
**FINAL REPORT
ROADWAY EXPLORATION REPORT
FRA-CEMETERY RD SUP
FRANKLIN COUNTY, OHIO
PID#: 120103**

Prepared For:

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NEAS PROJECT 24-0005

July 14, 2026



**ROADWAY EXPLORATION REPORT
FRA-CEMETERY ROAD SUP
FRANKLIN COUNTY, OHIO
PID: 120103**

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SECTION 1

GEOTECHNICAL PROJECT INFORMATION

SECTION 1: GEOTECHNICAL PROJECT INFORMATION

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1.1 INTRODUCTION

1.1.1. General

National Engineering & Architectural Services, Inc. (NEAS) presents our Roadway Exploration Report for the shared use path (SUP) and associated structures proposed as part of the Ohio Department of Transportation (ODOT) FRA-Cemetery Rd SUP (PID 120103) project located within the city of Hilliard, Franklin County, Ohio.

It is our understanding that the proposed project is intended to provide a SUP on the south side of Cemetery Road (Rd) in the city of Hilliard, Ohio to allow for pedestrian access between Britton Parkway (Pkw) and Trueman Boulevard (Blvd). Based on the FRA-Cemetery Rd SUP Stage 2 plans prepared by Burgess & Niple, Inc. (B&N), the proposed project will consist of: 1) the construction of approximately 3,090 lineal feet of path extending from Britton Pkw to Trueman Blvd; 2) the realignment and reconstruction of the existing Cemetery Rd eastbound (EB) to Interstate Route 270 (IR-270) southbound (SB) entrance ramp; 3) the construction of a new pedestrian bridge planned to parallel the existing Cemetery Rd Bridge (FRA-270-0419 L&R) over IR-270; 4) the construction of a pedestrian tunnel/culvert to carry foot traffic beneath the referenced newly realigned IR-270 SB entrance ramp; and, 5) the construction of a new retaining wall near the eastern end of the proposed path alignment to facilitate the construction of the path along a portion of existing Cemetery Rd embankment slope. This report presents a summary of the encountered surficial and subsurface conditions, our recommendations for structure foundation design and construction as well as our recommendations for subgrade (ramp and path) stabilization, pavement design parameters and embankment construction. Foundation recommendations and analysis were performed in accordance with Load and Resistance Factors Design (LRFD) method as set forth in AASHTO's Publication LRFD Bridge Design Specifications, 10th Edition (AASHTO, 2024), the 2026 revision of ODOT's Bridge Design Manual 2020 Edition (BDM) (ODOT [1], 2026) and ODOT's July 2026 revision of their Geotechnical Design Manual (GDM) (ODOT, 2026). The roadway analysis performed as part of this report has been performed in accordance with ODOT's July 2025 revision of the GDM and Pavement Design Manual (PDM) (ODOT, 2025).

The exploration was conducted in general accordance with Barr Engineering, Inc. DBA National Engineering & Architectural Services, Inc.'s (NEAS) proposal to Burgess & Niple, Inc. (B&N), dated November 15, 2023, with the provisions of ODOT's *Specifications for Geotechnical Explorations* (SGE) (ODOT, 2024).

The scope of work performed by NEAS as part of the referenced project included: a review of published geotechnical information; performing 16 total test borings; laboratory testing of soil samples in accordance with the SGE; performing geotechnical engineering analysis to assess proposed foundation design and construction considerations, embankment, subgrade stabilization requirements and recommended pavement design parameters; and development of this summary report.

1.2 GEOLOGY AND OBSERVATIONS OF THE PROJECT

1.2.1. Geology and Physiography

The project site is located within the Columbus Lowland physiographic region, part of the Till Plains (ODGS, 1998). This is a lowland area surrounded in all directions by relative uplands, having a broad regional slope toward the Scioto Valley with many larger streams and moderately low relief. The geology within this region consists of Loamy, high-lime in the west to medium-lime in the east, Wisconsinan-age

till and extensive outwash in Scioto Valley over deep Devonian- to Mississippian-age carbonate rocks, shales, and siltstones.

The geology at the project site is mapped as an average of 60 ft of Wisconsinan till atop Devonian-age limestone and some dolomite. The till in this region is comprised of an unsorted mix of silt, clay, sand, gravel, and boulders with variable carbonate content. The till at the site may contain silt, sand, and gravel lenses and deposited directly from several separate ice advances (Pavey, et al., 2013).

Based on the Bedrock Geologic Units Map of Ohio (USGS & ODGS, 2005), bedrock within the project area is primarily Columbus Limestone. The Columbus Limestone is comprised of fossiliferous, gray limestone in the upper two thirds and brown dolomite in the lower third, featuring massive bedding. The bedrock is described as gray to brown in color weathering to brown. Based on the ODNR bedrock topography map of Ohio, bedrock elevation at the bridge site can be expected to be between about 780 and 800 ft above mean sea level (amsl), putting bedrock at depths ranging from 70 to 110 ft below ground surface (bgs) (ODGS, 2003).

The soils at the project site have been mapped (Web Soil Survey) by the Natural Resources Conservation Service (USDA, 2015) as Urban land in the central portion, Celina silt loam in the eastern portion of the site, as well as Crosby silt loam and Kokomo silty clay loam in the western portion of the project. Soils classified as Urban Land 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 Celina series are characterized as very deep, moderately well drained soils, and moderately deep to dense till. The Celina series is comprised of both coarse- and fine-grained soils and classifies as A-6, and A-7 type soils according to the AASHTO method of soil classification. Soils in the Crosby series are characterized as very deep, poorly drained soils that are moderately deep to dense till. The Crosby series is comprised of primarily fine-grained soils and classifies as A-4, A-6, and A-7 type soils according to the AASHTO method of soil classification. Soils in the Kokomo series are characterized as very deep, very poorly drained soils that formed in loamy materials overlying till, and in depressions on till plains. The Kokomo series is comprised of primarily fine-grained soils and classifies as A-6 type soil according to the AASHTO method of soil classification.

1.2.2. Hydrology/Hydrogeology

Groundwater at the project site can be expected at an elevation consistent with that of the nearby Scioto River as it is the most dominant hydraulic influence in the vicinity of the project's boundaries. The water level of the river 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 proposed structures at the project site are not located within a flood hazard area based on available mapping by the Federal Emergency Management Agency's (FEMA) National Flood Hazard mapping program (FEMA, 2016).

1.2.3. Mining and Oil/Gas Production

No abandoned mines are noted on ODNR's Abandoned Underground Mine Locator within the immediate vicinity of the project's boundaries (ODNR [1], 2016).

No oil or gas wells are noted on ODNR's Ohio Oil & Gas Locator within the immediate vicinity of the project's boundaries (ODNR [2], 2016).

1.2.4. Historical Records and Previous Phases of Project Exploration

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

- Project Boring Logs, and Structural Foundation Investigation Sheets for project FRA-270-4.19, dated September 30, 1964;
- Project Boring Logs, and Structural Foundation Investigation Sheets for project FRA-270-3.67, dated November 12, 1964; and,
- Project Boring Logs, and Structural Foundation Investigation Sheets for project FRA-270-3.74, dated October 05, 1987.

Historical soil borings associated with the above plans were reviewed, however, they were not utilized for our analysis, and therefore, are not referenced within this report nor within the project developed Soil Profile Sheets.

1.2.5. Field Reconnaissance

A field reconnaissance visit for the project site was conducted on July 16th, 2024. Site conditions were noted and photographed during the visit. Photographs of notable features and a summary of our observations are provided below.

During our field reconnaissance, no geohazards were observed within the project limits. The land use of most of the project area consists of ODOT right-of-way (ROW) and commercial properties.

In general, the land located along the proposed SUP alignment consists of ODOT ROW property located south of the existing Cemetery Rd bridge over IR-270 (Photograph 1). Currently, the proposed bridge and SUP area consists of embankment slopes associated with the existing bridge as well as pavement, shoulders and drainage ditches associated with IR-270. The existing embankment slopes were vegetated with small to large/mature trees as well as heavy brush and bushes while the property immediately adjacent to IR-270 was generally mowed grass (Photographs 2). With respect to topography, the eastern Cemetery Rd embankment was observed to slope downward from roadway grades ranging from about 2 Horizontal to 1 Vertical (2H:1V) to 3.5H:1V. The western embankment was observed to slope downward at about a 2H:1V slope. No signs of geotechnical instability were observed along the existing embankments at the time of our reconnaissance.

The pavement conditions at the time of reconnaissance for the project area was observed to be in good condition with some signs of weathering and surface wear (Photograph 3). Crack sealing along Cemetery Rd was noted as well as low severity edge cracking and occasional raveling. Occasional low severity transverse cracking and raveling was observed along the existing IR-270 ramps to/from Cemetery Rd EB. Each roadway appeared to be relatively well drained, however, evidence of standing water was noted south of Cemetery Rd on the west side of the project area as well as along the west side of the existing Cemetery Rd EB to IR-270 SB entrance ramp. IR-270 and associated ramps appeared to be well drained to ditches on either side of the roadway. Overall, it appears that surface water drainage of the project area slopes to the south and likely drains to the Tudor Ditch (Photograph 4) and associated detention ponds within the area which likely drain into the Scioto River.

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Photograph 1: Overall view of bridge site located south of Cemetery Rd Bridge



Photograph 2: Property located at base of eastern embankment slope



Photograph 3: General pavement condition at the site



Photograph 4: Tudor Ditch located south of Cemetery Rd (west side)



1.3 GEOTECHNICAL EXPLORATION

1.3.1. Field Exploration Program

The exploration for the project was conducted by NEAS between July 30, 2024 and October 1, 2024, including 16 borings drilled to depths between 7.5 and 86.5 ft bgs. The boring locations were selected by B&N in general accordance with the guidelines contained in the SGE with the intent to evaluate subsurface soil and groundwater conditions at the site. Borings were typically located along proposed roadway/path alignments or at/near proposed (sub)structure locations that were not restricted by underground utilities or dictated by terrain (i.e., steep embankment slopes). Each as-drilled project boring location and corresponding ground surface elevation was surveyed in the field by NEAS prior to drilling operations utilizing a hand-held GPS unit. Each individual project boring log (included within Appendix 1B) includes the recorded boring latitude and longitude location (based on the surveyed Ohio State Plane South, NAD83, location) and the corresponding ground surface elevation. Coordinates, elevations and depths of the borings are shown in Table 1 below and boring locations are depicted on the Boring Location Plan provided in Appendix A.

Table 1: Project Boring Summary

Boring Number	Location (Sta/Offset)	Latitude	Longitude	Elevation (NAVD 88) (ft)	Depth (ft)	Structure / Boring Type
B-001-0-23	98+13, 35' RT.	40.033433	-83.117992	859.2	35.0	Retaining Wall
B-002-0-23	95+98, 42' RT.	40.033549	-83.118706	861.7	35.0	Retaining Wall
B-003-0-23	91+75, 51' RT.	40.033439	-83.120165	866.7	10.0	Roadway
B-004-0-23	87+02, 209' RT.	40.032821	-83.121755	871.2	7.5	Existing Subgrade
B-005-0-23	85+55, 48' RT.	40.033202	-83.122358	885.0	20.0	Roadway / Embankment
B-006-0-23	83+22, 104' RT.	40.032959	-83.123154	870.3	80.2	Bridge Forward Abutment
B-007-0-23	82+45, 84' RT.	40.032981	-83.123433	870.5	71.5	Bridge Pier 3
B-008-0-23	TBD	40.033007	-83.123850	870.6	65.0	Bridge Pier 2
B-009-0-23	80+50, 86' RT.	40.032899	-83.124121	870.6	71.5	Bridge Pier 1
B-010-0-23	79+62, 90' RT.	40.032855	-83.124432	873.6	86.5	Bridge Rear Abutment
B-011-0-23	108+32, 2' Left	40.031933	-83.124267	871.6	7.5	Existing Subgrade
B-012-0-23	110+40, 53' RT.	40.032447	-83.124687	872.4	51.5	Culvert / Tunnel
B-013-0-23	110+55, 25' Left	40.032294	-83.124890	871.5	50.0	Culvert / Tunnel
B-014-0-23	78+22, 101' RT.	40.032769	-83.124919	873.6	20.0	Roadway / Embankment
B-015-0-23	114+51, 3' Left	40.032654	-83.126175	876.4	7.5	Existing Subgrade
B-016-0-23	68+53, 39' RT.	40.032598	-83.128390	883.9	10.0	Roadway
Notes: 1. As-drilled boring location and corresponding ground surface elevation was surveyed in the field by B&N.						

Borings were drilled using a CME 55TB track-mounted drilling rig utilizing 3.25-inch diameter hollow stem augers. Soil samples were generally recovered using a split spoon sampler (AASHTO T-206 "Standard Method for Penetration Test and Split Barrel Sampling of Soils.") at intervals of 2.5-ft to a depth of 35 ft bgs and at 5-ft intervals thereafter until boring termination. The soil samples obtained from the exploration program were visually observed in the field by the NEAS field representative and preserved for review by a Geologist and possible laboratory testing. Standard penetration tests (SPT) were conducted using CME auto hammers that have been calibrated to be 89% efficient as indicated on the boring logs.

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.

1.3.2. Laboratory Testing Program

The laboratory testing program consisted of classification testing, moisture content determinations, and sulfate content testing. Data from the laboratory testing program were incorporated onto the boring logs (Appendix 1B). Soil samples are retained at the laboratory for 60 days following report submittal, after which time they will be discarded.

1.3.2.1. Classification Testing

Representative soil samples were selected for index property (Atterberg Limits) and gradation testing for classification purposes on approximately forty-seven percent (47%) of the samples. At each subgrade boring location, the upper two samples obtained below the proposed top of subgrade elevation were generally tested while additional samples were selected for testing with the intent of properly classifying the subsurface soil and groundwater conditions within the planned project limits. 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 and ODOT Supplements.

Final classification of soil strata in accordance with AASHTO M-145 "Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes," as modified by ODOT "Classification of Soils" was made once laboratory test results became available. The results of the soil classification are presented on the boring logs in Appendix 1B.

1.3.2.2. Standard Penetration Test Results

Standard Penetration Tests (SPT) and split-barrel (commonly known as split-spoon) sampling of soils were performed continuously 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 1B.

1.3.2.3. Sulfate Testing

Sulfate testing was generally performed on one sample from each subgrade 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 1 below, provided in Appendix 1C and presented on the boring logs within Appendix 1B.

Table 2: Sulfate Test Summary by Boring

Boring ID	Sample	Depth (ft)	Dilution Ratio	Average Sulfate Content (ppm)
B-001-0-23	SS-2	3.5 - 5.0	20	7
B-003-0-23	SS-2	3.5 - 5.0	20	673
B-004-0-23	SS-1	1.5 - 3.0	20	13
B-005-0-23	SS-1	1.0 - 2.5	20	20
B-011-0-23	SS-1	1.5 - 3.0	20	200
B-012-0-23	SS-2	3.5 - 5.0	20	20
B-013-0-23	SS-1	1.0 - 2.5	20	20
B-014-0-23	SS-1	1.0 - 2.5	20	20
B-015-0-23	SS-1	1.5 - 3.0	20	13
B-016-0-23	SS-1	1.0 - 2.5	20	20

1.3.2.4. Consolidation Testing

One (1) consolidation test was performed in accordance with ASTM D 2435-04 “Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading” on a relatively undisturbed cohesive soil sample collected from boring B-008-0-23; the results of the consolidation test are presented in Table 3 below, while the laboratory testing report is included with the associated boring log within Appendix 1B.

Table 3: Consolidation Test Results

Boring Number	Sample ID	Depth (ft)	Elevation (ft)	Compression Index (Cc)	Recompression Index (Cr)	Preconsolidation Pressure (psf)	Void Ratio
B-008-0-23	ST-7	16.8 - 16.9	853.8 - 853.7	0.046	0.007	7,000	0.330

1.4 FINDINGS, ANALYSES AND RECOMMENDATIONS

The pavement and subsurface conditions encountered during NEAS’s explorations are described in the following sections of this report and/or on each boring log presented in Appendix 1B. 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.

A summary of the subsurface conditions as well as analyses and recommendations for specific project design elements (i.e. roadways, structures, etc.) are provided within their dedicated section of this report. The specific design elements included within this report for the FRA-Cemetery Rd SUP include:

- Section 2: Project Roadway and Embankments;
- Section 3: Cemetery Rd Shared Use Path Bridge over IR-270;
- Section 4: Pedestrian Tunnel; and,
- Section 5: Retaining Wall.

1.5 QUALIFICATIONS

This investigation was performed in accordance with accepted geotechnical engineering practice for the purpose of characterizing the subsurface conditions along the referenced portions of roadways. This report has been prepared for Burgess & Niple, Inc., ODOT and their design consultants to be used solely in evaluating the subgrade soils within the project limits 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 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 construction, reconfiguration and rehabilitation 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 Burgess & Niple, Inc. in performing this geotechnical exploration for the FRA-Cemetery Road SUP project. Please call if there are any questions, or if we can be of further service.

Respectfully Submitted,

Brendan P. Andrews, P.E.
Project Geotechnical Engineer

Kevin C. Arens P.E.
Geotechnical Staff Engineer

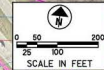
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APPENDIX 1A

BORING LOCATION PLAN

FRA-Cemetery SUP, PID 120103
Proposed Boring Locations

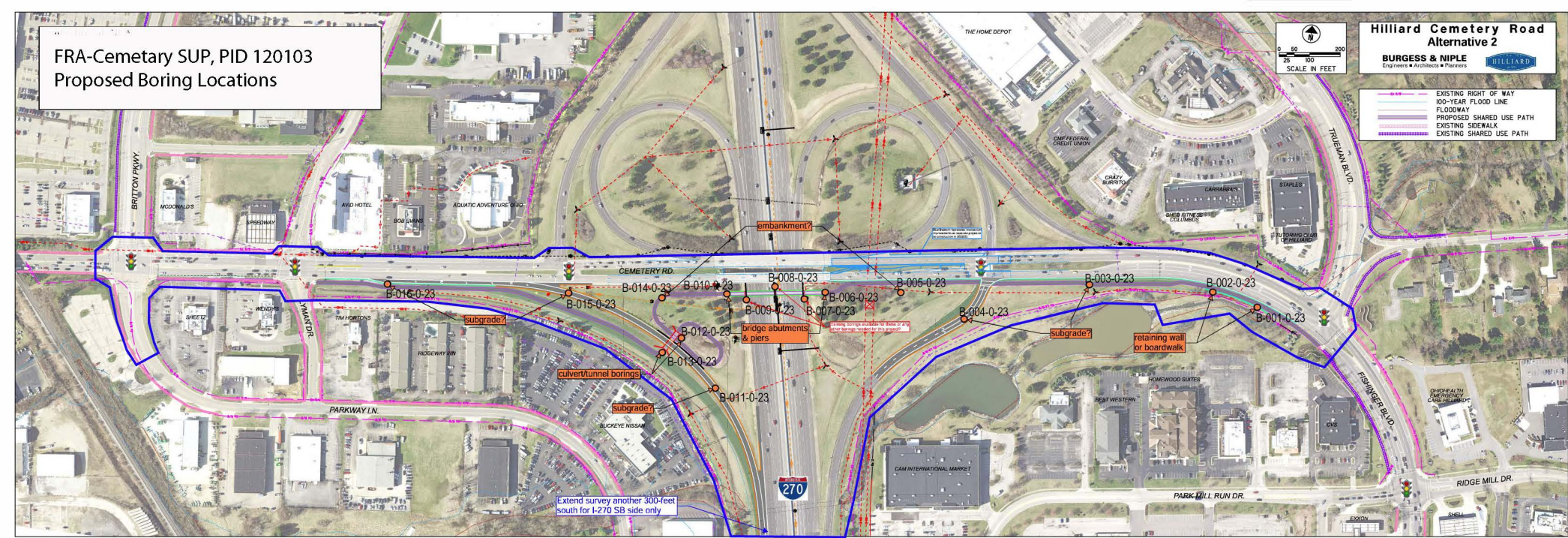


Hilliard Cemetery Road
Alternative 2

BURGESS & NIPLE
Engineers • Architects • Planners

HILLIARD

- EXISTING RIGHT OF WAY
- 100-YEAR FLOOD LINE
- FLOODWAY
- PROPOSED SHARED USE PATH
- EXISTING SIDEWALK
- EXISTING SHARED USE PATH



APPENDIX 1B

SOIL BORING LOGS & LABORATORY TEST RESULTS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 10/22/24 19:11 - C:\USERS\KARENSID\DESKTOP\GINT PROFILE LOGS\CEMETERY RD 81224.GPJ

