 \frac{\text{lb}}{\text{ft}}$

At the top of Foundation Preparation Layer

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$\Sigma V := V_{Ia}$

$\Sigma V = 24435.6 \frac{\text{lb}}{\text{ft}}$

Sum of Vertical Loads (Strength Ia)

$R_{td_top} := \Sigma V \cdot \tan(\phi_{Re})$

$R_{td_top} = 16482 \frac{\text{lb}}{\text{ft}}$

Nominal sliding resistance Drained Conditions

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$\phi_\tau := 1.0$

Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

Drained Conditions:

$\phi R_{n_d_top} := \phi_\tau \cdot R_{td_top}$

$R_{R_d_top} := \phi R_{n_d_top}$

$R_{R_d_top} = 16.5 \frac{\text{kip}}{\text{ft}}$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_top} := \frac{R_{R_d_top}}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding_d_top} = 1.99$$

At the bottom of Foundation Preparation Layer

$$\phi'_s := 29 \text{ deg}$$

Friction angle at bedrock, GDM 1303.3.5

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V := V_{Ia} \quad \Sigma V = 24435.6 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{\tau d_bottom} := \Sigma V \cdot \tan(\phi'_s) \quad R_{\tau d_bottom} = 13544.9 \frac{\text{lb}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_\tau := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

Drained Conditions: $\phi R_{n_d_bottom} := \phi_\tau \cdot R_{\tau d_bottom}$

$$R_{R_d_bottom} := \phi R_{n_d_bottom}$$

$$R_{R_d_bottom} = 13.5 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d_bottom} := \frac{R_{R_d_bottom}}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding_d_bottom} = 1.64$$

RETAINING WALL 5

Objective: To evaluate the external stability of MSE wall design with vertical wall face and broken backslope case.
Method: In accordance with ODOT Bridge Design Manual, 2026 [Sect. 307.4] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 3.11.5.8 and Sect. 11.10.5].

Assumptions:

- Horizontal backfill, Infinite backslope or Broken backslope behind MSE wall, see LRFD Sect. 3.11.5.8.1.
- For battered or vertical walls with a back face of wall angle of θ to horizontal.
- Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ .
- MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall

Estimate RMR:

PARAMETER		RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred	
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa
	Relative Rating		15	12	7	4	2	1
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%	
	Relative Rating		20	17	13	8	3	
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm	
	Relative Rating		30	25	20	10	5	
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints	
	Relative Rating		25	20	12	6	0	
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min	
		Ratio= joint water pressure/major principal stress	OR 0		OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5	
		General Conditions	OR Completely Dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems	
	Relative Rating		10		7	4	0	

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 2 + 13 + 20 + 12 + 7 = 54$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.

Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 1752 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_{rock} := 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (Hoek and Brown, 1988):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.1934266046$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.0904406983$$

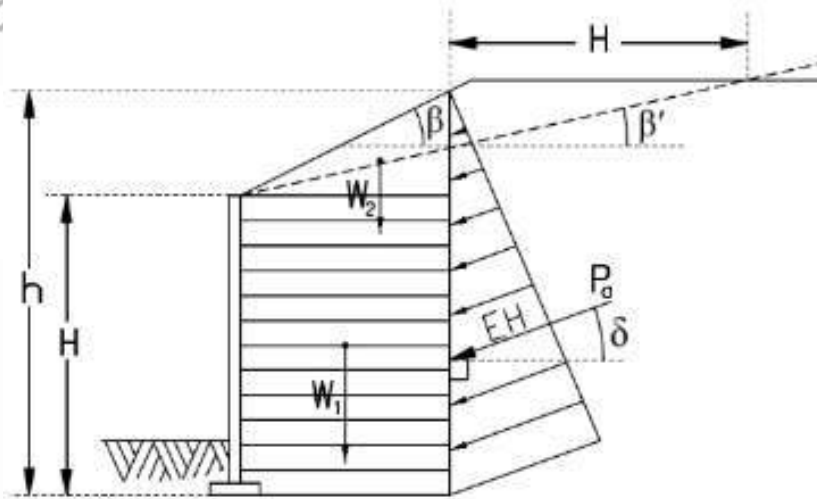
Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$\phi'_{fd} = 32 \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$c'_{fd} = 5616 \text{ psf}$$



Givens:

Wall Geometry:

$$H_e := 16.3 \text{ ft}$$

$$\theta := 90 \cdot \text{deg}$$

$$\beta_{BackSlope} := 0 \text{ deg}$$

$$\lambda := 0 \text{ ft}$$

Exposed wall height

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Inclination of ground slope behind face of wall. If it is horizontal backfill behind MSE wall, $\beta_{Backslope} = 0$ deg

Horizontal distance from the back of MSE wall to the top of slope. If it is infinite slope behind MSE Wall, input λ larger than 2H; If it is horizontal backfill behind MSE wall, $\lambda = 0$ ft

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \text{ deg}$$

$$\gamma_r := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_r := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \text{ deg}$$

$$\gamma_b := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_b := 0 \frac{\text{lbf}}{\text{ft}^2}$$

$$\delta := 0.67 \cdot \phi'_b$$

$$\delta = 20.1 \text{ deg}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Soil Design Parameters for Bearing Resistance (Average 1.5 B Below Leveling Pad)

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 32 \text{ deg}$$

Effective angle

$$\gamma_{fd} := \gamma_{rock} = 140 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 5616 \frac{\text{lb}_f}{\text{ft}^2}$$

Cohesion

Undercut & Replacement Design Parameters:

$$\phi_{Re} := 34 \text{ deg}$$

Angle of internal friction for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$c_{Re} := 0 \frac{\text{lb}_f}{\text{ft}^2}$$

Cohesion for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$\gamma_{Re} := 130 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit Weight for Replacement soil - Item 203 Granular Material Type C, C&MS 703.16.C. ODOT BDM Table 307-1.

$$\delta_{Re} := 0.67 \cdot \phi_{Re} \quad \delta_{Re} = 22.8 \text{ deg}$$

Friction angle between Replacement soils and footing taken as specified in LRFD BDS C3.11.5.3 (degrees)

$$D_{undercut} := 0.0 \text{ ft}$$

Depth of Undercut below bottom of footing

Foundation Surcharge Soil Parameters

$$\gamma_q := 120 \cdot \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Depth of Embedment Check:

$$d_{frost} := 3.25 \text{ ft}$$

$$d_{user} := 8.2 \text{ ft}$$

Local Frost Depth / User Input

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 8.2 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 24.5 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if}\left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if}\left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if}\left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5}\right)\right)\right)$$

$$d_{eSlope} = 1.2 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 8.2 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 24.5 \text{ ft}$$

Initial design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 0.7 \cdot H$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot \text{ft}, 0.7 \cdot H, L_{user})$$

$$L = 17.2 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L < \lambda, 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}^2}, 250 \frac{\text{lb} \cdot \text{ft}}{\text{ft}^2} \right) = 250 \frac{\text{lb} \cdot \text{ft}}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

$$SUR_{@MSE} := \text{if} \left(L < \lambda, 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}^2}, 250 \frac{\text{lb} \cdot \text{ft}}{\text{ft}^2} \right) = 250 \frac{\text{lb} \cdot \text{ft}}{\text{ft}^2}$$

Live load surcharge above the MSE Wall soil reinforcement (per LRFD BDS [3.11.6.4])

Calculations:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan(\beta_{BackSlope}) \quad H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h := \text{if} \left(L < \lambda, H + L \cdot \tan(\beta_{BackSlope}), H + H_{BackSlope} \right)$$

Height of retained fill at the back of the reinforced soil

$$h = 24.5 \text{ ft}$$

$$I := \text{atan} \left(\frac{H_{BackSlope}}{2 \cdot H} \right) \quad \beta := \text{if} \left(\lambda < 2 \cdot H, I, \beta_{BackSlope} \right)$$

$$I = 0 \text{ deg} \quad \beta = 0 \text{ deg}$$

Angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 2.6867$$

$$k_{ab} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right)$$

$$k_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot h^2 \cdot k_{ab}$$

$$F_T = 10705.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{ab}$$

$$F_{SUR} = 1820.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 50421 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_2 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_3 := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_3 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_4 := F_T \cdot \sin(\beta)$$

$$V_4 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_5 := F_{SUR} \cdot \sin(\beta)$$

$$V_5 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_6 := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_6 = 4287.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1} := \frac{L}{2}$$

$$d_{v1} = 8.6 \text{ ft}$$

$$d_{v2} := \text{if} \left(L < \lambda, \frac{2L}{3}, \frac{2 \cdot \lambda}{3} \right)$$

$$d_{v2} = 0 \text{ ft}$$

$$d_{v3} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right)$$

$$d_{v3} = 8.6 \text{ ft}$$

$$d_{v4} := L$$

$$d_{v4} = 17.2 \text{ ft}$$

$$d_{v5} := L$$

$$d_{v5} = 17.2 \text{ ft}$$

$$d_{v6} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right)$$

$$d_{v6} = 8.6 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1} \quad MV_1 = 432360.1 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_2 := V_2 \cdot d_{v2} \quad MV_2 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_3 := V_3 \cdot d_{v3} \quad MV_3 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_4 := V_4 \cdot d_{v4} \quad MV_4 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_5 := V_5 \cdot d_{v5} \quad MV_5 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_6 := V_6 \cdot d_{v6} \quad MV_6 = 36765.3 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T \cdot \cos(\beta) = 10705.8 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} \cdot \cos(\beta) = 1820.7 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

Moment:

$$d_{h1} := \frac{h}{3}$$

$$d_{h1} = 8.2 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 87430.9 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{2}$$

$$d_{h2} = 12.3 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 22303.8 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1 + V_2 + V_3$$

$$V_{EV} = 50421 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_4$$

$$V_{EH} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS} := V_5$$

$$V_{LS} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS@MSE} := V_6$$

$$V_{LS@MSE} = 4287.5 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 10705.8 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 1820.7 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1 + MV_2 + MV_3$$

$$M_{EV} = 432360.1 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_4$$

$$M_{EH1} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1} := MV_5$$

$$M_{LS1} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE} := MV_6$$

$$M_{LS1@MSE} = 36765.3 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 87430.9 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 22303.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1 \quad \text{LRFD Load Modifier}$$

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot \left(\langle Ia_{EV} \cdot V_{EV} \rangle + \langle Ia_{EH} \cdot V_{EH} \rangle + Ia_{LS} \cdot \langle V_{LS} + V_{LS@MSE} \rangle \right)$$

$$V_{Ia} = 57924.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot \left(\langle Ib_{EV} \cdot V_{EV} \rangle + \langle Ib_{EH} \cdot V_{EH} \rangle + Ib_{LS} \cdot \langle V_{LS} + V_{LS@MSE} \rangle \right)$$

$$V_{Ib} = 75571.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot \left(\langle Ia_{EH} \cdot H_{EH} \rangle + \langle Ia_{LS} \cdot H_{LS} \rangle \right)$$

$$H_{Ia} = 19245 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot \left(\langle Ib_{EH} \cdot H_{EH} \rangle + \langle Ib_{LS} \cdot H_{LS} \rangle \right)$$

$$H_{Ib} = 19245 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot \left(\langle Ia_{EV} \cdot M_{EV} \rangle + \langle Ia_{EH} \cdot M_{EH1} \rangle + Ia_{LS} \cdot \langle M_{LS1} + M_{LS1@MSE} \rangle \right)$$

$$MV_{Ia} = 496699.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot \left(\langle Ib_{EV} \cdot M_{EV} \rangle + \langle Ib_{EH} \cdot M_{EH1} \rangle + Ib_{LS} \cdot \langle M_{LS1} + M_{LS1@MSE} \rangle \right)$$

$$MV_{Ib} = 648025.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot \left(\langle Ia_{EH} \cdot M_{EH2} \rangle + \langle Ia_{LS} \cdot M_{LS2} \rangle \right)$$

$$MH_{Ia} = 170177.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot \left(\langle Ib_{EH} \cdot M_{EH2} \rangle + \langle Ib_{LS} \cdot M_{LS2} \rangle \right)$$

$$MH_{Ib} = 170177.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength Ib) about the Toe "O" of base length (the pivot):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 648025.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ib)

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 170177.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ib)

$$\Sigma V := V_{Ib} \quad \Sigma V = 75571.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ib)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 6.3 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \left| \frac{L}{2} - x \right|$$

$$e = 2.25 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

$$B' := L - 2 \cdot e + D_{\text{undercut}}$$

$$B' = 12.6 \text{ ft}$$

Effective Footing Width, Assumed at the bottom of Undercut and Replacement

Foundation Layout:

$$L_{\text{Wall}} := 580.85 \text{ ft}$$

Assumed Footing Length (Wall Section Length)

$$D_f := d_e$$

$$D_f = 8.2 \text{ ft}$$

Footing embedment

$$D_F := D_f + D_{\text{undercut}}$$

Embenment Depth at bottom of Undercut

$$d_w := D_f$$

Depth of Groundwater below ground surface in front of wall

Drained Conditions (Effective Stress):

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

$$N_q = 23.18$$

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

$$N_c = 35.49$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 30.2$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L_{\text{Wall}}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{\text{Wall}}} \right) \right)$$

$$s_c = 1.014$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L_{\text{Wall}}} \cdot \tan(\phi'_{fd}) \right), 1 \right)$$

$$s_q = 1.014$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{\text{Wall}}} \right), 1 \right)$$

$$s_\gamma = 0.991$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 1$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'} \right)$$

$$d_q = 1.16$$

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

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

$$N_{cm} = 35.995$$

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

$$N_{qm} = 27.224$$

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

$$N_{\gamma m} = 29.952$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 242192.8 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 157.4 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 157.4 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location about the toe "O" of base length (the pivot):

$$e := \left| \frac{L}{2} - x \right|$$

$$e = 2.25 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2]. The effective bearing width is equal to B-2e.

$$\sigma_v := \frac{\Sigma V}{B'}$$

$$\sigma_v = 5975.8 \frac{\text{lb}}{\text{ft}^2}$$

Bearing Stress

Bearing Resistance Capacity:Demand Ratio (CDR)

$$\text{Drained Conditions: } CDR_{\text{Bearing}_d} := \frac{q_{Rd}}{\sigma_v}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_d} = 26.34$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3}$$

$$e_{max} = 5.7 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]

$$\Sigma M_R := MV_{Ia}$$

$$\Sigma M_R = 496699.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia}$$

$$\Sigma M_O = 170177.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 57924.1 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 5.6 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \left| \frac{L}{2} - x \right|$$

$$e = 2.94 \text{ ft}$$

Wall eccentricity

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e}$$

Is the CDR > or = to 1.0?

$$CDR_{Eccentricity} = 1.95$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$R_u := H_{Ia}$$

$$R_u = 19245 \frac{lb_f}{ft}$$

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 57924.1 \frac{lb_f}{ft}$$

Sum of Vertical Loads (Strength Ia)

$$R_{td} := \Sigma V \cdot \tan(\phi'_{fd})$$

$$R_{td} = 36195 \frac{lb_f}{ft}$$

Nominal sliding resistance Drained Conditions

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_\tau := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

Drained Conditions:

$$\phi R_{n_d} := \phi_\tau \cdot R_{td}$$

$$R_{R_d} := \phi R_{n_d}$$

$$R_{R_d} = 36.2 \frac{kip}{ft}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d} := \frac{R_{R_d}}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding_d} = 1.88$$

RETAINING WALL 6

Objective: To evaluate the external stability of CIP wall design with sloping backfill.
Method: In accordance with ODOT Bridge Design Manual, 2024 LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Givens:

Backfill Soil Design Parameters:

$\phi'_f := 30 \text{ deg}$	Effective angle of internal friction
$\gamma_f := 120 \frac{\text{lbf}}{\text{ft}^3}$	Unit weight
$c'_f := 0 \frac{\text{lbf}}{\text{ft}^2}$	Effective Cohesion
$\delta := 0.67 \cdot \phi'_f$ $\delta = 20.1 \text{ deg}$	Friction angle between backfill and wall taken as specified in LRFD BDS C3.11.5.3 (degrees)

Foundation Soil Design Parameters:

<u>Drained Conditions (Effective Stress):</u>	
$\phi'_{fd} := 39 \text{ deg}$	Effective angle of internal friction
$\gamma_{fd} := 120 \frac{\text{lbf}}{\text{ft}^3}$	Unit weight
$c'_{fd} := 0 \frac{\text{lbf}}{\text{ft}^2}$	Effective Cohesion
$\delta_{fd} := 0.67 \cdot \phi'_{fd}$ $\delta_{fd} = 26.1 \text{ deg}$	Friction angle between foundation soils and footing taken as specified in LRFD BDS C3.11.5.3 (degrees)

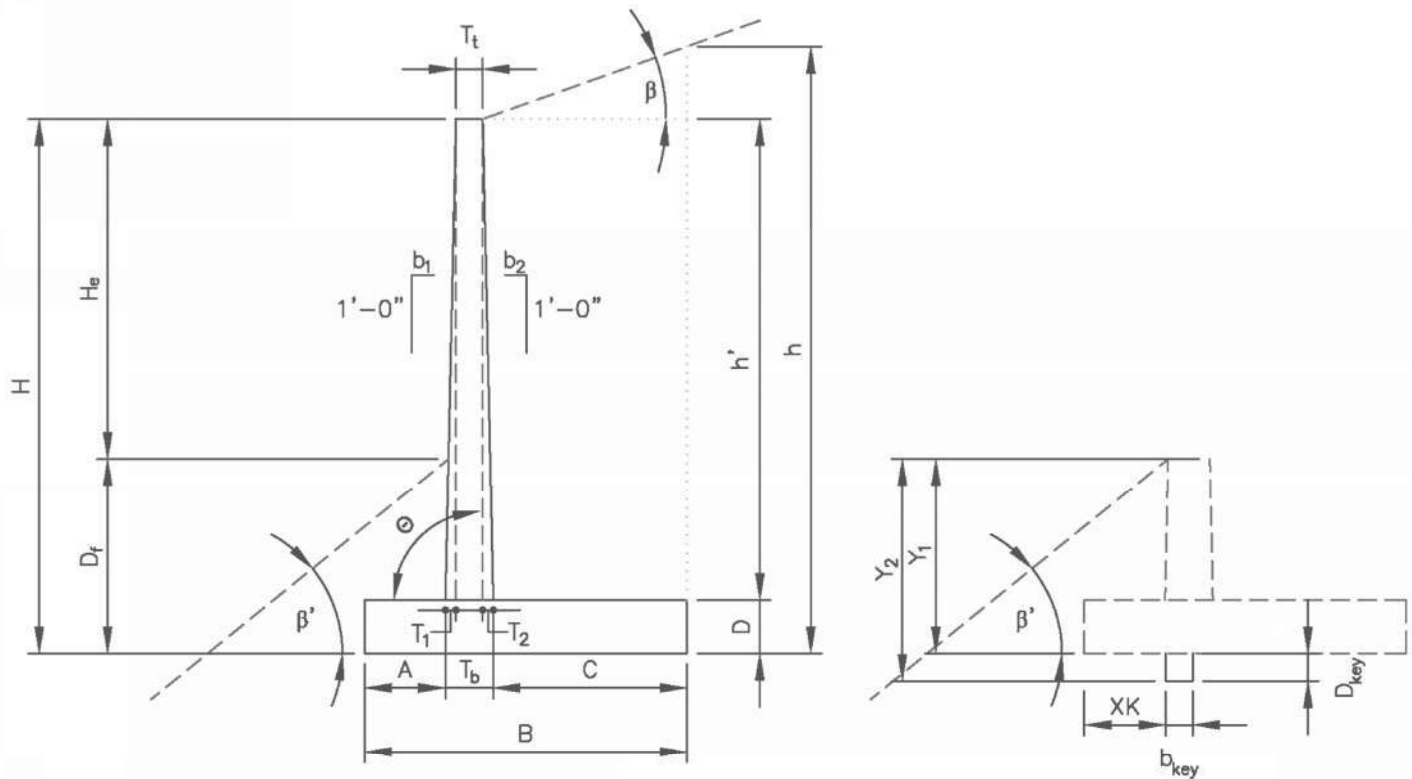
<u>Undrained Conditions (Total Stress):</u>	
$\phi_{fdu} := 39 \text{ deg}$	Angle of internal friction (Same as Drained Conditions if granular soils)
$\gamma_{fd} = 120 \frac{\text{lbf}}{\text{ft}^3}$	Unit weight
$Su_{fdu} := 0 \frac{\text{lbf}}{\text{ft}^2}$	Undrained Shear Strength
$\delta_{fdu} := 0.67 \cdot \phi_{fdu}$ $\delta_{fdu} = 26.1 \text{ deg}$	Friction angle between foundation soils and footing taken as specified in LRFD BDS C3.11.5.3 (degrees)

Foundation Surcharge Soil Parameters:

$\gamma_q := 120 \frac{\text{lbf}}{\text{ft}^3}$	Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)
--	---

Other Parameters:

$\gamma_c := 150 \frac{\text{lbf}}{\text{ft}^3}$	Concrete Unit weight
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Wall Geometry:

$$H_e := 9 \text{ ft}$$

$$D_f := 4.2 \text{ ft}$$

$$H := H_e + D_f$$

$$H = 13.2 \text{ ft}$$

$$T_t := 18 \text{ in}$$

$$b_1 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

$$b_2 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

$$\beta := 0 \text{ deg}$$

$$\beta' := 0 \text{ deg}$$

Inclination of ground slope:

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

Exposed wall height

Footing cover at Toe

Note: Where the potential for scour, erosion of undermining exists, spread footings shall be located to bear below the maximum depth of scour or undermining. Spread footings shall be located below the depth of potential frost. **LRFD BDS 10.6.1.2**

Design Wall Height

Stem thickness at top of wall

Frontwall batter, (b1H:12V)

Backwall batter, (b2H:12V)

Inclination of ground slope behind face of wall.
Horizontal backfill behind CIP wall, $\beta = 0 \text{ deg}$

Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, $\beta' = 0 \text{ deg}$. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

Preliminary Wall Dimensioning:

$B := 8 \text{ ft}$	$\frac{2}{5} \cdot H = 5.28 \text{ ft}$ to $\frac{3}{5} \cdot H = 7.92 \text{ ft}$	Footing base width (2/5H to 3/5H)
$A := 2.5 \text{ ft}$	$\frac{H}{8} = 1.65 \text{ ft}$ to $\frac{H}{5} = 2.64 \text{ ft}$	Toe projection (H/8 to H/5)
$D := 2.5 \text{ ft}$	$\frac{H}{8} = 1.65 \text{ ft}$ to $\frac{H}{5} = 2.64 \text{ ft}$	Footing thickness (H/8 to H/5)

Shear Key Dimensioning:

$D_{key} := 0 \text{ ft}$	Depth of shear key from bottom of footing Note: Footings on rock typically require shear key
$b_{key} := 0 \text{ ft}$	Width of shear key
$XK := 0 \text{ ft}$	Distance from toe to shear key

Other Wall Dimensions:

$h' := H - D$	$h' = 10.7 \text{ ft}$	Stem height
$T_1 := b_1 \cdot h'$	$T_1 = 0 \text{ ft}$	Stem front batter width
$T_2 := b_2 \cdot h'$	$T_2 = 0 \text{ ft}$	Stem back batter width
$T_b := T_1 + T_2 + T_t$	$T_b = 1.5 \text{ ft}$	Stem thickness at bottom of wall
$C := B - A - T_b$	$C = 4 \text{ ft}$	Heel projection
$\theta := 90 \text{ deg}$		Angle of back face of wall to horizontal = $\text{atan}(12/b_2)$
$b := 12 \text{ in}$	$b = 1 \text{ ft}$	Concrete strip width (for design)
$y_1 := D_f$	$y_1 = 4.2 \text{ ft}$	Depth to where passive pressure may begin to be utilized in front of wall. (Typically Df)
$y_2 := D_f + D_{key}$	$y_2 = 4.2 \text{ ft}$	Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.
$h := H + (T_2 + C) \cdot \tan(\beta)$	$h = 13.2 \text{ ft}$	Height of retained fill at back of heel

Live Load Surcharge Parameters:

$\lambda := 25 \text{ ft}$	Horizontal distance from the back of the wall to point of traffic surcharge load
$SUR := \text{if}\left(\lambda < \frac{H}{2}, 250 \frac{\text{lbf}}{\text{ft}^2}, 0 \frac{\text{lbf}}{\text{ft}^2}\right) = 0 \frac{\text{lbf}}{\text{ft}^2}$	Live load surcharge (per LRFD BDS [3.11.6.4]) Note: If vehicular loading is within 1 ft of the backface of the wall and with a design height, H, less than 20 ft, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2 for adjusted surcharge load calculation. Note: when $\lambda > H/2$, SUR equal 100 psf to account for construction loads

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{\sin(\phi'_f + \delta) \cdot \sin(\phi'_f - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}} \right)^2 \quad \Gamma = 2.687$$

$$k_{af} := \left(\frac{\sin(\theta + \phi'_f)^2}{\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta)} \right) \quad k_{af} = 0.297$$

Active Earth Pressure Coefficient
(per LRFD Sect. 3.11.5.3)

Foundation Soil Passive Earth:

Drained Conditions assuming ($\phi'_{fd} > 0$):

Input Parameters for **LRFD Figure 3.11.5.4-2**, assumes $\theta = 90$ degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0$$

$$\frac{-\delta_{fd}}{\phi'_{fd}} = -0.67$$

$$k'_p := 15$$

Passive Earth Pressure Coefficient
from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := .68$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p$$

$$k_{pd} = 10.2$$

Passive Earth Pressure Coefficient for
Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