PROJECT: CEMETERY RD		DRILLING FIRM / OPERATOR: NEAS INC. / RW		DRILL RIG: CME 45B		STATION / OFFSET:		EXPLORATION ID												
TYPE: ROADWAY		SAMPLING FIRM / LOGGER: NEAS INC. / NS		HAMMER: CME AUTOMATIC		ALIGNMENT:		B-003-0-23												
PID: SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 3/8/24		ELEVATION: 866.7 (MSL) EOB: 10.0 ft.		PAGE												
START: 8/2/24 END: 8/2/24		SAMPLING METHOD: SPT		ENERGY RATIO (%): 79		LAT / LONG: 40.033439, -83.120165		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
		866.7								GR	CS	FS	SI	CL	LL	PL	PI	WC		
6.0" TOPSOIL (DRILLERS DESCRIPTION)		866.2																		
MEDIUM DENSE, GRAY, GRAVEL WITH SAND, SILT, AND CLAY, DAMP			1		7															
			2		5	12	17	SS-1	-	-	-	-	-	-	-	-	-	11	A-2-6 (V)	
		863.7	3		4															
MEDIUM STIFF TO STIFF, DARK GRAY AND ORANGISH BROWN, CLAY, "AND" SILT, LITTLE SAND, LITTLE GRAVEL, MOIST			4		1	8	72	SS-2	1.00	11	6	10	36	37	43	22	21	26	A-7-6 (13)	
		861.2	5		5															
STIFF TO VERY STIFF, BROWN AND ORANGISH BROWN, CLAY, "AND" SILT, LITTLE SAND, TRACE GRAVEL, IRON STAINING, MOIST			6		3															
		858.7	7		3	9	22	SS-3	2.00	4	4	9	38	45	47	21	26	24	A-7-6 (16)	
		858.7	8		4															
HARD, BROWN, SANDY SILT, LITTLE CLAY, LITTLE GRAVEL, TRACE IRON STAINING, DAMP			9		7	20	56	SS-4	4.50	-	-	-	-	-	-	-	-	9	A-4a (V)	
		856.7	10		8															
			EOB																	
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																				
ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS																				

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 10/22/24 19:11 - C:\USERS\KARENSID\BORING\GINT PROFILE\LOGS\CEMETERY RD 81224.GPJ

PROJECT: CEMETERY RD		DRILLING FIRM / OPERATOR: NEAS INC. / RW		DRILL RIG: CME 45B		STATION / OFFSET:		EXPLORATION ID B-004-0-23												
TYPE: SUBGRADE		SAMPLING FIRM / LOGGER: NEAS INC. / NS		HAMMER: CME AUTOMATIC		ALIGNMENT:														
PID: SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 3/8/24		ELEVATION: 871.2 (MSL) EOB: 7.5 ft.		PAGE 1 OF 1												
START: 8/1/24 END: 8/1/24		SAMPLING METHOD: SPT		ENERGY RATIO (%): 79		LAT / LONG: 40.032821, -83.121755														
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS		SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
		871.2								GR	CS	FS	SI	CL	LL	PL	PI	WC		
6.0" TOPSOIL (DRILLERS DESCRIPTION)		870.7																		
SOFT TO MEDIUM STIFF, DARK BROWN BECOMING BROWN, CLAY, "AND" SILT, LITTLE TO SOME SAND, TRACE GRAVEL, MOIST TO DAMP			1																	
			2		2	8	39	SS-1	1.50	7	7	13	38	35	49	26	23	27	A-7-6 (15)	
			3		0	4														
			4		3	11	56	SS-2	0.50	9	8	13	36	34	42	22	20	22	A-7-6 (11)	
		866.7	5		2	5	21	50	SS-3	4.50	-	-	-	-	-	-	-	17	A-4a (V)	
HARD, BROWN BECOMING BROWN MOTTLED WITH GRAY, SANDY SILT, LITTLE CLAY, TRACE GRAVEL, MOIST TO DAMP			6		1	11														
			7		4	13	39	SS-4	4.50	-	-	-	-	-	-	-	-	13	A-4a (V)	
		863.7	EOB		6															
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																				
ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS																				

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 10/22/24 19:11 - C:\USERS\KARENSIDE\DESKTOP\GINT PROFILE LOGS\CEMETERY RD 81224.GPJ

PROJECT: CEMETERY RD		DRILLING FIRM / OPERATOR: NEAS INC. / RW		DRILL RIG: CME 45B		STATION / OFFSET:		EXPLORATION ID											
TYPE: ROADWAY		SAMPLING FIRM / LOGGER: NEAS INC. / NS		HAMMER: CME AUTOMATIC		ALIGNMENT:		B-005-0-23											
PID: SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 3/8/24		ELEVATION: 885.0 (MSL) EOB: 20.0 ft.		PAGE											
START: 8/3/24 END: 8/3/24		SAMPLING METHOD: SPT		ENERGY RATIO (%): 79		LAT / LONG: 40.033202, -83.122358		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
		885.0							GR	CS	FS	SI	CL	LL	PL	PI	WC		
6.0" TOPSOIL (DRILLERS DESCRIPTION)		884.5																	
VERY STIFF TO HARD, BROWN, SANDY SILT, LITTLE GRAVEL, LITTLE CLAY, DAMP			1	2															
			2	3	12	50	SS-1	2.50	20	17	21	30	12	19	14	5	10	A-4a (1)	
			3																
			4	3															
			5	5	26	44	SS-2	4.50	-	-	-	-	-	-	-	-	9	A-4a (V)	
			6																
			7	3															
			8	5	16	50	SS-3	4.00	-	-	-	-	-	-	-	-	11	A-4a (V)	
		877.0	9																
DENSE, BROWN AND GRAY, GRAVEL WITH SAND AND SILT, TRACE CLAY, DAMP			10	4															
			11	9	33	44	SS-4	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)	
		874.5	12																
			13	5															
STIFF TO HARD, BROWN AND GRAY, SILT AND CLAY, SOME SAND, LITTLE GRAVEL, TRACE IRON STAINING, DAMP			14	7	24	72	SS-5	4.50	14	12	15	35	24	28	17	11	14	A-6a (5)	
			15																
			16	4															
			17	8	28	67	SS-6	2.00	-	-	-	-	-	-	-	-	13	A-6a (V)	
		869.5	18																
			19	3															
VERY STIFF, BROWN MOTTLED WITH GRAY, SILTY CLAY, LITTLE SAND, LITTLE GRAVEL, DAMP			20	6	21	78	SS-7	3.50	11	8	12	36	33	36	17	19	17	A-6b (10)	
		867.0																	
VERY STIFF, BROWN, SANDY SILT, SOME CLAY, TRACE GRAVEL, DAMP				3															
				8	25	61	SS-8	4.00	-	-	-	-	-	-	-	-	13	A-4a (V)	
		865.0		11															
			EOB																
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE CAVED-IN AT 18.0'.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 10/22/24 19:11 - C:\USERS\KARENS\DESKTOP\GINT PROFILE LOGS\CEMETERY RD 81224.GPJ

PID:	SFN:	PROJECT:	CEMETERY RD	STATION / OFFSET:	START:	8/8/24	END:	8/9/24	PG 2 OF 3	B-006-0-23										
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS		SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL
										GR	CS	FS	SI	CL	LL	PL	PI	WC		
VERY STIFF, GRAY, SANDY SILT , LITTLE CLAY, LITTLE GRAVEL, DAMP (continued) @30.0'; ADDED WATER AS CIRCULATING FLUID		840.3		31	9 10 16	39	94	SS-12	3.50	-	-	-	-	-	-	-	-	11	A-4a (V)	
VERY DENSE, GRAY, GRAVEL WITH SAND AND SILT , LITTLE CLAY, MOIST @35.0'; ADDED WATER AS CIRCULATING FLUID		838.3		32																
				33	11 22 28	74	67	SS-13	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)	
				34																
				35	29 34 30	95	100	SS-14	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)	
				36																
		832.0		37																
				38																
VERY DENSE, GRAY, GRAVEL WITH SAND , LITTLE SILT, TRACE CLAY, MOIST				39																
				40	11 21 18	58	33	SS-15	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
				41																
		827.0		42																
				43																
VERY DENSE, GRAY, SANDY SILT , LITTLE GRAVEL, LITTLE CLAY, DAMP				44																
				45	10 15 22	55	100	SS-16	-	19	16	25	27	13	17	13	4	11	A-4a (1)	
				46																
		822.0		47																
				48																
VERY DENSE, GRAY, GRAVEL WITH SAND AND SILT , LITTLE CLAY, WET @50.0'; RIG CHATTER. POSSIBLE ENCOUNTER WITH BOULDER/COBBLE @50.0'; ADDED WATER AS CIRCULATING FLUID				49																
				50	16 21 32	79	44	SS-17	-	-	-	-	-	-	-	-	-	14	A-2-4 (V)	
				51																
		817.0		52																
				53																
VERY DENSE, GRAY, GRAVEL WITH SAND , LITTLE SILT, TRACE CLAY, MOIST				54																
				55	28 38 48	128	61	SS-18	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
				56																
		812.0		57																
				58																
				59																
VERY DENSE, GRAY, GRAVEL WITH SAND AND SILT , TRACE CLAY, DAMP				60	20 31 40	105	67	SS-19	-	27	20	21	22	10	18	14	4	10	A-2-4 (0)	
				61																

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 10/22/24 19:11 - C:\USERS\KARENS\DESKTOP\GINT PROFILE LOGS\CEMETERY RD 81224.GPJ

PID: _____	SFN: _____	PROJECT: _____	CEMETERY RD	STATION / OFFSET: _____				START: 8/7/24	END: 8/7/24	PG 2 OF 3				B-007-0-23						
MATERIAL DESCRIPTION AND NOTES		ELEV. 840.5	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			
DENSE, GRAY, COARSE AND FINE SAND , LITTLE TO SOME SILT, TRACE TO LITTLE GRAVEL, TRACE CLAY, WET (continued)		837.8	31	12 14 15	43	100	SS-12	-	-	-	-	-	-	-	-	-	19	A-3a (V)		
			32																	
			33	4 9 9	27	67	SS-13	-	38	29	21	9	3	NP	NP	NP	12	A-1-b (0)		
MEDIUM DENSE, GRAY, GRAVEL WITH SAND , TRACE SILT, TRACE CLAY, MOIST TO WET		832.2	34																	
			35	6 10 10	30	56	SS-14	-	-	-	-	-	-	-	-	15	A-1-b (V)			
			36																	
DENSE, GRAY, COARSE AND FINE SAND , TRACE TO LITTLE GRAVEL, TRACE TO LITTLE SILT, TRACE CLAY, WET @40.0'; ADDED WATER AS CIRCULATING FLUID		827.2	37																	
			38																	
			39																	
HARD, GRAY, SANDY SILT , SOME GRAVEL, LITTLE CLAY, DAMP		822.2	40	4 13 14	40	100	SS-15	-	-	-	-	-	-	-	-	-	19	A-3a (V)		
			41																	
			42																	
VERY DENSE, GRAY, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT , TRACE CLAY, MOIST		817.2	43																	
			44																	
			45	24 12 25	55	56	SS-16	4.50	33	16	14	26	11	19	14	5	11	A-4a (0)		
HARD, GRAY, SANDY SILT , SOME GRAVEL, LITTLE CLAY, DAMP @56.0'; RIG CHATTER. POSSIBLE ENCOUNTER WITH BOULDER/COBBLE		812.2	46																	
			47																	
			48																	
VERY DENSE, GRAY, GRAVEL WITH SAND , TRACE SILT, TRACE CLAY, MOIST @60.0'; ADDED WATER AS CIRCULATING FLUID @61.5': ENCOUNTERED 6" SAND HEAVE			49	29 50/3"	-	89	SS-17	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)		
			50																	
			51																	
			52																	
			53																	
			54																	
			55	4 34 50/5"	-	100	SS-18	4.50	-	-	-	-	-	-	-	-	8	A-4a (V)		
			56																	
			57																	
			58																	
			59																	
			60	15 25 28	79	100	SS-19	-	-	-	-	-	-	-	-	-	9	A-1-b (V)		
			61																	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 10/8/24 12:11 - X:\1\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\CEMETERY ROAD\GINT FILES\CEMETERY RD 81224.GPJ

PID: _____		SFN: _____		PROJECT: CEMETERY RD		STATION / OFFSET: _____				START: 9/19/24		END: 10/1/24		PG 3 OF 3		B-008-0-23							
MATERIAL DESCRIPTION AND NOTES				ELEV. 808.5	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					
LIMESTONE LIGHT TAN AND BROWN, SLIGHTLY WEATHERED, STRONG. (continued)				805.6	EOB	63																	
						64	50/2"	-	50	SS-20	-	-	-	-	-	-	-	-	-	-	7	Rock (V)	
						65																	
@65.0'; AUGER REFUSAL																							
NOTES: GROUNDWATER ENCOUNTERED AT 30.0' DURING DRILLING. HOLE CAVED-IN AT 13.0'. BORING OFFSET 4.0' WEST.																							
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 4 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																							

Consolidation Test

Project Name: Cemetery Rd
Source: B-008-0-23 ST-7 (16.8'-16.9')
Description: Hard, gray, SANDY SILT, some clay, trace gravel, damp.

Prepared by: LR
Checked by: ZM
Date: 10/7/2024

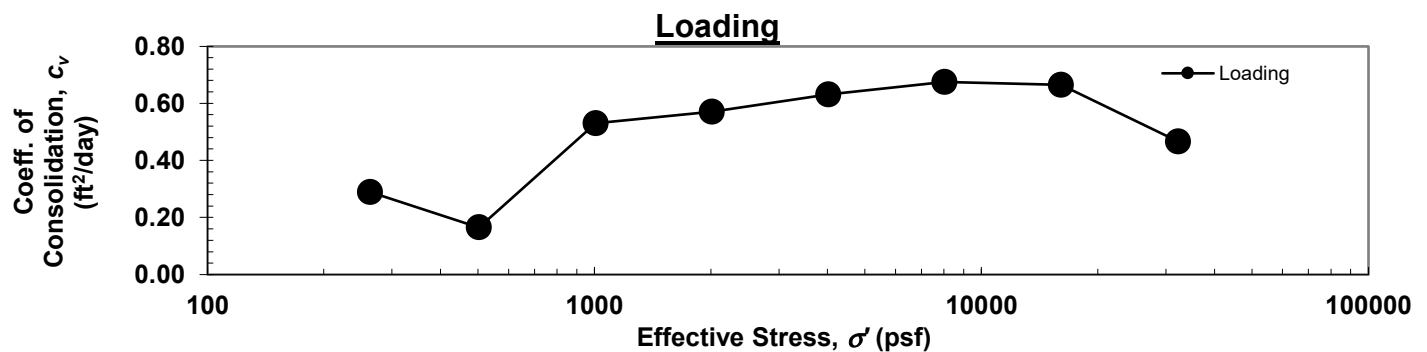
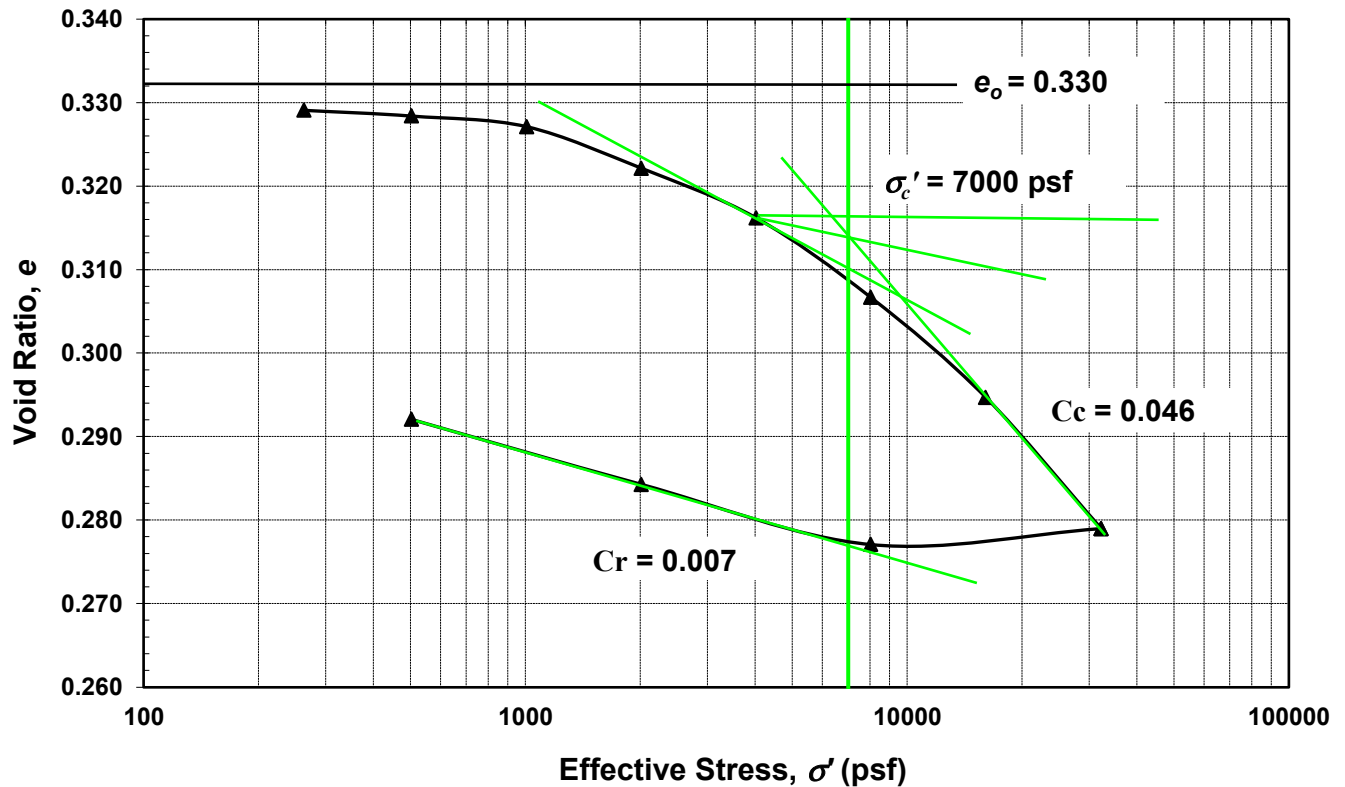
Test Specification: ASTM D 2435
Initial Void Ratio: 0.330
In-situ Vertical Effective Stress(psf): 2300

Initial Bulk Unit Weight (lb/ft³): 141
Dry Unit Weight (lb/ft³): 127

Compression and Swelling Index

Compression Index (C_c): 0.046
Recompression Index (C_r): 0.007
Preconsolidation Pressure (σ'_c)(psf): 7000
Over-Consolidation Ratio (OCR): 3.0

Consolidation Curve



STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 10/22/24 19:11 - C:\USERS\KARENS\DESKTOP\GINT PROFILE LOGS\CEMETERY RD 81224.GPJ

PID: _____	SFN: _____	PROJECT: _____	CEMETERY RD	STATION / OFFSET: _____						START: 8/6/24	END: 8/6/24		PG 2 OF 3		B-009-0-23							
MATERIAL DESCRIPTION AND NOTES			ELEV. 840.6	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					
STIFF TO HARD, GRAY, SANDY SILT , SOME CLAY, TRACE TO LITTLE GRAVEL, DAMP <i>(continued)</i>			840.6	W 830.6	31	6 7 10	25	100	SS-12	3.00	-	-	-	-	-	-	-	14	A-4a (V)			
					32																	
					33	4 6 8	21	100	SS-13	3.50	-	-	-	-	-	-	-	12	A-4a (V)			
					34																	
					35	5 6 7	19	100	SS-14	4.50	5	11	19	40	25	21	14	7	12	A-4a (6)		
					36																	
					37																	
					38																	
					39																	
					40	4 6 12	27	61	SS-15	2.00	-	-	-	-	-	-	-	-	9	A-4a (V)		
DENSE, GRAY, GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT , TRACE CLAY, MOIST			827.3		41																	
					42																	
					43																	
					44																	
					45	14 12 21	49	44	SS-16	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)		
					46																	
					47																	
					48																	
					49																	
					50	10 28 17	67	44	SS-17	-	43	23	15	14	5	NP	NP	NP	9	A-1-b (0)		
VERY DENSE, GRAY, GRAVEL WITH SAND , LITTLE SILT, TRACE CLAY, MOIST TO WET			822.3		51																	
					52																	
					53																	
					54																	
					55	10 20 24	65	61	SS-18	-	-	-	-	-	-	-	-	-	13	A-1-b (V)		
					56																	
					57																	
					58																	
					59																	
					60	16 24 28	77	61	SS-19	-	-	-	-	-	-	-	-	-	10	A-1-b (V)		
				61																		