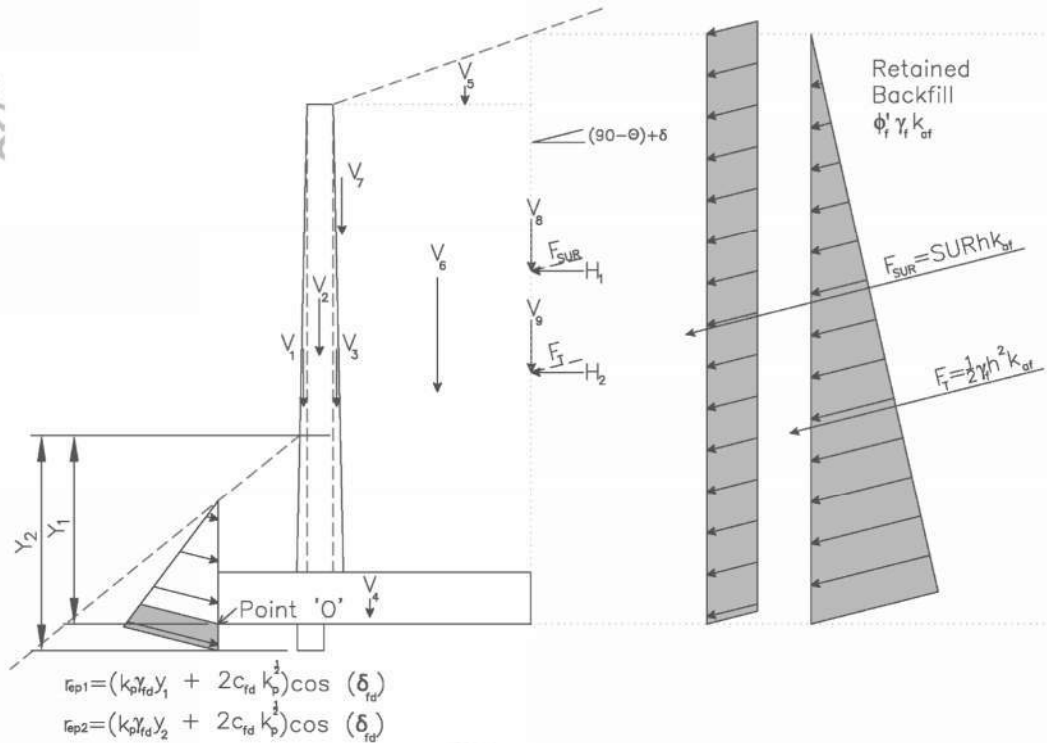
Undrained Conditions:

$$k_{pu} := \text{if}(\phi_{fdu} > 0, k_{pu}, 1)$$

$$k_{pu} = 5.978$$

Passive Earth Pressure Coefficient for
Resistance Undrained Conditions

Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:



$$F_T := \frac{1}{2} \cdot \gamma_f \cdot h^2 \cdot k_{af}$$

$$F_T = 3107.7 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{af}$$

$$F_{SUR} = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$$

$$V_1 = 0 \frac{\text{lbf}}{\text{ft}}$$

Wall stem front batter (DC)

$$V_2 := T_t \cdot h' \cdot \gamma_c$$

$$V_2 = 2407.5 \frac{\text{lbf}}{\text{ft}}$$

Wall stem (DC)+Type D barrier

$$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$$

$$V_3 = 0 \frac{\text{lbf}}{\text{ft}}$$

Wall stem back batter (DC)

$$V_4 := D \cdot B \cdot \gamma_c$$

$$V_4 = 3000 \frac{\text{lbf}}{\text{ft}}$$

Wall Footing (DC)

$$V_5 := \frac{1}{2} \cdot (T_2 + C) \cdot (T_2 + C) \cdot \tan(\beta) \cdot \gamma_f$$

$$V_5 = 0 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Backslope (EV)

$$V_6 := C \cdot h' \cdot \gamma_f$$

$$V_6 = 5136 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Heel (EV)

$$V_7 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_f$$

$$V_7 = 0 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Batter (EV)

$$V_8 := F_{SUR} \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_8 = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (vertical comp. - LS)

$$V_9 := F_T \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_9 = 1068 \frac{\text{lbf}}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

Moment Arm:

Moments produced from vertical loads about Point 'O'

$$d_{v1} := A + \frac{2}{3} \cdot T_1 = 2.5 \text{ ft}$$

$$d_{v2} := A + T_1 + \frac{T_1}{2} = 3.3 \text{ ft}$$

$$d_{v3} := A + T_1 + T_1 + \frac{T_2}{3} = 4 \text{ ft}$$

$$d_{v4} := \frac{B}{2} = 4 \text{ ft}$$

$$d_{v5} := A + T_1 + T_1 + \left(\frac{2}{3} (T_2 + C) \right) = 6.7 \text{ ft}$$

$$d_{v6} := B - \frac{C}{2} = 6 \text{ ft}$$

$$d_{v7} := A + T_1 + T_1 + \frac{2 T_2}{3} = 4 \text{ ft}$$

$$d_{v8} := B = 8 \text{ ft}$$

$$d_{v9} := B = 8 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1} = 0 \text{ lbf}$$

$$MV_2 := V_2 \cdot d_{v2} = 7824.4 \text{ lbf}$$

$$MV_3 := V_3 \cdot d_{v3} = 0 \text{ lbf}$$

$$MV_4 := V_4 \cdot d_{v4} = 12000 \text{ lbf}$$

$$MV_5 := V_5 \cdot d_{v5} = 0 \text{ lbf}$$

$$MV_6 := V_6 \cdot d_{v6} = 30816 \text{ lbf}$$

$$MV_7 := V_7 \cdot d_{v7} = 0 \text{ lbf}$$

$$MV_8 := V_8 \cdot d_{v8} = 0 \text{ lbf}$$

$$MV_9 := V_9 \cdot d_{v9} = 8543.9 \text{ lbf}$$

Horizontal Loads:

$$H_1 := F_{SUR} \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_1 = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

$$H_2 := F_T \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_2 = 2918.4 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm:

$$d_{h1} := \frac{h}{2} \quad d_{h1} = 6.6 \text{ ft}$$

$$d_{h2} := \frac{h}{3} \quad d_{h2} = 4.4 \text{ ft}$$

Moment:

$$MH_1 := H_1 \cdot d_{h1} \quad MH_1 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MH_2 := H_2 \cdot d_{h2} \quad MH_2 = 12841 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type:

$$V_{DC} := V_1 + V_2 + V_3 + V_4 \quad V_{DC} = 5407.5 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EV} := V_5 + V_6 + V_7 \quad V_{EV} = 5136 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS} := V_8 \quad V_{LS} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_9 \quad V_{EH} = 1068 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_2 \quad H_{EH} = 2918.4 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_1 \quad H_{LS} = 0 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$\begin{aligned}
 M_{DC} &:= MV_1 + MV_2 + MV_3 + MV_4 & M_{DC} &= 19824.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \\
 M_{EV} &:= MV_5 + MV_6 + MV_7 & M_{EV} &= 30816 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \\
 M_{LSV} &:= MV_8 & M_{LSV} &= 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \\
 M_{EH1} &:= MV_9 & M_{EH1} &= 8543.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \\
 M_{LSH} &:= MH_1 & M_{LSH} &= 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \\
 M_{EH2} &:= MH_2 & M_{EH2} &= 12841 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}
 \end{aligned}$$

Load Combination Limit States:

$\eta := 1$ LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity) $Ia_{DC} := 0.9$ $Ia_{EV} := 1$ $Ia_{EH} := 1.5$ $Ia_{LS} := 1.75$

Strength Limit State Ib:
(Bearing Capacity) $Ib_{DC} := 1.25$ $Ib_{EV} := 1.35$ $Ib_{EH} := 1.5$ $Ib_{LS} := 1.75$

Factored Vertical Loads by Limit State:

$$\begin{aligned}
 V_{Ia} &:= \eta \cdot ((Ia_{DC} \cdot V_{DC}) + (Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + (Ia_{LS} \cdot V_{LS})) & V_{Ia} &= 11604.7 \frac{\text{lb}}{\text{ft}} \\
 V_{Ib} &:= \eta \cdot ((Ib_{DC} \cdot V_{DC}) + (Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + (Ib_{LS} \cdot V_{LS})) & V_{Ib} &= 15294.9 \frac{\text{lb}}{\text{ft}}
 \end{aligned}$$

Factored Horizontal Loads by Limit State:

$$\begin{aligned}
 H_{Ia} &:= \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH})) & H_{Ia} &= 4377.6 \frac{\text{lb}}{\text{ft}} \\
 H_{Ib} &:= \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH})) & H_{Ib} &= 4377.6 \frac{\text{lb}}{\text{ft}}
 \end{aligned}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$\begin{aligned}
 MV_{Ia} &:= \eta \cdot ((Ia_{DC} \cdot M_{DC}) + (Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + (Ia_{LS} \cdot M_{LSV})) & MV_{Ia} &= 61473.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \\
 MV_{Ib} &:= \eta \cdot ((Ib_{DC} \cdot M_{DC}) + (Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + (Ib_{LS} \cdot M_{LSV})) & MV_{Ib} &= 79197.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}
 \end{aligned}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$\begin{aligned}
 MH_{Ia} &:= \eta \cdot ((Ia_{LS} \cdot M_{LSH}) + (Ia_{EH} \cdot M_{EH2})) & MH_{Ia} &= 19261.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \\
 MH_{Ib} &:= \eta \cdot ((Ib_{LS} \cdot M_{LSH}) + (Ib_{EH} \cdot M_{EH2})) & MH_{Ib} &= 19261.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}
 \end{aligned}$$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength lb:

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 79197.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 19261.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 15294.9 \frac{\text{lb}}{\text{ft}}$	Sum of Vertical Loads (Strength lb)
$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$	$x = 3.9 \text{ ft}$	Distance from Point "O" the resultant intersects the base
$e := \left \frac{B}{2} - x \right $	$e = 0.08 \text{ ft}$	Wall eccentricity, Note: The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2] . The effective bearing width is equal to B-2e. When the foundation eccentricity is negative the absolute value is used.

Foundation Layout:

$B' := B - 2 \cdot e$	$B' = 7.8 \text{ ft}$	Effective Footing Width
$L' := 408.53 \text{ ft}$		Effective Footing Length (Assumed)
$H' := H_{lb}$	$H' = 4377.6 \frac{\text{lb}}{\text{ft}}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 15294.9 \frac{\text{lb}}{\text{ft}}$	Summation of Vertical Loads (Strength lb)
$D_f = 4.2 \text{ ft}$		Footing embedment
$d_w := 0 \text{ ft}$		Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right)^2, 1.0 \right)$	$N_q = 55.96$
$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right)$	$N_c = 67.87$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$	$N_\gamma = 92.2$

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

$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right)$	$s_c = 1.016$
$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right)$	$s_q = 1.016$
$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right)$	$s_\gamma = 0.992$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)$$

$$d_q = 1.11$$

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

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

$$N_{cm} = 68.94$$

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

$$N_{qm} = 63.048$$

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

$$N_{\gamma m} = 91.539$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 37410.8 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .55$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 20.6 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 55.96$$

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

$$N_c = 67.87$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_{fdu})$$

$$N_\gamma = 92.2$$

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

$$s_c := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right) \quad s_c = 1.016$$

$$s_q := \text{if} \left(\phi_{fdu} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \tan(\phi_{fdu}), 1 \right) \quad s_q = 1.016$$

$$s_\gamma := \text{if} \left(\phi_{fdu} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right) \quad s_\gamma = 0.992$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see LRFD BDS C10.6.3.1.2a.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

$$i_c = 1$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 68.94$$

$$N_{qm} := N_q \cdot s_q \cdot i_q \quad N_{qm} = 56.827$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 91.539$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)$$

$$d_q = 1.11$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 68.94$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 63.048$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 91.539$$

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

$$q_{nu} := Su_{fdu} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 37410.8 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .55$$

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 20.6 \text{ ksf}$$

Bearing resistance factor LRFD Table 11.5.7-1.

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 20.6 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 20.6 \text{ ksf}$$

Evaluate External Stability of Wall:

Compute the ultimate bearing stress :

$$e = 0.08 \text{ ft}$$

$$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e}$$

$$\sigma_V = 1.952 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions:

$$CDR_{Bearing_D} := \frac{q_{Rd}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{Bearing_D} = 10.54$$

Undrained Conditions:

$$CDR_{Bearing_U} := \frac{q_{Ru}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{Bearing_U} = 10.54$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{B}{3}$$

$$e_{max} = 2.7 \text{ ft}$$

Maximum Eccentricity **LRFD [C11.6.3.3.]**
Equals B/3 for soil.

$$\Sigma M_R := MV_{Ia}$$

$$\Sigma M_R = 61473.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia}$$

$$\Sigma M_O = 19261.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 11604.7 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 3.6 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \left| \frac{B}{2} - x \right|$$

$$e = 0.36 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e. .

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e}$$

Is the CDR > or = to 1.0?

$$CDR_{Eccentricity} = 7.36$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$R_u := H_{Ia} \quad R_u = 4377.6 \frac{\text{lb}}{\text{ft}}$$

Drained Conditions (Effective Stress):

Compute passive resistance throughout the design life of the wall LRFD [Eq 3.11.5.4-1]:

$$r_{ep1} := (k_{pd} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}) \cdot \cos(\delta_{fd})$$

Nominal passive pressure at y1

$$r_{ep2} := (k_{pd} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}) \cdot \cos(\delta_{fd})$$

Nominal passive pressure at y2

$$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot (y_2 - y_1) \quad R_{ep} = 0 \frac{\text{lb}}{\text{ft}}$$

Nominal passive resistance Drained Conditions

416 Note: Passive Resistance shall be neglected in stability computations, unless the base of the wall extends below the depth of maximum scour, freeze-thaw or other disturbances. In the latter case, only the embedment below the greater of these depths shall be considered effective LRFD [11.6.3.5].

Compute sliding resistance between soil and foundation:

$$c := 1.0$$

c = 1.0 for Cast-in-Place
c = 0.8 for Precast

$$\Sigma V := V_{Ia} \quad \Sigma V = 11604.7 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{\tau} := c \cdot \Sigma V \cdot \tan(\phi'_{fd}) \quad R_{\tau} = 9397.3 \frac{\text{lb}}{\text{ft}}$$

Nominal sliding resistance Cohesionless Soils

Compute factored resistance against failure by sliding LRFD [10.6.3.4]:

$$\phi_{ep} := 0.5$$

Resistance factor for passive resistance specified in LRFD Table 10.5.5.2.2-1

$$\phi_{\tau} := 1.0$$

Resistance factor for sliding resistance specified in LRFD Table 11.5.7-1.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau} + \phi_{ep} \cdot R_{ep}$$

$$R_R := \phi R_n$$

Factored Sliding Resistance to be used in CDR Calculations:

$$R_R = 9397.32 \frac{\text{lb}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding} = 2.15$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT Bridge Design Manual [Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3 m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	OR 0		OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5		
		General Conditions	OR Completely Dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 4 + 13 + 20 + 12 + 7 = 56$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 5536 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_r := 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (Hoek and Brown, 1988):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.2077481871 \quad m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.1129467638$$

Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2}\right) \text{ deg} \quad c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 33 \text{ deg}$$

$$c'_{fd} = 5824 \text{ psf}$$

Backfill Soil Design Parameters:

$$\phi'_f := 32 \text{ deg}$$

Effective angle of internal friction

$$\gamma_f := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective Cohesion

$$\delta := \frac{2}{3} \cdot \phi'_f$$

$$\delta = 21.3 \text{ deg}$$

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 33 \text{ deg}$$

Effective angle of internal friction, from above RMR estimation

$$\gamma_{fd} := \gamma_r = 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 5824 \text{ psf}$$

Effective Cohesion, from above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd}$$

$$\delta_{fd} = 22 \text{ deg}$$

Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Foundation Surcharge Soil Parameters:

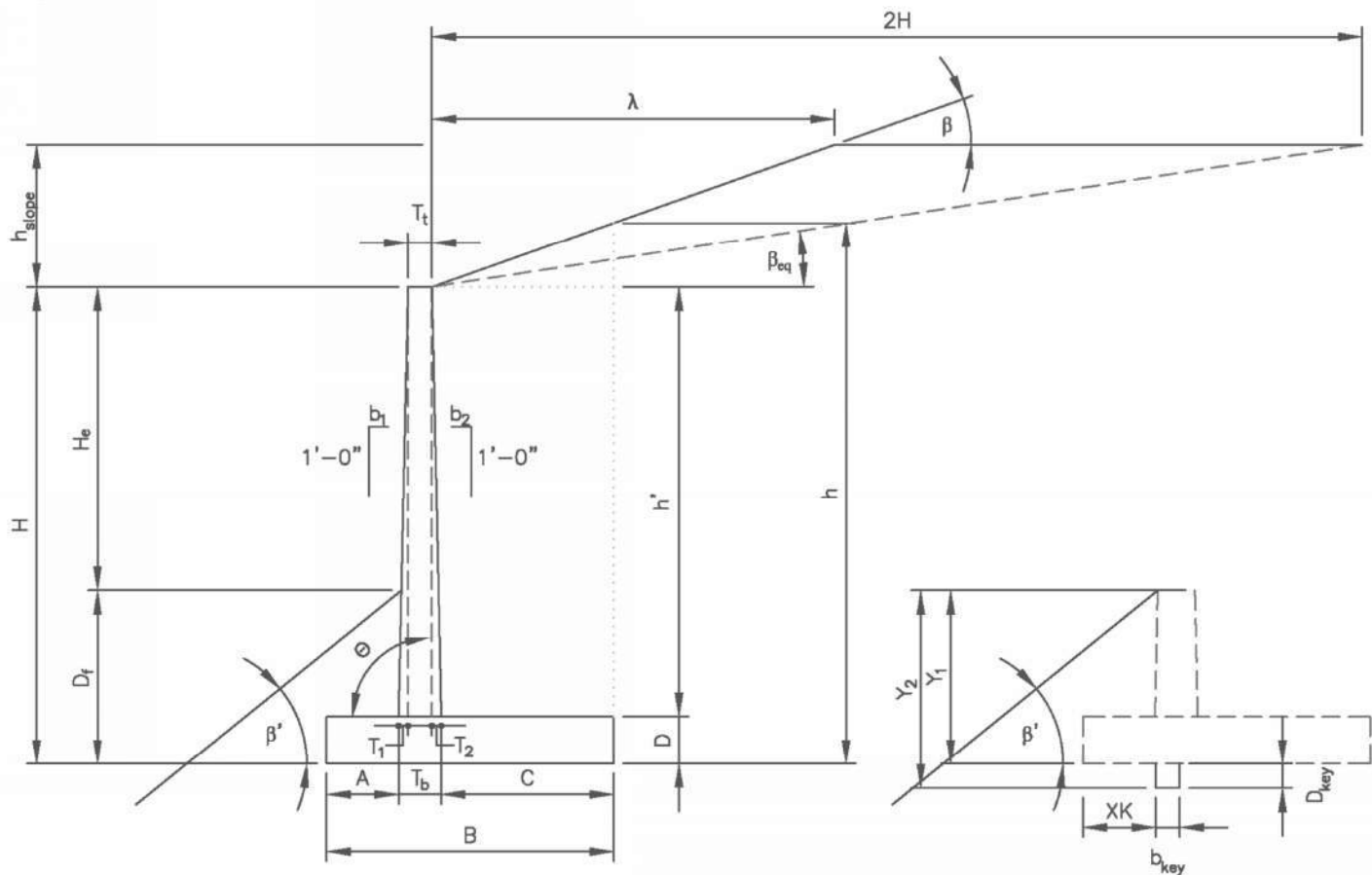
$$\gamma_q := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Other Parameters:

$$\gamma_c := 150 \frac{\text{lbf}}{\text{ft}^3}$$

Concrete Unit weight



Wall Geometry:

$$H_e := 11.9 \text{ ft}$$

$$D_f := 6.2 \text{ ft}$$

$$H := H_e + D_f \qquad H = 18.1 \text{ ft}$$

$$T_t := 18 \text{ in}$$

$$b_I := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

$$b_2 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

Exposed wall height

Footing cover at Toe

Note: Where the potential for scour, erosion of undermining exists, spread footings shall be located to bear below the maximum depth of scour or undermining. Spread footings shall be located below the depth of potential frost. **LRFD BDS 10.6.1.2**

Design Wall Height

Stem thickness at top of wall

Frontwall batter, (b1H:12V)

Backwall batter, (b2H:12V)

Preliminary Wall Dimensioning:

$$B := 8 \text{ ft} \quad \frac{2}{5} \cdot H = 7.24 \text{ ft} \text{ to } \frac{3}{5} \cdot H = 10.86 \text{ ft} \quad \text{Footing base width (2/5H to 3/5H)}$$

$A := 2.5$ *ft* $\frac{H}{8} = 2.26$ *ft* to $\frac{H}{5} = 3.62$ *ft* Toe projection (H/8 to H/5)

$$D := 2.5 \text{ ft} \qquad \frac{H}{8} = 2.26 \text{ ft} \quad \text{to} \quad \frac{H}{5} = 3.62 \text{ ft} \qquad \text{Footing thickness (H/8 to H/5)}$$

Shear Key Dimensioning:

$$D_{key} := 0 \text{ ft}$$

$$b_{key} := 0 \text{ ft}$$

$$XK := 0 \text{ ft}$$

Depth of shear key from bottom of footing

Note: Footings on rock typically require shear key

Width of shear key

Distance from toe to shear key

Other Wall Dimensions:

$$h' := H - D \quad h' = 15.6 \text{ ft}$$

$$T_1 := b_1 \cdot h' \quad T_1 = 0 \text{ ft}$$

$$T_2 := b_2 \cdot h' \quad T_2 = 0 \text{ ft}$$

$$T_b := T_1 + T_2 + T_t \quad T_b = 1.5 \text{ ft}$$

$$C := B - A - T_b \quad C = 4 \text{ ft}$$

$$\theta := 90 \text{ deg}$$

$$b := 12 \text{ in} \quad b = 1 \text{ ft}$$

$$y_1 := 3.25 \text{ ft} \quad y_1 = 3.25 \text{ ft}$$

$$y_2 := D_f + D_{key} \quad y_2 = 6.2 \text{ ft}$$

Stem height

Stem front batter width

Stem back batter width

Stem thickness at bottom of wall

Heel projection

Angle of back face of wall to horizontal = $\text{atan}(12/b2)$

Concrete strip width (for design)

Depth to where passive pressure may begin to be utilized in front of wall.

Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

Site Grading and Slope Dimensions:

$$\beta := 0 \text{ deg}$$

Inclination of ground slope:

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

Inclination of ground slope behind face of wall.
Horizontal backfill behind CIP wall, $\beta = 0 \text{ deg}$

$$\beta' := 0 \text{ deg}$$

Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, $\beta' = 0 \text{ deg}$. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

$$\lambda := 0 \text{ ft}$$

Horizontal distance from the back of the wall to point of slope crest .

$$L_{Traffic} := 0 \text{ ft}$$

Horizontal distance from assumed traffic surcharge load to Backface of Wall.

$$2 \cdot H = 36.2 \text{ ft}$$

IF λ IS GREATER THAN $2 \cdot H$ - USE INFINITE SLOPE CALCULATION SHEET

$$h_{slope} := \lambda \cdot \tan(\beta) \quad h_{slope} = 0 \text{ ft}$$

Height of broken slope behind wall

$$\beta_{eq} := \text{atan}\left(\frac{h_{slope}}{2 \cdot H}\right) = 0 \text{ deg}$$

Equivalent backslope angle

$$h := \text{if}(\lambda \leq T_2 + C, H + h_{slope}, H + (T_2 + C) \cdot \tan(\beta)) = 18.1 \text{ ft}$$

Height of retained fill at back of heel

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L_{Traffic} < \frac{H}{2}, 250 \frac{\text{lbf}}{\text{ft}^2}, 0 \frac{\text{lbf}}{\text{ft}^2} \right) = 250 \frac{\text{lbf}}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

Note: A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2.

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_f + \delta) \cdot \sin(\phi'_f - \beta_{eq}))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta_{eq}))}} \right)^2 \quad \Gamma = 2.807$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_f))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.275$$

Active Earth Pressure Coefficient
(per LRFD Sect. 3.11.5.3)

Foundation Soil Passive Earth:

Drained Conditions assuming ($\phi'_{fd} > 0$):

Input Parameters for **LRFD Figure 3.11.5.4-2**, assumes $\theta = 90$ degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0$$

$$\frac{-\delta_{fd}}{\phi'_{fd}} = -0.667$$

$$k'_p := 7.5$$

Passive Earth Pressure Coefficient
from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.752$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p$$

$$k_{pd} = 5.64$$

Passive Earth Pressure Coefficient for
Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:

$$F_T := \frac{1}{2} \cdot \gamma_f \cdot h^2 \cdot k_{af}$$

$$F_T = 5406.004 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{af}$$

$$F_{SUR} = 1244.476 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$$

$$V_1 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Wall stem front batter (DC)

$$V_2 := T_t \cdot h' \cdot \gamma_c$$

$$V_2 = 3510 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Wall stem (DC)

$$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$$

$$V_3 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Wall stem back batter (DC)

$$V_4 := D \cdot B \cdot \gamma_c$$

$$V_4 = 3000 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Wall Footing (DC)

$$V_5 := C \cdot h' \cdot \gamma_f$$

$$V_5 = 7488 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil Backfill - Heel (EV)

$$V_6 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_f$$

$$V_6 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil Backfill - Batter (EV)

$$V_7 := \text{if} \left(\lambda \leq T_2 + C, \frac{1}{2} \cdot \lambda^2 \cdot \tan(\beta) \cdot \gamma_f, \frac{1}{2} \cdot (T_2 + C)^2 \cdot \tan(\beta) \cdot \gamma_f \right)$$

$$V_7 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil Backfill - Backslope
Triangle Portion (EV)

$$V_8 := \text{if} \left(\lambda \leq T_2 + C, (T_2 + C - \lambda) \cdot h_{\text{slope}} \cdot \gamma_f, 0 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \right)$$

$$V_8 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil Backfill - Backslope
Rectangular Portion (EV)

$$V_9 := \text{if} \left(\lambda \leq T_2 + C, (T_2 + C - \lambda) \cdot SUR, 0 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \right)$$

$$V_9 = 1000 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge Above Heel - (LS)

$$V_{10} := F_{SUR} \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{10} = 452.732 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge Resultant (vertical
comp. - LS)

$$V_{11} := F_T \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{11} = 1966.667 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active earth force resultant (vertical
component - EH)

Moment Arm:

Moments produced from vertical loads about Point 'O'

$$d_{v1} := A + \frac{2}{3} \cdot T_1 = 2.5 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1} = 0 \text{ lb} \cdot \text{ft}$$

$$d_{v2} := A + T_1 + \frac{T_t}{2} = 3.25 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2} = 11407.5 \text{ lb} \cdot \text{ft}$$

$$d_{v3} := A + T_1 + T_t + \frac{T_2}{3} = 4 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3} = 0 \text{ lb} \cdot \text{ft}$$

Moment:

$$d_{v4} := \frac{B}{2} = 4 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4} = 12000 \text{ lbf}$$

$$d_{v5} := B - \frac{C}{2} = 6 \text{ ft}$$

$$MV_5 := V_5 \cdot d_{v5} = 44928 \text{ lbf}$$

$$d_{v6} := A + T_1 + T_t + \frac{2 T_2}{3} = 4 \text{ ft}$$

$$MV_6 := V_6 \cdot d_{v6} = 0 \text{ lbf}$$

$$d_{v7} := \text{if} \left(\lambda \leq T_2 + C, A + T_1 + T_t + \left(\frac{2}{3} (\lambda) \right), A + T_1 + T_t + \left(\frac{2}{3} (T_2 + C) \right) \right) = 4 \text{ ft}$$

$$MV_7 := V_7 \cdot d_{v7} = 0 \text{ lbf}$$

$$d_{v8} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2} \right) = 6 \text{ ft}$$

$$MV_8 := V_8 \cdot d_{v8} = 0 \text{ lbf}$$

$$d_{v9} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2} \right) = 6 \text{ ft}$$

$$MV_9 := V_9 \cdot d_{v9} = 6000 \text{ lbf}$$

$$d_{v10} := B = 8 \text{ ft}$$

$$MV_{10} := V_{10} \cdot d_{v10} = 3621.855 \text{ lbf}$$

$$d_{v11} := B = 8 \text{ ft}$$

$$MV_{11} := V_{11} \cdot d_{v11} = 15733.339 \text{ lbf}$$

Horizontal Loads:

$$H_1 := F_{SUR} \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_1 = 1159.204 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

$$H_2 := F_T \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_2 = 5035.583 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm:

$$d_{h1} := \frac{h}{2} \quad d_{h1} = 9.05 \text{ ft}$$

Moment:

$$MH_1 := H_1 \cdot d_{h1} \quad MH_1 = 10490.798 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{3} \quad d_{h2} = 6.033 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2} \quad MH_2 = 30381.35 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type:

$$V_{DC} := V_1 + V_2 + V_3 + V_4 \quad V_{DC} = 6510 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EV} := V_5 + V_6 + V_7 + V_8 \quad V_{EV} = 7488 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_1a} := V_{10} \quad V_{LS_1a} = 452.732 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_1b} := V_9 + V_{10} \quad V_{LS_1a} = 452.732 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_{11} \quad V_{EH} = 1966.667 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_2 \quad H_{EH} = 5035.583 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_1 \quad H_{LS} = 1159.204 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4$$

$$M_{DC} = 23407.5 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EV} := MV_5 + MV_6 + MV_7 + MV_8$$

$$M_{EV} = 44928 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_1a} := MV_{10}$$

$$M_{LSV_1a} = 3621.855 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ib} := MV_9 + MV_{10}$$

$$M_{LSV_Ib} = 9621.855 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_{11}$$

$$M_{EH1} = 15733.339 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSH} := MH_1$$

$$M_{LSH} = 10490.798 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_2$$

$$M_{EH2} = 30381.35 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1$$

LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{DC} := 0.9$$

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{DC} := 1.25$$

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot ((Ia_{DC} \cdot V_{DC}) + (Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + (Ia_{LS} \cdot V_{LS_Ia}))$$

$$V_{Ia} = 17089.282 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{DC} \cdot V_{DC}) + (Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + (Ib_{LS} \cdot V_{LS_Ib}))$$

$$V_{Ib} = 23738.582 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 9581.982 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 9581.982 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot ((Ia_{DC} \cdot M_{DC}) + (Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + (Ia_{LS} \cdot M_{LSV_Ia}))$$

$$MV_{Ia} = 95933.006 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{DC} \cdot M_{DC}) + (Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + (Ib_{LS} \cdot M_{LSV_Ib}))$$

$$MV_{Ib} = 130350.431 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LSH}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 63930.922 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LSH}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 63930.922 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength Ib:

$$\Sigma M_R := MV_{Ib}$$

$$\Sigma M_R = 130350.431 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ib)

$$\Sigma M_O := MH_{Ib}$$

$$\Sigma M_O = 63930.922 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ib)

$$\Sigma V := V_{Ib}$$

$$\Sigma V = 23738.582 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ib)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 2.798 \text{ ft}$$

Distance from Point "O" the resultant

intersects the base

$$e := \left| \frac{B}{2} - x \right|$$

$$e = 1.2 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

Foundation Layout:

$$B' := B - 2 \cdot e$$

$$B' = 5.596 \text{ ft}$$

Effective Footing Width

$$L' := 411.9 \text{ ft}$$

Effective Footing Length (Assumed)

$$H' := H_{lb}$$

$$H' = 9581.982 \frac{\text{lb}}{\text{ft}}$$

Summation of Horizontal Loads (Strength lb)

$$V' := V_{lb}$$

$$V' = 23738.582 \frac{\text{lb}}{\text{ft}}$$

Summation of Vertical Loads (Strength lb)

$$D_f = 6.2 \text{ ft}$$

Footing embedment

$$d_w := 0 \text{ ft}$$

Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

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

$$N_q = 26.09$$

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

$$N_c = 38.64$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 35.188$$

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

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

$$s_c = 1.009$$

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

$$s_q = 1.009$$

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

$$s_\gamma = 0.995$$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**. "Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{wy} := \text{if}(d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5)$$

$$C_{wy} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)$$

$$d_q = 1.23$$

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

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

$$N_{cm} = 38.993$$

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

$$N_{qm} = 32.253$$

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

$$N_{\gamma m} = 34.996$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{wy}$$

$$q_{nd} = 245946.211 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 110.676 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Evaluate External Stability of Wall:

Compute the factored bearing stress :

$$e = 1.2 \text{ ft}$$

$$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e}$$

$$\sigma_V = 4.242 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions:

$$CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 26.09$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{\max} := \frac{9 \cdot B}{20}$$

$$e_{\max} = 3.6 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]
Within 9B/10 for rock.

$$\Sigma M_R := MV_{Ia}$$

$$\Sigma M_R = 95933.006 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia}$$

$$\Sigma M_O = 63930.922 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 17089.282 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 1.873 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \left| \frac{B}{2} - x \right|$$

$$e = 2.13 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B' , since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to $B - 2e$.

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e}$$

Is the CDR > or = to 1.0?

$$CDR_{Eccentricity} = 1.69$$

$$CDR_{Sliding} := \frac{R_R}{R_u}$$

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RETAINING WALL 7

Objective: To evaluate the external stability of MSE wall design with vertical wall face and broken backslope case.
Method: In accordance with ODOT Bridge Design Manual, 2026 [Sect. 307.4] and LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 3.11.5.8 and Sect. 11.10.5].

Assumptions:

- Horizontal backfill, Infinite backslope or Broken backslope behind MSE wall, see LRFD Sect. 3.11.5.8.1.
- For battered or vertical walls with a back face of wall angle of θ to horizontal.
- Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ .
- MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall

Estimate RMR:

PARAMETER			RANGES OF VALUES					
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred	
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa
	Relative Rating		15	12	7	4	2	1
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%	
	Relative Rating		20	17	13	8	3	
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm	
	Relative Rating		30	25	20	10	5	
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints	
	Relative Rating		25	20	12	6	0	
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min	
		Ratio= joint water pressure/major principal stress	OR 0		OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5	
		General Conditions	OR Completely Dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems	
	Relative Rating		10		7	4	0	

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 4 + 8 + 20 + 12 + 7 = 51$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.

Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 6540 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_{rock} := 140 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight of bedrock

Empirical parameters (undisturbed rock), s and m (Hoek and Brown, 1988):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.1737739435$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.0648035921$$

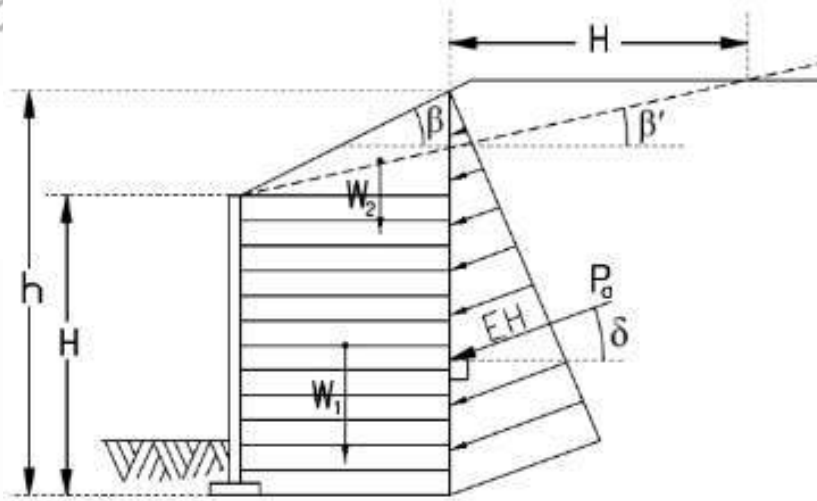
Empirical rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2} \right) \text{ deg}$$

$$\phi'_{fd} = 30.5 \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$c'_{fd} = 5304 \text{ psf}$$



Givens:

Wall Geometry:

$$H_e := 16.2 \text{ ft}$$

$$\theta := 90 \cdot \text{deg}$$

$$\beta_{BackSlope} := 0 \text{ deg}$$

$$\lambda := 0 \text{ ft}$$

Exposed wall height

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Inclination of ground slope behind face of wall. If it is horizontal backfill behind MSE wall, $\beta_{Backslope} = 0$ deg

Horizontal distance from the back of MSE wall to the top of slope. If it is infinite slope behind MSE Wall, input λ larger than 2H; If it is horizontal backfill behind MSE wall, $\lambda = 0$ ft

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \text{ deg}$$

$$\gamma_r := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_r := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \text{ deg}$$

$$\gamma_b := 120 \frac{\text{lbf}}{\text{ft}^3}$$

$$c'_b := 0 \frac{\text{lbf}}{\text{ft}^2}$$

$$\delta := \frac{2}{3} \cdot \phi'_b$$

$$\delta = 20 \text{ deg}$$

Effective angle of internal friction (Per BDM Table 307-1)

Unit weight (Per BDM Table 307-1)

Effective Cohesion

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Soil Design Parameters for Bearing Resistance (Average 1.5 B Below Leveling Pad)

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 30.5 \text{ deg} \quad \text{Effective angle}$$

$$\gamma_{fd} := \gamma_{rock} = 140 \frac{\text{lb}_f}{\text{ft}^3} \quad \text{Unit weight}$$

$$c'_{fd} = 5304 \frac{\text{lb}_f}{\text{ft}^2} \quad \text{Cohesion}$$

Undercut & Replacement Design Parameters:

$$\phi_{Re} := 34 \text{ deg} \quad \text{Angle of internal friction for Replacement soil - Item 203 Granular Material Type C, C\&MS 703.16.C. ODOT BDM Table 307-1.}$$

$$c_{Re} := 0 \frac{\text{lb}_f}{\text{ft}^2} \quad \text{Cohesion for Replacement soil - Item 203 Granular Material Type C, C\&MS 703.16.C. ODOT BDM Table 307-1.}$$

$$\gamma_{Re} := 130 \frac{\text{lb}_f}{\text{ft}^3} \quad \text{Unit Weight for Replacement soil - Item 203 Granular Material Type C, C\&MS 703.16.C. ODOT BDM Table 307-1.}$$

$$\delta_{Re} := 0.67 \cdot \phi_{Re} \quad \delta_{Re} = 22.8 \text{ deg} \quad \text{Friction angle between Replacement soils and footing taken as specified in LRFD BDS C3.11.5.3 (degrees)}$$

$$D_{undercut} := 0.0 \text{ ft} \quad \text{Depth of Undercut below bottom of footing}$$

Foundation Surcharge Soil Parameters

$$\gamma_q := 120 \cdot \frac{\text{lb}_f}{\text{ft}^3} \quad \text{Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)}$$

Depth of Embedment Check:

$$d_{frost} := 3.25 \text{ ft} \quad d_{user} := 7.1 \text{ ft} \quad \text{Local Frost Depth / User Input}$$

$$Slope_{fw} := 0 \text{ deg}$$

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user}) \quad d_{est} = 7.1 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e \quad H_{est} = 23.3 \text{ ft}$$

Inclination of ground slope in front of wall :

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if}\left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if}\left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if}\left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5}\right)\right)\right)$$

$$d_{eSlope} = 1.2 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope}) \quad d_e = 7.1 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e \quad H = 23.3 \text{ ft}$$

Initial design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 0.7 \cdot H$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot \text{ft}, 0.7 \cdot H, L_{user}) \quad L = 16.3 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L < \lambda, 0 \frac{\text{lb}f}{\text{ft}^2}, 250 \frac{\text{lb}f}{\text{ft}^2} \right) = 250 \frac{\text{lb}f}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

$$SUR_{@MSE} := \text{if} \left(L < \lambda, 0 \frac{\text{lb}f}{\text{ft}^2}, 250 \frac{\text{lb}f}{\text{ft}^2} \right) = 250 \frac{\text{lb}f}{\text{ft}^2}$$

Live load surcharge above the MSE Wall soil reinforcement (per LRFD BDS [3.11.6.4])

Calculations:

Active Earth Pressure:

$$H_{BackSlope} := \lambda \cdot \tan(\beta_{BackSlope}) \quad H_{BackSlope} = 0 \text{ ft}$$

Height of Slope behind the MSE wall

$$h := \text{if} \left(L < \lambda, H + L \cdot \tan(\beta_{BackSlope}), H + H_{BackSlope} \right)$$

Height of retained fill at the back of the reinforced soil

$$I := \text{atan} \left(\frac{H_{BackSlope}}{2 \cdot H} \right) \quad \beta := \text{if} \left(\lambda < 2 \cdot H, I, \beta_{BackSlope} \right)$$

$$I = 0 \text{ deg} \quad \beta = 0 \text{ deg}$$

Angle of friction between retained backfill and reinforced soil

$$K := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 = 2.6845$$

$$K_{ab} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right)$$

$$K_{ab} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot h^2 \cdot K_{ab}$$

$$F_T = 9684.5 \frac{\text{lb}f}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot K_{ab}$$

$$F_{SUR} = 1731.9 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 45602.8 \frac{\text{lb}f}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := \text{if} \left(L < \lambda, \frac{1}{2} \cdot \gamma_b \cdot L \cdot (h - H), \frac{1}{2} \cdot \gamma_b \cdot \lambda \cdot (h - H) \right)$$

$$V_2 = 0 \frac{\text{lb}f}{\text{ft}}$$

Triangular Soil backfill - backslope soil (EV)

$$V_3 := \text{if} \left(L < \lambda, 0 \cdot \frac{\text{lb}f}{\text{ft}}, \gamma_b \cdot (L - \lambda) \cdot (h - H) \right)$$

$$V_3 = 0 \frac{\text{lb}f}{\text{ft}}$$

Rectangular Soil backfill - backslope soil (EV)

$$V_4 := F_T \cdot \sin(\beta)$$

$$V_4 = 0 \frac{\text{lb}f}{\text{ft}}$$

Active earth force resultant (vertical component - EH)

$$V_5 := F_{SUR} \cdot \sin(\beta)$$

$$V_5 = 0 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge (vertical component - LS)

$$V_6 := SUR_{@MSE} \cdot (L - \lambda)$$

$$V_6 = 4077.5 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge above MSE Wall soil reinforcement

Moment Arm (The pivot is at the toe of MSE Wall "O"):

$$d_{v1} := \frac{L}{2}$$

$$d_{v1} = 8.2 \text{ ft}$$

$$d_{v2} := \text{if} \left(L < \lambda, \frac{2L}{3}, \frac{2 \cdot \lambda}{3} \right)$$

$$d_{v2} = 0 \text{ ft}$$

$$d_{v3} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right)$$

$$d_{v3} = 8.2 \text{ ft}$$

$$d_{v4} := L$$

$$d_{v4} = 16.3 \text{ ft}$$

$$d_{v5} := L$$

$$d_{v5} = 16.3 \text{ ft}$$

$$d_{v6} := \text{if} \left(L < \lambda, 0 \cdot \text{ft}, \lambda + \frac{(L - \lambda)}{2} \right)$$

$$d_{v6} = 8.2 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 371890.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_3 := V_3 \cdot d_{v3}$$

$$MV_3 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_4 := V_4 \cdot d_{v4}$$

$$MV_4 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_5 := V_5 \cdot d_{v5}$$

$$MV_5 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_6 := V_6 \cdot d_{v6}$$

$$MV_6 = 33252 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T \cdot \cos(\beta) = 9684.5 \frac{\text{lb}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} \cdot \cos(\beta) = 1731.9 \frac{\text{lb}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

Moment:

$$d_{h1} := \frac{h}{3}$$

$$d_{h1} = 7.8 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 75216.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{2}$$

$$d_{h2} = 11.7 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 20176.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1 + V_2 + V_3$$

$$V_{EV} = 45602.8 \frac{\text{lb}}{\text{ft}}$$

$$V_{EH} := V_4$$

$$V_{EH} = 0 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS} := V_5$$

$$V_{LS} = 0 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS@MSE} := V_6$$

$$V_{LS@MSE} = 4077.5 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 9684.5 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 1731.9 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1 + MV_2 + MV_3$$

$$M_{EV} = 371890.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_4$$

$$M_{EH1} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1} := MV_5$$

$$M_{LS1} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS1@MSE} := MV_6$$

$$M_{LS1@MSE} = 33252 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 75216.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 20176.1 \frac{lb \cdot ft}{ft}$$

Load Combination Limit States:

$\eta := 1$ LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot \left((Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + Ia_{LS} \cdot (V_{LS} + V_{LS@MSE}) \right)$$

$$V_{Ia} = 52738.4 \frac{lb}{ft}$$

$$V_{Ib} := \eta \cdot \left((Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + Ib_{LS} \cdot (V_{LS} + V_{LS@MSE}) \right)$$

$$V_{Ib} = 68699.4 \frac{lb}{ft}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot \left((Ia_{EH} \cdot H_{EH}) + (Ia_{LS} \cdot H_{LS}) \right)$$

$$H_{Ia} = 17557.5 \frac{lb}{ft}$$

$$H_{Ib} := \eta \cdot \left((Ib_{EH} \cdot H_{EH}) + (Ib_{LS} \cdot H_{LS}) \right)$$

$$H_{Ib} = 17557.5 \frac{lb}{ft}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot \left((Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + Ia_{LS} \cdot (M_{LS1} + M_{LS1@MSE}) \right)$$

$$MV_{Ia} = 430081.5 \frac{lb \cdot ft}{ft}$$

$$MV_{Ib} := \eta \cdot \left((Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + Ib_{LS} \cdot (M_{LS1} + M_{LS1@MSE}) \right)$$

$$MV_{Ib} = 560243.2 \frac{lb \cdot ft}{ft}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot \left((Ia_{EH} \cdot M_{EH2}) + (Ia_{LS} \cdot M_{LS2}) \right)$$

$$MH_{Ia} = 148132.9 \frac{lb \cdot ft}{ft}$$

$$MH_{Ib} := \eta \cdot \left((Ib_{EH} \cdot M_{EH2}) + (Ib_{LS} \cdot M_{LS2}) \right)$$

$$MH_{Ib} = 148132.9 \frac{lb \cdot ft}{ft}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength Ib) about the Toe "O" of base length (the pivot):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 560243.2 \frac{lb \cdot ft}{ft}$$

Sum of Resisting Moments (Strength Ib)

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 148132.9 \frac{lb \cdot ft}{ft}$$

Sum of Overturning Moments (Strength Ib)

$$\Sigma V := V_{Ib} \quad \Sigma V = 68699.4 \frac{lb}{ft}$$

Sum of Vertical Loads (Strength Ib)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 6 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \left| \frac{L}{2} - x \right|$$

$$e = 2.16 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B' , since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to $B-2e$.

$$B' := L - 2 \cdot e + D_{\text{undercut}}$$

$$B' = 12 \text{ ft}$$

Effective Footing Width, Assumed at the
bottom of Undercut and Replacement

Foundation Layout:

$$L_{\text{Wall}} := 218.75 \text{ ft}$$

Assumed Footing Length (Wall Section Length)

$$D_f := d_e$$

$$D_f = 7.1 \text{ ft}$$

Footing embedment

$$D_F := D_f + D_{\text{undercut}}$$

Embenment Depth at bottom of Undercut

$$d_w := 0$$

Depth of Groundwater below ground surface in front
of wall

Drained Conditions (Effective Stress):

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

$$N_q = 19.48$$

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

$$N_c = 31.37$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$$

$$N_\gamma = 24.1$$

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

$$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L_{\text{Wall}}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{\text{Wall}}} \right) \right)$$

$$s_c = 1.034$$

$$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L_{\text{Wall}}} \cdot \tan(\phi'_{fd}) \right), 1 \right)$$

$$s_q = 1.032$$

$$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{\text{Wall}}} \right), 1 \right)$$

$$s_\gamma = 0.978$$

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

$$i_q := 1$$

$$i_q = 1$$

$$i_\gamma := 1$$

$$i_\gamma = 1$$

$$i_c := 1$$

$$i_c = 1$$

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

$$C_{wq} := \text{if} (d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot L) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan} \left(\frac{D_F}{B'} \right)$$

$$d_q = 1.15$$

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

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

$$N_{cm} = 32.44$$

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

$$N_{qm} = 23.179$$

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

$$N_{\gamma m} = 23.597$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_F \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma'_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 191845.3 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 124.7 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Bearing Resistance at Base of the Wall:

Compute the resultant location about the toe "O" of base length (the pivot):

$$e := \left| \frac{L}{2} - x \right|$$

$$e = 2.16 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2]. The effective bearing width is equal to B-2e.

$$\sigma_v := \frac{\Sigma V}{B'}$$

$$\sigma_v = 5726.1 \frac{\text{lb}}{\text{ft}^2}$$

Bearing Stress

Bearing Resistance Capacity:Demand Ratio (CDR)

$$\text{Drained Conditions: } CDR_{\text{Bearing}_d} := \frac{q_{Rd}}{\sigma_v}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_d} = 21.78$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3}$$

$$e_{max} = 5.4 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]

$$\Sigma M_R := MV_{Ia}$$

$$\Sigma M_R = 430081.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia}$$

$$\Sigma M_O = 148132.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 52738.4 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 5.3 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \left| \frac{L}{2} - x \right|$$

$$e = 2.81 \text{ ft}$$

Wall eccentricity

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{\text{Eccentricity}} := \frac{e_{max}}{e}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Eccentricity}} = 1.94$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$R_u := H_{Ia} \quad R_u = 17557.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Drained Conditions (Effective Stress):

Compute sliding resistance between soil and foundation:

$$\Sigma V := V_{Ia} \quad \Sigma V = 52738.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$R_{td} := \Sigma V \cdot \tan(\phi'_{fd}) \quad R_{td} = 31065.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Nominal sliding resistance Drained Conditions}$$

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]:**

$$\phi_\tau := 1.0$$

Resistance factor for sliding resistance specified in
LRFD Table 11.5.7-1.

$$\text{Drained Conditions:} \quad \phi R_{n_d} := \phi_\tau \cdot R_{td} \quad R_{R_d} := \phi R_{n_d} \quad R_{R_d} = 31.1 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding_d} := \frac{R_{R_d}}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding_d} = 1.77$$

RETAINING WALL 8

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT Bridge Design Manual [Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 Mpa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	0		0.0 to 0.2	0.2 to 0.5	>0.5		
		General Conditions	Completely Dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 4 + 8 + 20 + 12 + 7 = 51$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 4620 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_r := 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of bedrock

Empiracle parameters (undisturbed rock), s and m (Hoek and Brown, 1988):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.1737739435 \quad m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.0648035921$$

Empiracle rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2}\right) \text{ deg} \quad c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 30.5 \text{ deg}$$

$$c'_{fd} = 5304 \text{ psf}$$

Backfill Soil Design Parameters:

$$\phi'_f := 32 \text{ deg}$$

Effective angle of internal friction

$$\gamma_f := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective Cohesion

$$\delta := \frac{2}{3} \cdot \phi'_f$$

$$\delta = 21.3 \text{ deg}$$

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 30.5 \text{ deg}$$

Effective angle of internal friction, from above RMR estimation

$$\gamma_{fd} := \gamma_r = 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 5304 \text{ psf}$$

Effective Cohesion, from above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd}$$

$$\delta_{fd} = 20.3 \text{ deg}$$

Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Foundation Surcharge Soil Parameters:

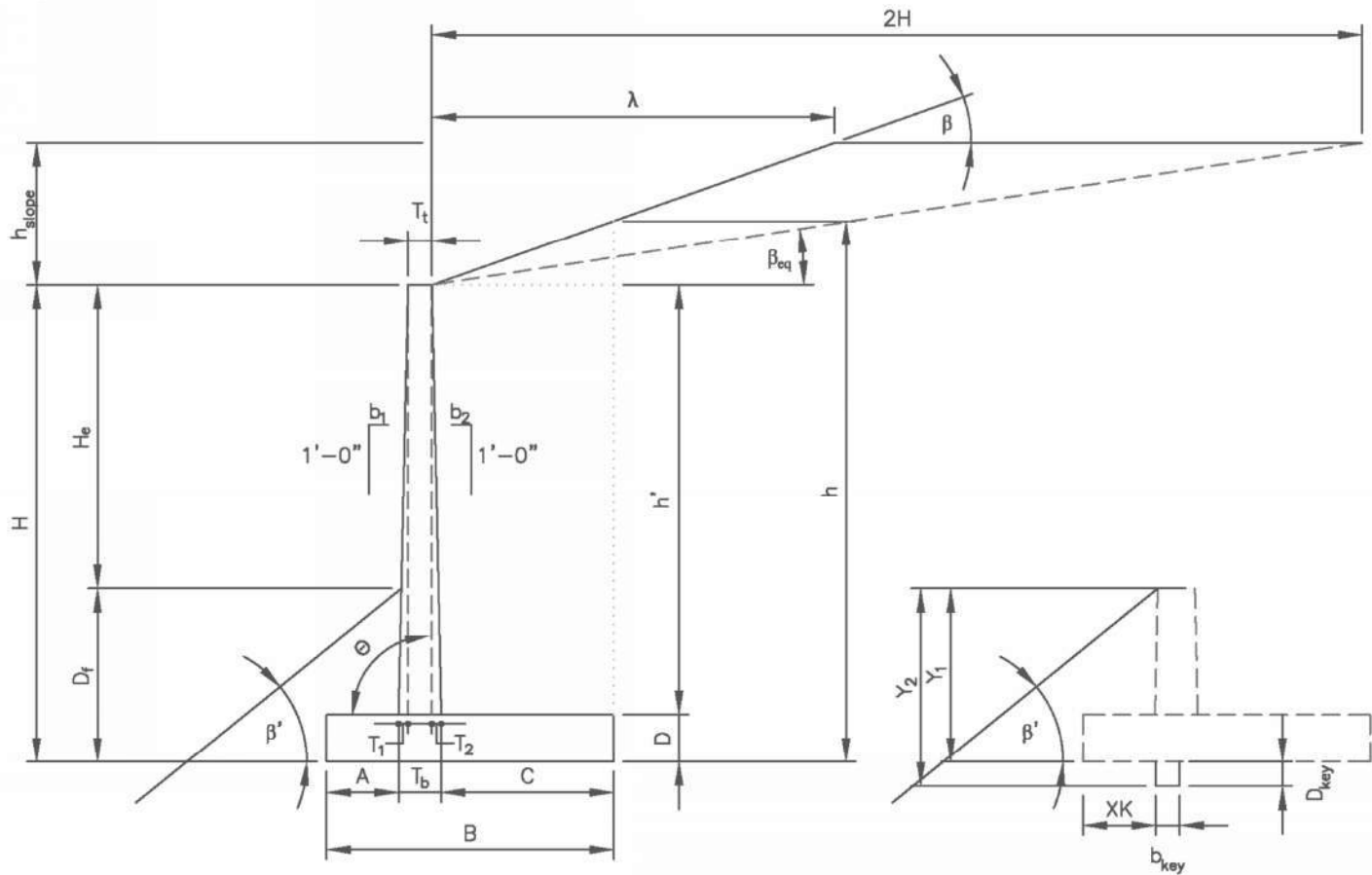
$$\gamma_q := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Other Parameters:

$$\gamma_c := 150 \frac{\text{lbf}}{\text{ft}^3}$$

Concrete Unit weight



Wall Geometry:

$$H_e := 6.3 \text{ ft}$$

$$D_f := 4.2 \text{ ft}$$

$$H := H_e + D_f$$

$$H = 10.5 \text{ ft}$$

$$T_t := 20 \text{ in}$$

$$b_1 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

$$b_2 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

Preliminary Wall Dimensioning:

$$B := 7 \text{ ft} \quad \frac{2}{5} \cdot H = 4.2 \text{ ft} \quad \text{to} \quad \frac{3}{5} \cdot H = 6.3 \text{ ft}$$

$$A := 1.5 \text{ ft} \quad \frac{H}{8} = 1.31 \text{ ft} \quad \text{to} \quad \frac{H}{5} = 2.1 \text{ ft}$$

$$D := 2 \text{ ft} \quad \frac{H}{8} = 1.31 \text{ ft} \quad \text{to} \quad \frac{H}{5} = 2.1 \text{ ft}$$