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 10/22/24 19:11 - C:\USERS\KARENSID\BORING\PROFILE\LOGS\CEMETERY RD 81224.GPJ

PID: _____	SFN: _____	PROJECT: _____	CEMETERY RD	STATION / OFFSET: _____	START: 7/30/24	END: 7/31/24	PG 2 OF 3	B-010-0-23													
MATERIAL DESCRIPTION AND NOTES		ELEV. 843.6	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					
VERY STIFF TO HARD, GRAY, SANDY SILT , SOME CLAY, TRACE TO LITTLE GRAVEL, DAMP (continued)		833.1	31	5 7 10	25	89	SS-12	2.50	-	-	-	-	-	-	-	-	14	A-4a (V)			
			32																		
			33	5 7 10	25	89	SS-13	2.75	9	9	18	39	25	23	15	8	13	A-4a (6)			
			34																		
			35	5 7 13	30	100	SS-14	3.00	-	-	-	-	-	-	-	-	12	A-4a (V)			
			36																		
			37																		
			38																		
			39																		
			40	5 7 10	25	100	SS-15	-	-	-	-	-	-	-	-	-	10	A-2-4 (V)			
MEDIUM DENSE TO VERY DENSE, GRAY, GRAVEL WITH SAND AND SILT , TRACE CLAY, MOIST		833.1	41																		
			42																		
			43																		
			44																		
			45	3 12 48	89	100	SS-16	-	-	-	-	-	-	-	-	-	10	A-2-4 (V)			
			46																		
			47																		
			48																		
			49																		
			50	16 17 16	49	89	SS-17	-	38	21	14	19	8	NP	NP	NP	8	A-2-4 (0)			
@50.0'; RIG CHATTER. POSSIBLE ENCOUNTER WITH BOULDER/COBBLE		820.3	51																		
			52																		
			53																		
			54																		
			55	17 21 19	59	100	SS-18	-	37	21	22	14	6	NP	NP	NP	9	A-1-b (0)			
			56																		
			57																		
			58																		
			59																		
			60	3 10 13	34	78	SS-19	-	-	-	-	-	-	-	-	-	12	A-1-b (V)			
@60.0'; RIG CHATTER. POSSIBLE ENCOUNTER WITH BOULDER/COBBLE		820.3	61																		
@61.5': ENCOUNTERED 14" SAND HEAVE																					

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 10/22/24 19:11 - C:\USERS\KARENSID\DESKTOP\GINT PROFILE LOGS\CEMETER RD 81224.GPJ

PROJECT: CEMETERY RD		DRILLING FIRM / OPERATOR: NEAS INC. / RW		DRILL RIG: CME 45B		STATION / OFFSET:		EXPLORATION ID													
TYPE: SUBGRADE		SAMPLING FIRM / LOGGER: NEAS INC. / NS		HAMMER: CME AUTOMATIC		ALIGNMENT:		B-011-0-23													
PID: SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 3/8/24		ELEVATION: 871.6 (MSL) EOB: 7.5 ft.		PAGE													
START: 8/1/24 END: 8/1/24		SAMPLING METHOD: SPT		ENERGY RATIO (%): 79		LAT / LONG: 40.031933, -83.124267		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
			871.6								GR	CS	FS	SI	CL	LL	PL	PI	WC		
6" TOPSOIL (DRILLERS DESCRIPTION)			871.1																		
HARD, BROWN, SILT AND CLAY, SOME SAND, TRACE GRAVEL, DAMP				1																	
				2		7															
			868.6	3		10	32	50	SS-1	4.50	10	8	13	36	33	33	18	15	11	A-6a (9)	
				4		14															
HARD, BROWN, SANDY SILT, LITTLE CLAY, LITTLE GRAVEL, SS-3 CONTAINS TRACE IRON STAINING, DAMP				5		2	7	25	28	SS-2	4.50	11	14	20	38	17	20	15	5	10	A-4a (4)
				6		12															
			865.6	7		7															
						10	30	33	SS-3	4.50	-	-	-	-	-	-	-	-	10	A-4a (V)	
						13															
MEDIUM DENSE, BROWN, GRAVEL WITH SAND AND SILT, TRACE CLAY, WET			864.1	7		2	6	17	39	SS-4	-	-	-	-	-	-	-	-	12	A-2-4 (V)	
				EOB		7															
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.																					
ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS																					

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT GDT - 10/22/24 19:12 - C:\USERS\KAREN\DESKTOP\GINT PROFILE LOGS\CEMETERY RD 81224.GPJ

PROJECT: CEMETERY RD		DRILLING FIRM / OPERATOR: NEAS INC. / NS		DRILL RIG: CME 45B		STATION / OFFSET:		EXPLORATION ID											
TYPE: ROADWAY		SAMPLING FIRM / LOGGER: NEAS INC. / NS		HAMMER: CME AUTOMATIC		ALIGNMENT:		B-014-0-23											
PID: SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 3/8/24		ELEVATION: 873.6 (MSL) EOB: 20.0 ft.		PAGE											
START: 8/6/24 END: 8/6/24		SAMPLING METHOD: SPT		ENERGY RATIO (%): 79		LAT / LONG: 40.032769, -83.124919		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
		873.6							GR	CS	FS	SI	CL	LL	PL	PI	WC		
HARD, BROWN MOTTLED WITH GRAY AND ORANGISH BROWN, CLAY , "AND" SILT, LITTLE SAND, TRACE GRAVEL, IRON STAINING, DAMP			1	3															
			2	6	17	39	SS-1	4.50	7	5	10	40	38	51	27	24	16	A-7-6 (16)	
			3																
			4	2															
			5	4	13	39	SS-2	4.50	3	3	9	39	46	50	21	29	17	A-7-6 (18)	
		868.1	6																
HARD, BROWN MOTTLED WITH GRAY, SANDY SILT , SOME GRAVEL, LITTLE CLAY, DAMP			7	4	17	33	SS-3	4.25	29	13	13	28	17	27	17	10	12	A-4a (2)	
			8																
		865.6	9	2															
VERY STIFF TO HARD, GRAY, SANDY SILT , SOME CLAY, TRACE TO LITTLE GRAVEL, DAMP			10	4	16	61	SS-4	4.50	-	-	-	-	-	-	-	-	12	A-4a (V)	
			11																
			12	9	38	72	SS-5	3.00	-	-	-	-	-	-	-	-	12	A-4a (V)	
			13																
			14	2															
			15	7	20	78	SS-6	4.50	-	-	-	-	-	-	-	-	10	A-4a (V)	
		W 858.6	16																
			17	7	24	72	SS-7	4.50	18	9	15	35	23	25	15	10	11	A-4a (5)	
			18																
			19	4	18	78	SS-8	3.50	-	-	-	-	-	-	-	-	13	A-4a (V)	
		853.6	20	6	8														
			EOB																
NOTES: GROUNDWATER ENCOUNTERED AT 15.0' DURING DRILLING. HOLE DID NOT CAVE.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS																			

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 10/22/24 19:12 - C:\USERS\KARENSID\DESKTOP\GINT PROFILE LOGS\CEMETERY RD 81224.GPJ

PROJECT: CEMETERY RD		DRILLING FIRM / OPERATOR: NEAS INC. / RW		DRILL RIG: CME 45B		STATION / OFFSET:		EXPLORATION ID														
TYPE: SUBGRADE		SAMPLING FIRM / LOGGER: NEAS INC. / RW		HAMMER: CME AUTOMATIC		ALIGNMENT:		B-015-0-23														
PID: SFN:		DRILLING METHOD: 3.25" HSA		CALIBRATION DATE: 3/8/24		ELEVATION: 876.4 (MSL) EOB: 7.5 ft.		PAGE														
START: 8/6/24 END: 8/6/24		SAMPLING METHOD: SPT		ENERGY RATIO (%): 79		LAT / LONG: 40.032654, -83.126175		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	
			876.4								GR	CS	FS	SI	CL	LL	PL	PI	WC			
STIFF TO HARD, BROWN AND GRAY, CLAY , "AND" SILT, LITTLE SAND, TRACE GRAVEL, DAMP TO MOIST				1																		
				2	3	5	14	72	SS-1	4.50	7	3	9	45	36	47	25	22	18	A-7-6 (14)		
				3	2	5	13	67	SS-2	2.25	1	2	9	38	50	54	22	32	26	A-7-6 (19)		
				4	5																	
				5	4	3	9	61	SS-3	1.50	-	-	-	-	-	-	-	-	-	22	A-7-6 (V)	
VERY STIFF, BROWN AND GRAY, SILTY CLAY , LITTLE SAND, TRACE GRAVEL, IRON STAINING, DAMP			870.4	6	3	4																
			868.9	7	5	8	17	89	SS-4	3.00	-	-	-	-	-	-	-	-	13	A-6b (V)		
				EOB																		
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE CAVED-IN AT 3.0'.																						
ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS																						

APPENDIX 1C

SULFATE TESTING RESULTS



OHIO DEPARTMENT OF TRANSPORTATION

DETERMINING SULFATE CONTENT IN SOILS

SUPPLEMENT 1122

Project C-R-S: Cemetery Rd
 PID No:
 Report Date: 9/27/2024
 Consultant: NEAS Inc.
 Technician: P. Johnson

Boring ID & Sample #	Station	Offset	Latitude & Longitude or State Plane Coordinates	Elevation	Soaking Time (hr)	Replicate Sample Readings						Sulfate Content (ppm)
						1		2		3		
						Dilution	Reading	Dilution	Reading	Dilution	Reading	
B-001-0-23 SS-2					18.7	20	0	20	0	20	1	7
B-003-0-23 SS-2					18.7	20	32	20	34	20	35	673
B-004-0-23 SS-1					19.5	20	0	20	1	20	1	13
B-005-0-23 SS-1					19.5	20	1	20	1	20	1	20
B-011-0-23 SS-1					18.9667	20	10	20	9	20	11	200
B-012-0-23 SS-2					18.9833	20	1	20	1	20	1	20
B-013-0-23 SS-1					19	20	1	20	1	20	1	20
B-014-0-23 SS-1					19.15	20	1	20	1	20	1	20
B-015-0-23 SS-1					19.0333	20	1	20	0	20	1	13
B-016-0-23 SS-1					19.0167	20	1	20	1	20	1	20

SECTION 2

PROJECT ROADWAY AND EMBANKMENTS

SECTION 2: PROJECT ROADWAY AND EMBANKMENTS

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2.1 PROJECT ROADWAY AND EMBANKMENT INTRODUCTION

2.1.1. General

We understand the proposed project is intended to provide a SUP on the south side of Cemetery Rd in the city of Hilliard, Ohio to allow for pedestrian access between Britton Pkwy and Trueman Blvd. For this purpose, a subgrade exploration and subsequent analysis was completed for the construction of approximately 3,090 lineal feet of SUP as well as the realignment and reconstruction of the existing Cemetery Rd EB to IR-270 SB entrance ramp. The analysis completed for this portion of the proposed project included a subgrade analysis and embankment stability analyses (i.e., global stability and settlement). The subgrade analysis performed in accordance with ODOT's GDM criteria utilizing the ODOT provided *Roadway Subgrade Analysis Spreadsheet* (Subgrade_Analysis_2024-04-04.xlsm, Version 14.7 dated April 4, 2024). Input information for the spreadsheet was based on the soil characteristics gathered during NEAS's subgrade exploration (i.e., SPT results, laboratory test results, etc.). A subgrade analysis was performed for the entire project as well as for each of the referenced segments individually.

Based on our evaluation of the subsurface conditions and our geotechnical engineering analyses of the proposed construction, it is our opinion that subgrade conditions are generally satisfactory, and pavement can be designed without the need for extreme levels of remediation. In general, the subgrade soils throughout the project can be stabilized by being reworked with the use of standard compaction (subgrade preparation) practices. However, it is our opinion that the project should include contingency quantities for subgrade improvement as some of the existing and proposed pavement "tie-in" points might encounter material that requires stabilization via excavate and replace with geotextile. The following sections provide further details about the analysis performed and the recommendations provided.

2.2 FINDINGS

The pavement and subsurface conditions encountered during NEAS's explorations are described in the following subsections and/or on each boring log presented in Appendix 1B. 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. It should be noted that for the purposes of this report and our analysis the term 'subgrade' has been assumed to represent soils and/or soil conditions from 1.5 ft below proposed final pavement grades to a depth of 7.5 ft below the proposed pavement grades.

2.2.1. Existing Pavement

The pavement section thicknesses in terms of asphalt and concrete were measured at project borings performed within existing pavement during the subsurface exploration for the project and are recorded on the test boring logs provided in Appendix 1B. A summary of these measurements is provided in Table 1 below.

Table 1: Measured Pavement Thickness at Boring Locations

Boring ID	Existing Alignment	Proposed Alignment	Drilled Depth (ft)	Asphalt Thickness (in)	Concrete Thickness (in)
B-001-0-23	Cemetery Rd	SUP	35.0	14.0	-
B-002-0-23	Cemetery Rd	SUP	35	14.0	-

2.2.2. Subgrade Conditions

The subgrade conditions within the project limits are relatively consistent and are generally comprised of either fill soils (i.e., embankment fill) or natural soils generally consisting fine-grained soils comprised of sand, silt, clay combinations. The subgrade soils encountered within the project limits are generally classified as either A-4a, A-4b, A-6a, A-6b or A-7-6 type soils. With respect to sulfate within the subgrade soil, based on the project laboratory testing program, each subgrade soil sample tested was determined to have a sulfate content of less than 5,000 parts per million (ppm) (i.e., the level which ODOT considers high and may prevent the use of chemical stabilization).

The following subsections present a summary of the subsurface conditions by path/roadway segment with problem areas highlighted where present.

2.2.2.1. Shared Use Path

The proposed SUP will extend between Britton Pkwy and Trueman Blvd. The borings performed along the proposed SUP alignment included borings B-001-0-23 through B-003-0-23, B-005-0-23, borings B-012-0-23 through B-014-0-23, and B-016-0-23.

Along the SUP, eighty-two percent (82%) of the soil samples were identified as fine-grained soils and were comprised of: 1) cohesive Sandy Silt (A-4a, 35% of samples); 2) Silty Clay (A-6b, 16% of samples); 3) Clay (A-7-6, 16% of samples); and, 4) Silt and Clay (A-6a, 15% of samples). With respect to the consistency of the fine-grained soils, the descriptions varied from soft to hard correlating to converted SPT-N values (N_{60}) between 4 and 65 blows per foot (bpf). Natural moisture contents ranged from 9 to 26 percent. Based on Atterberg Limit tests performed on representative samples of the fine-grained subgrade soils obtained along the SUP alignment, the liquid and plastic limits ranged from 19 to 51 percent and from 13 to 27 percent, respectively.

Eighteen percent (18%) of the samples taken along the proposed SUP were classified as coarse-grained, non-cohesive soils and were comprised of: 1) Gravel (A-1-a, 6% of samples); 2) Sandy Silt (A-4a, 1 sample); 3) Gravel and Stone Fragments with Sand and Silt (A-2-4, 1 sample); 4) Coarse and Fine Sand (A-3a, 1 sample); and 5) Gravel with Sand, Silt, and Clay (A-2-6, 1 sample). With respect to the relative density of the coarse-grained soils, the descriptions varied from medium dense to dense correlating to N_{60} values between 13 and 33 bpf. Natural moisture contents ranged from 4 to 14 percent.

2.2.2.2. Cemetery Road EB to IR-270 SB Ramp

The Cemetery Rd EB to IR-270 SB Ramp is planned to undergo full reconstruction. The borings performed along this portion of roadway included borings B-011-0-23 and B-015-0-23.

Along the Cemetery Rd EB to IR-270 SB Ramp, eighty-eight percent (88%) of the soil samples were identified as fine-grained soils and were comprised of: 1) Clay (A-7-6, 38% of samples); 2) cohesive Sandy Silt (A-4a, 25% of samples); 3) Silt and Clay (A-6a, 1 samples); and, Silty Clay (A-6b, 1 sample). With respect to the consistency of the fine-grained soils, the descriptions varied from stiff to hard correlating to N_{60} values between 9 and 32 bpf. Natural moisture contents ranged from 10 to 26 percent. Based on

Atterberg Limit tests performed on representative samples of the fine-grained subgrade soils obtained along the ramp, the liquid and plastic limits ranged from 20 to 54 percent and from 15 to 25 percent, respectively.

Twelve percent (12%) of the samples taken along the ramp were classified as coarse-grained, non-cohesive soils and were comprised of Gravel and/or Stone Fragments with Sand and Silt (A-2-4). With respect to the relative density of the coarse-grained soils, they can be described as medium dense correlating to a N_{60} value of 17 bpf. Natural moisture content was 12 percent.

2.2.2.3. IR-270 NB to Cemetery Rd Ramp

The IR-270 NB to Cemetery Rd Ramp is planned to undergo partial reconstruction. The boring performed along this portion of roadway included boring B-004-0-23.

Along the IR-270 NB to Cemetery Rd Ramp, one hundred percent (100%) of the soil samples were identified as fine-grained soils and were comprised of Clay (A-7-6, 50% of samples) and cohesive Sandy Silt (A-4a, 50% of samples). With respect to the consistency of the fine-grained soils, the descriptions varied from medium stiff to stiff correlating to N_{60} values between 8 and 21 bpf. Natural moisture contents ranged from 13 to 27 percent. Based on Atterberg Limit tests performed on representative samples of the fine-grained subgrade soils obtained along the ramp, the liquid and plastic limits ranged from 42 to 49 percent and from 22 to 26 percent, respectively.

2.3 ANALYSES AND RECOMMENDATIONS

2.3.1. Subgrade Analysis and Recommendations

A subgrade analysis was 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 analysis including pavement design parameters and unsuitable subgrade conditions identified within the project limits. Subgrade analysis spreadsheets are provided in Appendix 2A.

Again, 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 from 1.5 ft below proposed final pavement grades to a depth of 7.5 ft below the proposed pavement grades.

2.3.1.1. Pavement Design Recommendations

It is our understanding that pavement analysis and design is to be performed to determine the proposed pavement sections for the new SUP as well as roadway segments within the project limits to undergo full depth replacement. A subgrade analysis was performed per ODOT's GDM using the subgrade soil data obtained during our field exploration program to evaluate the soil characteristics and develop pavement parameters for use in pavement design. The subgrade parameters recommended for use in pavement design are presented in Table 2 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 2: Pavement Design Values

Segment	Maximum N _{60L}	Minimum N _{60L}	Average N _{60L}	Average PI Values	Design CBR
Shared Use Path	21	4	12	17	6
Cemetery Road EB to IR-270 SB Ramp	9	9	9	19	4
IR-270 NB to Cemetery Rd Ramp	8	8	8	22	4

2.3.1.2. Unsuitable Subgrade

Per ODOT's GDM, the presence of select subgrade conditions (i.e., unsuitable) are prohibited within the subgrade zone for new pavement construction. These unsuitable subgrade conditions generally include the presence of rock and specific soil types which include A-4b, A-2-5, A-5, A-7-5, A-8a, A-8b, and soils with liquid limits greater than 65. With respect to the proposed reconstruction/rehabilitation project these unsuitable subgrade conditions were not encountered within the subgrade of the referenced project path/roadway segments.

2.3.1.3. Unstable Soils

The ODOT GDM recommends subgrade stabilization for soils considered unstable in which the N₆₀ 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 N₆₀ value at the subject boring, *Figure 600-1: Subgrade Stabilization* within the ODOT GDM recommends a depth of subgrade stabilization for ODOT standard stabilization methods. Additionally, although a soil sample's N₆₀ value may meet the criteria to be considered an unstable soil, the depth in which the unstable soil is encountered in relation to the proposed subgrade is considered when each individual subgrade boring is analyzed. For example, if the ODOT 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.

Based on N₆₀ values encountered within the project borings, our subgrade analysis suggests the need for 12 to 18 inches of either excavate and replace or chemical stabilization at one location. A summary of the boring location where unstable soils were encountered and determined to have a potential impact on subgrade performance are shown in Table 3 below. Also included is the associated ODOT GDM recommended remediation depth at that location.