Exposed wall height

Footing cover at Toe

Note: Where the potential for scour, erosion of undermining exists, spread footings shall be located to bear below the maximum depth of scour or undermining. Spread footings shall be located below the depth of potential frost. **LRFD BDS 10.6.1.2**

Design Wall Height

Stem thickness at top of wall

Frontwall batter, (b1H:12V)

Backwall batter, (b2H:12V)

Footing base width (2/5H to 3/5H)

Toe projection (H/8 to H/5)

Footing thickness (H/8 to H/5)

Shear Key Dimensioning:

$$D_{key} := 0 \text{ ft}$$

$$b_{key} := 0 \text{ ft}$$

$$XK := 0 \text{ ft}$$

Depth of shear key from bottom of footing

Note: Footings on rock typically require shear key

Width of shear key

Distance from toe to shear key

Other Wall Dimensions:

$$h' := H - D$$

$$h' = 8.5 \text{ ft}$$

Stem height

$$T_1 := b_1 \cdot h'$$

$$T_1 = 0 \text{ ft}$$

Stem front batter width

$$T_2 := b_2 \cdot h'$$

$$T_2 = 0 \text{ ft}$$

Stem back batter width

$$T_b := T_1 + T_2 + T_t$$

$$T_b = 1.667 \text{ ft}$$

Stem thickness at bottom of wall

$$C := B - A - T_b$$

$$C = 3.833 \text{ ft}$$

Heel projection

$$\theta := 90 \text{ deg}$$

Angle of back face of wall to horizontal = $\text{atan}(12/b_2)$

$$b := 12 \text{ in}$$

$$b = 1 \text{ ft}$$

Concrete strip width (for design)

$$y_1 := 3.25 \text{ ft}$$

$$y_1 = 3.25 \text{ ft}$$

Depth to where passive pressure may begin to be utilized in front of wall.

$$y_2 := D_f + D_{key}$$

$$y_2 = 4.2 \text{ ft}$$

Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

Site Grading and Slope Dimensions:

$$\beta := 0 \text{ deg}$$

Inclination of ground slope:

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

Inclination of ground slope behind face of wall.
Horizontal backfill behind CIP wall, $\beta = 0 \text{ deg}$

$$\beta' := 0 \text{ deg}$$

Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, $\beta' = 0 \text{ deg}$. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

$$\lambda := 0 \text{ ft}$$

Horizontal distance from the back of the wall to point of slope crest .

$$L_{Traffic} := 0 \text{ ft}$$

Horizontal distance from assumed traffic surcharge load to Backface of Wall.

$$2 \cdot H = 21 \text{ ft}$$

IF λ IS GREATER THAN $2 \cdot H$ - USE INFINITE SLOPE CALCULATION SHEET

$$h_{slope} := \lambda \cdot \tan(\beta)$$

$$h_{slope} = 0 \text{ ft}$$

Height of broken slope behind wall

$$\beta_{eq} := \text{atan}\left(\frac{h_{slope}}{2 \cdot H}\right) = 0 \text{ deg}$$

Equivalent backslope angle

$$h := \text{if}(\lambda \leq T_2 + C, H + h_{slope}, H + (T_2 + C) \cdot \tan(\beta)) = 10.5 \text{ ft}$$

Height of retained fill at back of heel

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L_{Traffic} < \frac{H}{2}, 250 \frac{\text{lbf}}{\text{ft}^2}, 0 \frac{\text{lbf}}{\text{ft}^2} \right) = 250 \frac{\text{lbf}}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

Note: A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2 .

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_f + \delta) \cdot \sin(\phi'_f - \beta_{eq}))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta_{eq}))}} \right)^2 \quad \Gamma = 2.807$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_f))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.275$$

Active Earth Pressure Coefficient
(per LRFD Sect. 3.11.5.3)

Foundation Soil Passive Earth:

Drained Conditions assuming ($\phi'_{fd} > 0$):

Input Parameters for **LRFD Figure 3.11.5.4-2**, assumes $\theta = 90$ degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0$$

$$\frac{-\delta_{fd}}{\phi'_{fd}} = -0.667$$

$$k'_p := 6.5$$

Passive Earth Pressure Coefficient
from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.752$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p$$

$$k_{pd} = 4.888$$

Passive Earth Pressure Coefficient for
Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:

$$F_T := \frac{1}{2} \cdot \gamma_f \cdot h^2 \cdot k_{af}$$

$$F_T = 1819.273 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{af}$$

$$F_{SUR} = 721.934 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$$

$$V_1 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Wall stem front batter (DC)

$$V_2 := T_t \cdot h' \cdot \gamma_c + \gamma_c \cdot \frac{(1 \text{ ft} + 1.67 \text{ ft}) \cdot 3.5 \text{ ft}}{2}$$

$$V_2 = 2825.875 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Wall stem (DC)

$$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$$

$$V_3 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Wall stem back batter (DC)

$$V_4 := D \cdot B \cdot \gamma_c$$

$$V_4 = 2100 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Wall Footing (DC)

$$V_5 := C \cdot h' \cdot \gamma_f$$

$$V_5 = 3910 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil Backfill - Heel (EV)

$$V_6 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_f$$

$$V_6 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil Backfill - Batter (EV)

$$V_7 := \text{if} \left(\lambda \leq T_2 + C, \frac{1}{2} \cdot \lambda^2 \cdot \tan(\beta) \cdot \gamma_f, \frac{1}{2} \cdot (T_2 + C)^2 \cdot \tan(\beta) \cdot \gamma_f \right)$$

$$V_7 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil Backfill - Backslope
Triangle Portion (EV)

$$V_8 := \text{if} \left(\lambda \leq T_2 + C, (T_2 + C - \lambda) \cdot h_{\text{slope}} \cdot \gamma_f, 0 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \right)$$

$$V_8 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil Backfill - Backslope
Rectangular Portion (EV)

$$V_9 := \text{if} \left(\lambda \leq T_2 + C, (T_2 + C - \lambda) \cdot SUR, 0 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \right)$$

$$V_9 = 958.333 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge Above Heel - (LS)

$$V_{10} := F_{SUR} \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{10} = 262.635 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge Resultant (vertical
comp. - LS)

$$V_{11} := F_T \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{11} = 661.839 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active earth force resultant (vertical
component - EH)

Moment Arm:

Moments produced from vertical loads about Point 'O'

$$d_{v1} := A + \frac{2}{3} \cdot T_1 = 1.5 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1} = 0 \text{ lb} \cdot \text{ft}$$

$$d_{v2} := A + T_1 + \frac{T_t}{2} = 2.333 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2} = 6593.708 \text{ lb} \cdot \text{ft}$$

$$d_{v3} := A + T_1 + T_t + \frac{T_2}{3} = 3.167 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3} = 0 \text{ lb} \cdot \text{ft}$$

Moment:

$$d_{v4} := \frac{B}{2} = 3.5 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4} = 7350 \text{ lbf}$$

$$d_{v5} := B - \frac{C}{2} = 5.083 \text{ ft}$$

$$MV_5 := V_5 \cdot d_{v5} = 19875.833 \text{ lbf}$$

$$d_{v6} := A + T_1 + T_t + \frac{2 T_2}{3} = 3.167 \text{ ft}$$

$$MV_6 := V_6 \cdot d_{v6} = 0 \text{ lbf}$$

$$d_{v7} := \text{if} \left(\lambda \leq T_2 + C, A + T_1 + T_t + \left(\frac{2}{3} (\lambda) \right), A + T_1 + T_t + \left(\frac{2}{3} (T_2 + C) \right) \right) = 3.167 \text{ ft}$$

$$MV_7 := V_7 \cdot d_{v7} = 0 \text{ lbf}$$

$$d_{v8} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2} \right) = 5.083 \text{ ft}$$

$$MV_8 := V_8 \cdot d_{v8} = 0 \text{ lbf}$$

$$d_{v9} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2} \right) = 5.083 \text{ ft}$$

$$MV_9 := V_9 \cdot d_{v9} = 4871.528 \text{ lbf}$$

$$d_{v10} := B = 7 \text{ ft}$$

$$MV_{10} := V_{10} \cdot d_{v10} = 1838.442 \text{ lbf}$$

$$d_{v11} := B = 7 \text{ ft}$$

$$MV_{11} := V_{11} \cdot d_{v11} = 4632.873 \text{ lbf}$$

Horizontal Loads:

$$H_1 := F_{SUR} \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_1 = 672.467 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

$$H_2 := F_T \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_2 = 1694.616 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm:

$$d_{h1} := \frac{h}{2} \quad d_{h1} = 5.25 \text{ ft}$$

Moment:

$$MH_1 := H_1 \cdot d_{h1} \quad MH_1 = 3530.449 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{3} \quad d_{h2} = 3.5 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2} \quad MH_2 = 5931.155 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type:

$$V_{DC} := V_1 + V_2 + V_3 + V_4 \quad V_{DC} = 4925.875 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EV} := V_5 + V_6 + V_7 + V_8 \quad V_{EV} = 3910 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_1a} := V_{10} \quad V_{LS_1a} = 262.635 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_1b} := V_9 + V_{10} \quad V_{LS_1a} = 262.635 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_{11} \quad V_{EH} = 661.839 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_2 \quad H_{EH} = 1694.616 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_1 \quad H_{LS} = 672.467 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4$$

$$M_{DC} = 13943.708 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EV} := MV_5 + MV_6 + MV_7 + MV_8$$

$$M_{EV} = 19875.833 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS_1a} := MV_{10}$$

$$M_{LS_1a} = 1838.442 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ib} := MV_9 + MV_{10}$$

$$M_{EH1} := MV_{11}$$

$$M_{LSH} := MH_1$$

$$M_{EH2} := MH_2$$

$$M_{LSV_Ib} = 6709.97 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} = 4632.873 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSH} = 3530.449 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} = 5931.155 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$\eta := 1$ LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{DC} := 0.9$$

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{DC} := 1.25$$

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot \left(\langle Ia_{DC} \cdot V_{DC} \rangle + \langle Ia_{EV} \cdot V_{EV} \rangle + \langle Ia_{EH} \cdot V_{EH} \rangle + \langle Ia_{LS} \cdot V_{LS_Ia} \rangle \right) \quad V_{Ia} = 9795.656 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot \left(\langle Ib_{DC} \cdot V_{DC} \rangle + \langle Ib_{EV} \cdot V_{EV} \rangle + \langle Ib_{EH} \cdot V_{EH} \rangle + \langle Ib_{LS} \cdot V_{LS_Ib} \rangle \right) \quad V_{Ib} = 14565.296 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot \left(\langle Ia_{LS} \cdot H_{LS} \rangle + \langle Ia_{EH} \cdot H_{EH} \rangle \right) \quad H_{Ia} = 3718.74 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot \left(\langle Ib_{LS} \cdot H_{LS} \rangle + \langle Ib_{EH} \cdot H_{EH} \rangle \right) \quad H_{Ib} = 3718.74 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot \left(\langle Ia_{DC} \cdot M_{DC} \rangle + \langle Ia_{EV} \cdot M_{EV} \rangle + \langle Ia_{EH} \cdot M_{EH1} \rangle + \langle Ia_{LS} \cdot M_{LSV_Ia} \rangle \right) \quad MV_{Ia} = 42591.754 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot \left(\langle Ib_{DC} \cdot M_{DC} \rangle + \langle Ib_{EV} \cdot M_{EV} \rangle + \langle Ib_{EH} \cdot M_{EH1} \rangle + \langle Ib_{LS} \cdot M_{LSV_Ib} \rangle \right) \quad MV_{Ib} = 62953.767 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot \left(\langle Ia_{LS} \cdot M_{LSH} \rangle + \langle Ia_{EH} \cdot M_{EH2} \rangle \right) \quad MH_{Ia} = 15075.018 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot \left(\langle Ib_{LS} \cdot M_{LSH} \rangle + \langle Ib_{EH} \cdot M_{EH2} \rangle \right) \quad MH_{Ib} = 15075.018 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength lb:

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 62953.767 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 15075.018 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 14565.296 \frac{\text{lb}}{\text{ft}}$	Sum of Vertical Loads (Strength lb)
$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$	$x = 3.287 \text{ ft}$	Distance from Point "O" the resultant intersects the base
$e := \left \frac{B}{2} - x \right $	$e = 0.21 \text{ ft}$	Wall eccentricity, Note: The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2] . The effective bearing width is equal to B-2e.

Foundation Layout:

$B' := B - 2 \cdot e$	$B' = 6.574 \text{ ft}$	Effective Footing Width
$L' := 150.39 \text{ ft}$		Effective Footing Length (Assumed)
$H' := H_{lb}$	$H' = 3718.74 \frac{\text{lb}}{\text{ft}}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 14565.296 \frac{\text{lb}}{\text{ft}}$	Summation of Vertical Loads (Strength lb)
$D_f = 4.2 \text{ ft}$		Footing embedment
$d_w := 0 \text{ ft}$		Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right)$	$N_q = 19.48$
$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right)$	$N_c = 31.37$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$	$N_\gamma = 24.127$

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

$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right)$	$s_c = 1.027$
$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right)$	$s_q = 1.026$
$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right)$	$s_\gamma = 0.983$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5) \quad C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \quad C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)$$

$$d_q = 1.16$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 32.223$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 23.226$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 23.705$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nd} = 182219.96 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd} \quad q_{Rd} = 81.999 \text{ ksf} \quad \text{Factored bearing resistance Drained Conditions}$$

Evaluate External Stability of Wall:

Compute the factored bearing stress :

$$e = 0.21 \text{ ft}$$

$$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e} \quad \sigma_V = 2.215 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions:

$$CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 37.01$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 B}{20}$$

$$e_{max} = 3.15 \text{ ft}$$

Maximum Eccentricity **LRFD [C11.6.3.3.]**
Within 9B/10 for rock.

$$\Sigma M_R := MV_{Ia}$$

$$\Sigma M_R = 42591.754 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia}$$

$$\Sigma M_O = 15075.018 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 9795.656 \frac{\text{lbf}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 2.809 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \left| \frac{B}{2} - x \right|$$

$$e = 0.69 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e. .

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e}$$

Is the CDR > or = to 1.0?

$$CDR_{Eccentricity} = 4.56$$

RETAINING WALL 9

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT Bridge Design Manual [Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 Mpa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	0		0.0 to 0.2	0.2 to 0.5	>0.5		
		General Conditions	Completely Dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 2 + 13 + 20 + 12 + 7 = 54$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 2331 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_r := 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of bedrock

Empiracle parameters (undisturbed rock), s and m (Hoek and Brown, 1988):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.1934266046 \quad m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.0904406983$$

Empiracle rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2}\right) \text{ deg} \quad c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 32 \text{ deg}$$

$$c'_{fd} = 5616 \text{ psf}$$

Backfill Soil Design Parameters:

$$\phi'_f := 32 \text{ deg}$$

Effective angle of internal friction

$$\gamma_f := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective Cohesion

$$\delta := \frac{2}{3} \cdot \phi'_f \quad \delta = 21.3 \text{ deg}$$

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 32 \text{ deg}$$

Effective angle of internal friction, from above RMR estimation

$$\gamma_{fd} := \gamma_r = 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 5616 \text{ psf}$$

Effective Cohesion, from above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd} \quad \delta_{fd} = 21.3 \text{ deg}$$

Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Foundation Surcharge Soil Parameters:

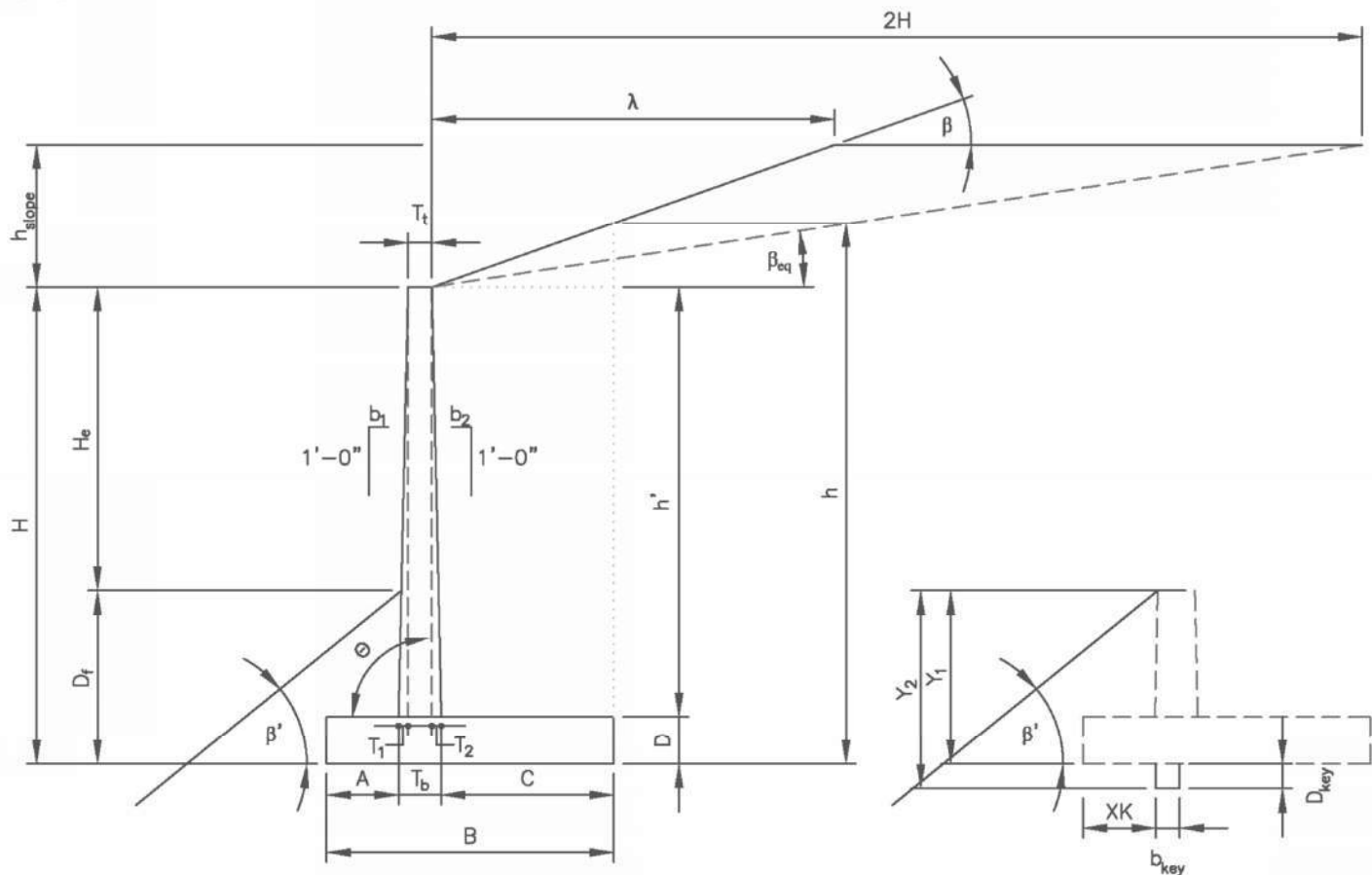
$$\gamma_q := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Other Parameters:

$$\gamma_c := 150 \frac{\text{lbf}}{\text{ft}^3}$$

Concrete Unit weight



Wall Geometry:

$$H_e := 20.3 \text{ ft}$$

$$D_f := 5.3 \text{ ft}$$

$$H := H_e + D_f$$

$$H = 25.6 \text{ ft}$$

$$T_t := 18 \text{ in}$$

$$b_1 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

$$b_2 := \frac{8}{14} \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

Preliminary Wall Dimensioning:

$$B := 15 \text{ ft}$$

$$\frac{2}{5} \cdot H = 10.24 \text{ ft to } \frac{3}{5} \cdot H = 15.36 \text{ ft}$$

Footing base width (2/5H to 3/5H)

$$A := 4 \text{ ft}$$

$$\frac{H}{8} = 3.2 \text{ ft to } \frac{H}{5} = 5.12 \text{ ft}$$

Toe projection (H/8 to H/5)

$$D := 3 \text{ ft}$$

$$\frac{H}{8} = 3.2 \text{ ft to } \frac{H}{5} = 5.12 \text{ ft}$$

Footing thickness (H/8 to H/5)

Exposed wall height

Footing cover at Toe

Note: Where the potential for scour, erosion of undermining exists, spread footings shall be located to bear below the maximum depth of scour or undermining. Spread footings shall be located below the depth of potential frost. **LRFD BDS 10.6.1.2**

Design Wall Height

Stem thickness at top of wall

Frontwall batter, (b1H:12V)

Backwall batter, (b2H:12V)

Shear Key Dimensioning:

$$D_{key} := 0 \text{ ft}$$

$$b_{key} := 0 \text{ ft}$$

$$XK := 0 \text{ ft}$$

Depth of shear key from bottom of footing

Note: Footings on rock typically require shear key

Width of shear key

Distance from toe to shear key

Other Wall Dimensions:

$$h' := H - D$$

$$h' = 22.6 \text{ ft}$$

Stem height

$$T_1 := b_1 \cdot h'$$

$$T_1 = 0 \text{ ft}$$

Stem front batter width

$$T_2 := b_2 \cdot 14 \text{ ft}$$

$$T_2 = 0.667 \text{ ft}$$

Stem back batter width

$$T_b := T_1 + T_2 + T_t$$

$$T_b = 2.167 \text{ ft}$$

Stem thickness at bottom of wall

$$C := B - A - T_b$$

$$C = 8.833 \text{ ft}$$

Heel projection

$$\theta := 90 \text{ deg}$$

Angle of back face of wall to horizontal = $\text{atan}(12/b_2)$

$$b := 12 \text{ in}$$

$$b = 1 \text{ ft}$$

Concrete strip width (for design)

$$y_1 := 3.25 \text{ ft}$$

$$y_1 = 3.25 \text{ ft}$$

Depth to where passive pressure may begin to be utilized in front of wall.

$$y_2 := D_f + D_{key}$$

$$y_2 = 5.3 \text{ ft}$$

Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

Site Grading and Slope Dimensions:

$$\beta := 26.565 \text{ deg}$$

Inclination of ground slope:

- Horizontal: 0
- 3H:1V: 18.435
- 2H:1V: 26.565
- 1.5H:1V: 33.690

Inclination of ground slope behind face of wall.
Horizontal backfill behind CIP wall, $\beta = 0 \text{ deg}$

$$\beta' := 0 \text{ deg}$$

Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, $\beta' = 0 \text{ deg}$. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

$$\lambda := 0 \text{ ft}$$

Horizontal distance from the back of the wall to point of slope crest.

$$L_{Traffic} := 0 \text{ ft}$$

Horizontal distance from assumed traffic surcharge load to Backface of Wall.

$$2 \cdot H = 51.2 \text{ ft}$$

$$h_{slope} := \lambda \cdot \tan(\beta)$$

$$h_{slope} = 0 \text{ ft}$$

Height of broken slope behind wall

$$\beta_{eq} := \text{atan}\left(\frac{h_{slope}}{2 \cdot H}\right) = 0 \text{ deg}$$

Equivalent backslope angle

$$h := \text{if}(\lambda \leq T_2 + C, H + h_{slope}, H + (T_2 + C) \cdot \tan(\beta)) = 25.6 \text{ ft}$$

Height of retained fill at back of heel

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L_{Traffic} < \frac{H}{2}, 250 \frac{\text{lb}}{\text{ft}^2}, 0 \frac{\text{lb}}{\text{ft}^2} \right) = 250 \frac{\text{lb}}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

Note: A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2.

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_f + \delta) \cdot \sin(\phi'_f - \beta_{eq}))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta_{eq}))}} \right)^2 \quad \Gamma = 2.807$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_f))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.275$$

Active Earth Pressure Coefficient
(per LRFD Sect. 3.11.5.3)

Foundation Soil Passive Earth:

Drained Conditions assuming ($\phi'_{fd} > 0$):

Input Parameters for **LRFD Figure 3.11.5.4-2**, assumes $\theta = 90$ degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0$$

$$\frac{-\delta_{fd}}{\phi'_{fd}} = -0.667$$

$$k'_p := 8$$

Passive Earth Pressure Coefficient
from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.752$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p$$

$$k_{pd} = 6.016$$

Passive Earth Pressure Coefficient for
Drained Conditions

Undrained Conditions ($\phi'_{fdu} > 0$): **Note:** Expand window below to complete calculation

Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:

$$F_T := \frac{1}{2} \cdot \gamma_f \cdot h^2 \cdot k_{af}$$

$$F_T = 10814.318 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{af}$$

$$F_{SUR} = 1760.143 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$$

$$V_1 = 0 \frac{\text{lbf}}{\text{ft}}$$

Wall stem front batter (DC)

$$V_2 := T_t \cdot h' \cdot \gamma_c$$

$$V_2 = 5085 \frac{\text{lbf}}{\text{ft}}$$

Wall stem (DC)

$$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$$

$$V_3 = 1130 \frac{\text{lbf}}{\text{ft}}$$

Wall stem back batter (DC)

$$V_4 := D \cdot B \cdot \gamma_c$$

$$V_4 = 6750 \frac{\text{lbf}}{\text{ft}}$$

Wall Footing (DC)

$$V_5 := C \cdot h' \cdot \gamma_f$$

$$V_5 = 23956 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Heel (EV)

$$V_6 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_f$$

$$V_6 = 904 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Batter (EV)

$$V_7 := \text{if} \left(\lambda \leq T_2 + C, \frac{1}{2} \cdot \lambda^2 \cdot \tan(\beta) \cdot \gamma_f, \frac{1}{2} \cdot (T_2 + C)^2 \cdot \tan(\beta) \cdot \gamma_f \right)$$

$$V_7 = 0 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Backslope
Triangle Portion (EV)

$$V_8 := \text{if} \left(\lambda \leq T_2 + C, (T_2 + C - \lambda) \cdot h_{\text{slope}} \cdot \gamma_f, 0 \cdot \frac{\text{lbf}}{\text{ft}} \right)$$

$$V_8 = 0 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Backslope
Rectangular Portion (EV)

$$V_9 := \text{if} \left(\lambda \leq T_2 + C, (T_2 + C - \lambda) \cdot SUR, 0 \cdot \frac{\text{lbf}}{\text{ft}} \right)$$

$$V_9 = 2375 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Above Heel - (LS)

$$V_{10} := F_{SUR} \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{10} = 640.328 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (vertical
comp. - LS)

$$V_{11} := F_T \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{11} = 3934.175 \frac{\text{lbf}}{\text{ft}}$$

Active earth force resultant (vertical
component - EH)

Moment Arm:

Moments produced from vertical loads about Point 'O'

$$d_{v1} := A + \frac{2}{3} \cdot T_1 = 4 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1} = 0 \text{ lbf}$$

$$d_{v2} := A + T_1 + \frac{T_t}{2} = 4.75 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2} = 24153.75 \text{ lbf}$$

$$d_{v3} := A + T_1 + T_t + \frac{T_2}{3} = 5.722 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3} = 6466.111 \text{ lbf}$$

$$d_{v4} := \frac{B}{2} = 7.5 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4} = 50625 \text{ lbf}$$

$$d_{v5} := B - \frac{C}{2} = 10.583 \text{ ft}$$

$$MV_5 := V_5 \cdot d_{v5} = 253534.333 \text{ lbf}$$

$$d_{v6} := A + T_l + T_t + \frac{2 T_2}{3} = 5.944 \text{ ft}$$

$$MV_6 := V_6 \cdot d_{v6} = 5373.778 \text{ lbf}$$

$$d_{v7} := \text{if} \left(\lambda \leq T_2 + C, A + T_l + T_t + \left(\frac{2}{3} (\lambda) \right), A + T_l + T_t + \left(\frac{2}{3} (T_2 + C) \right) \right) = 5.5 \text{ ft}$$

$$MV_7 := V_7 \cdot d_{v7} = 0 \text{ lbf}$$

$$d_{v8} := A + T_l + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2} \right) = 10.25 \text{ ft}$$

$$MV_8 := V_8 \cdot d_{v8} = 0 \text{ lbf}$$

$$d_{v9} := A + T_l + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2} \right) = 10.25 \text{ ft}$$

$$MV_9 := V_9 \cdot d_{v9} = 24343.75 \text{ lbf}$$

$$d_{v10} := B = 15 \text{ ft}$$

$$MV_{10} := V_{10} \cdot d_{v10} = 9604.92 \text{ lbf}$$

$$d_{v11} := B = 15 \text{ ft}$$

$$MV_{11} := V_{11} \cdot d_{v11} = 59012.629 \text{ lbf}$$

Horizontal Loads:

$$H_l := F_{SUR} \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_l = 1639.537 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

$$H_2 := F_T \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_2 = 10073.318 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm:

$$d_{h1} := \frac{h}{2} \quad d_{h1} = 12.8 \text{ ft}$$

Moment:

$$MH_l := H_l \cdot d_{h1} \quad MH_l = 20986.079 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{3} \quad d_{h2} = 8.533 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2} \quad MH_2 = 85958.978 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type:

$$V_{DC} := V_1 + V_2 + V_3 + V_4 \quad V_{DC} = 12965 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EV} := V_5 + V_6 + V_7 + V_8 \quad V_{EV} = 24860 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_Ia} := V_{10} \quad V_{LS_Ia} = 640.328 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_Ib} := V_9 + V_{10} \quad V_{LS_Ia} = 640.328 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_{11} \quad V_{EH} = 3934.175 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_2 \quad H_{EH} = 10073.318 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_l \quad H_{LS} = 1639.537 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4$$

$$M_{DC} = 81244.861 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EV} := MV_5 + MV_6 + MV_7 + MV_8$$

$$M_{EV} = 258908.111 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ia} := MV_{10}$$

$$M_{LSV_Ia} = 9604.92 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ib} := MV_9 + MV_{10}$$

$$M_{LSV_Ib} = 33948.67 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_{11}$$

$$M_{EH1} = 59012.629 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$M_{LSH} := MH_1$$

$$M_{LSH} = 20986.079 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_2$$

$$M_{EH2} = 85958.978 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

Load Combination Limit States:

$\eta := 1$ LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{DC} := 0.9$$

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{DC} := 1.25$$

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot \left(\langle Ia_{DC} \cdot V_{DC} \rangle + \langle Ia_{EV} \cdot V_{EV} \rangle + \langle Ia_{EH} \cdot V_{EH} \rangle + \langle Ia_{LS} \cdot V_{LS_Ia} \rangle \right)$$

$$V_{Ia} = 43550.337 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot \left(\langle Ib_{DC} \cdot V_{DC} \rangle + \langle Ib_{EV} \cdot V_{EV} \rangle + \langle Ib_{EH} \cdot V_{EH} \rangle + \langle Ib_{LS} \cdot V_{LS_Ib} \rangle \right)$$

$$V_{Ib} = 60945.337 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot \left(\langle Ia_{LS} \cdot H_{LS} \rangle + \langle Ia_{EH} \cdot H_{EH} \rangle \right)$$

$$H_{Ia} = 17979.167 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot \left(\langle Ib_{LS} \cdot H_{LS} \rangle + \langle Ib_{EH} \cdot H_{EH} \rangle \right)$$

$$H_{Ib} = 17979.167 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot \left(\langle Ia_{DC} \cdot M_{DC} \rangle + \langle Ia_{EV} \cdot M_{EV} \rangle + \langle Ia_{EH} \cdot M_{EH1} \rangle + \langle Ia_{LS} \cdot M_{LSV_Ia} \rangle \right)$$

$$MV_{Ia} = 437356.04 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot \left(\langle Ib_{DC} \cdot M_{DC} \rangle + \langle Ib_{EV} \cdot M_{EV} \rangle + \langle Ib_{EH} \cdot M_{EH1} \rangle + \langle Ib_{LS} \cdot M_{LSV_Ib} \rangle \right)$$

$$MV_{Ib} = 599011.143 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot \left(\langle Ia_{LS} \cdot M_{LSH} \rangle + \langle Ia_{EH} \cdot M_{EH2} \rangle \right)$$

$$MH_{Ia} = 165664.105 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot \left(\langle Ib_{LS} \cdot M_{LSH} \rangle + \langle Ib_{EH} \cdot M_{EH2} \rangle \right)$$

$$MH_{Ib} = 165664.105 \frac{\text{lb}\cdot\text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength lb:

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 599011.143 \frac{lb \cdot ft}{ft}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 165664.105 \frac{lb \cdot ft}{ft}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 60945.337 \frac{lb}{ft}$	Sum of Vertical Loads (Strength lb)
$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$	$x = 7.11 \text{ ft}$	Distance from Point "O" the resultant intersects the base
$e := \left \frac{B}{2} - x \right $	$e = 0.39 \text{ ft}$	Wall eccentricity, Note: The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2] . The effective bearing width is equal to B-2e.

Foundation Layout:

$B' := B - 2 \cdot e$	$B' = 14.221 \text{ ft}$	Effective Footing Width
$L' := 325.2 \text{ ft}$		Effective Footing Length (Assumed)
$H' := H_{lb}$	$H' = 17979.167 \frac{lb}{ft}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 60945.337 \frac{lb}{ft}$	Summation of Vertical Loads (Strength lb)
$D_f = 5.3 \text{ ft}$		Footing embedment
$d_w := 0 \text{ ft}$		Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right)^2, 1.0 \right)$	$N_q = 23.18$
$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right)$	$N_c = 35.49$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$	$N_\gamma = 30.215$

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

$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right)$	$s_c = 1.029$
$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right)$	$s_q = 1.027$
$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right)$	$s_\gamma = 0.983$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 36.504$$

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

$$N_{qm} = 26.156$$

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

$$N_{\gamma m} = 29.686$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 228098.404 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 102.644 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Evaluate External Stability of Wall:

Compute the factored bearing stress :

$$e = 0.39 \text{ ft}$$

$$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e}$$

$$\sigma_V = 4.286 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions:

$$CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 23.95$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 B}{20}$$

$$e_{max} = 6.75 \text{ ft}$$

Maximum Eccentricity **LRFD [C11.6.3.3.]**
Within 9B/10 for rock.

$$\Sigma M_R := MV_{Ia}$$

$$\Sigma M_R = 437356.04 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia}$$

$$\Sigma M_O = 165664.105 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 43550.337 \frac{\text{lbf}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 6.239 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \left| \frac{B}{2} - x \right|$$

$$e = 1.26 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e. .

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e}$$

Is the CDR > or = to 1.0?

$$CDR_{Eccentricity} = 5.35$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT Bridge Design Manual [Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3 m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	OR 0		OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5		
		General Conditions	OR Completely Dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 4 + 17 + 20 + 12 + 7 = 60$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 6587 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_r := 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of bedrock

Empiracle parameters (undisturbed rock), s and m (Hoek and Brown, 1988):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.2396510364$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.1761544269$$

Empiracle rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2}\right) \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 35 \text{ deg}$$

$$c'_{fd} = 6240 \text{ psf}$$

Backfill Soil Design Parameters:

$$\phi'_f := 32 \text{ deg}$$

Effective angle of internal friction

$$\gamma_f := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 0 \frac{\text{lbf}}{\text{ft}^2}$$

Effective Cohesion

$$\delta := \frac{2}{3} \cdot \phi'_f$$

$$\delta = 21.3 \text{ deg}$$

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 35 \text{ deg}$$

Effective angle of internal friction, from above RMR estimation

$$\gamma_{fd} := \gamma_r = 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 6240 \text{ psf}$$

Effective Cohesion, from above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd}$$

$$\delta_{fd} = 23.3 \text{ deg}$$

Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Foundation Surcharge Soil Parameters:

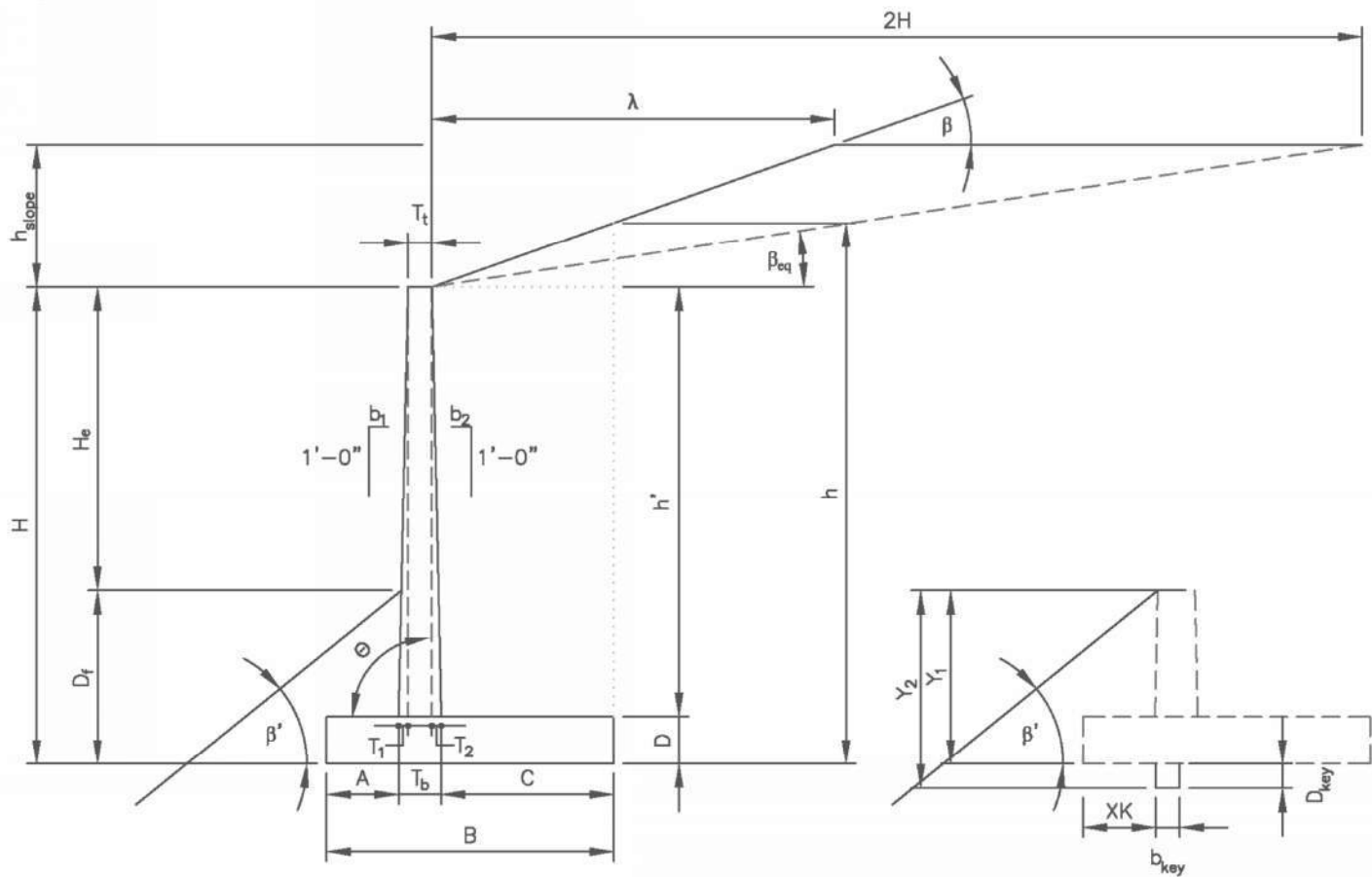
$$\gamma_q := 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Other Parameters:

$$\gamma_c := 150 \frac{\text{lbf}}{\text{ft}^3}$$

Concrete Unit weight



Wall Geometry:

$$H_e := 9.3 \text{ ft}$$

$$D_f := 4.3 \text{ ft}$$

$$H := H_e + D_f \quad H = 13.6 \text{ ft}$$

$$T_t := 18 \text{ in}$$

$$b_1 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

$$b_2 := \frac{8}{10} \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

Preliminary Wall Dimensioning:

$$B := 10 \text{ ft} \quad \frac{2}{5} \cdot H = 5.44 \text{ ft} \quad \text{to} \quad \frac{3}{5} \cdot H = 8.16 \text{ ft}$$

$$A := 2.5 \text{ ft} \quad \frac{H}{8} = 1.7 \text{ ft} \quad \text{to} \quad \frac{H}{5} = 2.72 \text{ ft}$$

$$D := 2.5 \text{ ft} \quad \frac{H}{8} = 1.7 \text{ ft} \quad \text{to} \quad \frac{H}{5} = 2.72 \text{ ft}$$

Exposed wall height

Footing cover at Toe

Note: Where the potential for scour, erosion of undermining exists, spread footings shall be located to bear below the maximum depth of scour or undermining. Spread footings shall be located below the depth of potential frost. **LRFD BDS 10.6.1.2**

Design Wall Height

Stem thickness at top of wall

Frontwall batter, (b1H:12V)

Backwall batter, (b2H:12V)

Footing base width (2/5H to 3/5H)

Toe projection (H/8 to H/5)

Footing thickness (H/8 to H/5)

Shear Key Dimensioning:

$$D_{key} := 0 \text{ ft}$$

$$b_{key} := 0 \text{ ft}$$

$$XK := 0 \text{ ft}$$

Depth of shear key from bottom of footing

Note: Footings on rock typically require shear key

Width of shear key

Distance from toe to shear key

Other Wall Dimensions:

$$h' := H - D$$

$$h' = 11.1 \text{ ft}$$

Stem height

$$T_1 := b_1 \cdot h'$$

$$T_1 = 0 \text{ ft}$$

Stem front batter width

$$T_2 := b_2 \cdot 10 \text{ ft}$$

$$T_2 = 8 \text{ in}$$

Stem back batter width

$$T_b := T_1 + T_2 + T_t$$

$$T_b = 2.167 \text{ ft}$$

Stem thickness at bottom of wall

$$C := B - A - T_b$$

$$C = 5.333 \text{ ft}$$

Heel projection

$$\theta := 90 \text{ deg}$$

Angle of back face of wall to horizontal = $\text{atan}(12/b_2)$

$$b := 12 \text{ in}$$

$$b = 1 \text{ ft}$$

Concrete strip width (for design)

$$y_1 := 3.25 \text{ ft}$$

$$y_1 = 3.25 \text{ ft}$$

Depth to where passive pressure may begin to be utilized in front of wall.

$$y_2 := D_f + D_{key}$$

$$y_2 = 4.3 \text{ ft}$$

Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

Site Grading and Slope Dimensions:

$$\beta := 26.565 \text{ deg}$$

Inclination of ground slope:

- Horizontal: 0
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

Inclination of ground slope behind face of wall.

Horizontal backfill behind CIP wall, $\beta = 0 \text{ deg}$

$$\beta' := 0 \text{ deg}$$

Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, $\beta' = 0 \text{ deg}$. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

$$\lambda := 35 \text{ ft}$$

Horizontal distance from the back of the wall to point of slope crest .

$$L_{Traffic} := 35 \text{ ft}$$

Horizontal distance from assumed traffic surcharge load to Backface of Wall.

$$2 \cdot H = 27.2 \text{ ft}$$

$$h_{slope} := \lambda \cdot \tan(\beta)$$

$$h_{slope} = 17.5 \text{ ft}$$

Height of broken slope behind wall

$$\beta_{eq} := \text{atan}\left(\frac{h_{slope}}{2 \cdot H}\right) = 32.756 \text{ deg}$$

Equivalent backslope angle

$$h := \text{if}(\lambda \leq T_2 + C, H + h_{slope}, H + (T_2 + C) \cdot \tan(\beta)) = 16.6 \text{ ft}$$

Height of retained fill at back of heel

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L_{Traffic} < \frac{H}{2}, 250 \frac{\text{lb}}{\text{ft}^2}, 0 \frac{\text{lb}}{\text{ft}^2} \right) = 0 \frac{\text{lb}}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

Note: A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2.

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_f + \delta) \cdot \sin(\phi'_f - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 \quad \Gamma = 1.695$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_f))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.455$$

Active Earth Pressure Coefficient
(per LRFD Sect. 3.11.5.3)

Foundation Soil Passive Earth:

Drained Conditions assuming ($\phi'_{fd} > 0$):

Input Parameters for **LRFD Figure 3.11.5.4-2**, assumes $\theta = 90$ degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0$$

$$\frac{-\delta_{fd}}{\phi'_{fd}} = -0.667$$

$$k'_p := 12.5$$

Passive Earth Pressure Coefficient
from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.752$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p$$

$$k_{pd} = 9.4$$

Passive Earth Pressure Coefficient for
Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:

$$F_T := \frac{1}{2} \cdot \gamma_f \cdot h^2 \cdot k_{af}$$

$$F_T = 7530.572 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{af}$$

$$F_{SUR} = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$$

$$V_1 = 0 \frac{\text{lbf}}{\text{ft}}$$

Wall stem front batter (DC)

$$V_2 := T_t \cdot h' \cdot \gamma_c$$

$$V_2 = 2497.5 \frac{\text{lbf}}{\text{ft}}$$

Wall stem (DC)

$$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$$

$$V_3 = 555 \frac{\text{lbf}}{\text{ft}}$$

Wall stem back batter (DC)

$$V_4 := D \cdot B \cdot \gamma_c$$

$$V_4 = 3750 \frac{\text{lbf}}{\text{ft}}$$

Wall Footing (DC)

$$V_5 := C \cdot h' \cdot \gamma_f$$

$$V_5 = 7104 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Heel (EV)

$$V_6 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_f$$

$$V_6 = 444 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Batter (EV)

$$V_7 := \text{if} \left(\lambda \leq T_2 + C, \frac{1}{2} \cdot \lambda^2 \cdot \tan(\beta) \cdot \gamma_f, \frac{1}{2} \cdot (T_2 + C)^2 \cdot \tan(\beta) \cdot \gamma_f \right)$$

$$V_7 = 1079.998 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Backslope
Triangle Portion (EV)

$$V_8 := \text{if} \left(\lambda \leq T_2 + C, (T_2 + C - \lambda) \cdot h_{\text{slope}} \cdot \gamma_f, 0 \cdot \frac{\text{lbf}}{\text{ft}} \right)$$

$$V_8 = 0 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Backslope
Rectangular Portion (EV)

$$V_9 := \text{if} \left(\lambda \leq T_2 + C, (T_2 + C - \lambda) \cdot SUR, 0 \cdot \frac{\text{lbf}}{\text{ft}} \right)$$

$$V_9 = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Above Heel - (LS)

$$V_{10} := F_{SUR} \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{10} = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (vertical
comp. - LS)

$$V_{11} := F_T \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{11} = 2739.571 \frac{\text{lbf}}{\text{ft}}$$

Active earth force resultant (vertical
component - EH)

Moment Arm:

Moments produced from vertical loads about Point 'O'

$$d_{v1} := A + \frac{2}{3} \cdot T_1 = 2.5 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1} = 0 \text{ lbf}$$

$$d_{v2} := A + T_1 + \frac{T_t}{2} = 3.25 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2} = 8116.875 \text{ lbf}$$

$$d_{v3} := A + T_1 + T_t + \frac{T_2}{3} = 4.222 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3} = 2343.333 \text{ lbf}$$

Moment:

$$d_{v4} := \frac{B}{2} = 5 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4} = 18750 \text{ lbf}$$

$$d_{v5} := B - \frac{C}{2} = 7.333 \text{ ft}$$

$$MV_5 := V_5 \cdot d_{v5} = 52096 \text{ lbf}$$

$$d_{v6} := A + T_1 + T_t + \frac{2 T_2}{3} = 4.444 \text{ ft}$$

$$MV_6 := V_6 \cdot d_{v6} = 1973.333 \text{ lbf}$$

$$d_{v7} := \text{if} \left(\lambda \leq T_2 + C, A + T_1 + T_t + \left(\frac{2}{3} (\lambda) \right), A + T_1 + T_t + \left(\frac{2}{3} (T_2 + C) \right) \right) = 8 \text{ ft}$$

$$MV_7 := V_7 \cdot d_{v7} = 8639.981 \text{ lbf}$$

$$d_{v8} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2} \right) = 24.5 \text{ ft}$$

$$MV_8 := V_8 \cdot d_{v8} = 0 \text{ lbf}$$

$$d_{v9} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2} \right) = 24.5 \text{ ft}$$

$$MV_9 := V_9 \cdot d_{v9} = 0 \text{ lbf}$$

$$d_{v10} := B = 10 \text{ ft}$$

$$MV_{10} := V_{10} \cdot d_{v10} = 0 \text{ lbf}$$

$$d_{v11} := B = 10 \text{ ft}$$

$$MV_{11} := V_{11} \cdot d_{v11} = 27395.71 \text{ lbf}$$

Horizontal Loads:

$$H_1 := F_{SUR} \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_1 = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

$$H_2 := F_T \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_2 = 7014.575 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm:

$$d_{h1} := \frac{h}{2} \quad d_{h1} = 8.3 \text{ ft}$$

Moment:

$$MH_1 := H_1 \cdot d_{h1} \quad MH_1 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{3} \quad d_{h2} = 5.533 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2} \quad MH_2 = 38813.968 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type:

$$V_{DC} := V_1 + V_2 + V_3 + V_4 \quad V_{DC} = 6802.5 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EV} := V_5 + V_6 + V_7 + V_8 \quad V_{EV} = 8627.998 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_1a} := V_{10} \quad V_{LS_1a} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_1b} := V_9 + V_{10} \quad V_{LS_1a} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_{11} \quad V_{EH} = 2739.571 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_2 \quad H_{EH} = 7014.575 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_1 \quad H_{LS} = 0 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4$$

$$M_{DC} = 29210.208 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EV} := MV_5 + MV_6 + MV_7 + MV_8$$

$$M_{EV} = 62709.314 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ia} := MV_{10}$$

$$M_{LSV_Ia} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ib} := MV_9 + MV_{10}$$

$$M_{LSV_Ib} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_{11}$$

$$M_{EH1} = 27395.71 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSH} := MH_1$$

$$M_{LSH} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_2$$

$$M_{EH2} = 38813.968 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$$\eta := 1$$

LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{DC} := 0.9$$

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{DC} := 1.25$$

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot ((Ia_{DC} \cdot V_{DC}) + (Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + (Ia_{LS} \cdot V_{LS_Ia}))$$

$$V_{Ia} = 18859.604 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{DC} \cdot V_{DC}) + (Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + (Ib_{LS} \cdot V_{LS_Ib}))$$

$$V_{Ib} = 24260.278 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 10521.863 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 10521.863 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot ((Ia_{DC} \cdot M_{DC}) + (Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + (Ia_{LS} \cdot M_{LSV_Ia}))$$

$$MV_{Ia} = 130092.066 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{DC} \cdot M_{DC}) + (Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + (Ib_{LS} \cdot M_{LSV_Ib}))$$

$$MV_{Ib} = 162263.899 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LSH}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 58220.952 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LSH}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 58220.952 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength lb:

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 162263.899 \frac{lb \cdot ft}{ft}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 58220.952 \frac{lb \cdot ft}{ft}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 24260.278 \frac{lb}{ft}$	Sum of Vertical Loads (Strength lb)
$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$	$x = 4.289 \text{ ft}$	Distance from Point "O" the resultant intersects the base
$e := \left \frac{B}{2} - x \right $	$e = 0.71 \text{ ft}$	Wall eccentricity, Note: The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2] . The effective bearing width is equal to B-2e.

Foundation Layout:

$B' := B - 2 \cdot e$	$B' = 8.577 \text{ ft}$	Effective Footing Width
$L' := 325.2 \text{ ft}$		Effective Footing Length (Assumed)
$H' := H_{lb}$	$H' = 10521.863 \frac{lb}{ft}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 24260.278 \frac{lb}{ft}$	Summation of Vertical Loads (Strength lb)
$D_f = 4.3 \text{ ft}$		Footing embedment
$d_w := 0 \text{ ft}$		Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right)$	$N_q = 33.3$
$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right)$	$N_c = 46.12$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$	$N_\gamma = 48.029$

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

$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right)$	$s_c = 1.019$
$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right)$	$s_q = 1.018$
$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right)$	$s_\gamma = 0.989$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5) \quad C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \quad C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)$$

$$d_q = 1.12$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 47.002$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 37.924$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 47.522$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nd} = 317341.81 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd} \quad q_{Rd} = 142.804 \text{ ksf} \quad \text{Factored bearing resistance Drained Conditions}$$

Evaluate External Stability of Wall:

Compute the factored bearing stress :

$$e = 0.71 \text{ ft}$$

$$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e} \quad \sigma_V = 2.828 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$ Is the CDR > or = to 1.0? $CDR_{\text{Bearing}_D} = 50.49$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 B}{20}$$

$$e_{max} = 4.5 \text{ ft}$$

Maximum Eccentricity **LRFD [C11.6.3.3.]**
Within 9B/10 for rock.

$$\Sigma M_R := MV_{Ia}$$

$$\Sigma M_R = 130092.066 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia}$$

$$\Sigma M_O = 58220.952 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 18859.604 \frac{\text{lbf}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 3.811 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \left| \frac{B}{2} - x \right|$$

$$e = 1.19 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e. .