Table 3: Unstable Soil Locations Summary

Boring ID	Average HP (tsf)	N _{60L} (bpf)	Moisture Above Optimum (%)	Depth Below Subgrade (ft)	Remediation Depth (inches) Excavate and Replace (Item 204 w/ Geotextile)
Shared Use Path					
B-016-0-23	4.00	9	2	0.0 - 1.0	12

It should be noted that Figure B - Subgrade Stabilization does not apply to soil types A-1-a, A-1-b, A-3, or A-3a, nor to soils with N_{60L} values of 15 or more. Per ODOT GDM guidance, these soils should be reworked to stabilize the subgrade.

2.3.1.3.1. High Moisture Content Soils

High moisture content soils are defined by the ODOT GDM as soils that exceed the estimated optimum moisture content (per Table 600-1: Optimum Moisture Content within the ODOT GDM) for a given

classification by more than 3 percent. Per the ODOT 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 unstable 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. Summaries of the boring locations where high moisture content conditions were encountered in the top 3 ft of subgrade within the limits of each proposed roadway segment are shown in Table 4 below.

Table 4: High Moisture Content Soils Location Summary

Boring ID	Moisture Content (%)	Optimum Moisture Content (%)	Depth Below Subgrade (ft)
Shared Use Path			
B-015-0-23	26	19	1.5 - 3.0
B-013-0-23	21	16	0.9 - 2.4

2.3.1.4. Subgrade Stabilization Recommendations

Based on the results of our analysis, subgrade soils designated by ODOT's GB1 as "unstable" were present at one location within the limits of the project. Therefore, due to the results of our analysis and because a majority of the project will have new fill placed atop the existing subgrade, it is recommended that the existing subgrade either be: 1) "reworked" and prepared in accordance with typical Subgrade Compaction and Proof Rolling (Item 204) procedures and specifications; or, 2) prepared in accordance with typical Roadway Excavation and Embankment (Item 203) procedures and specifications. However, as some of the existing "tie-in" points where new fill is minimal might encounter material that requires stabilization, we recommend that project plans include contingency quantities for subgrade improvement. Actual depths and limits of subgrade stabilization should be determined in the field by the Project Engineer based on ODOT's Subgrade Compaction and Proof Rolling specifications (Item 204).

2.3.2. Embankment Stability Analysis and Recommendations

2.3.2.1. Embankment Stability Analysis

For purposes of evaluating stability of the planned roadway embankments proposed as part of the project, NEAS reviewed cross-sections along the length of the project roadway segments to identify sections that were interpreted to represent conditions that posed the greatest potential for slope instability as a result of the planned construction. In general, cross-sections along each of the proposed roadway alignments were reviewed to identify planned alterations that may present a combination of existing subsurface conditions and planned site grading (i.e., cutting and/or filling) that would potentially be critical to the stability of the existing and/or proposed slopes at the site. Based on our review of the available information along the referenced alignments and the associated soil properties, two cross-sections were estimated to be most "critical" along the proposed roadways and were analyzed for global stability. The cross-sections selected to be evaluated include cross-sections along Ramp WS at approximate STA. 108+50 and STA. 111+00.

For each 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 project's Stage 1 plans set dated October 23, 2024 prepared by B&N; 2) a live load surcharge of 250 pounds per square foot (psf), accounting for traffic induced loads; and, 3) test borings and laboratory data developed as part of this report.

Roadway Exploration Report
FRA-Cemetery Road SUP
Franklin County, Ohio
PID: 120103

For analysis purposes, borings performed along or adjacent to the indicated proposed embankment sections were reviewed and a generalized material profile was developed for analysis to represent worse case conditions at the selected cross-section locations. Utilizing the generalized soil profile, engineering properties for each soil strata were estimated based on the 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 properties for use in analysis (with sited correlation/reference material) at each of the referenced cross-section locations are summarized within Tables 6 and 7 below.

Table 5: Soil Profile and Estimated Engineering Properties – Ramp WS – STA. 108+50

Embankment Stability Analysis: STA. 108+50 (Ramp WS), B-015-0-23 & B-013-0-23				
Soil Description	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Clay Depth (876.4 ft - 870.4 ft)	110	1500	150	22
Silty Clay Depth (870.4 ft - 866 ft)	120	2050	200	24
Sandy Silt Depth (866 ft - 863 ft)	120	-	-	31
Sandy Silt Depth (863 ft - 850.5 ft)	122	2200	210	25
Sandy Silt Depth (850.5 ft - 824.8 ft)	132	5250	370	28
Gravel with Sand and Silt Depth (824.8 ft - 821.5 ft)	135	-	-	41
Notes: 1. Values calculated per ODOT GDM Section 404/1304 and/or ODOT BDM Table 305-2.				

Table 6: Soil Profile and Estimated Engineering Properties – Ramp WS – STA. 111+00

Embankment Stability Analysis: STA. 111+00 (Ramp WS), B-013-0-23				
Soil Description	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Silty Clay Depth (871.5 ft - 866 ft)	112	2350	220	25
Sandy Silt Depth (866 ft - 863 ft)	120	-	-	32
Sandy Silt Depth (863 ft - 850.5 ft)	122	2200	210	25
Sandy Silt Depth (850.5 ft - 824.8 ft)	132	5250	370	28
Gravel with Sand and Silt Depth (824.8 ft - 821.5 ft)	135	-	-	41
Notes: 1. Values calculated per ODOT GDM Section 404/1304 and/or ODOT BDM Table 305-2.				

The above referenced slope stability model was analyzed for long-term (Effective Stress) and short-term (Total Stress) slope stability utilizing the software entitled *Slide2* by Rocscience, Inc. Specifically, the Spencer analysis methods were used to calculate a factor of safety (FOS) for circular and non-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 1.33 which equates to an AASHTO resistance factor less than 0.75 (per AASHTO's LRFD BDS the specified resistance factors are essentially the inverse of the FOS that should be targeted in slope stability programs). For this analysis, a resistance factor of 0.75 or lower is targeted as the slope analyzed does not contain or support a structural element.

Based on our slope stability analyses for the above referenced roadway embankment sections, the minimum slope stability safety factor is about 1.83 (0.55 resistance factor) for the sections analyzed. The graphical outputs of the slope stability program (cross-sectional model, calculated safety factor, and critical failure plane) for the analyzed sections are included within Appendix 2B.

2.3.2.2. Settlement Analysis

For purposes of evaluating potential for settlement of the planned roadway embankments proposed as part of the project, NEAS reviewed the project's Stage 1 Plans dated October 23, 2024 prepared by B&N to identify the embankment grading and soil profile characteristics that represent conditions that could induce the greatest potential for settlement as a result of the planned construction (e.g., presence of compressible soils, large embankment fill depths, etc.). Based on our review of the available information along the project roadway and the associated soil properties, a generalized soil profile was generated along the Shared Use Path and Ramp WS alignments where relatively significant amounts of new fill is planned. Settlement parameters were estimated and assigned to each soil layer supporting the analyzed roadway segments based on average field and laboratory testing results from borings performed in the area. The generalized soil profile and estimated settlement parameters per identified soil layer underlying the evaluated roadway segments is provided in Table 8 below. Utilizing this information and the software entitled *Settle3* by Rocscience, a settlement model was developed and analyzed for both elastic (short-term) and primary consolidation (long-term) settlement. *Settle3* outputs of our settlement analyses are included within Appendix 2C.

Table 7: Soil Profile and Settlement Parameters for Analysis – Shared Use Path and Ramp WS

Shared Use Path and Ramp WS: Settlement Analysis B-012-0-23 & B-013-0-23								
Soil Description	Unit Weight (pcf)	Elastic Modulus ⁽¹⁾ (psf)	Poissons Ratio ⁽¹⁾ , ν	Void Ratio e_o	Compression Index ⁽²⁾ , C_c	Recompression Index ⁽³⁾ , C_r	OCR ⁽⁴⁾	Coeff. of Consol. ⁽⁵⁾ , C_v (ft ² /day)
Cohesive Soil 1	120	1032000	0.45	0.621	0.22	0.045	48	0.5
Granular Soil 1	118	261000	0.30	-	-	-	-	-
Cohesive Soil 2 ⁽⁶⁾	135	1866000	0.45	0.33	0.05	0.007	3.0	0.6
Granular Soil 2	140	416000	0.40	-	-	-	-	-
NS: 1. Values interpreted from 2017 AASHTO LRFD BDS Table C10.4.6.3-1 2. Values calculated from Kulhawy and Mayne, 1990, Equation 6-6. 3. Values calculated from Kulhawy and Mayne, 1990, Equation 6-9. 4. Values interpreted from Mayne and Kemper, 1988, Figure 7. 5. Values interpreted from FHWA GEC No. 5, Boeckmann, et al., 2016, Figure 6-37. 6. Based on laboratory testing results of sample ST-7 in boring B-008-0-23.								

Based on our analyses, the estimated maximum total settlement associated with the loads induced by the proposed embankment is about 3.9 inches. The settlement will begin as the embankment load is applied and will dissipate with time. However, the amount of settlement and the time required for the settlement to occur is mostly dependent on the thickness of the underlying compressible soil, the uniformity and properties of these layers (i.e., compaction, material type, compressibility, etc.), and the proposed embankment fill height/surcharge load. Of the total settlement, about 1.4 inches is expected to be elastic (immediate) and take place during construction. The remaining 2.5 inches of settlement is anticipated to be long-term with the majority (i.e., 90 percent) of long-term settlement anticipated to take place in the first 45 days following construction. Due to the potential maximum settlement and the expected variability, NEAS concludes that the at-grade structures (i.e., guardrails, signage, etc.), existing utilities (i.e., known gas, water and electrical lines) and pavements constructed in the referenced area may be subjected to potentially damaging settlement. However, provided that the project schedule can tolerate a delay between substantial completion of the site earthwork and the commencement of the at-grade structure/pavement construction, postponing construction and allowing the potentially damaging settlements to take place should be considered to prevent additional costs associated with repairs and maintenance.

2.3.2.3. Embankment Construction Recommendations

In areas where additional embankment material is proposed along existing slopes (i.e., side-hill sliver fills) that are steeper than 8H:1V but flatter than 4H:1V, it is recommended that the proposed embankment be benched into the existing slopes in accordance with Item 203.05 “Embankment Construction Methods” of the ODOT CMS.

For areas where additional embankment material is proposed along existing slopes that are steeper than 4H:1V, it is recommended that the proposed embankment be designed and constructed in accordance with ODOT’s GDM. For sidehill fills planned on existing slopes steeper than 4H:1V, ODOT’s GDM recommends that *the embankment slopes be constructed utilizing special benching special benching (as detailed in the ODOT Construction Inspection Manual of Procedures, Section 203.05, “Benching”) to assure the embankment is “knitted” together* (ODOT, 2023). A special benching scheme similar to that shown in Figures 800-1, 800-3 or 800-4, as appropriate, of the ODOT GDM should be used in areas where special benching is recommended. The height and width dimensions of the special benching scheme shown in these figures should be arranged to minimize the required cut and fill quantities, though the height of a single bench shall not exceed 20 ft without a stability analysis and design per OSHA requirements. Additionally, it may be appropriate to adjust the bench slope shown from a 1H:1V to a 1.75H:1V slope if the existing slope is made up of primarily granular materials. The benched material should be replaced with compacted engineered fill per Item 203 of the ODOT CMS, while proper lift thicknesses and material density should be maintained in the proposed fill per Item 203.06 of the ODOT CMS. In situations where it is not practical to extend the final bench through the existing roadway due to maintenance of traffic concerns, a benching scheme similar to that shown in Figure 800-2 of the ODOT GDM can be used in order to avoid impacting the existing roadway, guardrail or shoulder. This scheme results in the placement of a temporary over-steepened fill that can later be “shaved-off” to bring the slope to the final proposed grade.

REFERENCES

ODOT. (2023). *Geotechnical Design Manual*. Columbus, Ohio: Ohio Department of Transportation: Office of Geotechnical Engineering.

APPENDIX 2A

SUBGRADE ANALYSIS SPREADSHEETS

OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES

Geotechnical Design Manual Section 600

Instructions: Enter data in the shaded cells only.

(Enter state route number, project description, county, consultant's name, prepared by name, and date prepared. This information will be transferred to all other sheets. The date prepared must be entered in the appropriate cell on this sheet to remove these instructions prior to printing.)

**FRA-CemeteryRd SUP
120103**

SUP

Provide a SUP on the south side of Cemetery Road (Rd) in the city of Hilliard, Ohio

NEAS Inc.

Prepared By: Brendan Andrews, PE
Date prepared: Wednesday, September 17, 2025

**Brendan Andrews
1329 East Kemper Road, Suite 4104B
Cincinnati, OH 45246**

**513.337.9823 Ext. 701
brendan.andrews@neasinc.com**

NO. OF BORINGS: **6**

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	B-016-0-23	SUP	7+98.9	1	Right	CME 45B	79	883.9	882.4	1.5 C
2	B-015-0-23	SUP	13+92.3	29	Left	CME 45B	79	876.4	874.9	1.5 C
3	B-013-0-23	SUP	17+85.3	4	Left	CME 55TB	89	871.5	871.4	0.1 C
4	B-012-0-23	SUP	18+64.2	7	Right	CME 55TB	89	872.4	873.1	0.8 F
5	B-014-0-23	SUP	24+50.8	2	Right	CME 45B	79	873.6	887.5	14.0 F
6	B-005-0-23	SUP	31+96.6	3	Left	CME 45B	79	885.0	883.5	1.5 C

#	Boring	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 016-0 23	SS-1	1.0	2.5	-0.5	1.0	9	4	4	41	22	19	41	39	80	21	19	A-7-6	12	20		N ₆₀		12"	
		SS-2	3.5	5.0	2.0	3.5	40		4.25	23	15	8	33	18	51	11	10	A-4a	3						
		SS-3	6.0	7.5	4.5	6.0	4		4							12	14	A-6a	10						
		SS-4	8.5	10.0	7.0	8.5	30		4.5							12	10	A-4a							
2	B 015-0 23	SS-1	1.5	3.0	0.0	1.5	14	9	4.5	47	25	22	45	36	81	18	22	A-7-6	14	200					
		SS-2	3.0	4.5	1.5	3.0	13		2.25	54	22	32	38	50	88	26	19	A-7-6	19		N ₆₀ & M _C				
		SS-3	4.5	6.0	3.0	4.5	9		1.5							22	18	A-7-6	16						
		SS-4	6.0	7.5	4.5	6.0	17		3							13	16	A-6b	16						
3	B 013-0 23	SS-1	1.0	2.5	0.9	2.4	16	13	4	40	21	19	38	45	83	19	16	A-6b	12	20		Mc			
		SS-2	3.5	5.0	3.4	4.9	22		2							21	16	A-6b	16						
		SS-3	6.0	7.5	5.9	7.4	13			NP	NP	NP	34	13	47	11	11	A-4a							
		SS-4	8.5	10.0	8.4	9.9	36		4.5							10	10	A-4a							
4	B 012-0 23	SS-1	2.5	4.0	3.3	4.8	21	21	4.5	34	20	14	46	30	76	14	15	A-6a	10	20					
		SS-2	5.0	6.5	5.8	7.3	22									14	8	A-3a							
		SS-3	7.5	9.0	8.3	9.8	7		3	21	16	5	37	15	52	13	11	A-4a							
		SS-4																							
5	B 014-0 23	SS-1	1.0	2.5	15.0	16.5	17		4.5	51	27	24	40	38	78	16	24	A-7-6		20					
		SS-2	3.5	5.0	17.5	19.0	13		4.5	50	21	29	39	46	85	17	18	A-7-6							
		SS-3	6.0	7.5	20.0	21.5	17		4.25	27	17	10	28	17	45	12	12	A-4a							
		SS-4	8.5	10.0	22.5	24.0	16		4.5							12	10	A-4a							
6	B 005-0 23	SS-1	1.0	2.5	-0.5	1.0	12	12	2.5	19	14	5	30	12	42	10	10	A-4a	1	20					
		SS-2	3.5	5.0	2.0	3.5	26		4.5							9	10	A-4a	8						
		SS-3	6.0	7.5	4.5	6.0	16		4							11	10	A-4a	8						
		SS-4	8.5	10.0	7.0	8.5	33								8	10	A-2-4								

PID: 120103

County-Route-Section: FRA-CemeteryRd SUP

No. of Borings: 6

Geotechnical Consultant: NEAS Inc.

Prepared By: Brendan Andrews, PE

Date prepared: 9/17/2025

Chemical Stabilization Options		
320	Rubblize & Roll	No
206	Cement Stabilization	Option
	Lime Stabilization	Option
206	Depth	12"

Excavate and Replace Stabilization Options	
Global Geotextile Average(N60L): Average(HP):	12" 0"
Global Geogrid Average(N60L): Average(HP):	0" 0"

Design CBR	6
---------------	---

% Samples within 3 feet of subgrade			
$N_{60} \leq 5$	0%	$HP \leq 0.5$	0%
$N_{60} < 12$	13%	$0.5 < HP \leq 1$	0%
$12 \leq N_{60} < 15$	20%	$1 < HP \leq 2$	7%
$N_{60} \geq 20$	13%	$HP > 2$	47%
M+	13%		
Rock	0%		
Unsuitable Soil	0%		

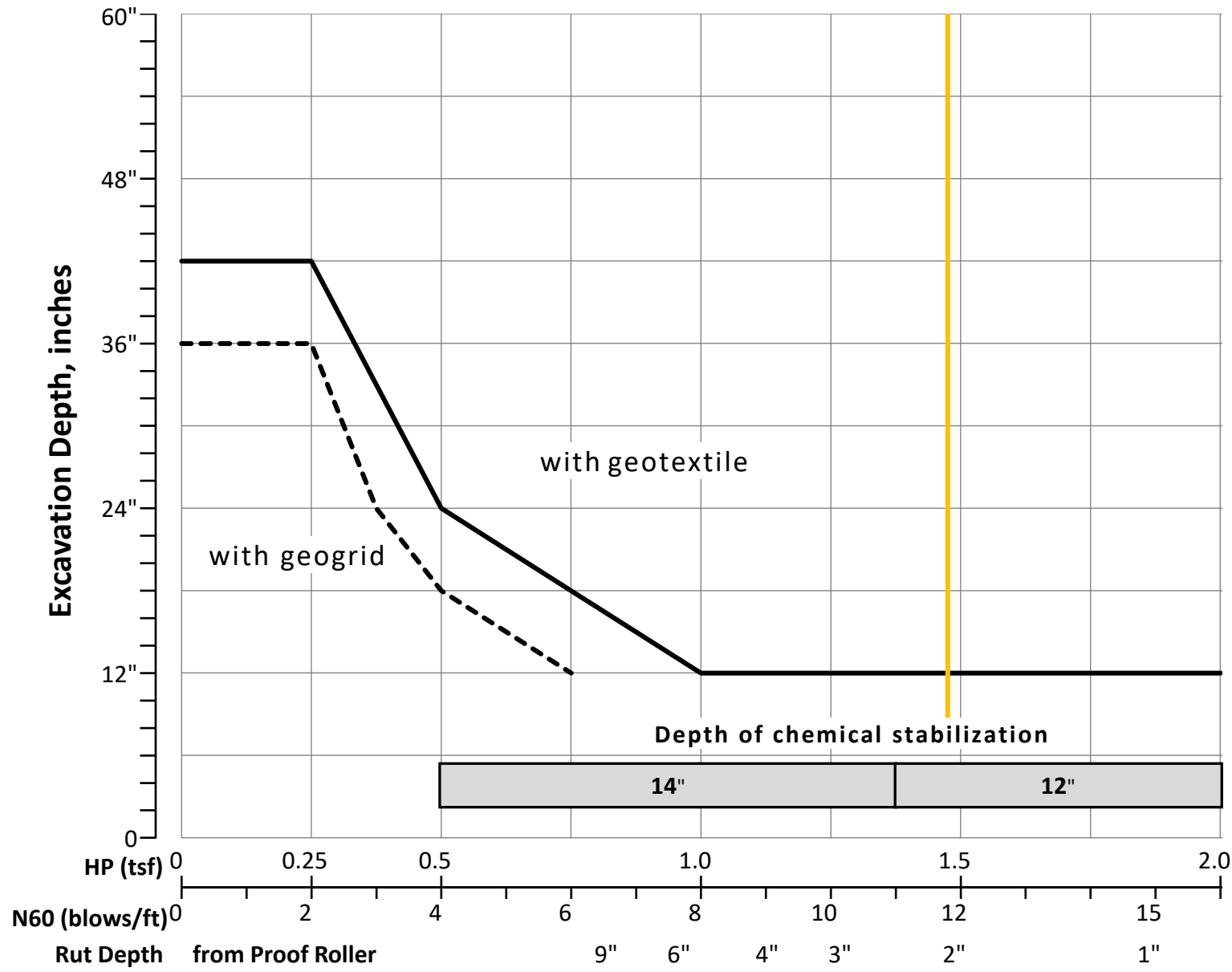
Excavate and Replace at Surface	
Average	0"
Maximum	0"
Minimum	0"

% Proposed Subgrade Surface	
Unstable & Unsuitable	43%
Unstable	43%
Unsuitable (Soil & Rock)	0%

	N_{60}	N_{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M_C	M_{OPT}	GI
Average	18	12	3.74	37	20	17	37	30	67	14	14	11
Maximum	40	21	4.50	54	27	32	46	50	88	26	24	19
Minimum	4	4	1.50	19	14	5	28	12	42	8	8	1

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	0	0	0	1	0	0	0	0	1	10	0	0	2	3	0	6	0	0	23
Percent	0%	0%	0%	0%	4%	0%	0%	0%	0%	4%	43%	0%	0%	9%	13%	0%	26%	0%	0%	100%
% Rock Granular Cohesive	0%	0%	52%									48%								100%
Surface Class Count	0	0	0	0	0	0	0	0	0	0	3	0	0	0	1	0	3	0	0	7
Surface Class Percent	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	43%	0%	0%	0%	14%	0%	43%	0%	0%	100%

Fig. 600-1 – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
3.74	0.50	☐ HP
11.80	6.00	☐ N60L

Average HP

Average N_{60L}



OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES

Geotechnical Design Manual Section 600

Instructions: Enter data in the shaded cells only.

(Enter state route number, project description, county, consultant's name, prepared by name, and date prepared. This information will be transferred to all other sheets. The date prepared must be entered in the appropriate cell on this sheet to remove these instructions prior to printing.)

**FRA-CemeteryRd SUP
120103**

Ramp WS - Cemetery Rd EB to IR-270 SB Ramp

Provide a SUP on the south side of Cemetery Road (Rd) in the city of Hilliard, Ohio

NEAS Inc.

Prepared By: Brendan Andrews, PE
Date prepared: Wednesday, October 2, 2024

Brendan Andrews
1329 East Kemper Road, Suite 4104B
Cincinnati, OH 45246

513.337.9823 Ext. 701
brendan.andrews@neasinc.com

NO. OF BORINGS: 2

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	B-015-0-23	Ramp WS	114+50.6	3	Left	CME 45B	79	876.4	877.9	1.5 F
2	B-011-0-23	Ramp WS	108+32.2	2	Left	CME 55TB	89	871.6	879.9	8.4 F

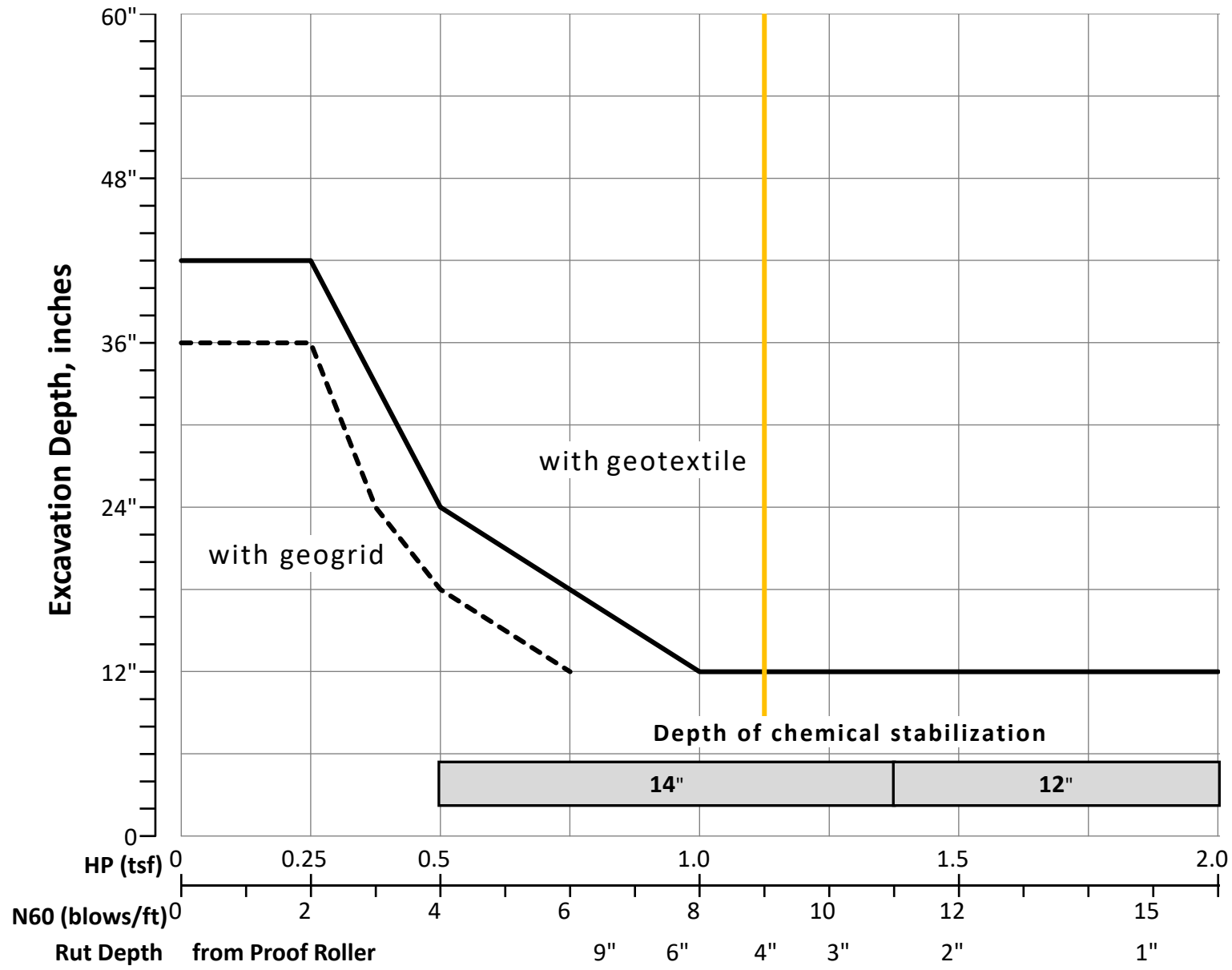
#	Boring	Sample	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _C	M _{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
1	B 015-0 23	SS-1	1.5	3.0	3.0	4.5	14	9	4.5	47	25	22	45	36	81	18	22	A-7-6	14	13					
		SS-2	3.0	4.5	4.5	6.0	13		2.25	54	22	32	38	50	88	26	19	A-7-6	19						
		SS-3	4.5	6.0	6.0	7.5	9		1.5							22	18	A-7-6							
		SS-4	6.0	7.5	7.5	9.0	17		3							13	16	A-6b							
2	B 011-0 23	SS-1	1.5	3.0	9.9	11.4	32		4.5	33	18	15	36	33	69	11	14	A-6a		200					
		SS-2	3.0	4.5	11.4	12.9	25		4.5	20	15	5	38	17	55	10	10	A-4a							
		SS-3	4.5	6.0	12.9	14.4	30		4.5							10	10	A-4a							
		SS-4	6.0	7.5	14.4	15.9	17									12	10	A-2-4							

Date prepared: 10/2/2024

[illegible]



Fig. 600-1 – Subgrade Stabilization



Override Table

Calculated Average	New Values	Check to Override
3.54	0.50	☐ HP
9.00	6.00	☐ N60L

Average HP

Average N_{60L}



OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES

Geotechnical Design Manual Section 600

Instructions: Enter data in the shaded cells only.

(Enter state route number, project description, county, consultant's name, prepared by name, and date prepared. This information will be transferred to all other sheets. The date prepared must be entered in the appropriate cell on this sheet to remove these instructions prior to printing.)

**FRA-CemeteryRd SUP
120103**

Ramp SE - IR-270 NB to Cemetery Rd Ramp

Provide a SUP on the south side of Cemetery Road (Rd) in the city of Hilliard, Ohio

NEAS Inc.

Prepared By: Brendan Andrews, PE
Date prepared: Wednesday, October 2, 2024

Brendan Andrews
1329 East Kemper Road, Suite 4104B
Cincinnati, OH 45246

513.337.9823 Ext. 701
brendan.andrews@neasinc.com

NO. OF BORINGS: 1

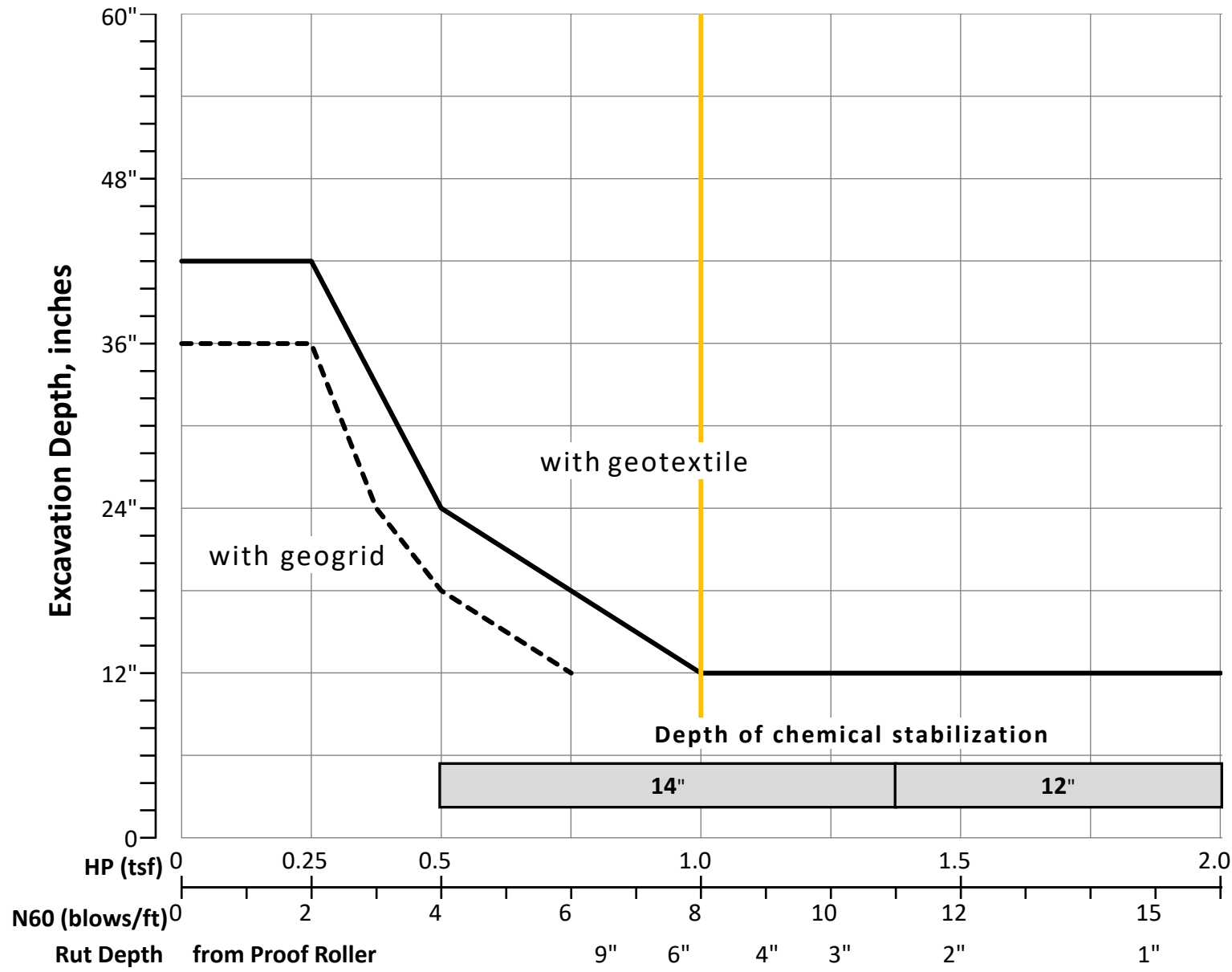
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1	B-004-0-23	Ramp SE	202+31	12	Right	CME 45B	79	871.2	873.4	2.2 F

#	Boring	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 23	SS-1	1.5	3.0	3.7	5.2	8		1.5	49	26	23	38	35	73	27	23	A-7-6	15	13					
		SS-2	3.0	4.5	5.2	6.7	11		0.5	42	22	20	36	34	70	22	19	A-7-6							
		SS-3	4.5	6.0	6.7	8.2	21		4.5							17	10	A-4a							
		SS-4	6.0	7.5	8.2	9.7	13		8	4.5							13	10	A-4a						

Date prepared: 10/2/2024

[illegible]

Fig. 600-1 – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
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8.00	6.00	☐ N60L