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e}$$

Is the CDR > or = to 1.0?

$$CDR_{Eccentricity} = 3.78$$

RETAINING WALL 10

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT Bridge Design Manual [Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3 m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	OR 0		OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5		
		General Conditions	OR Completely Dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 4 + 20 + 20 + 12 + 7 = 63$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 5520 \text{ psi}$$

Unconfined compressive strength of rock core, based on testing

$$\gamma_r := 140 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of bedrock

Empiracle parameters (undisturbed rock), s and m (Hoek and Brown, 1988):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.2667539526$$

$$m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.2458433069$$

Empiracle rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2}\right) \text{ deg}$$

$$c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 36.5 \text{ deg}$$

$$c'_{fd} = 6552 \text{ psf}$$

Backfill Soil Design Parameters:

$$\phi'_f := 32 \text{ deg}$$

Effective angle of internal friction

$$\gamma_f := 120 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight

$$c'_f := 0 \frac{\text{lb}_f}{\text{ft}^2}$$

Effective Cohesion

$$\delta := \frac{2}{3} \cdot \phi'_f$$

$$\delta = 21.3 \text{ deg}$$

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 36.5 \text{ deg}$$

Effective angle of internal friction, from above RMR estimation

$$\gamma_{fd} := \gamma_r = 140 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 6552 \text{ psf}$$

Effective Cohesion, from above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd}$$

$$\delta_{fd} = 24.3 \text{ deg}$$

Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Foundation Surcharge Soil Parameters:

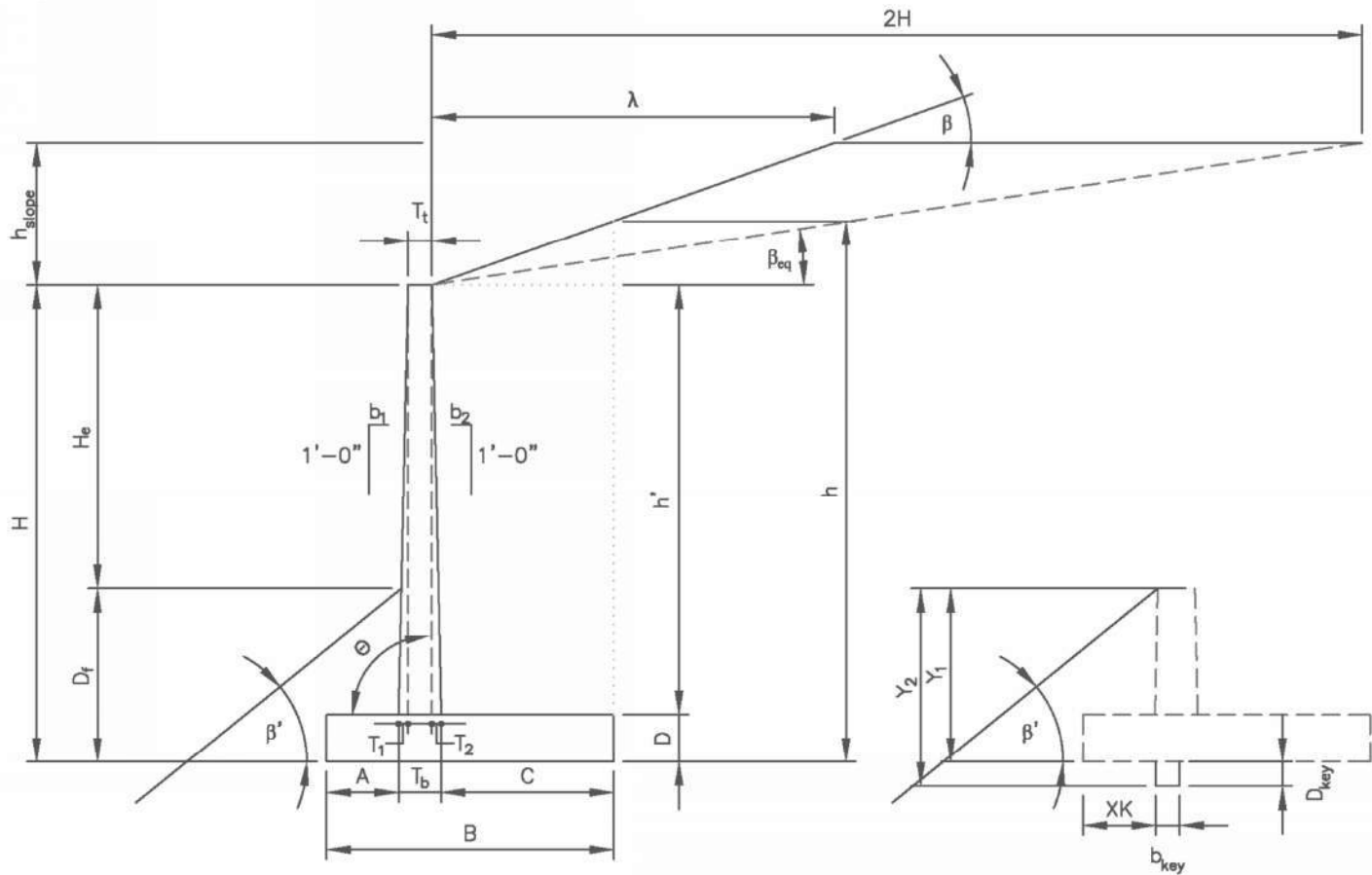
$$\gamma_q := 120 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Other Parameters:

$$\gamma_c := 150 \frac{\text{lb}_f}{\text{ft}^3}$$

Concrete Unit weight



Wall Geometry:

$$H_e := 22.5 \text{ ft}$$

$$D_f := 4 \text{ ft}$$

$$H := H_e + D_f$$

$$H = 26.5 \text{ ft}$$

$$T_t := 18 \text{ in}$$

$$b_1 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

$$b_2 := \frac{8}{10} \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

Preliminary Wall Dimensioning:

$$B := 15 \text{ ft}$$

$$\frac{2}{5} \cdot H = 10.6 \text{ ft} \text{ to } \frac{3}{5} \cdot H = 15.9 \text{ ft}$$

Footing base width (2/5H to 3/5H)

$$A := 4 \text{ ft}$$

$$\frac{H}{8} = 3.31 \text{ ft} \text{ to } \frac{H}{5} = 5.3 \text{ ft}$$

Toe projection (H/8 to H/5)

$$D := 3 \text{ ft}$$

$$\frac{H}{8} = 3.31 \text{ ft} \text{ to } \frac{H}{5} = 5.3 \text{ ft}$$

Footing thickness (H/8 to H/5)

Exposed wall height

Footing cover at Toe

Note: Where the potential for scour, erosion of undermining exists, spread footings shall be located to bear below the maximum depth of scour or undermining. Spread footings shall be located below the depth of potential frost. **LRFD BDS 10.6.1.2**

Design Wall Height

Stem thickness at top of wall

Frontwall batter, (b1H:12V)

Backwall batter, (b2H:12V)

Shear Key Dimensioning:

$$D_{key} := 0 \text{ ft}$$

$$b_{key} := 0 \text{ ft}$$

$$XK := 0 \text{ ft}$$

Depth of shear key from bottom of footing

Note: Footings on rock typically require shear key

Width of shear key

Distance from toe to shear key

Other Wall Dimensions:

$$h' := H - D$$

$$h' = 23.5 \text{ ft}$$

Stem height

$$T_1 := b_1 \cdot h'$$

$$T_1 = 0 \text{ ft}$$

Stem front batter width

$$T_2 := b_2 \cdot 10 \text{ ft}$$

$$T_2 = 8 \text{ in}$$

Stem back batter width

$$T_b := T_1 + T_2 + T_t$$

$$T_b = 2.167 \text{ ft}$$

Stem thickness at bottom of wall

$$C := B - A - T_b$$

$$C = 8.833 \text{ ft}$$

Heel projection

$$\theta := 90 \text{ deg}$$

Angle of back face of wall to horizontal = $\text{atan}(12/b_2)$

$$b := 12 \text{ in}$$

$$b = 1 \text{ ft}$$

Concrete strip width (for design)

$$y_1 := 3.25 \text{ ft}$$

$$y_1 = 3.25 \text{ ft}$$

Depth to where passive pressure may begin to be utilized in front of wall.

$$y_2 := D_f + D_{key}$$

$$y_2 = 4 \text{ ft}$$

Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

Site Grading and Slope Dimensions:

$$\beta := 26.565 \text{ deg}$$

Inclination of ground slope:

- Horizontal: 0
- 3H:1V: 18.435
- 2H:1V: 26.565
- 1.5H:1V: 33.690

Inclination of ground slope behind face of wall.

Horizontal backfill behind CIP wall, $\beta = 0 \text{ deg}$

$$\beta' := 0 \text{ deg}$$

Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, $\beta' = 0 \text{ deg}$. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

$$\lambda := 40 \text{ ft}$$

Horizontal distance from the back of the wall to point of slope crest .

$$L_{Traffic} := 40 \text{ ft}$$

Horizontal distance from assumed traffic surcharge load to Backface of Wall.

$$2 \cdot H = 53 \text{ ft}$$

$$h_{slope} := \lambda \cdot \tan(\beta)$$

$$h_{slope} = 20 \text{ ft}$$

Height of broken slope behind wall

$$\beta_{eq} := \text{atan}\left(\frac{h_{slope}}{2 \cdot H}\right) = 20.674 \text{ deg}$$

Equivalent backslope angle

$$h := \text{if}(\lambda \leq T_2 + C, H + h_{slope}, H + (T_2 + C) \cdot \tan(\beta)) = 31.25 \text{ ft}$$

Height of retained fill at back of heel

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L_{Traffic} < \frac{H}{2}, 250 \frac{lb_f}{ft^2}, 0 \frac{lb_f}{ft^2} \right) = 0 \frac{lb_f}{ft^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

Note: A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2.

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_f + \delta) \cdot \sin(\phi'_f - \beta_{eq}))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta_{eq}))}} \right)^2 \quad \Gamma = 2.031$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_f))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.38$$

Active Earth Pressure Coefficient
(per LRFD Sect. 3.11.5.3)

Foundation Soil Passive Earth:

Drained Conditions assuming ($\phi'_{fd} > 0$):

Input Parameters for **LRFD Figure 3.11.5.4-2**, assumes $\theta = 90$ degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0$$

$$\frac{-\delta_{fd}}{\phi'_{fd}} = -0.667$$

$$k'_p := 14$$

Passive Earth Pressure Coefficient
from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.682$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p$$

$$k_{pd} = 9.548$$

Passive Earth Pressure Coefficient for
Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:

$$F_T := \frac{1}{2} \cdot \gamma_f \cdot h^2 \cdot k_{af}$$

$$F_T = 22273.966 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{af}$$

$$F_{SUR} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$$

$$V_1 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Wall stem front batter (DC)

$$V_2 := T_t \cdot h' \cdot \gamma_c$$

$$V_2 = 5287.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Wall stem (DC)

$$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$$

$$V_3 = 1175 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Wall stem back batter (DC)

$$V_4 := D \cdot B \cdot \gamma_c$$

$$V_4 = 6750 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Wall Footing (DC)

$$V_5 := C \cdot h' \cdot \gamma_f$$

$$V_5 = 24910 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil Backfill - Heel (EV)

$$V_6 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_f$$

$$V_6 = 940 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil Backfill - Batter (EV)

$$V_7 := \text{if} \left(\lambda \leq T_2 + C, \frac{1}{2} \cdot \lambda^2 \cdot \tan(\beta) \cdot \gamma_f, \frac{1}{2} \cdot (T_2 + C)^2 \cdot \tan(\beta) \cdot \gamma_f \right)$$

$$V_7 = 2707.494 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil Backfill - Backslope
Triangle Portion (EV)

$$V_8 := \text{if} \left(\lambda \leq T_2 + C, (T_2 + C - \lambda) \cdot h_{\text{slope}} \cdot \gamma_f, 0 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \right)$$

$$V_8 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil Backfill - Backslope
Rectangular Portion (EV)

$$V_9 := \text{if} \left(\lambda \leq T_2 + C, (T_2 + C - \lambda) \cdot SUR, 0 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \right)$$

$$V_9 = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge Above Heel - (LS)

$$V_{10} := F_{SUR} \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{10} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge Resultant (vertical
comp. - LS)

$$V_{11} := F_T \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{11} = 8103.117 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active earth force resultant (vertical
component - EH)

Moment Arm:

Moments produced from vertical loads about Point 'O'

$$d_{v1} := A + \frac{2}{3} \cdot T_1 = 4 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1} = 0 \text{ lb} \cdot \text{ft}$$

$$d_{v2} := A + T_1 + \frac{T_t}{2} = 4.75 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2} = 25115.625 \text{ lb} \cdot \text{ft}$$

$$d_{v3} := A + T_1 + T_t + \frac{T_2}{3} = 5.722 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3} = 6723.611 \text{ lb} \cdot \text{ft}$$

Moment:

$$d_{v4} := \frac{B}{2} = 7.5 \text{ ft}$$

$$d_{v5} := B - \frac{C}{2} = 10.583 \text{ ft}$$

$$d_{v6} := A + T_l + T_t + \frac{2 T_2}{3} = 5.944 \text{ ft}$$

$$d_{v7} := \text{if} \left(\lambda \leq T_2 + C, A + T_l + T_t + \left(\frac{2}{3} (\lambda) \right), A + T_l + T_t + \left(\frac{2}{3} (T_2 + C) \right) \right) = 11.833 \text{ ft}$$

$$d_{v8} := A + T_l + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2} \right) = 30.25 \text{ ft}$$

$$d_{v9} := A + T_l + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2} \right) = 30.25 \text{ ft}$$

$$d_{v10} := B = 15 \text{ ft}$$

$$d_{v11} := B = 15 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4} = 50625 \text{ lbf}$$

$$MV_5 := V_5 \cdot d_{v5} = 263630.833 \text{ lbf}$$

$$MV_6 := V_6 \cdot d_{v6} = 5587.778 \text{ lbf}$$

$$MV_7 := V_7 \cdot d_{v7} = 32038.678 \text{ lbf}$$

$$MV_8 := V_8 \cdot d_{v8} = 0 \text{ lbf}$$

$$MV_9 := V_9 \cdot d_{v9} = 0 \text{ lbf}$$

$$MV_{10} := V_{10} \cdot d_{v10} = 0 \text{ lbf}$$

$$MV_{11} := V_{11} \cdot d_{v11} = 121546.759 \text{ lbf}$$

Horizontal Loads:

$$H_1 := F_{SUR} \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_1 = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

$$H_2 := F_T \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_2 = 20747.748 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm:

$$d_{h1} := \frac{h}{2} \quad d_{h1} = 15.625 \text{ ft}$$

Moment:

$$MH_1 := H_1 \cdot d_{h1} \quad MH_1 = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{3} \quad d_{h2} = 10.417 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2} \quad MH_2 = 216122.297 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type:

$$V_{DC} := V_1 + V_2 + V_3 + V_4 \quad V_{DC} = 13212.5 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EV} := V_5 + V_6 + V_7 + V_8 \quad V_{EV} = 28557.494 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_1a} := V_{10} \quad V_{LS_1a} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_1b} := V_9 + V_{10} \quad V_{LS_1a} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_{11} \quad V_{EH} = 8103.117 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_2 \quad H_{EH} = 20747.748 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_1 \quad H_{LS} = 0 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4$$

$$M_{DC} = 82464.236 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EV} := MV_5 + MV_6 + MV_7 + MV_8$$

$$M_{EV} = 301257.29 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ia} := MV_{10}$$

$$M_{LSV_Ia} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ib} := MV_9 + MV_{10}$$

$$M_{LSV_Ib} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_{11}$$

$$M_{EH1} = 121546.759 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSH} := MH_1$$

$$M_{LSH} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_2$$

$$M_{EH2} = 216122.297 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$\eta := 1$ LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{DC} := 0.9$$

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{DC} := 1.25$$

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot ((Ia_{DC} \cdot V_{DC}) + (Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + (Ia_{LS} \cdot V_{LS_Ia}))$$

$$V_{Ia} = 52603.42 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{DC} \cdot V_{DC}) + (Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + (Ib_{LS} \cdot V_{LS_Ib}))$$

$$V_{Ib} = 67222.918 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 31121.621 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 31121.621 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot ((Ia_{DC} \cdot M_{DC}) + (Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + (Ia_{LS} \cdot M_{LSV_Ia}))$$

$$MV_{Ia} = 557795.241 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{DC} \cdot M_{DC}) + (Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + (Ib_{LS} \cdot M_{LSV_Ib}))$$

$$MV_{Ib} = 692097.775 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LSH}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 324183.446 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LSH}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 324183.446 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength lb:

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 692097.775 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 324183.446 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 67222.918 \frac{\text{lb}}{\text{ft}}$	Sum of Vertical Loads (Strength lb)
$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$	$x = 5.473 \text{ ft}$	Distance from Point "O" the resultant intersects the base
$e := \left \frac{B}{2} - x \right $	$e = 2.03 \text{ ft}$	Wall eccentricity, Note: The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2] . The effective bearing width is equal to B-2e.

Foundation Layout:

$B' := B - 2 \cdot e$	$B' = 10.946 \text{ ft}$	Effective Footing Width
$L' := 398.88 \text{ ft}$		Effective Footing Length (Assumed)
$H' := H_{lb}$	$H' = 31121.621 \frac{\text{lb}}{\text{ft}}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 67222.918 \frac{\text{lb}}{\text{ft}}$	Summation of Vertical Loads (Strength lb)
$D_f = 4 \text{ ft}$		Footing embedment
$d_w := 0 \text{ ft}$		Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right)$	$N_q = 40.24$
$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right)$	$N_c = 53.03$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$	$N_\gamma = 61.031$

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

$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right)$	$s_c = 1.021$
$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right)$	$s_q = 1.02$
$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right)$	$s_\gamma = 0.989$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \tan\left(\frac{D_f}{B'}\right)$$

$$d_q = 1.09$$

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

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

$$N_{cm} = 54.133$$

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

$$N_{qm} = 44.551$$

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

$$N_{\gamma m} = 60.361$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 388499.788 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor **LRFD Table 10.5.5.2.2-1.**

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 174.825 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 174.825 \text{ ksf}$$

Evaluate External Stability of Wall:

Compute the factored bearing stress :

$$e = 2.03 \text{ ft}$$

$$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e} \quad \sigma_V = 6.141 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions:

$$CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 28.47$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{\max} := \frac{9 \cdot B}{20}$$

$$e_{\max} = 6.75 \text{ ft}$$

Maximum Eccentricity **LRFD [C11.6.3.3.]**
Within 9B/10 for rock.

$$\Sigma M_R := MV_{Ia}$$

$$\Sigma M_R = 557795.241 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia}$$

$$\Sigma M_O = 324183.446 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 52603.42 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 4.441 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \left| \frac{B}{2} - x \right|$$

$$e = 3.06 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e. .

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{\text{Eccentricity}} := \frac{e_{\max}}{e}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Eccentricity}} = 2.21$$

Objective: To evaluate the external stability of shallow foundation on bedrock with RMR system design.
Method: In accordance with ODOT Bridge Design Manual [Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Estimate RMR:

PARAMETER		RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 MPa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred	
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa
	Relative Rating		15	12	7	4	2	1
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%	
	Relative Rating		20	17	13	8	3	
3	Spacing of joints		>3 m	1 to 3 m	0.3 to 1 m	50 to 300 mm	<50mm	
	Relative Rating		30	25	20	10	5	
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints	
	Relative Rating		25	20	12	6	0	
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min	
		Ratio= joint water pressure/major principal stress	OR 0		OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5	
		General Conditions	OR Completely Dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems	
	Relative Rating		10		7	4	0	

Given:

Rock Strength Parameters (Average Below Footing):

$$RMR := 2 + 13 + 20 + 12 + 7 = 54$$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above)**.
Do NOT apply adjustment for joint orientation

$$m_i := 15$$

Rock strength constant using **GDM Section 1303.3.3**.
Sandstone, breccia, and conglomerate: $m_i=15$; Claystone (mudstone), shale, clay-shale, and siltstone: $m_i=10$; Dolomite and limestone: $m_i=7$; Coal: $m_i=1$

$$q_u := 2250 \text{ psi}$$

Unconfined compressive strength of rock core, based on GDM Table 400-6

$$\gamma_r := 140 \frac{\text{lb}}{\text{ft}^3}$$

Unit weight of bedrock

Empiracle parameters (undisturbed rock), s and m (Hoek and Brown, 1988):

$$s := e^{\left(\frac{RMR - 100}{28}\right)} \quad s = 0.1934266046 \quad m := m_i \cdot e^{\left(\frac{RMR - 100}{9}\right)} \quad m = 0.0904406983$$

Empiracle rock strength parameters, ϕ and c (Bieniawski, 1989):

$$\phi'_{fd} := \left(5 + \frac{RMR}{2}\right) \text{ deg} \quad c'_{fd} := (0.104 \cdot RMR) \text{ ksf}$$

$$\phi'_{fd} = 32 \text{ deg}$$

$$c'_{fd} = 5616 \text{ psf}$$

Backfill Soil Design Parameters:

$$\phi'_f := 32 \text{ deg}$$

Effective angle of internal friction

$$\gamma_f := 120 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight

$$c'_f := 0 \frac{\text{lb}_f}{\text{ft}^2}$$

Effective Cohesion

$$\delta := \frac{2}{3} \cdot \phi'_f \quad \delta = 21.3 \text{ deg}$$

Friction angle between backfill and wall taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Foundation Soil/Rock Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} = 32 \text{ deg}$$

Effective angle of internal friction, from above RMR estimation

$$\gamma_{fd} := \gamma_r = 140 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight

$$c'_{fd} = 5616 \text{ psf}$$

Effective Cohesion, from above RMR estimation

$$\delta_{fd} := \frac{2}{3} \cdot \phi'_{fd} \quad \delta_{fd} = 21.3 \text{ deg}$$

Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Foundation Surcharge Soil Parameters:

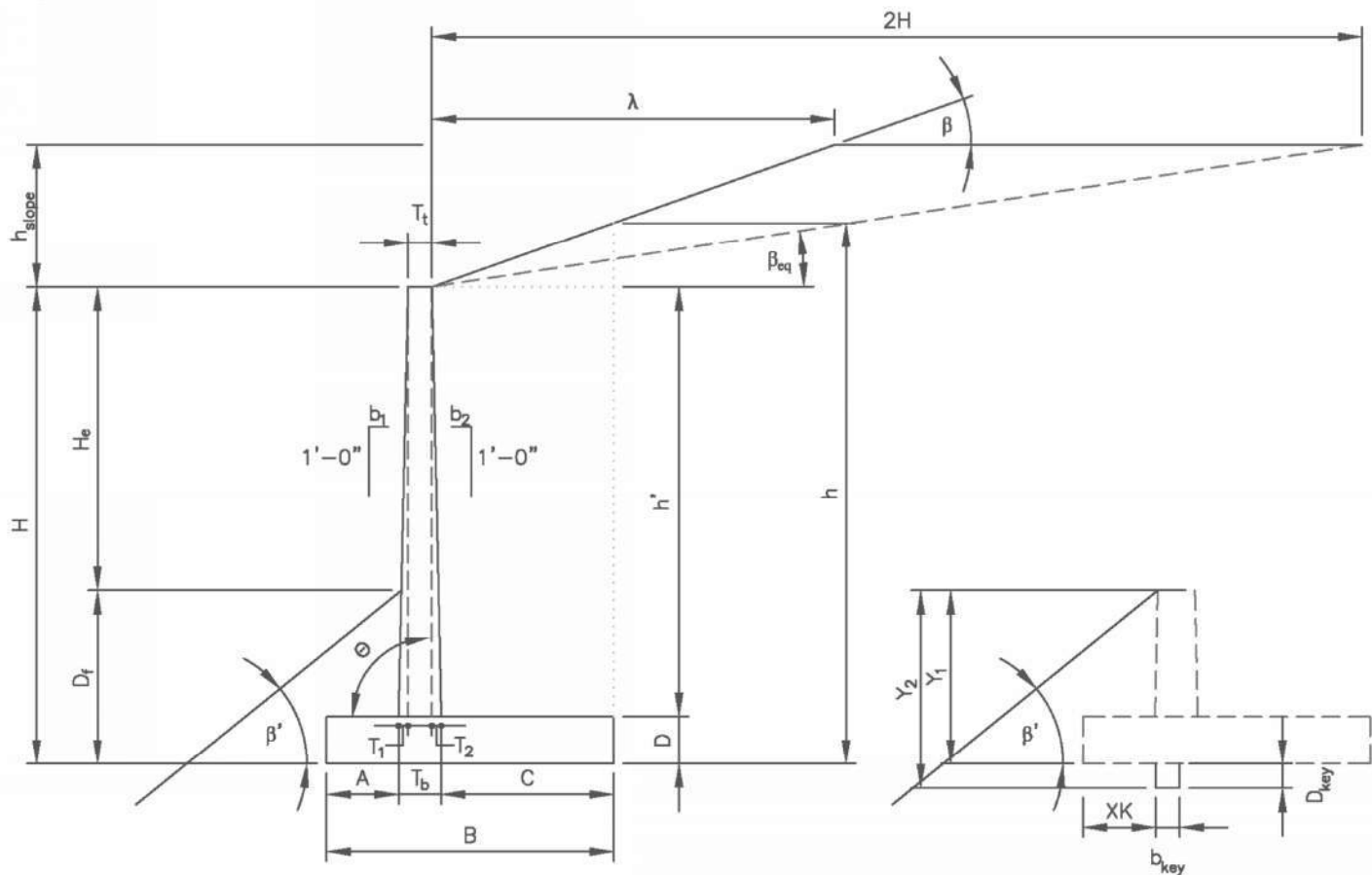
$$\gamma_q := 120 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Other Parameters:

$$\gamma_c := 150 \frac{\text{lb}_f}{\text{ft}^3}$$

Concrete Unit weight



Wall Geometry:

$$H_e := 6.9 \text{ ft}$$

$$D_f := 4.3 \text{ ft}$$

$$H := H_e + D_f$$

$$H = 11.2 \text{ ft}$$

$$T_t := 18 \text{ in}$$

$$b_1 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

$$b_2 := \frac{8}{7} \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

Preliminary Wall Dimensioning:

$$B := 9 \text{ ft} \quad \frac{2}{5} \cdot H = 4.48 \text{ ft} \quad \text{to} \quad \frac{3}{5} \cdot H = 6.72 \text{ ft}$$

$$A := 2.5 \text{ ft} \quad \frac{H}{8} = 1.4 \text{ ft} \quad \text{to} \quad \frac{H}{5} = 2.24 \text{ ft}$$

$$D := 2.5 \text{ ft} \quad \frac{H}{8} = 1.4 \text{ ft} \quad \text{to} \quad \frac{H}{5} = 2.24 \text{ ft}$$

Exposed wall height

Footing cover at Toe

Note: Where the potential for scour, erosion of undermining exists, spread footings shall be located to bear below the maximum depth of scour or undermining. Spread footings shall be located below the depth of potential frost. **LRFD BDS 10.6.1.2**

Design Wall Height

Stem thickness at top of wall

Frontwall batter, (b1H:12V)

Backwall batter, (b2H:12V)

Footing base width (2/5H to 3/5H)

Toe projection (H/8 to H/5)

Footing thickness (H/8 to H/5)

Shear Key Dimensioning:

$$D_{key} := 0 \text{ ft}$$

$$b_{key} := 0 \text{ ft}$$

$$XK := 0 \text{ ft}$$

Depth of shear key from bottom of footing

Note: Footings on rock typically require shear key

Width of shear key

Distance from toe to shear key

Other Wall Dimensions:

$$h' := H - D$$

$$h' = 8.7 \text{ ft}$$

Stem height

$$T_1 := b_1 \cdot h'$$

$$T_1 = 0 \text{ ft}$$

Stem front batter width

$$T_2 := b_2 \cdot 10 \text{ ft}$$

$$T_2 = 11.429 \text{ in}$$

Stem back batter width

$$T_b := T_1 + T_2 + T_t$$

$$T_b = 2.452 \text{ ft}$$

Stem thickness at bottom of wall

$$C := B - A - T_b$$

$$C = 4.048 \text{ ft}$$

Heel projection

$$\theta := 90 \text{ deg}$$

Angle of back face of wall to horizontal = $\text{atan}(12/b_2)$

$$b := 12 \text{ in}$$

$$b = 1 \text{ ft}$$

Concrete strip width (for design)

$$y_1 := 3.25 \text{ ft}$$

$$y_1 = 3.25 \text{ ft}$$

Depth to where passive pressure may begin to be utilized in front of wall.

$$y_2 := D_f + D_{key}$$

$$y_2 = 4.3 \text{ ft}$$

Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

Site Grading and Slope Dimensions:

$$\beta := 26.565 \text{ deg}$$

Inclination of ground slope:

- Horizontal: 0
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

Inclination of ground slope behind face of wall.
Horizontal backfill behind CIP wall, $\beta = 0 \text{ deg}$

$$\beta' := 0 \text{ deg}$$

Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, $\beta' = 0 \text{ deg}$. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

$$\lambda := 44 \text{ ft}$$

Horizontal distance from the back of the wall to point of slope crest .

$$L_{Traffic} := 44 \text{ ft}$$

Horizontal distance from assumed traffic surcharge load to Backface of Wall.

$$2 \cdot H = 22.4 \text{ ft}$$

$$h_{slope} := \lambda \cdot \tan(\beta)$$

$$h_{slope} = 22 \text{ ft}$$

Height of broken slope behind wall

$$\beta_{eq} := \text{atan}\left(\frac{h_{slope}}{2 \cdot H}\right) = 44.484 \text{ deg}$$

Equivalent backslope angle

$$h := \text{if}(\lambda \leq T_2 + C, H + h_{slope}, H + (T_2 + C) \cdot \tan(\beta)) = 13.7 \text{ ft}$$

Height of retained fill at back of heel

Live Load Surcharge Parameters:

$$SUR := \text{if} \left(L_{Traffic} < \frac{H}{2}, 250 \frac{\text{lbf}}{\text{ft}^2}, 0 \frac{\text{lbf}}{\text{ft}^2} \right) = 0 \frac{\text{lbf}}{\text{ft}^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])

Note: A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2.

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_f + \delta) \cdot \sin(\phi'_f - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2 \quad \Gamma = 1.695$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_f))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.455$$

Active Earth Pressure Coefficient
(per LRFD Sect. 3.11.5.3)

Foundation Soil Passive Earth:

Drained Conditions assuming ($\phi'_{fd} > 0$):

Input Parameters for **LRFD Figure 3.11.5.4-2**, assumes $\theta = 90$ degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0$$

$$\frac{-\delta_{fd}}{\phi'_{fd}} = -0.667$$

$$k'_p := 8$$

Passive Earth Pressure Coefficient
from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.752$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p$$

$$k_{pd} = 6.016$$

Passive Earth Pressure Coefficient for
Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:

$$F_T := \frac{1}{2} \cdot \gamma_f \cdot h^2 \cdot k_{af}$$

$$F_T = 5129.239 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot h \cdot k_{af}$$

$$F_{SUR} = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$$

$$V_1 = 0 \frac{\text{lbf}}{\text{ft}}$$

Wall stem front batter (DC)

$$V_2 := T_t \cdot h' \cdot \gamma_c$$

$$V_2 = 1957.5 \frac{\text{lbf}}{\text{ft}}$$

Wall stem (DC)

$$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$$

$$V_3 = 621.429 \frac{\text{lbf}}{\text{ft}}$$

Wall stem back batter (DC)

$$V_4 := D \cdot B \cdot \gamma_c$$

$$V_4 = 3375 \frac{\text{lbf}}{\text{ft}}$$

Wall Footing (DC)

$$V_5 := C \cdot h' \cdot \gamma_f$$

$$V_5 = 4225.714 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Heel (EV)

$$V_6 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_f$$

$$V_6 = 497.143 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Batter (EV)

$$V_7 := \text{if} \left(\lambda \leq T_2 + C, \frac{1}{2} \cdot \lambda^2 \cdot \tan(\beta) \cdot \gamma_f, \frac{1}{2} \cdot (T_2 + C)^2 \cdot \tan(\beta) \cdot \gamma_f \right)$$

$$V_7 = 749.998 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Backslope
Triangle Portion (EV)

$$V_8 := \text{if} \left(\lambda \leq T_2 + C, (T_2 + C - \lambda) \cdot h_{\text{slope}} \cdot \gamma_f, 0 \cdot \frac{\text{lbf}}{\text{ft}} \right)$$

$$V_8 = 0 \frac{\text{lbf}}{\text{ft}}$$

Soil Backfill - Backslope
Rectangular Portion (EV)

$$V_9 := \text{if} \left(\lambda \leq T_2 + C, (T_2 + C - \lambda) \cdot SUR, 0 \cdot \frac{\text{lbf}}{\text{ft}} \right)$$

$$V_9 = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Above Heel - (LS)

$$V_{10} := F_{SUR} \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{10} = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (vertical
comp. - LS)

$$V_{11} := F_T \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{11} = 1865.982 \frac{\text{lbf}}{\text{ft}}$$

Active earth force resultant (vertical
component - EH)

Moment Arm:

Moments produced from vertical loads about Point 'O'

$$d_{v1} := A + \frac{2}{3} \cdot T_1 = 2.5 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1} = 0 \text{ lbf}$$

$$d_{v2} := A + T_1 + \frac{T_t}{2} = 3.25 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2} = 6361.875 \text{ lbf}$$

$$d_{v3} := A + T_1 + T_t + \frac{T_2}{3} = 4.317 \text{ ft}$$

$$MV_3 := V_3 \cdot d_{v3} = 2682.993 \text{ lbf}$$

$$d_{v4} := \frac{B}{2} = 4.5 \text{ ft}$$

$$MV_4 := V_4 \cdot d_{v4} = 15187.5 \text{ lbf}$$

Moment:

$$d_{v5} := B - \frac{C}{2} = 6.976 \text{ ft}$$

$$MV_5 := V_5 \cdot d_{v5} = 29479.388 \text{ lbf}$$

$$d_{v6} := A + T_l + T_t + \frac{2 T_2}{3} = 4.635 \text{ ft}$$

$$MV_6 := V_6 \cdot d_{v6} = 2304.218 \text{ lbf}$$

$$d_{v7} := \text{if} \left(\lambda \leq T_2 + C, A + T_l + T_t + \left(\frac{2}{3} (\lambda) \right), A + T_l + T_t + \left(\frac{2}{3} (T_2 + C) \right) \right) = 7.333 \text{ ft}$$

$$MV_7 := V_7 \cdot d_{v7} = 5499.988 \text{ lbf}$$

$$d_{v8} := A + T_l + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2} \right) = 28.5 \text{ ft}$$

$$MV_8 := V_8 \cdot d_{v8} = 0 \text{ lbf}$$

$$d_{v9} := A + T_l + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2} \right) = 28.5 \text{ ft}$$

$$MV_9 := V_9 \cdot d_{v9} = 0 \text{ lbf}$$

$$d_{v10} := B = 9 \text{ ft}$$

$$MV_{10} := V_{10} \cdot d_{v10} = 0 \text{ lbf}$$

$$d_{v11} := B = 9 \text{ ft}$$

$$MV_{11} := V_{11} \cdot d_{v11} = 16793.84 \text{ lbf}$$

Horizontal Loads:

$$H_l := F_{SUR} \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_l = 0 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

$$H_2 := F_T \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_2 = 4777.782 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm:

$$d_{h1} := \frac{h}{2} \quad d_{h1} = 6.85 \text{ ft}$$

Moment:

$$MH_l := H_l \cdot d_{h1} \quad MH_l = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{h}{3} \quad d_{h2} = 4.567 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2} \quad MH_2 = 21818.529 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type:

$$V_{DC} := V_1 + V_2 + V_3 + V_4 \quad V_{DC} = 5953.929 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EV} := V_5 + V_6 + V_7 + V_8 \quad V_{EV} = 5472.855 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_Ia} := V_{10} \quad V_{LS_Ia} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_Ib} := V_9 + V_{10} \quad V_{LS_Ia} = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_{11} \quad V_{EH} = 1865.982 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_2 \quad H_{EH} = 4777.782 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_l \quad H_{LS} = 0 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4$$

$$M_{DC} = 24232.368 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EV} := MV_5 + MV_6 + MV_7 + MV_8$$

$$M_{EV} = 37283.593 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ia} := MV_{10}$$

$$M_{LSV_Ia} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ib} := MV_9 + MV_{10}$$

$$M_{LSV_Ib} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_{11}$$

$$M_{EH1} = 16793.84 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSH} := MH_1$$

$$M_{LSH} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_2$$

$$M_{EH2} = 21818.529 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Load Combination Limit States:

$\eta := 1$ LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia:
(Sliding and Eccentricity)

$$Ia_{DC} := 0.9$$

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

Strength Limit State Ib:
(Bearing Capacity)

$$Ib_{DC} := 1.25$$

$$Ib_{EV} := 1.35$$

$$Ib_{EH} := 1.5$$

$$Ib_{LS} := 1.75$$

Factored Vertical Loads by Limit State:

$$V_{Ia} := \eta \cdot ((Ia_{DC} \cdot V_{DC}) + (Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + (Ia_{LS} \cdot V_{LS_Ia}))$$

$$V_{Ia} = 13630.365 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{DC} \cdot V_{DC}) + (Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + (Ib_{LS} \cdot V_{LS_Ib}))$$

$$V_{Ib} = 17629.739 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 7166.673 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 7166.673 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot ((Ia_{DC} \cdot M_{DC}) + (Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + (Ia_{LS} \cdot M_{LSV_Ia}))$$

$$MV_{Ia} = 84283.485 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{DC} \cdot M_{DC}) + (Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + (Ib_{LS} \cdot M_{LSV_Ib}))$$

$$MV_{Ib} = 105814.071 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LSH}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 32727.794 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LSH}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 32727.794 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength lb:

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 105814.071 \frac{lb \cdot ft}{ft}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 32727.794 \frac{lb \cdot ft}{ft}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 17629.739 \frac{lb \cdot ft}{ft}$	Sum of Vertical Loads (Strength lb)
$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$	$x = 4.146 \text{ ft}$	Distance from Point "O" the resultant intersects the base
$e := \left \frac{B}{2} - x \right $	$e = 0.35 \text{ ft}$	Wall eccentricity, Note: The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation LRFD [11.6.3.2] . The effective bearing width is equal to B-2e.

Foundation Layout:

$B' := B - 2 \cdot e$	$B' = 8.291 \text{ ft}$	Effective Footing Width
$L' := 398.88 \text{ ft}$		Effective Footing Length (Assumed)
$H' := H_{lb}$	$H' = 7166.673 \frac{lb \cdot ft}{ft}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 17629.739 \frac{lb \cdot ft}{ft}$	Summation of Vertical Loads (Strength lb)
$D_f = 4.3 \text{ ft}$		Footing embedment
$d_w := 0 \text{ ft}$		Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

$N_q := \text{if} \left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan \left(45 \text{ deg} + \frac{\phi'_{fd}}{2} \right), 1.0 \right)$	$N_q = 23.18$
$N_c := \text{if} \left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14 \right)$	$N_c = 35.49$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$	$N_\gamma = 30.215$

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

$s_c := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L'} \right) \right)$	$s_c = 1.014$
$s_q := \text{if} \left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd}) \right), 1 \right)$	$s_q = 1.013$
$s_\gamma := \text{if} \left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'} \right), 1 \right)$	$s_\gamma = 0.992$

Load inclination factors:

$$i_q := 1$$

$$i_\gamma := 1$$

$$i_c := 1$$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**.
"Most geotechnical engineers do not use the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5) \quad C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \quad C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)$$

$$d_q = 1.13$$

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 35.972$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{qm} = 26.58$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 29.963$$

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

$$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nd} = 217571.66 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance, LRFD [Eq 10.6.3.1.1]:

$$\phi_b := .45$$

Bearing resistance factor LRFD Table 10.5.5.2.2-1.

$$q_{Rd} := \phi_b \cdot q_{nd} \quad q_{Rd} = 97.907 \text{ ksf} \quad \text{Factored bearing resistance Drained Conditions}$$

Evaluate External Stability of Wall:

Compute the factored bearing stress :

$$e = 0.35 \text{ ft}$$

$$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e} \quad \sigma_V = 2.126 \text{ ksf}$$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions:

$$CDR_{\text{Bearing}_D} := \frac{q_{Rd}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Bearing}_D} = 46.05$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{9 B}{20}$$

$$e_{max} = 4.05 \text{ ft}$$

Maximum Eccentricity **LRFD [C11.6.3.3.]**
Within 9B/10 for rock.

$$\Sigma M_R := MV_{Ia}$$

$$\Sigma M_R = 84283.485 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia}$$

$$\Sigma M_O = 32727.794 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 13630.365 \frac{\text{lbf}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 3.782 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \left| \frac{B}{2} - x \right|$$

$$e = 0.72 \text{ ft}$$

Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e. .

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e}$$

Is the CDR > or = to 1.0?

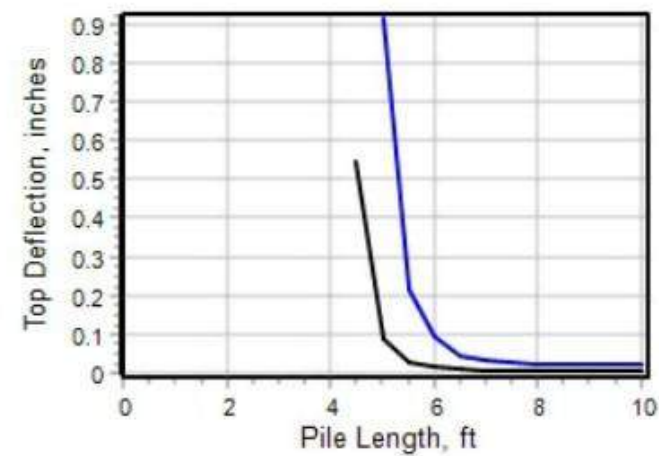
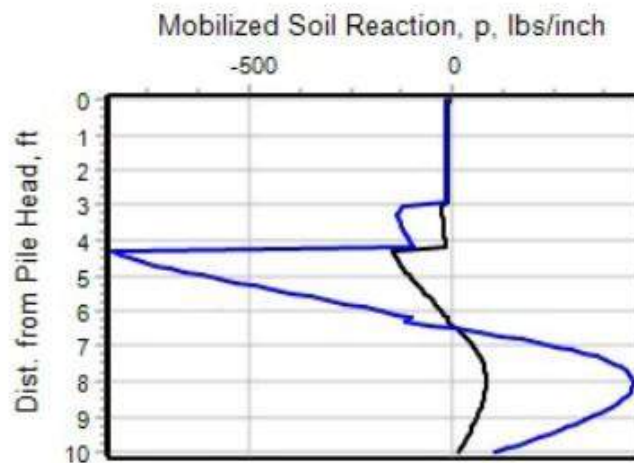
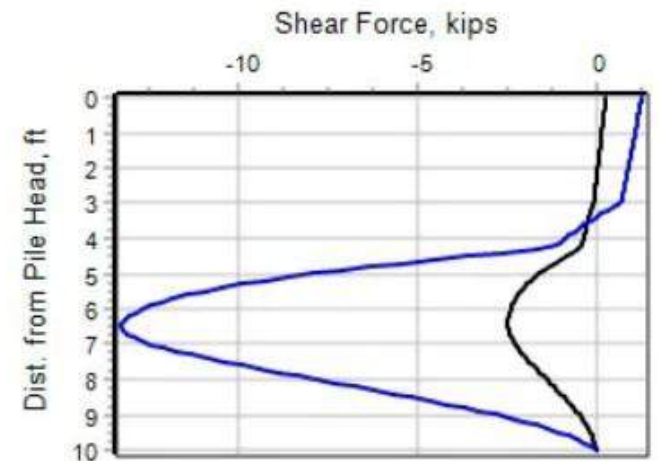
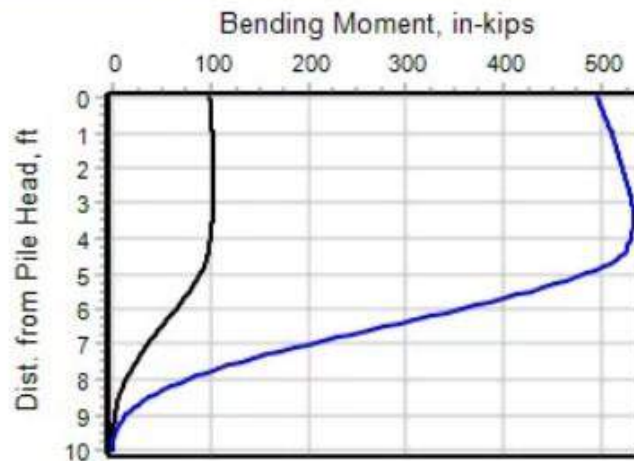
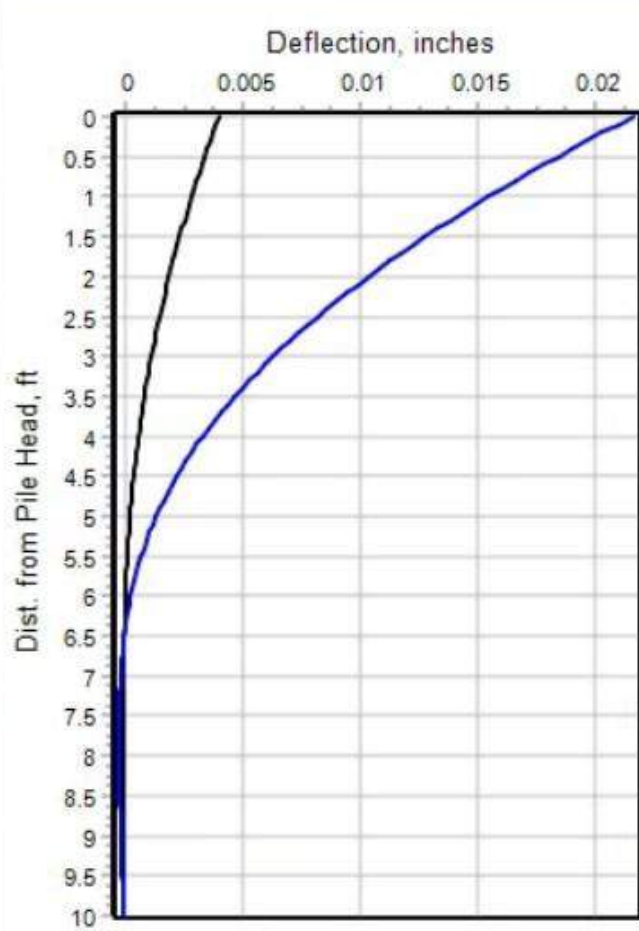
$$CDR_{Eccentricity} = 5.64$$

APPENDIX F

CCTV POLE ANALYSES



Results for 70ft CCTV Camera_STA. 357+05_L= 10 ft_B-058.lp12d



☐ Show All Legends

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Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Columbus, OH, USA

=====

This model was prepared by:
Zmankoci

Files Used for Analysis

Path to file locations:
\22-0033 (113208-114865 SUM-77 SR8-11.650.00 & SUM-77SR8-9.650.00)\113208-114865\geotechnical\light
towers\Lpile\Files\

Name of input data file:
70ft CCTV Camera_STA. 357+05_L=10 ft_B-058.lp12d

Name of output report file:
70ft CCTV Camera_STA. 357+05_L=10 ft_B-058.lp12o

Name of plot output file:

70ft CCTV Camera_STA. 357+05_L=10 ft_B-058.lp12p

Name of runtime message file:

70ft CCTV Camera_STA. 357+05_L=10 ft_B-058.lp12r

Date and Time of Analysis

Date: August 10, 2026

Time: 22:38:18

Problem Title

Project Name: SUM-77/SR8

Job Number: 114865

Client: Arcadis/ODOT

Engineer: ZM

Description: 70 ft Foundation Design for CCTV Camera

Program Options and Settings

Computational Options:

- Conventional Analysis

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed = 500
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 100

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 10.000 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	23.1000
2	10.000	24.9000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile

Cross-sectional Shape	=	Circular Pipe
Length of section	=	10.000000 ft
Width of top of section	=	23.100000 in
Width of bottom of section	=	24.900000 in
Wall Thickness at Top	=	3.550000 in
Wall Thickness at Bottom	=	3.550000 in
Top Area	=	218.034384 sq. in
Bottom Area	=	238.109161 sq. in
Moment of Inertia at Top	=	10760. in^4

Moment of Inertia at Bottom	=	13942. in^4
Elastic Modulus	=	6244000. psi

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer	=	0.0000 ft
Distance from top of pile to bottom of layer	=	3.000000 ft
Effective unit weight at top of layer	=	98.000000 pcf
Effective unit weight at bottom of layer	=	98.000000 pcf
Undrained cohesion at top of layer	=	250.000000 psf
Undrained cohesion at bottom of layer	=	250.000000 psf
Epsilon-50 at top of layer	=	0.026800
Epsilon-50 at bottom of layer	=	0.026800

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	3.000000 ft
Distance from top of pile to bottom of layer	=	4.200000 ft
Effective unit weight at top of layer	=	122.000000 pcf
Effective unit weight at bottom of layer	=	122.000000 pcf
Friction angle at top of layer	=	40.000000 deg.
Friction angle at bottom of layer	=	40.000000 deg.
Subgrade k at top of layer	=	656.000000 pci
Subgrade k at bottom of layer	=	656.000000 pci

Layer 3 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	4.200000 ft
---	---	-------------

Distance from top of pile to bottom of layer	=	6.300000 ft
Effective unit weight at top of layer	=	155.000000 pcf
Effective unit weight at bottom of layer	=	155.000000 pcf
Uniaxial compressive strength at top of layer	=	1125. psi
Uniaxial compressive strength at bottom of layer	=	1125. psi
Initial modulus of rock at top of layer	=	3000. psi
Initial modulus of rock at bottom of layer	=	3000. psi
RQD of rock at top of layer	=	0.0000 %
RQD of rock at bottom of layer	=	0.0000 %
k _{rm} of rock at top of layer	=	0.0001600
k _{rm} of rock at bottom of layer	=	0.0001600

Layer 4 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer	=	6.300000 ft
Distance from top of pile to bottom of layer	=	16.300000 ft
Effective unit weight at top of layer	=	150.000000 pcf
Effective unit weight at bottom of layer	=	150.000000 pcf
Uniaxial compressive strength at top of layer	=	1067. psi
Uniaxial compressive strength at bottom of layer	=	1067. psi
Initial modulus of rock at top of layer	=	5760. psi
Initial modulus of rock at bottom of layer	=	5760. psi
RQD of rock at top of layer	=	26.000000 %
RQD of rock at bottom of layer	=	26.000000 %
k _{rm} of rock at top of layer	=	0.0001700
k _{rm} of rock at bottom of layer	=	0.0001700

(Depth of the lowest soil layer extends 6.300 ft below the pile tip)

**** Warning - Possible Input Data Error ****

Values entered for effective unit weight of rock were outside the limits of 50 pcf to 150 pcf.

The maximum input value, in layer 4, for effective unit weight = 155.00 pcf

This data may be erroneous. Please check your data.

Summary of Input Soil Properties

Layer E50 Num. or krm	Soil Type Name kpy (p-y Curve Type) pci	Layer Rock Mass Depth Modulus ft psi	Effective Unit Wt. pcf	Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	RQD %
1 0.02680	Soft --	0.00 --	98.0000	250.0000	--	--	--
0.02680	Clay --	3.0000 --	98.0000	250.0000	--	--	--
2 --	Sand 656.0000 (Reese, et al.)	3.0000 --	122.0000	--	40.0000	--	--
--	656.0000	4.2000 --	122.0000	--	40.0000	--	--
3 1.60E-04	Weak --	4.2000 3000.	155.0000	--	--	1125.	0.00
1.60E-04	Rock --	6.3000 3000.	155.0000	--	--	1125.	0.00
4 1.70E-04	Weak --	6.3000 5760.	150.0000	--	--	1067.	26.0000
1.70E-04	Rock --	16.3000 5760.	150.0000	--	--	1067.	26.0000

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Analysis No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run
1	1	V = 253.000000 lbs	M = 99208. in-lbs	11061.	Yes	
2	1	V = 1264. lbs	M = 496038. in-lbs	11061.	Yes	

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	5807.
2	3.0000	1.8809	No	No	5807.	11633.
3	4.2000	4.2000	No	Yes	N.A.	N.A.
4	6.3000	6.3000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head	=	253.0 lbs
Applied moment at pile head	=	99208.0 in-lbs
Axial thrust load on pile head	=	11061.0 lbs

Depth	Deflect.	Bending	Shear	Slope	Total	Bending	Soil Res.	Soil Spr.	Distrib.
X	y	Moment	Force	S	Stress	Stiffness	p	Es*H	Lat.
Load									
feet	inches	in-lbs	lbs	radians	psi*	lb-in^2	lb/inch	lb/inch	lb/inch
-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
0.00	0.00402	99208.	253.0000	-1.06E-04	157.2211	6.72E+10	-8.270	1234.	
0.00									
0.10000	0.00389	99507.	243.0198	-1.05E-04	157.2902	6.72E+10	-8.364	2578.	
0.00									
0.2000	0.00377	99794.	232.9291	-1.03E-04	157.3458	6.76E+10	-8.454	2691.	
0.00									
0.3000	0.00365	100069.	222.7331	-1.01E-04	157.3879	6.77E+10	-8.540	2810.	
0.00									
0.4000	0.00353	100331.	212.4368	-9.94E-05	157.4165	6.79E+10	-8.621	2934.	
0.00									
0.5000	0.00341	100581.	202.0455	-9.76E-05	157.4314	6.81E+10	-8.698	3062.	
0.00									
0.6000	0.00329	100819.	191.5644	-9.59E-05	157.4326	6.83E+10	-8.770	3197.	
0.00									
0.7000	0.00318	101044.	180.9990	-9.41E-05	157.4201	6.85E+10	-8.839	3337.	
0.00									
0.8000	0.00307	101256.	170.3544	-9.23E-05	157.3939	6.86E+10	-8.902	3484.	
0.00									
0.9000	0.00296	101455.	159.6361	-9.05E-05	157.3540	6.88E+10	-8.961	3637.	
0.00									
1.0000	0.00285	101641.	148.8496	-8.88E-05	157.3003	6.90E+10	-9.016	3797.	
0.00									
1.1000	0.00274	101814.	138.0004	-8.70E-05	157.2328	6.92E+10	-9.066	3965.	
0.00									
1.2000	0.00264	101975.	127.0939	-8.52E-05	157.1516	6.94E+10	-9.111	4141.	