Average HP

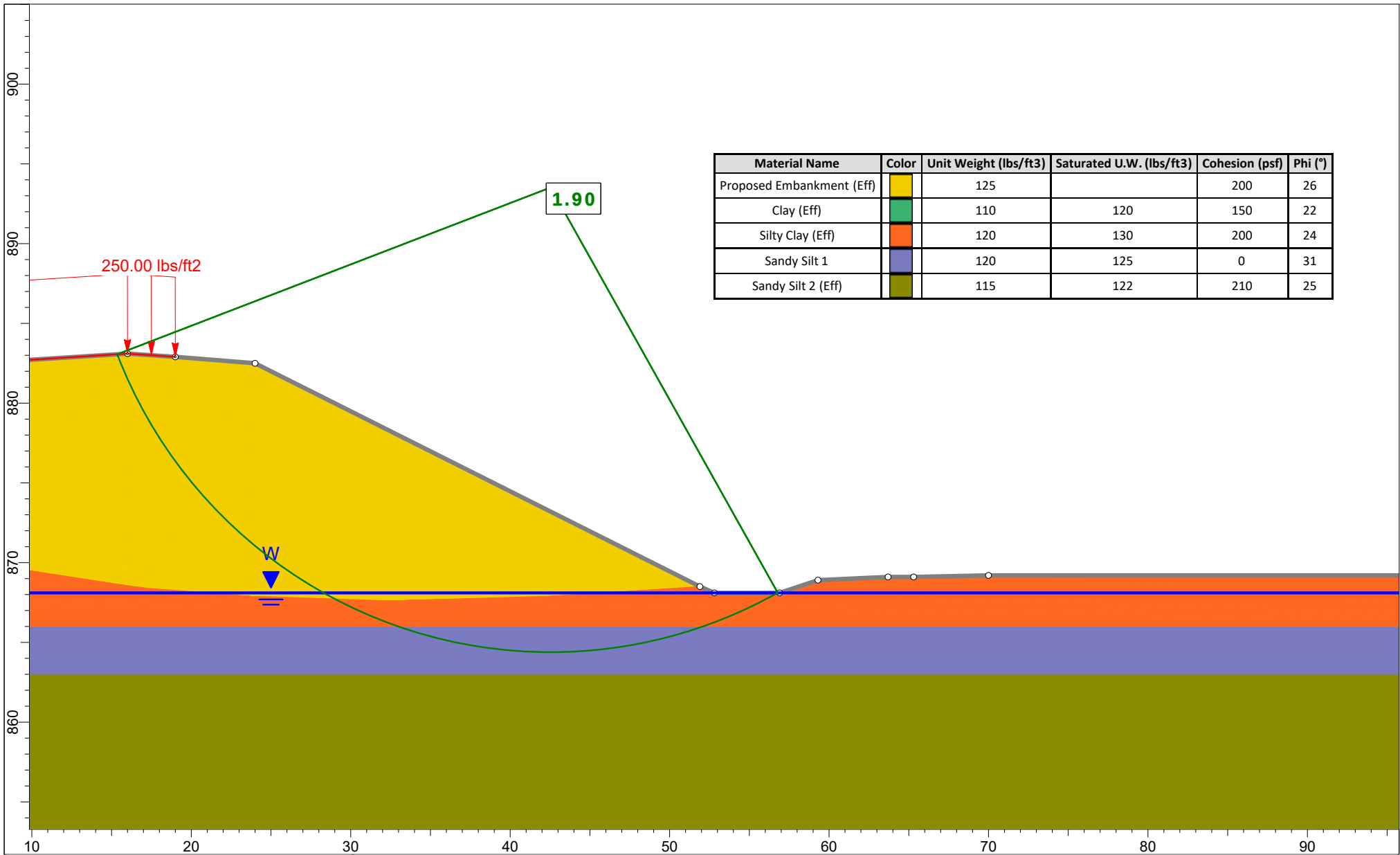
Average N_{60L}



APPENDIX 2B

EMBANKMENT GLOBAL STABILITY ANALYSIS

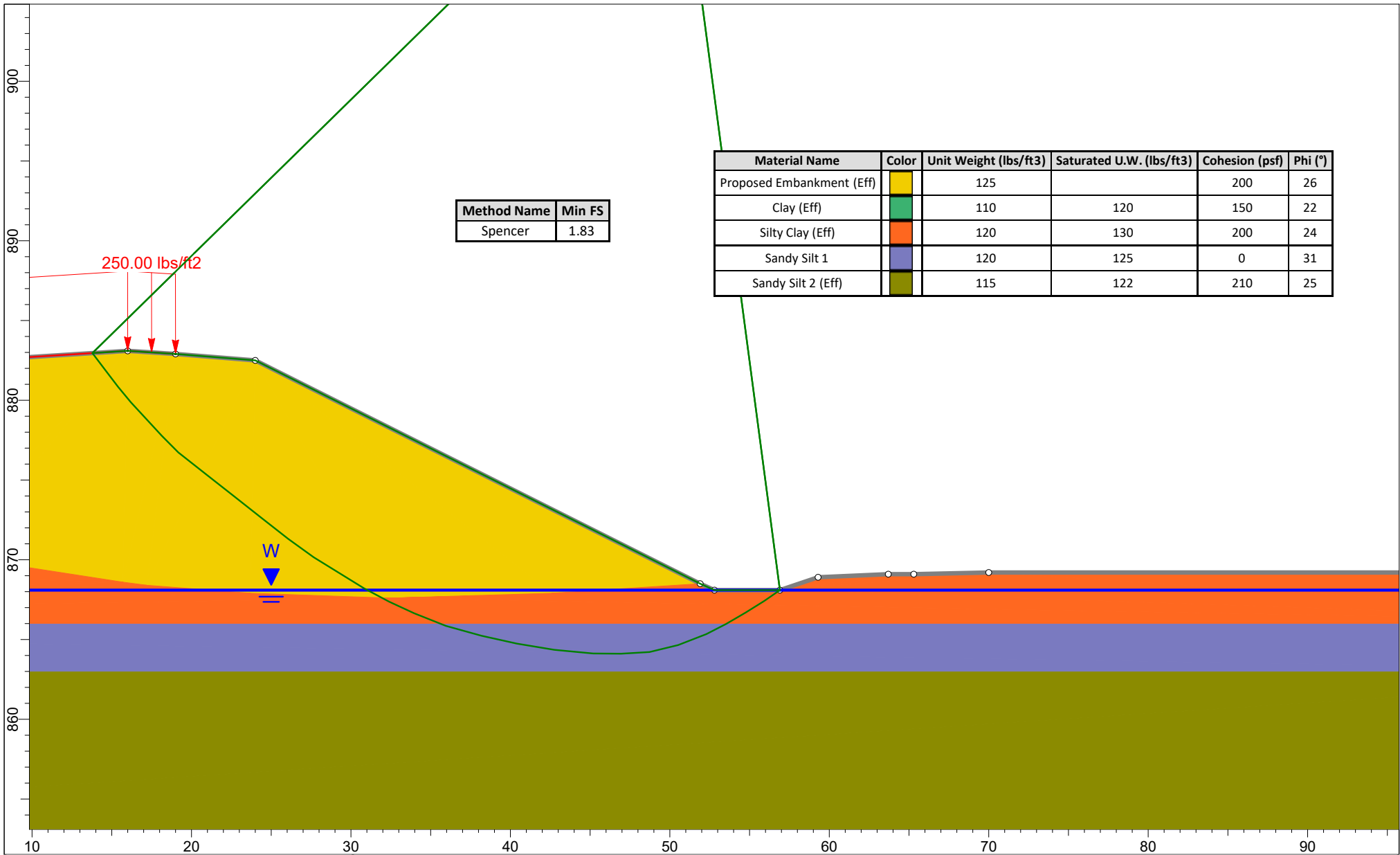
RAMP WS – STA.108+50



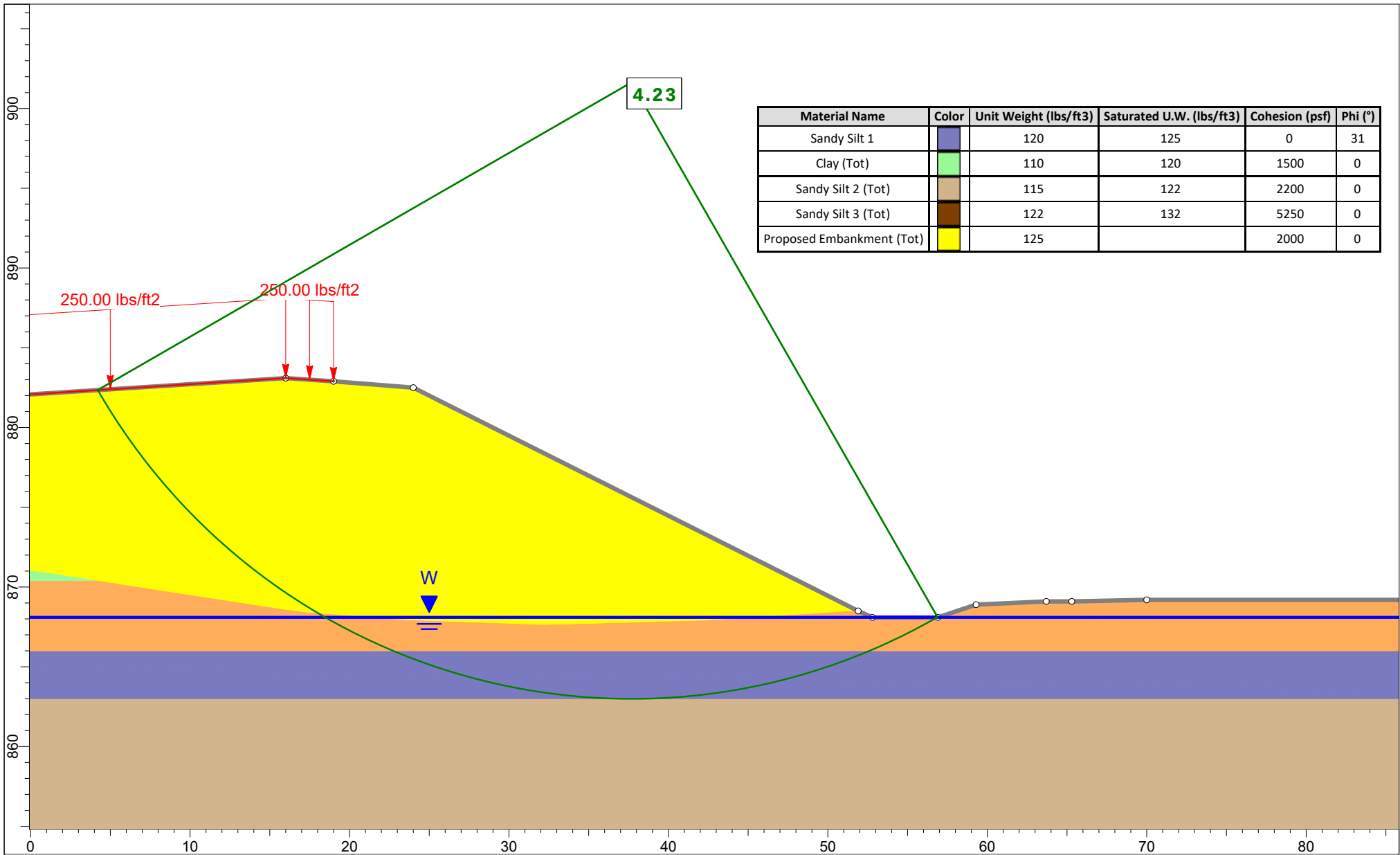
Material Name	Color	Unit Weight (lbs/ft ³)	Saturated U.W. (lbs/ft ³)	Cohesion (psf)	Phi (°)
Proposed Embankment (Eff)		125		200	26
Clay (Eff)		110	120	150	22
Silty Clay (Eff)		120	130	200	24
Sandy Silt 1		120	125	0	31
Sandy Silt 2 (Eff)		115	122	210	25



Project	FRA-CEMETERY RD-SUP, PID 120103				
Group	Effective Stress		Scenario	Circular Failure	
Drawn By	KCA		Company	NEAS Inc.	
Date	3/13/2025, 11:32:58 AM		File Name	RampWS_STA108+50_031325.slmd	

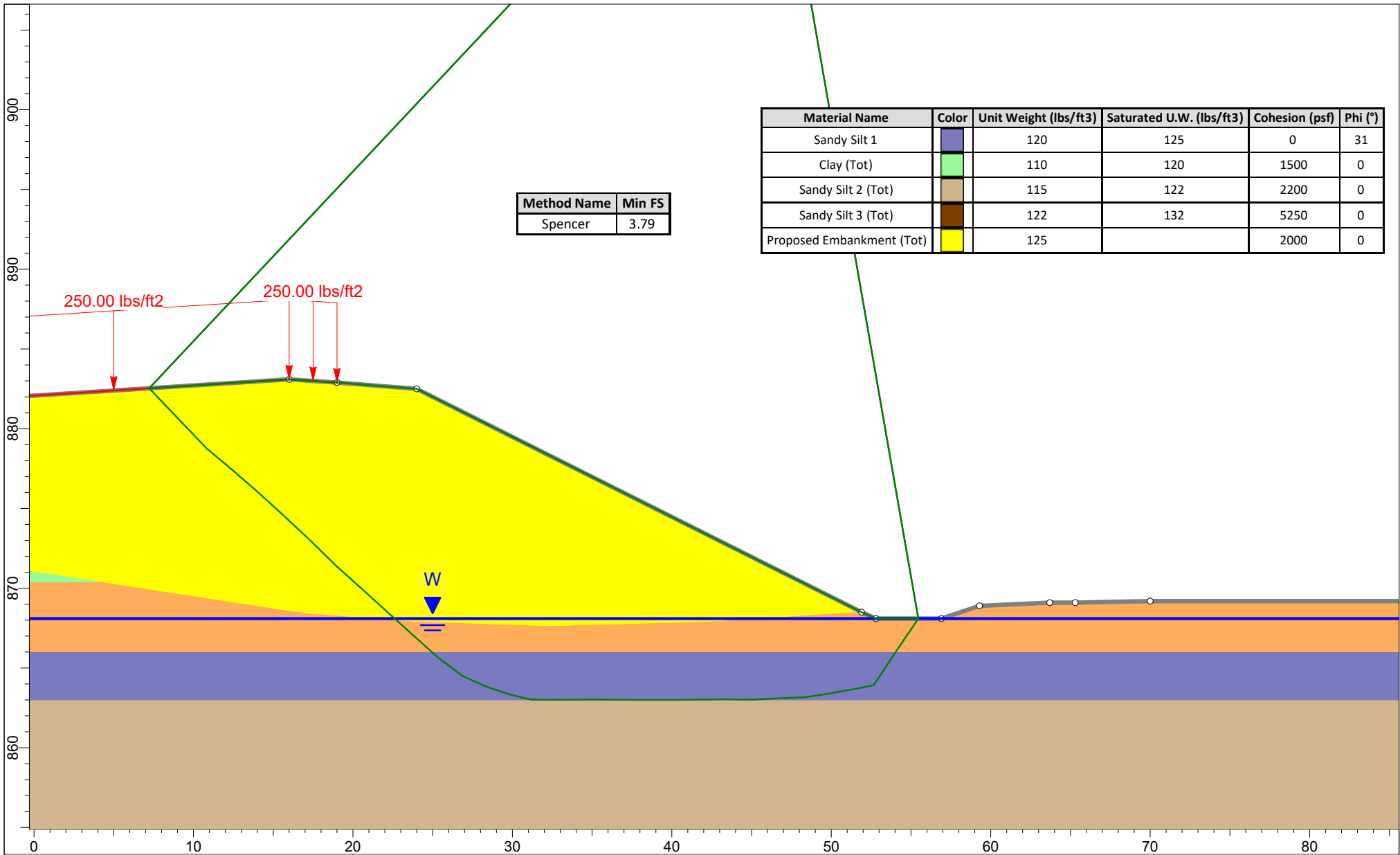


	Project		FRA-CEMETERY RD-SUP, PID 120103	
	Group		Effective Stress	Scenario Non-Circular Failure
	Drawn By		KCA	Company NEAS Inc.
	Date		3/13/2025, 11:32:58 AM	File Name RampWS_STA108+50_031325.slmd
	SLIDEINTERPRET 9.038			



Material Name	Color	Unit Weight (lbs/ft ³)	Saturated U.W. (lbs/ft ³)	Cohesion (psf)	Phi (°)
Sandy Silt 1		120	125	0	31
Clay (Tot)		110	120	1500	0
Sandy Silt 2 (Tot)		115	122	2200	0
Sandy Silt 3 (Tot)		122	132	5250	0
Proposed Embankment (Tot)		125		2000	0

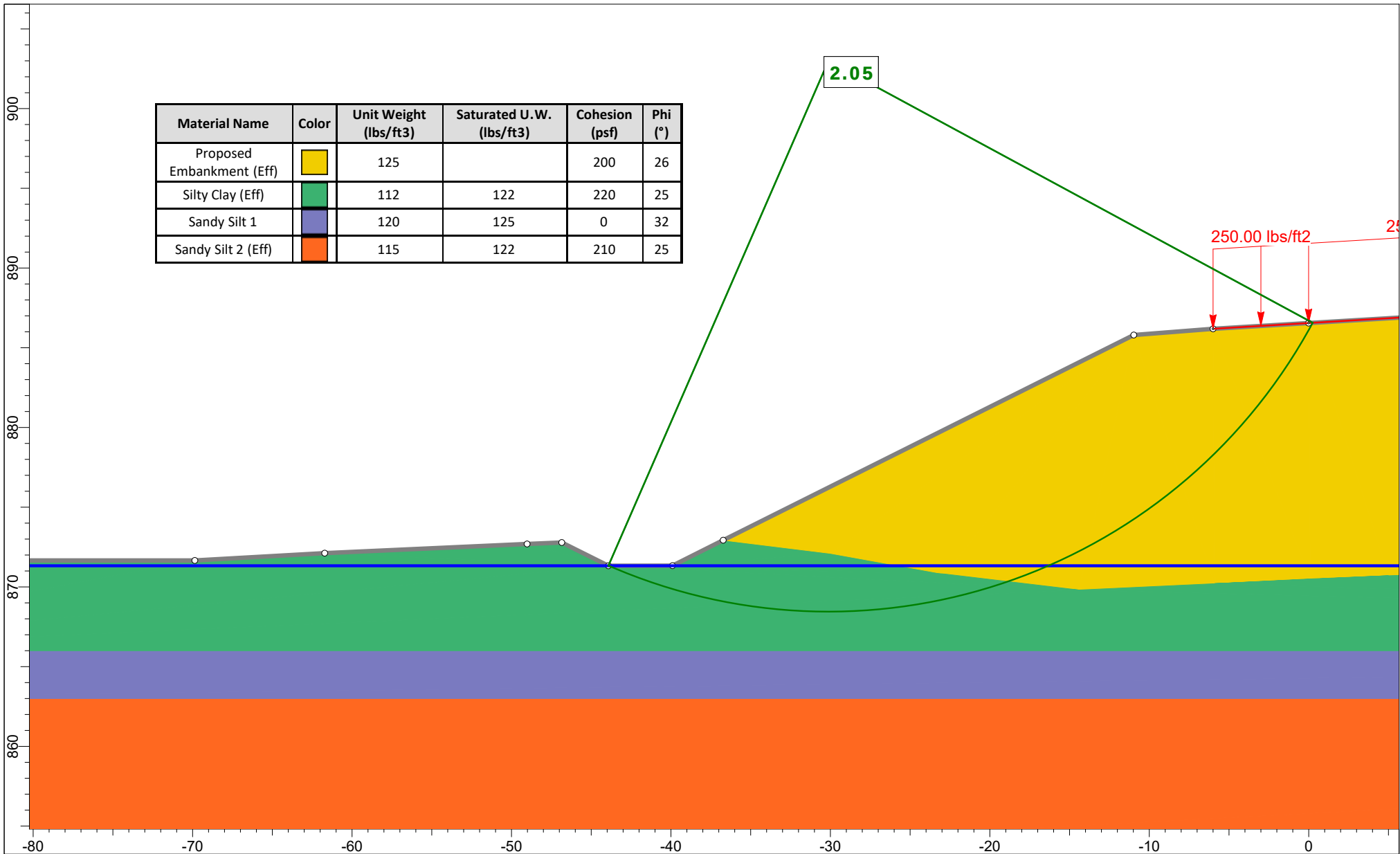
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	Group		Total Stress	Scenario
	Drawn By		KCA	Company
	Date		3/13/2025, 11:32:58 AM	File Name
				RampWS_STA108+50_031325.slmd



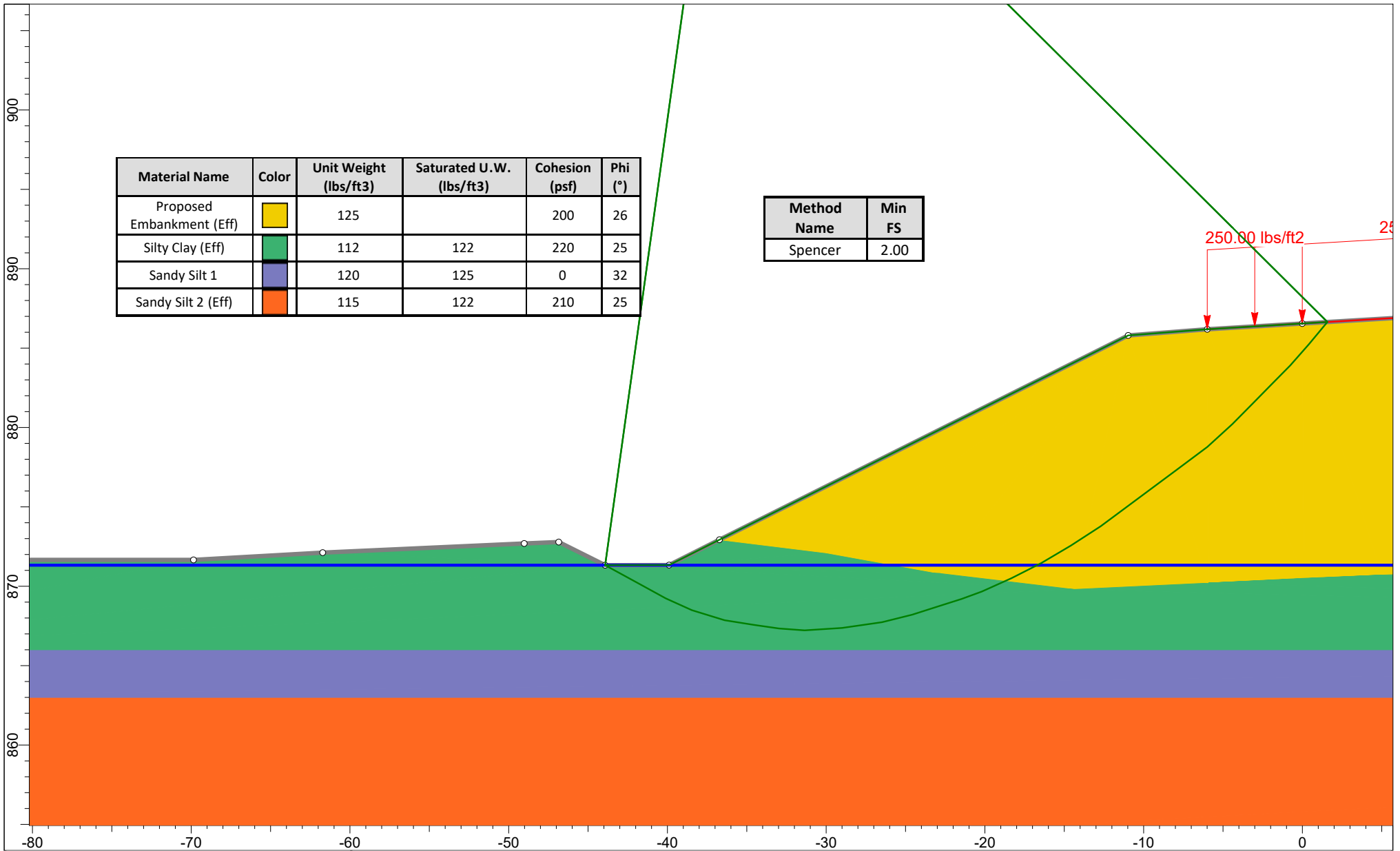
Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Cohesion (psf)	Phi (°)
Sandy Silt 1		120	125	0	31
Clay (Tot)		110	120	1500	0
Sandy Silt 2 (Tot)		115	122	2200	0
Sandy Silt 3 (Tot)		122	132	5250	0
Proposed Embankment (Tot)		125		2000	0

	Project		FRA-CEMETERY RD-SUP, PID 120103	
	Group	Total Stress	Scenario	Non-Circular Failure
	Drawn By	KCA	Company	NEAS Inc.
	Date	3/13/2025, 11:32:58 AM	File Name	RampWS_STA108+50_031325.slmd
	SLIDEINTERPRET 9.038			

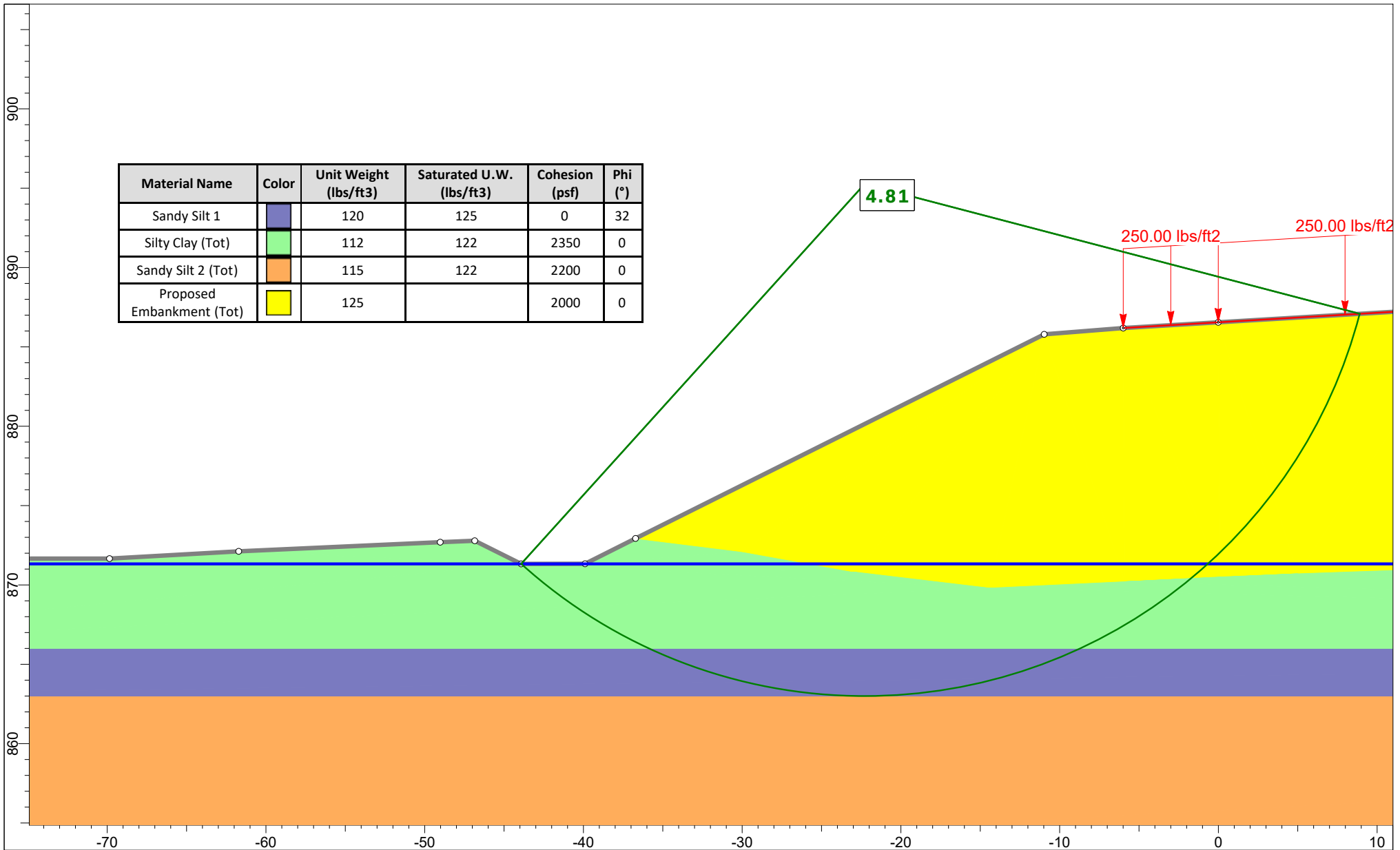
RAMP WS – STA.111+00



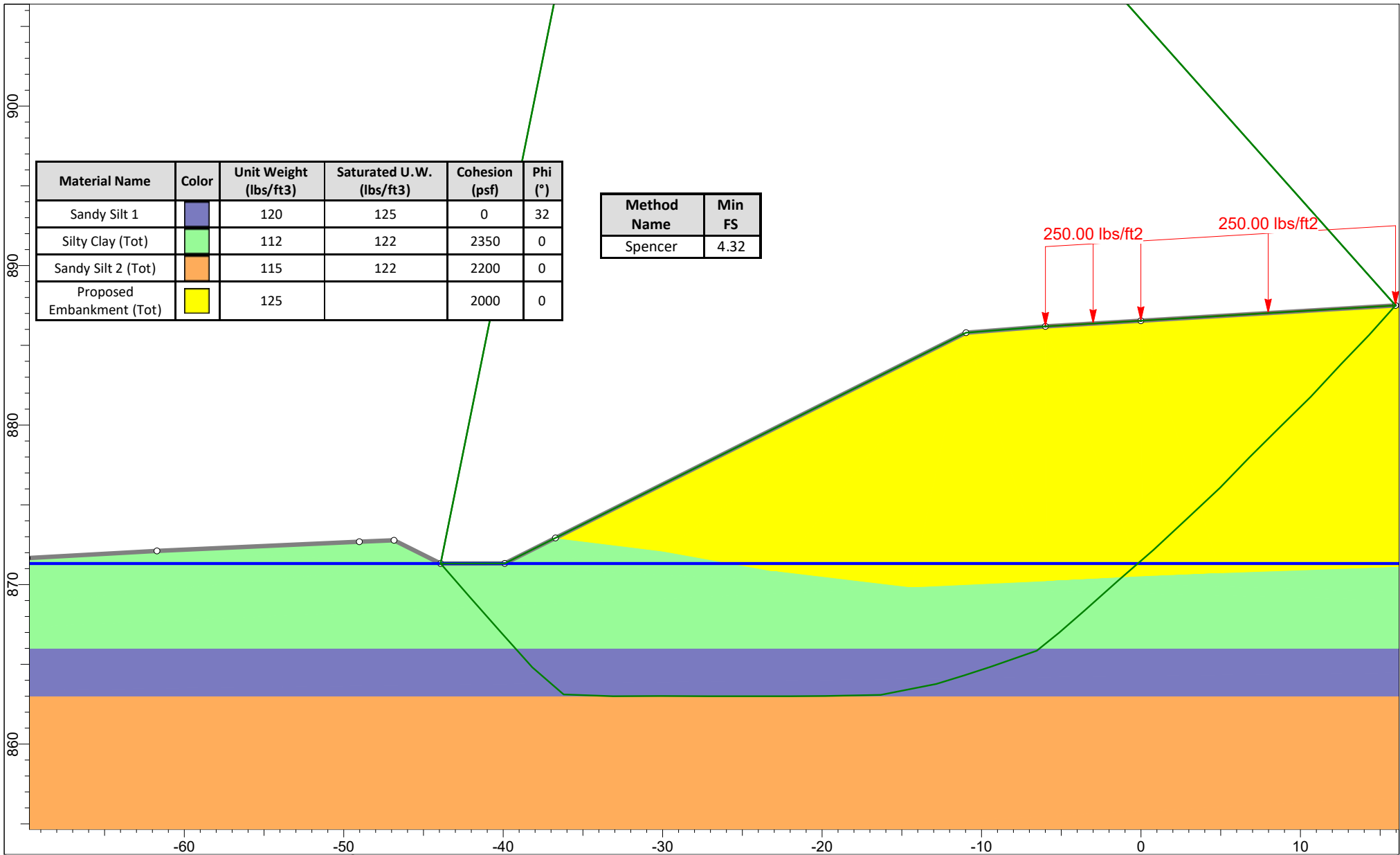
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	Group		Effective Stress	Scenario
	Drawn By		KCA	Company
	Date		3/13/2025, 11:32:58 AM	NEAS Inc.
			File Name	RampWS_STA111+00_031325.slmd



 SLIDEINTERPRET 9.038	ProjectFRA-CEMETERY RD-SUP, PID 120103	
	GroupEffective Stress	ScenarioNon-Circular Failure
	Drawn ByKCA	CompanyNEAS Inc.
	Date3/13/2025, 11:32:58 AM	File NameRampWS_STA111+00_031325.slmd



	Project		FRA-CEMETERY RD-SUP, PID 120103	
	Group	Total Stress	Scenario	Circular Failure
	Drawn By	KCA	Company	NEAS Inc.
	Date	3/13/2025, 11:32:58 AM	File Name	RampWS_STA111+00_031325.slmd
	SLIDEINTERPRET 9.038			



Material Name	Color	Unit Weight (lbs/ft3)	Saturated U. W. (lbs/ft3)	Cohesion (psf)	Phi (°)
Sandy Silt 1		120	125	0	32
Silty Clay (Tot)		112	122	2350	0
Sandy Silt 2 (Tot)		115	122	2200	0
Proposed Embankment (Tot)		125		2000	0

Method Name	Min FS
Spencer	4.32

	Project		FRA-CEMETERY RD-SUP, PID 120103	
	Group	Total Stress	Scenario	Non-Circular Failure
	Drawn By	KCA	Company	NEAS Inc.
	Date	3/13/2025, 11:32:58 AM	File Name	RampWS_STA111+00_031325.slmd
	SLIDEINTERPRET 9.038			

APPENDIX 2C

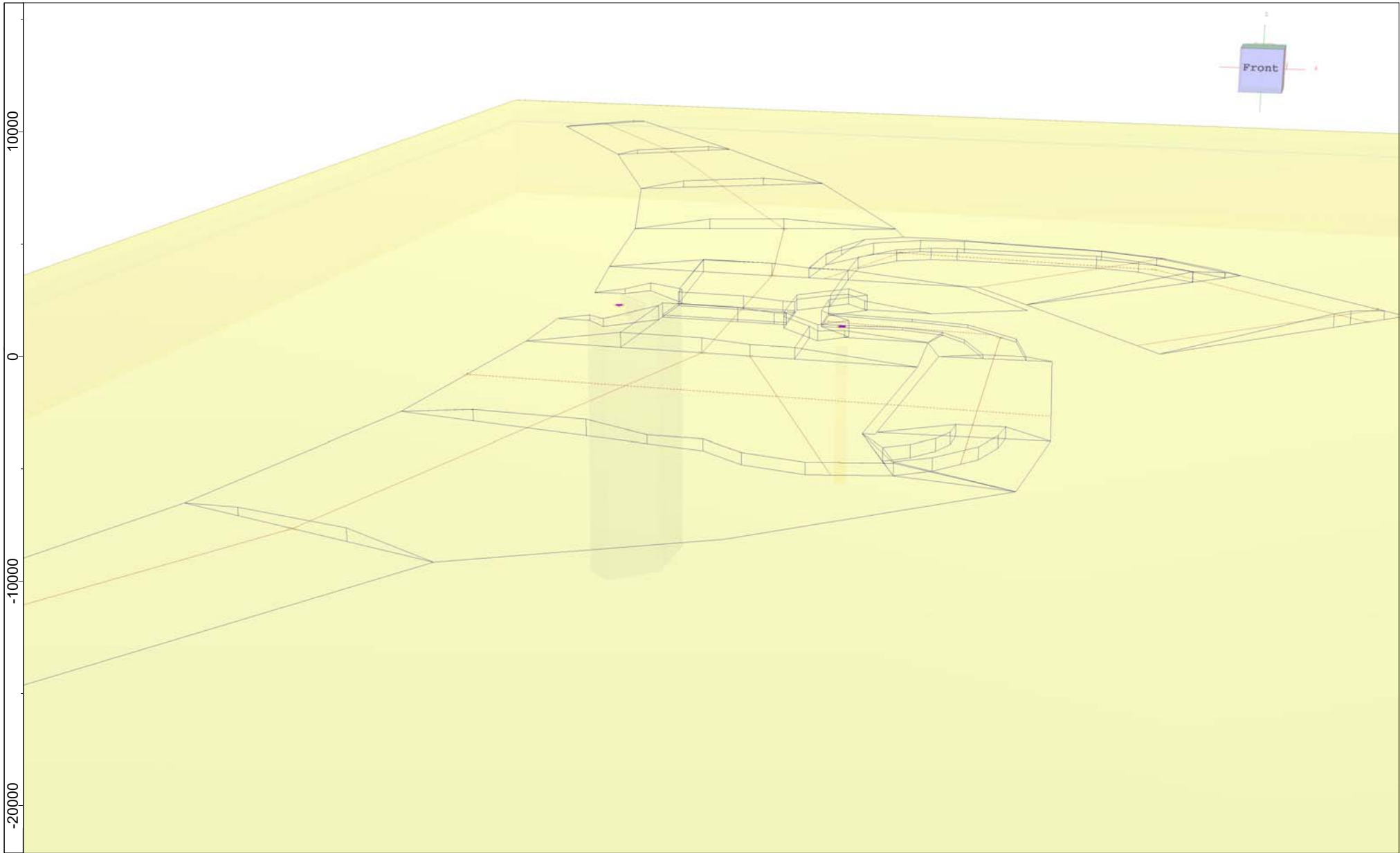
SETTLEMENT ANALYSIS



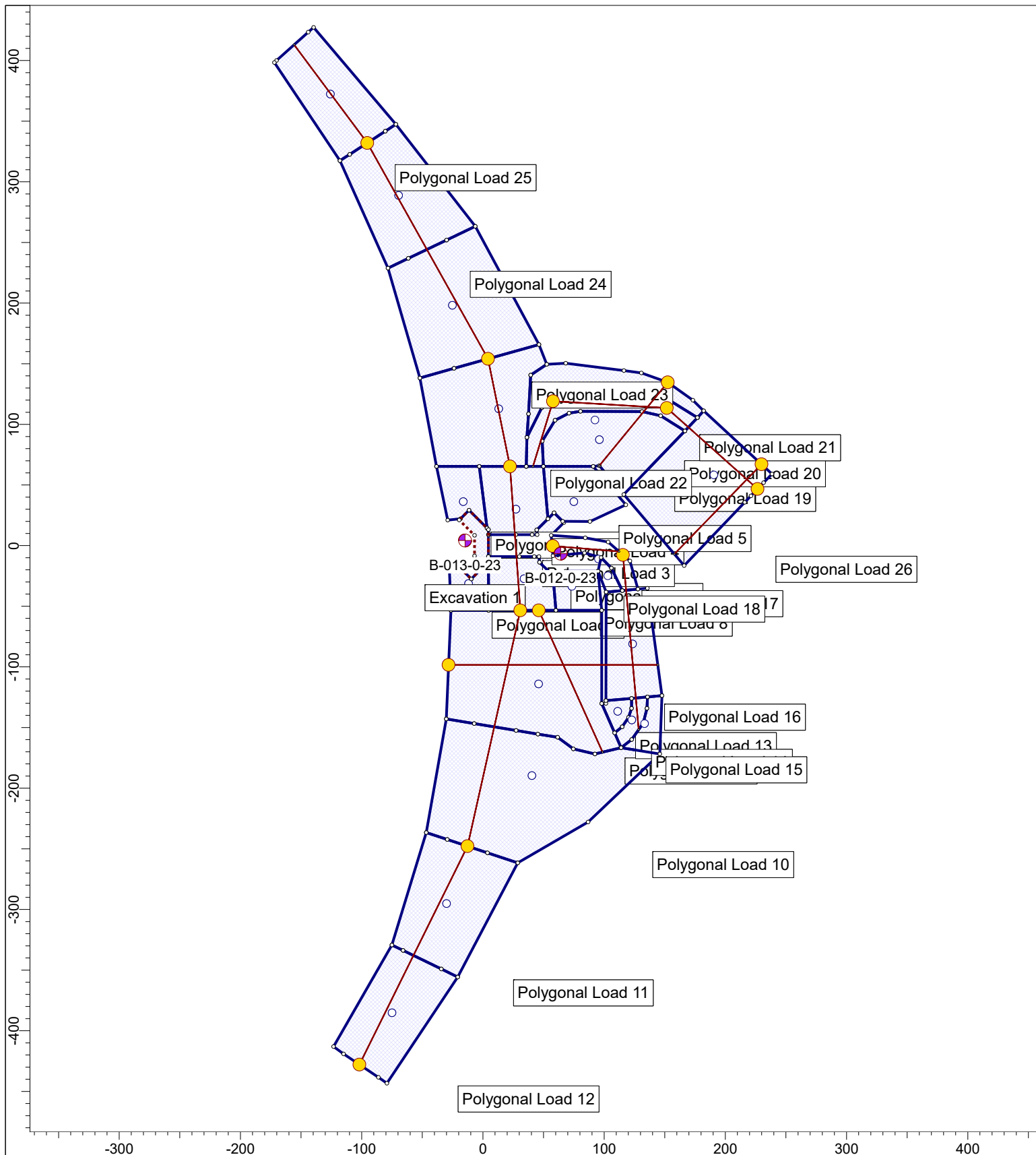
FRA-CEMETERY RS-SUP

NEAS Inc.

Report Creation Date: 2025/03/28, 10:01:10



Project		FRA-CEMETERY RS-SUP	
Analysis Description		Settlement at Culvert FRA-TUNNEL Under Southbound IR-270 On-Ramp	
Drawn By		KCA	Company NEAS Inc.
Date		2/28/2025, 12:12:02 PM	File Name CemeteryRd_Roadway_Settle031225.s3z



	Project			FRA-CEMETERY RS-SUP
	Analysis Description			Settlement at Culvert FRA-TUNNEL Under Southbound IR-270 On-Ramp
	Drawn By		Company	KCA NEAS Inc.
	Date		File Name	2/28/2025, 12:12:02 PM CemeteryRd_Roadway_Settle031225.s3z

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Settle3 Analysis Information

FRA-CEMETERY RS-SUP

Project Settings

Document Name	CemeteryRd_Roadway_Settle031225.s3z
Project Title	FRA-CEMETERY RS-SUP
Analysis	Settlement at Culvert FRA-TUNNEL Under Southbound IR-270 On-Ramp
Author	KCA
Company	NEAS Inc.
Date Created	2/28/2025, 12:12:02 PM
Last saved with Settle3 version	5.024
Stress Computation Method	Boussinesq
Stress Units	Imperial, stress as ksf
Settlement Units	inches
Time-dependent Consolidation Analysis	
Time Units	days
Permeability Units	feet/day

Advanced Settings

Start of secondary consolidation (% of primary)	95
Min. stress for secondary consolidation (% of initial)	1
Reset time when load changes for secondary consolidation	No
Minimum settlement ratio for subgrade modulus	0.9
Use average poisson's ratio to calculate layered stresses	
Update Cv in each time step (improves consolidation accuracy)	
Ignore negative effective stresses in settlement calculations	
Add field points to load edges	

Soil Profile

Layer Option	Non-Horizontal Layers
Interpolation Method	TIN Triangulation
Use Non-Horizontal Ground Surface	No
Vertical Axis	Elevation
Ground Elevation (ft)	871.3

Stage Settings

Stage #	Name	Time [days]
1	Stage 1	0
2	Stage 2	45
3	Stage 3	20000

Results

Time taken to compute: 9.56135 seconds

Stage: Stage 1 = 0 d

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.43903
Total Consolidation Settlement [in]	0	0
Virgin Consolidation Settlement [in]	0	0
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	1.43903
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-3.96044e-05	2.10001
Loading Stress XX [ksf]	-0.836324	3.01629
Loading Stress YY [ksf]	-0.568585	2.63054
Effective Stress ZZ [ksf]	0	4.18916
Effective Stress XX [ksf]	-0.320523	7.19151
Effective Stress YY [ksf]	-0.568585	6.67732
Total Stress ZZ [ksf]	-3.96044e-05	8.01644
Total Stress XX [ksf]	-0.299483	11.0011
Total Stress YY [ksf]	-0.2586	10.1493
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.00141177	28.6262
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.00141177	28.6262
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	5.07592e-09	0.00789152
Pore Water Pressure [ksf]	-3.96044e-05	3.8298
Excess Pore Water Pressure [ksf]	-3.96044e-05	2.10001
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.308096	33.2194
Over-consolidation Ratio	1	48
Void Ratio	0	0.621
Permeability [ft/d]	0	0.0586693
Coefficient of Consolidation [ft ² /d]	0	0.6
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	2
Undrained Shear Strength	-1.11022e-16	2.22045e-16

Stage: Stage 2 = 45 d

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.69223
Total Consolidation Settlement [in]	0	2.25539
Virgin Consolidation Settlement [in]	0	0.224506
Recompression Consolidation Settlement [in]	0	2.03094
Immediate Settlement [in]	0	1.43903
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-3.96044e-05	2.10001
Loading Stress XX [ksf]	-0.836324	3.01629
Loading Stress YY [ksf]	-0.568585	2.63054
Effective Stress ZZ [ksf]	0	5.58243
Effective Stress XX [ksf]	-0.299485	8.56608
Effective Stress YY [ksf]	-0.2586	7.72746
Total Stress ZZ [ksf]	0	8.01644
Total Stress XX [ksf]	-0.299483	11.0011
Total Stress YY [ksf]	-0.2586	10.1493
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.000907408	8.38067
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.00141177	28.6262
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.00253996	13.0829
Total Strain	4.25904e-05	0.158745
Pore Water Pressure [ksf]	0	2.8988
Excess Pore Water Pressure [ksf]	0	1.62339
Degree of Consolidation [%]	0	99.8331
Pre-consolidation Stress [ksf]	0.308096	33.2194
Over-consolidation Ratio	1	47.8308
Void Ratio	0	0.620931
Permeability [ft/d]	0	0.286612
Coefficient of Consolidation [ft ² /d]	0	0.6
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	99.9972
Undrained Shear Strength	0	0.568343

Stage: Stage 3 = 20000 d

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.9291
Total Consolidation Settlement [in]	0	2.49763
Virgin Consolidation Settlement [in]	0	0.224506
Recompression Consolidation Settlement [in]	0	2.2732
Immediate Settlement [in]	0	1.43903
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-3.96044e-05	2.10001
Loading Stress XX [ksf]	-0.836324	3.01629
Loading Stress YY [ksf]	-0.568585	2.63054
Effective Stress ZZ [ksf]	0	5.58243
Effective Stress XX [ksf]	-0.299483	8.56608
Effective Stress YY [ksf]	-0.2586	7.72746
Total Stress ZZ [ksf]	0	8.01644
Total Stress XX [ksf]	-0.299483	11.0011
Total Stress YY [ksf]	-0.2586	10.1493
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.000779246	7.74797
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.00141177	28.6262
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.00173925	11.6836
Total Strain	4.26228e-05	0.158745
Pore Water Pressure [ksf]	-2.53975e-13	2.43984
Excess Pore Water Pressure [ksf]	-2.53975e-13	3.47062e-13
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.308096	33.2194
Over-consolidation Ratio	1	47.8306
Void Ratio	0	0.620931
Permeability [ft/d]	0	0.286612
Coefficient of Consolidation [ft ² /d]	0	0.6
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	100
Undrained Shear Strength	0	0.568343

Loads

1. Polygonal Load: "Polygonal Load 2"

Label	Polygonal Load 2
Load Type	Flexible
Area of Load	665.685 ft2
Load	2.3 ksf
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates

X [ft]	Y [ft]
58.7	27.1
44.4	12.9
46.7	-13.7
61.3	-28.2
69.4	-20
57.8	-8.4
56.3	8.5
66.8	19