0.00									
1.3000	0.00254	102122.	116.1359	-8.35E-05	157.0567	6.96E+10	-9.152	4325.	
0.00									
1.4000	0.00244	102256.	105.1319	-8.17E-05	156.9480	6.98E+10	-9.188	4519.	
0.00									
1.5000	0.00234	102376.	94.0877	-8.00E-05	156.8256	6.99E+10	-9.219	4722.	
0.00									
1.6000	0.00225	102484.	83.0090	-7.82E-05	156.6896	7.01E+10	-9.245	4935.	
0.00									
1.7000	0.00216	102578.	71.9017	-7.65E-05	156.5399	7.03E+10	-9.267	5160.	
0.00									
1.8000	0.00206	102658.	60.7716	-7.47E-05	156.3766	7.05E+10	-9.283	5396.	
0.00									
1.9000	0.00198	102725.	49.6248	-7.30E-05	156.1998	7.07E+10	-9.295	5645.	
0.00									
2.0000	0.00189	102779.	38.4671	-7.12E-05	156.0095	7.09E+10	-9.301	5907.	
0.00									
2.1000	0.00181	102820.	27.3048	-6.95E-05	155.8058	7.11E+10	-9.303	6185.	
0.00									
2.2000	0.00172	102847.	16.1438	-6.77E-05	155.5888	7.13E+10	-9.299	6478.	
0.00									
2.3000	0.00164	102860.	4.9905	-6.60E-05	155.3586	7.15E+10	-9.290	6787.	
0.00									
2.4000	0.00156	102860.	-6.149	-6.43E-05	155.1152	7.16E+10	-9.276	7116.	
0.00									
2.5000	0.00149	102847.	-17.268	-6.26E-05	154.8587	7.18E+10	-9.256	7464.	
0.00									
2.6000	0.00141	102821.	-28.360	-6.09E-05	154.5892	7.20E+10	-9.231	7834.	
0.00									
2.7000	0.00134	102781.	-39.419	-5.91E-05	154.3070	7.22E+10	-9.201	8227.	
0.00									
2.8000	0.00127	102728.	-50.439	-5.74E-05	154.0119	7.24E+10	-9.165	8645.	
0.00									
2.9000	0.00120	102661.	-61.411	-5.57E-05	153.7043	7.26E+10	-9.123	9091.	
0.00									
3.0000	0.00114	102582.	-83.015	-5.40E-05	153.3842	7.28E+10	-26.883	28339.	
0.00									
3.1000	0.00107	102463.	-114.878	-5.24E-05	153.0259	7.30E+10	-26.222	29284.	

0.00									
3.2000	0.00101	102307.	-145.917	-5.07E-05	152.6304	7.32E+10	-25.510	30228.	
0.00									
3.3000	9.53E-04	102114.	-176.076	-4.90E-05	152.1991	7.34E+10	-24.754	31173.	
0.00									
3.4000	8.95E-04	101886.	-205.302	-4.73E-05	151.7332	7.36E+10	-23.957	32118.	
0.00									
3.5000	8.39E-04	101623.	-233.551	-4.57E-05	151.2341	7.37E+10	-23.124	33062.	
0.00									
3.6000	7.85E-04	101327.	-260.781	-4.40E-05	150.7032	7.39E+10	-22.259	34007.	
0.00									
3.7000	7.34E-04	100998.	-286.957	-4.24E-05	150.1419	7.41E+10	-21.368	34952.	
0.00									
3.8000	6.84E-04	100639.	-312.049	-4.08E-05	149.5516	7.43E+10	-20.453	35896.	
0.00									
3.9000	6.36E-04	100250.	-336.033	-3.91E-05	148.9339	7.45E+10	-19.519	36841.	
0.00									
4.0000	5.90E-04	99834.	-358.888	-3.75E-05	148.2902	7.47E+10	-18.572	37786.	
0.00									
4.1000	5.46E-04	99390.	-380.598	-3.59E-05	147.6220	7.49E+10	-17.613	38730.	
0.00									
4.2000	5.04E-04	98921.	-401.156	-3.43E-05	146.9309	7.51E+10	-16.649	39675.	
0.00									
4.3000	4.63E-04	98428.	-500.126	-3.28E-05	146.2183	7.53E+10	-148.302	384127.	
0.00									
4.4000	4.25E-04	97722.	-675.834	-3.12E-05	145.2971	7.55E+10	-144.545	408217.	
0.00									
4.5000	3.88E-04	96807.	-846.505	-2.97E-05	144.1738	7.57E+10	-139.906	432271.	
0.00									
4.6000	3.54E-04	95691.	-1011.	-2.81E-05	142.8561	7.59E+10	-134.492	456289.	
0.00									
4.7000	3.21E-04	94381.	-1169.	-2.66E-05	141.3527	7.61E+10	-128.408	480271.	
0.00									
4.8000	2.90E-04	92886.	-1319.	-2.52E-05	139.6732	7.63E+10	-121.751	504216.	
0.00									
4.9000	2.60E-04	91216.	-1461.	-2.37E-05	137.8280	7.65E+10	-114.618	528126.	
0.00									
5.0000	2.33E-04	89381.	-1594.	-2.23E-05	135.8281	7.67E+10	-107.100	552000.	

0.00									
5.1000	2.07E-04	87392.	-1718.	-2.09E-05	133.6848	7.69E+10	-99.282	575838.	
0.00									
5.2000	1.83E-04	85259.	-1832.	-1.96E-05	131.4100	7.71E+10	-91.246	599641.	
0.00									
5.3000	1.60E-04	82995.	-1937.	-1.83E-05	129.0156	7.73E+10	-83.070	623407.	
0.00									
5.4000	1.39E-04	80612.	-2031.	-1.70E-05	126.5138	7.75E+10	-74.824	647139.	
0.00									
5.5000	1.19E-04	78121.	-2116.	-1.58E-05	123.9165	7.77E+10	-66.576	670834.	
0.00									
5.6000	1.01E-04	75534.	-2191.	-1.46E-05	121.2358	7.79E+10	-58.386	694495.	
0.00									
5.7000	8.41E-05	72862.	-2256.	-1.34E-05	118.4837	7.81E+10	-50.312	718120.	
0.00									
5.8000	6.86E-05	70119.	-2312.	-1.24E-05	115.6716	7.83E+10	-42.404	741710.	
0.00									
5.9000	5.44E-05	67314.	-2358.	-1.13E-05	112.8108	7.85E+10	-34.709	765264.	
0.00									
6.0000	4.15E-05	64459.	-2395.	-1.03E-05	109.9123	7.87E+10	-27.268	788784.	
0.00									
6.1000	2.97E-05	61565.	-2424.	-9.33E-06	106.9866	7.89E+10	-20.117	812269.	
0.00									
6.2000	1.91E-05	58642.	-2444.	-8.42E-06	104.0436	7.91E+10	-13.288	835719.	
0.00									
6.3000	9.51E-06	55700.	-2460.	-7.56E-06	101.0929	7.93E+10	-13.071	1649536.	
0.00									
6.4000	9.48E-07	52739.	-2468.	-6.74E-06	98.1348	7.95E+10	-1.339	1694426.	
0.00									
6.5000	-6.66E-06	49776.	-2463.	-5.96E-06	95.1853	7.97E+10	9.6490	1739249.	
0.00									
6.6000	-1.34E-05	46827.	-2446.	-5.24E-06	92.2596	7.99E+10	19.8670	1784006.	
0.00									
6.7000	-1.92E-05	43907.	-2416.	-4.56E-06	89.3715	8.01E+10	29.2981	1828697.	
0.00									
6.8000	-2.43E-05	41029.	-2376.	-3.92E-06	86.5335	8.03E+10	37.9325	1873321.	
0.00									
6.9000	-2.86E-05	38205.	-2326.	-3.33E-06	83.7571	8.05E+10	45.7669	1917880.	

0.00								
7.0000	-3.23E-05	35447.	-2266.	-2.78E-06	81.0528	8.07E+10	52.8043	1962372.
0.00								
7.1000	-3.53E-05	32765.	-2199.	-2.28E-06	78.4295	8.09E+10	59.0533	2006799.
0.00								
7.2000	-3.78E-05	30169.	-2125.	-1.81E-06	75.8954	8.11E+10	64.5275	2051161.
0.00								
7.3000	-3.97E-05	27665.	-2045.	-1.38E-06	73.4574	8.13E+10	69.2449	2095457.
0.00								
7.4000	-4.11E-05	25261.	-1959.	-9.92E-07	71.1212	8.16E+10	73.2273	2139687.
0.00								
7.5000	-4.20E-05	22962.	-1870.	-6.38E-07	68.8918	8.18E+10	76.4999	2183853.
0.00								
7.6000	-4.26E-05	20774.	-1776.	-3.17E-07	66.7729	8.20E+10	79.0906	2227953.
0.00								
7.7000	-4.28E-05	18699.	-1680.	-2.87E-08	64.7675	8.22E+10	81.0293	2271989.
0.00								
7.8000	-4.27E-05	16742.	-1582.	2.30E-07	62.8775	8.24E+10	82.3479	2315960.
0.00								
7.9000	-4.22E-05	14902.	-1483.	4.60E-07	61.1041	8.26E+10	83.0792	2359867.
0.00								
8.0000	-4.16E-05	13183.	-1383.	6.64E-07	59.4477	8.28E+10	83.2570	2403709.
0.00								
8.1000	-4.07E-05	11583.	-1283.	8.43E-07	57.9077	8.30E+10	82.9152	2447486.
0.00								
8.2000	-3.95E-05	10102.	-1184.	9.99E-07	56.4833	8.32E+10	82.0876	2491200.
0.00								
8.3000	-3.83E-05	8740.	-1087.	1.14E-06	55.1727	8.34E+10	80.8077	2534850.
0.00								
8.4000	-3.68E-05	7494.	-990.714	1.25E-06	53.9735	8.36E+10	79.1081	2578435.
0.00								
8.5000	-3.53E-05	6362.	-897.037	1.35E-06	52.8830	8.39E+10	77.0203	2621958.
0.00								
8.6000	-3.36E-05	5341.	-806.080	1.43E-06	51.8979	8.41E+10	74.5744	2665416.
0.00								
8.7000	-3.18E-05	4428.	-718.256	1.50E-06	51.0143	8.43E+10	71.7988	2708811.
0.00								
8.8000	-3.00E-05	3618.	-633.945	1.56E-06	50.2280	8.45E+10	68.7203	2752143.

0.00									
8.9000	-2.81E-05	2906.	-553.494	1.61E-06	49.5346	8.47E+10	65.3636	2795412.	
0.00									
9.0000	-2.61E-05	2289.	-477.225	1.64E-06	48.9291	8.49E+10	61.7513	2838617.	
0.00									
9.1000	-2.41E-05	1761.	-405.432	1.67E-06	48.4063	8.51E+10	57.9038	2881760.	
0.00									
9.2000	-2.21E-05	1316.	-338.386	1.69E-06	47.9608	8.53E+10	53.8392	2924840.	
0.00									
9.3000	-2.00E-05	948.6901	-276.339	1.71E-06	47.5870	8.55E+10	49.5733	2967858.	
0.00									
9.4000	-1.80E-05	652.7534	-219.523	1.72E-06	47.2788	8.58E+10	45.1199	3010813.	
0.00									
9.5000	-1.59E-05	421.7894	-168.157	1.73E-06	47.0301	8.60E+10	40.4902	3053705.	
0.00									
9.6000	-1.38E-05	249.1311	-122.447	1.73E-06	46.8347	8.62E+10	35.6935	3096536.	
0.00									
9.7000	-1.17E-05	127.8715	-82.588	1.74E-06	46.6861	8.64E+10	30.7368	3139304.	
0.00									
9.8000	-9.66E-06	50.8728	-48.771	1.74E-06	46.5775	8.66E+10	25.6255	3182011.	
0.00									
9.9000	-7.58E-06	10.7749	-21.178	1.74E-06	46.5023	8.68E+10	20.3632	3224656.	
0.00									
10.0000	-5.49E-06	0.00	0.00	1.74E-06	46.4535	8.71E+10	14.9517	1633619.	
0.00									

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection	=	0.00402059 inches
Computed slope at pile head	=	-0.0001065 radians
Maximum bending moment	=	102860. inch-lbs
Maximum shear force	=	-2468. lbs
Depth of maximum bending moment	=	2.40000000 feet below pile head
Depth of maximum shear force	=	6.40000000 feet below pile head
Number of iterations	=	6

Number of zero deflection points = 1

Pile-head Deflection vs. Pile Length for Load Case 1

Boundary Condition Type 1, Shear and Moment

Shear = 253. lbs
Moment = 99208. in-lbs
Axial Load = 11061. lbs

Pile Length feet	Pile Head Deflection inches	Maximum Moment ln-lbs	Maximum Shear lbs
10.00000	0.00402059	102860.	-2468.
9.50000	0.00401872	102862.	-2497.
9.00000	0.00401664	102863.	-2592.
8.50000	0.00409692	102835.	-2803.
8.00000	0.00425322	102787.	-3135.
7.50000	0.00465598	102674.	-3603.
7.00000	0.00546937	102491.	-4128.
6.50000	0.00738670	102187.	-4670.
6.00000	0.01302904	101707.	-5540.
5.50000	0.02604840	101253.	-6675.
5.00000	0.08477952	100742.	-8413.
4.50000	0.54675362	100768.	-9173.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 2

0.00	1.2000	0.01434	512756.	1043.	-4.51E-04	588.0776	6.94E+10	-16.015	1340.
0.00	1.3000	0.01380	514002.	1024.	-4.43E-04	588.3145	6.96E+10	-16.092	1399.
0.00	1.4000	0.01328	515225.	1004.	-4.34E-04	588.5250	6.98E+10	-16.160	1461.
0.00	1.5000	0.01276	516425.	985.0445	-4.25E-04	588.7093	6.99E+10	-16.220	1525.
0.00	1.6000	0.01226	517601.	965.5492	-4.16E-04	588.8673	7.01E+10	-16.272	1593.
0.00	1.7000	0.01176	518753.	945.9968	-4.07E-04	588.9991	7.03E+10	-16.315	1664.
0.00	1.8000	0.01128	519882.	926.3978	-3.98E-04	589.1049	7.05E+10	-16.350	1739.
0.00	1.9000	0.01081	520987.	906.7623	-3.89E-04	589.1847	7.07E+10	-16.376	1818.
0.00	2.0000	0.01034	522068.	887.1008	-3.81E-04	589.2386	7.09E+10	-16.393	1902.
0.00	2.1000	0.00989	523126.	867.4240	-3.72E-04	589.2667	7.11E+10	-16.402	1989.
0.00	2.2000	0.00945	524160.	847.7425	-3.63E-04	589.2692	7.13E+10	-16.401	2082.
0.00	2.3000	0.00902	525170.	828.0673	-3.54E-04	589.2463	7.15E+10	-16.391	2180.
0.00	2.4000	0.00860	526157.	808.4092	-3.45E-04	589.1980	7.16E+10	-16.372	2284.
0.00	2.5000	0.00819	527120.	788.7794	-3.37E-04	589.1246	7.18E+10	-16.344	2394.
0.00	2.6000	0.00779	528059.	769.1892	-3.28E-04	589.0262	7.20E+10	-16.306	2510.
0.00	2.7000	0.00741	528974.	749.6499	-3.19E-04	588.9030	7.22E+10	-16.259	2634.
0.00	2.8000	0.00703	529866.	730.1732	-3.10E-04	588.7553	7.24E+10	-16.202	2766.
0.00	2.9000	0.00666	530735.	710.7707	-3.01E-04	588.5833	7.26E+10	-16.135	2906.
0.00	3.0000	0.00631	531580.	627.1751	-2.93E-04	588.3871	7.28E+10	-123.191	23442.

0.00	3.1000	0.00596	532248.	475.9252	-2.84E-04	588.0110	7.30E+10	-128.892	25950.
0.00	3.2000	0.00562	532730.	317.8544	-2.75E-04	587.4476	7.32E+10	-134.559	28706.
0.00	3.3000	0.00530	533018.	154.5091	-2.66E-04	586.6897	7.34E+10	-137.683	31173.
0.00	3.4000	0.00499	533108.	-8.165	-2.58E-04	585.7339	7.36E+10	-133.441	32118.
0.00	3.5000	0.00468	533005.	-165.624	-2.49E-04	584.5875	7.37E+10	-128.991	33062.
0.00	3.6000	0.00439	532717.	-317.632	-2.40E-04	583.2580	7.39E+10	-124.357	34007.
0.00	3.7000	0.00410	532249.	-463.984	-2.32E-04	581.7530	7.41E+10	-119.563	34952.
0.00	3.8000	0.00383	531610.	-604.502	-2.23E-04	580.0804	7.43E+10	-114.633	35896.
0.00	3.9000	0.00357	530805.	-739.035	-2.15E-04	578.2484	7.45E+10	-109.589	36841.
0.00	4.0000	0.00332	529842.	-867.462	-2.06E-04	576.2649	7.47E+10	-104.455	37786.
0.00	4.1000	0.00308	528728.	-989.687	-1.97E-04	574.1381	7.49E+10	-99.253	38730.
0.00	4.2000	0.00284	527472.	-1106.	-1.89E-04	571.8765	7.51E+10	-94.007	39675.
0.00	4.3000	0.00262	526080.	-1666.	-1.81E-04	569.4880	7.53E+10	-839.160	384127.
0.00	4.4000	0.00241	523479.	-2661.	-1.72E-04	565.9135	7.55E+10	-819.757	408217.
0.00	4.5000	0.00221	519698.	-3630.	-1.64E-04	561.1870	7.57E+10	-795.383	432271.
0.00	4.6000	0.00202	514772.	-4567.	-1.56E-04	555.3495	7.59E+10	-766.618	456289.
0.00	4.7000	0.00183	508741.	-5468.	-1.48E-04	548.4478	7.61E+10	-734.028	480271.
0.00	4.8000	0.00166	501653.	-6327.	-1.40E-04	540.5340	7.63E+10	-698.154	504216.
0.00	4.9000	0.00150	493560.	-7141.	-1.32E-04	531.6641	7.65E+10	-659.520	528126.

0.00	5.0000	0.00134	484517.	-7908.	-1.24E-04	521.8978	7.67E+10	-618.624	552000.
0.00	5.1000	0.00120	474583.	-8625.	-1.17E-04	511.2977	7.69E+10	-575.942	575838.
0.00	5.2000	0.00106	463820.	-9290.	-1.09E-04	499.9280	7.71E+10	-531.925	599641.
0.00	5.3000	9.37E-04	452291.	-9901.	-1.02E-04	487.8548	7.73E+10	-486.996	623407.
0.00	5.4000	8.19E-04	440060.	-10458.	-9.55E-05	475.1447	7.75E+10	-441.555	647139.
0.00	5.5000	7.08E-04	427193.	-10961.	-8.88E-05	461.8646	7.77E+10	-395.973	670834.
0.00	5.6000	6.06E-04	413756.	-11409.	-8.23E-05	448.0811	7.79E+10	-350.594	694495.
0.00	5.7000	5.11E-04	399814.	-11803.	-7.60E-05	433.8602	7.81E+10	-305.734	718120.
0.00	5.8000	4.23E-04	385432.	-12143.	-7.00E-05	419.2664	7.83E+10	-261.683	741710.
0.00	5.9000	3.43E-04	370673.	-12431.	-6.42E-05	404.3629	7.85E+10	-218.703	765264.
0.00	6.0000	2.69E-04	355599.	-12669.	-5.86E-05	389.2107	7.87E+10	-177.027	788784.
0.00	6.1000	2.02E-04	340270.	-12857.	-5.33E-05	373.8689	7.89E+10	-136.864	812269.
0.00	6.2000	1.41E-04	324744.	-12998.	-4.83E-05	358.3938	7.91E+10	-98.396	835719.
0.00	6.3000	8.63E-05	309076.	-13128.	-4.35E-05	342.8390	7.93E+10	-118.612	1649536.
0.00	6.4000	3.69E-05	293237.	-13231.	-3.89E-05	327.1774	7.95E+10	-52.107	1694426.
0.00	6.5000	-7.17E-06	277323.	-13256.	-3.46E-05	311.5006	7.97E+10	10.3937	1739249.
0.00	6.6000	-4.62E-05	261424.	-13208.	-3.06E-05	295.8942	7.99E+10	68.7357	1784006.
0.00	6.7000	-8.06E-05	245623.	-13093.	-2.68E-05	280.4376	8.01E+10	122.8078	1828697.
0.00	6.8000	-1.11E-04	230000.	-12916.	-2.32E-05	265.2035	8.03E+10	172.5401	1873321.

0.00	6.9000	-1.36E-04	214625.	-12682.	-1.99E-05	250.2585	8.05E+10	217.9007	1917880.
0.00	7.0000	-1.58E-04	199564.	-12396.	-1.68E-05	235.6626	8.07E+10	258.8929	1962372.
0.00	7.1000	-1.77E-04	184875.	-12063.	-1.40E-05	221.4695	8.09E+10	295.5517	2006799.
0.00	7.2000	-1.92E-04	170612.	-11689.	-1.13E-05	207.7267	8.11E+10	327.9410	2051161.
0.00	7.3000	-2.04E-04	156822.	-11279.	-8.93E-06	194.4754	8.13E+10	356.1500	2095457.
0.00	7.4000	-2.13E-04	143544.	-10837.	-6.71E-06	181.7512	8.16E+10	380.2906	2139687.
0.00	7.5000	-2.20E-04	130814.	-10368.	-4.70E-06	169.5836	8.18E+10	400.4937	2183853.
0.00	7.6000	-2.25E-04	118660.	-9878.	-2.87E-06	157.9969	8.20E+10	416.9067	2227953.
0.00	7.7000	-2.27E-04	107107.	-9370.	-1.22E-06	147.0100	8.22E+10	429.6900	2271989.
0.00	7.8000	-2.27E-04	96172.	-8849.	2.65E-07	136.6368	8.24E+10	439.0147	2315960.
0.00	7.9000	-2.26E-04	85870.	-8318.	1.59E-06	126.8865	8.26E+10	445.0597	2359867.
0.00	8.0000	-2.24E-04	76208.	-7782.	2.77E-06	117.7639	8.28E+10	448.0089	2403709.
0.00	8.1000	-2.20E-04	67192.	-7245.	3.80E-06	109.2695	8.30E+10	448.0491	2447486.
0.00	8.2000	-2.15E-04	58821.	-6709.	4.71E-06	101.4002	8.32E+10	445.3676	2491200.
0.00	8.3000	-2.08E-04	51091.	-6177.	5.50E-06	94.1490	8.34E+10	440.1500	2534850.
0.00	8.4000	-2.01E-04	43995.	-5654.	6.19E-06	87.5058	8.36E+10	432.5783	2578435.
0.00	8.5000	-1.94E-04	37522.	-5141.	6.77E-06	81.4573	8.39E+10	422.8296	2621958.
0.00	8.6000	-1.85E-04	31657.	-4640.	7.27E-06	75.9877	8.41E+10	411.0737	2665416.
0.00	8.7000	-1.76E-04	26385.	-4155.	7.68E-06	71.0782	8.43E+10	397.4724	2708811.

8.8000	-1.67E-04	21685.	-3687.	8.02E-06	66.7082	8.45E+10	382.1781	2752143.
0.00								
8.9000	-1.57E-04	17535.	-3239.	8.30E-06	62.8547	8.47E+10	365.3327	2795412.
0.00								
9.0000	-1.47E-04	13912.	-2811.	8.52E-06	59.4929	8.49E+10	347.0669	2838617.
0.00								
9.1000	-1.36E-04	10788.	-2407.	8.70E-06	56.5964	8.51E+10	327.4994	2881760.
0.00								
9.2000	-1.26E-04	8136.	-2026.	8.83E-06	54.1372	8.53E+10	306.7367	2924840.
0.00								
9.3000	-1.15E-04	5925.	-1671.	8.93E-06	52.0860	8.55E+10	284.8726	2967858.
0.00								
9.4000	-1.04E-04	4125.	-1343.	9.00E-06	50.4121	8.58E+10	261.9882	3010813.
0.00								
9.5000	-9.36E-05	2702.	-1043.	9.05E-06	49.0840	8.60E+10	238.1521	3053705.
0.00								
9.6000	-8.27E-05	1621.	-771.949	9.08E-06	48.0689	8.62E+10	213.4205	3096536.
0.00								
9.7000	-7.18E-05	848.6875	-531.194	9.09E-06	47.3332	8.64E+10	187.8377	3139304.
0.00								
9.8000	-6.09E-05	346.3766	-321.630	9.10E-06	46.8423	8.66E+10	161.4370	3182011.
0.00								
9.9000	-5.00E-05	76.5349	-144.223	9.11E-06	46.5611	8.68E+10	134.2409	3224656.
0.00								
10.0000	-3.90E-05	0.00	0.00	9.11E-06	46.4535	8.71E+10	106.2628	1633619.
0.00								

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 2:

Pile-head deflection	=	0.02160622 inches
Computed slope at pile head	=	-0.0005579 radians
Maximum bending moment	=	533108. inch-lbs
Maximum shear force	=	-13256. lbs
Depth of maximum bending moment	=	3.40000000 feet below pile head
Depth of maximum shear force	=	6.50000000 feet below pile head

Number of iterations = 6
Number of zero deflection points = 1

Pile-head Deflection vs. Pile Length for Load Case 2

Boundary Condition Type 1, Shear and Moment

Shear = 1264. lbs
Moment = 496038. in-lbs
Axial Load = 11061. lbs

Pile Length feet	Pile Head Deflection inches	Maximum Moment ln-lbs	Maximum Shear lbs
10.00000	0.02160622	533108.	-13256.
9.50000	0.02160020	533392.	-13432.
9.00000	0.02159364	533534.	-14013.
8.50000	0.02208109	533393.	-15240.
8.00000	0.02302539	533018.	-17121.
7.50000	0.02550849	532187.	-19823.
7.00000	0.03069607	531252.	-23078.
6.50000	0.04362889	529630.	-26808.
6.00000	0.09029829	525450.	-34865.
5.50000	0.21429439	520542.	-46286.
5.00000	0.91738162	515082.	-70742.

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	253.0000	M, in-lb	99208.	11061.	0.00402	-1.06E-04	-2468.	102860.
2	V, lb	1264.	M, in-lb	496038.	11061.	0.02161	-5.58E-04	-13256.	533108.

Maximum pile-head deflection = 0.0216062218 inches

Maximum pile-head rotation = -0.0005579208 radians = -0.031967 deg.

 Summary of Warning Messages

The following warning was reported 3406 times

**** Warning ****

An unreasonable input value for unconfined compressive strength has been specified for a soil defined using the weak rock criteria. The input value is greater than 500 psi. Please check your input data for correctness.

The analysis ended normally.