2. Polygonal Load: "Polygonal Load 3"

Label	Polygonal Load 3
Load Type	Flexible
Area of Load	736.2 ft2
Load	0.31 ksf
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates

X [ft]	Y [ft]
4.6	-9
46.3	-9
44.7	9
4.6	9

3. Polygonal Load: "Polygonal Load 1"

Label	Polygonal Load 1
Load Type	Flexible
Area of Load	1427.33 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-11.4	29.2	1.21
3.4	14.4	2.06
-3	65.3	1.91
-38.3	65.3	0
-29	21	0
-18.4	22.2	0.74

4. Polygonal Load: "Polygonal Load 4"

Label	Polygonal Load 4
Load Type	Flexible
Area of Load	2825.84 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
4.6	13.2	2.06
4.6	9	2.06
27.1	9	2.06
40.6	9	1.63
44.7	9	1.63
44.4	12.9	1.63
53.6	22.1	1.63
49.9	65.3	1.56
35.8	65.3	1.56
22.3	65.3	1.98
-3	65.3	1.98
3.4	14.4	2.06

5. Polygonal Load: "Polygonal Load 5"

Label	Polygonal Load 5
Load Type	Flexible
Area of Load	2319.01 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
53.6	22.1	1.63
58.7	27.1	1.46
65.8	20	1.3
88.3	19.9	0
117.9	33.6	0
91	65.3	0
49.9	65.3	1.56

6. Polygonal Load: "Polygonal Load 6"

Label	Polygonal Load 6
Load Type	Flexible
Area of Load	2280.35 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
4.6	-13.2	2.1
4.7	-53.4	2.1
30.9	-53.4	2.1
46.1	-53.4	1.59
60.2	-53.4	1.59
57.8	-24.7	1.64
46.7	-13.7	1.65
46.3	-9	1.65
42.3	-9	1.65
30.1	-9	2.1
4.6	-9	2.1

7. Polygonal Load: "Polygonal Load 7"

Label	Polygonal Load 7
Load Type	Flexible
Area of Load	1020.31 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-16.7	-20.4	0.81
-27.6	-20.4	0
-26.4	-53.4	0
4.7	-53.4	2.1
4.6	-13.2	2.1
-9.6	-27.5	1.13

8. Polygonal Load: "Polygonal Load 8"

Label	Polygonal Load 8
Load Type	Flexible
Area of Load	1171.99 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
60.2	-53.4	1.59
98	-53.4	0
95	-21.1	0
68.3	-21.1	0.96
61.3	-28.2	1.44
57.8	-24.7	1.64

9. Polygonal Load: "Polygonal Load 9"

Label	Polygonal Load 9
Load Type	Flexible
Area of Load	13215 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-7.2	-146.7	1.425
27.4	-152.4	2.0125
45.4	-155.4	1.075
61.6	-158	1.425
74.6	-167.6	1.4375
92.4	-171.8	1.45
114	-166.6	1.4625
98	-130.2	0
98	-53.4	0
60.2	-53.4	1.5875
46.1	-53.4	1.5875
30.9	-53.4	2.1
4.7	-53.4	2.1
-26.4	-53.4	0
-30.2	-142.9	0

10. Polygonal Load: "Polygonal Load 10"

Label	Polygonal Load 10
Load Type	Flexible
Area of Load	13677.4 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-46.7	-236.6	0
-29.4	-242.3	0.94
3.8	-253.3	1.46
28.7	-261.6	0
86.8	-227.9	0
145.9	-171.8	0
114	-166.6	1.46
92.4	-171.8	1.45
74.6	-167.6	1.44
61.6	-158	1.43
45.4	-155.4	1.08
27.4	-152.4	2.01
-7.2	-146.7	1.43
-30.2	-142.9	0

11. Polygonal Load: "Polygonal Load 11"

Label	Polygonal Load 11
Load Type	Flexible
Area of Load	7058.93 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-75.1	-329.3	0
-65.8	-333.8	0.75
-34.3	-349.1	1.03
-20.8	-355.7	0
28.7	-261.6	0
3.8	-253.3	1.46
-29.4	-242.3	0.94
-46.7	-236.6	0

12. Polygonal Load: "Polygonal Load 12"

Label	Polygonal Load 12
Load Type	Flexible
Area of Load	5724.29 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-20.8	-355.7	0
-34.3	-349.1	1.03
-65.8	-333.8	0.75
-75.1	-329.3	0
-123.2	-413.1	0
-114.9	-419.1	0.19
-86.2	-438.4	0.19
-79.2	-443.3	0

13. Polygonal Load: "Polygonal Load 13"

Label	Polygonal Load 13
Load Type	Flexible
Area of Load	433.01 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
122.6	-126	1.46
101.5	-127.8	0
101.5	-130.1	0
98	-130.2	0
108.7	-154.4	1.61
114.9	-149.4	1.58
120.1	-141.7	1.54
122.6	-134.2	1.5

14. Polygonal Load: "Polygonal Load 14"

Label	Polygonal Load 14
Load Type	Flexible
Area of Load	527.365 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
114	-166.6	1.61
122.7	-159.9	1.58
132.2	-146.5	1.54
135.3	-134.2	1.5
135.8	-124.8	1.46
122.6	-126	1.46
122.6	-134.2	1.5
120.1	-141.7	1.54
114.9	-149.4	1.58
108.7	-154.4	1.61

15. Polygonal Load: "Polygonal Load 15"

Label	Polygonal Load 15
Load Type	Flexible
Area of Load	809.345 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
114	-166.6	1.61
145.9	-171.8	0
147.7	-123.7	0
135.8	-124.8	1.46
135.3	-134.2	1.5
132.2	-146.5	1.54
122.7	-159.9	1.58

16. Polygonal Load: "Polygonal Load 16"

Label	Polygonal Load 16
Load Type	Flexible
Area of Load	3596.42 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
101.5	-127.8	0
122.6	-126	1.46
135.8	-124.8	1.46
147.7	-123.7	0
135.7	-35.2	0
127.9	-35.9	0.51
115	-37.1	0.51
101.6	-38.3	0

17. Polygonal Load: "Polygonal Load 17"

Label	Polygonal Load 17
Load Type	Flexible
Area of Load	1187.17 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
57.8	-8.4	0.3
83.8	-6	0.35
97.5	-9.8	0.4
106	-18.5	0.45
115	-37.1	0.51
127.9	-35.9	0.51
121.3	-12.7	0.45
103.3	3	0.4
84.4	6.4	0.35
56.3	8.5	0.3

18. Polygonal Load: "Polygonal Load 18"

Label	Polygonal Load 18
Load Type	Flexible
Area of Load	292.1 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
97.5	-9.8	0.4
95	-21.1	0
101.6	-38.3	0
115	-37.1	0.51
106	-18.5	0.45

19. Polygonal Load: "Polygonal Load 19"

Label	Polygonal Load 19
Load Type	Flexible
Area of Load	4945.09 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
91	65.3	0
96	65.9	0
116.2	42.2	0
166.6	94.5	1.68
146.6	107.1	1.68
131.2	110.5	1.91
80.5	110.7	1.9
71.1	109.2	1.86
59.5	103.3	1.85
48.9	86.2	1.79
49.9	65.3	1.56

20. Polygonal Load: "Polygonal Load 20"

Label	Polygonal Load 20
Load Type	Flexible
Area of Load	2465.88 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
48.9	86.2	1.79
59.5	103.3	1.85
71.1	109.2	1.86
80.5	110.7	1.9
131.2	110.5	1.91
146.6	107.1	1.68
166.6	94.5	1.68
177	105.5	0.84
151.6	121.2	1.18
131.2	125.5	1.43
80.5	125.7	1.9
64.4	122.9	1.86
48.6	113.7	1.85
36.3	89.2	1.79
35.8	65.3	1.53
49.9	65.3	1.56

21. Polygonal Load: "Polygonal Load 21"

Label	Polygonal Load 21
Load Type	Flexible
Area of Load	3005.72 ft ²
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
48.6	113.7	1.85
64.4	122.9	1.86
80.5	125.7	1.9
131.2	125.5	1.43
151.6	121.2	1.18
177	105.5	0.84
182.1	111.1	0
173.2	119.9	0
152.5	134.6	0
130.7	142.4	0
116.3	144.3	0
68.3	150.4	0
52.7	149.5	0
39.4	140.6	0
37.6	108.6	1.4
36.3	89.2	1.79

22. Polygonal Load: "Polygonal Load 22"

Label	Polygonal Load 22
Load Type	Flexible
Area of Load	7397.82 ft ²
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
35.8	65.3	1.56
36.3	89.2	1.79
37.6	108.6	1.4
39.4	140.6	0
52.7	149.5	0
46.3	165.7	0
4.2	154	1.76
-23.8	146.2	1.71
-52	138.2	0
-38.3	65.3	0
-3	65.3	1.91
22.3	65.3	1.98

23. Polygonal Load: "Polygonal Load 23"

Label	Polygonal Load 23
Load Type	Flexible
Area of Load	9225.8 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-52	138.2	0
-23.8	146.2	1.71
4.2	154	1.76
46.3	165.7	0
-6.3	263.3	0
-29.9	251.9	1.35
-61.5035	236.831	1.26
-78.2802	228.8	0

24. Polygonal Load: "Polygonal Load 24"

Label	Polygonal Load 24
Load Type	Flexible
Area of Load	6789.47 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-78.2802	228.8	0
-61.5035	236.831	1.26
-29.9343	251.944	1.35
-6.3026	263.257	0
-71.7742	347.332	0
-80.4947	341.665	0.75
-109.843	322.594	0.75
-117.892	317.363	0

25. Polygonal Load: "Polygonal Load 25"

Label	Polygonal Load 25
Load Type	Flexible
Area of Load	4958.63 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-117.892	317.363	0
-109.843	322.594	0.75
-80.4947	341.665	0.75
-71.7742	347.332	0
-139.593	427.311	0
-144.077	423.325	0.29
-170.173	400	0.09
-172.027	398.323	0

26. Polygonal Load: "Polygonal Load 26"

Label	Polygonal Load 26
Load Type	Flexible
Area of Load	7645.56 ft ²
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
165.993	-16.469	0
216	35.6849	1.96
221.121	41.0263	1.99
231.502	51.8534	1.28
238.423	59.0715	0
182.051	111.132	0
176.958	105.526	0.84
166.649	94.5201	1.68
116.199	42.2116	0

Excavations

1. Excavation: "Excavation 1"

Elevation	870.1 ft
Installation Stage	Stage 1 = 0 d
Load at Bottom of Excavation	2.3 ksf
Load Installation Stage	Stage 1 = 0 d

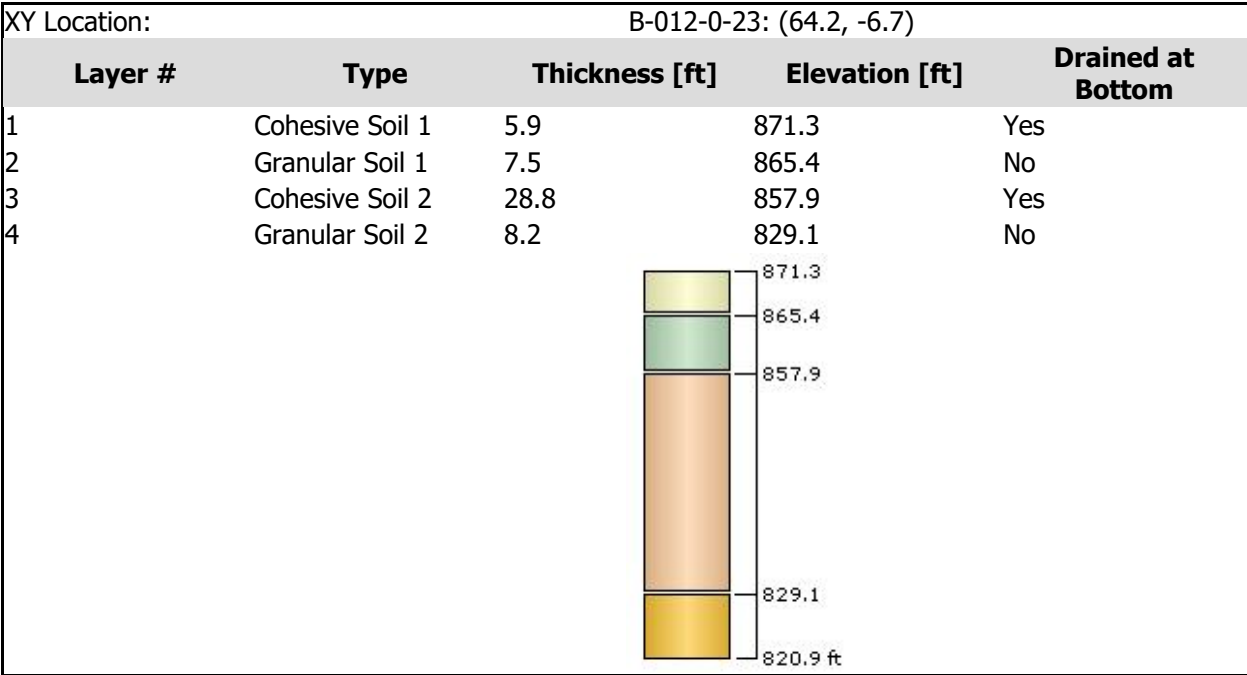
Coordinates

X [ft]		Y [ft]	
-6.88		8.5	
-6.9		-8.5	
-17.8		-19.3	
-9.6		-27.5	
4.6		-13.2	
4.6		-9	
4.6		13.2	
-11.4		29.2	
-19.5		21.1	

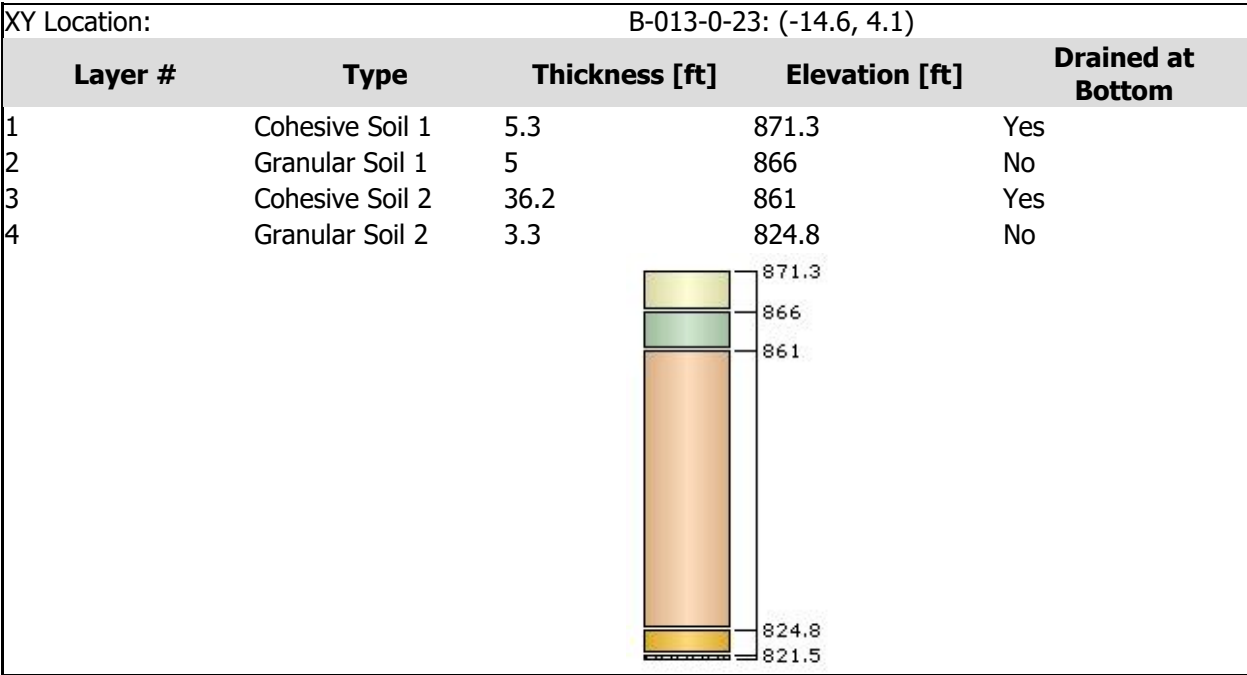
Soil Layers

Ground Surface Drained: Yes

B-012-0-23



B-013-0-23



Soil Properties

Property	Cohesive Soil 1	Granular Soil 1	Cohesive Soil 2	Granular Soil 2
Color				
Unit Weight [kips/ft ³]	0.12	0.118	0.125	0.135
Saturated Unit Weight [kips/ft ³]	0.12	0.118	0.135	0.14
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	1032	261	1866	416
Esur [ksf]	1032	261	1866	416
Primary Consolidation	Enabled	Disabled	Enabled	Disabled
Material Type	Non-Linear		Non-Linear	
Cc	0.22	-	0.05	-
Cr	0.045	-	0.007	-
e0	0.621	-	0.33	-
OCR	48	-	3	-
Cv [ft ² /d]	0.5	-	0.6	-
Cvr [ft ² /d]	0.5	-	0.6	-
B-bar	1	-	1	-
Undrained Su A [kips/ft ²]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1

Groundwater

Groundwater method

Piezometric Lines

Water Unit Weight

0.0624 kips/ft3

Generating excess pore pressure above water table

Piezometric Line Entities

ID	Elevation (ft)
1	860 ft

Query

Query Lines

Line #	Query Line Name	Start Location	End Location	Horizontal Divisions	Vertical Divisions
1	Query Line 1	-101.85, - 427.876	-12.6826, - 247.839	20	Auto: 63
2	Query Line 2	-12.6826, - 247.839	30.9, -53.4	20	Auto: 67
3	Query Line 3	30.9, -53.4	22.3, 65.3	20	Auto: 55
4	Query Line 4	22.3, 65.3	4.2, 154	20	Auto: 63
5	Query Line 5	4.2, 154	-95.3329, 332.023	20	Auto: 63
6	Query Line 6	-95.3329, 332.023	-155.677, 412.957	20	Auto: 63
7	Query Line 7	226.548, 46.6866	151.728, 113.471	20	Auto: 55
8	Query Line 8	151.728, 113.471	57.8009, 119.057	20	Auto: 55
9	Query Line 9	57.8009, 119.057	41.3048, 65.3	20	Auto: 63
10	Query Line 10	152.5, 134.6	96, 65.9	20	Auto: 55
11	Query Line 11	229.803, 67.0331	157.937, - 6.97519	20	Auto: 55
12	Query Line 12	115.53, - 7.66712	128.486, - 151.739	20	Auto: 55
13	Query Line 13	46.1, -53.4	98.7077, - 170.281	20	Auto: 55
14	Query Line 14	-28.3096, - 98.3769	144.266, - 98.3769	20	Auto: 67
15	Query Line 15	57.675, -0.35	112.261, - 4.8164	20	Auto: 55

SECTION 3

CEMETERY RD SHARED USE PATH BRIDGE OVER IR-270

SECTION 3: CEMETERY RD SHARED USE PATH BRIDGE OVER IR-270

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3.1. INTRODUCTION

3.1.1. Proposed Construction

It is our understanding that ODOT is planning the addition of a shared use path (SUP) which parallels the south side of Cemetery Rd in the city of Hilliard, Ohio to provide pedestrian connectivity between Britton Pkwy and Trueman Blvd. As part of the planned SUP, a new structure is proposed to carry the SUP over IR-270. Based on the Stage 2 Bridge FRA-270-1262 site plan developed by B&N dated May 5, 2025, the proposed structure will likely consist of a two-span, pre-stressed concrete beam bridge with a reinforced composite concrete deck. The proposed structure is planned to be approximately 244 ft in length (abutment to abutment) and 13-ft in width (toe to toe of curb) with semi-integral, mechanically stabilized earth (MSE) wall abutments and a cap-and-column type pier. The proposed substructures will likely be supported by a driven pile foundation consisting of 12-inch diameter and 14-inch diameter closed-ended cast-in-place reinforced concrete pipe piles (CIP piles) at the abutments and pier, respectively.

3.2. GEOTECHNICAL FINDINGS

The subsurface conditions encountered during NEAS's explorations are described in the following subsections and on each boring log presented in Appendix 1B. 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, results of historical explorations, and consideration of the geological history of the site.

3.2.1. Subsurface Conditions

3.2.1.1. Overburden Soil

At the proposed bridge site, two different materials were generally encountered below the existing pavement section, topsoil or ground surface. In general, the two different overburden materials consisted of either embankment "man-made" fill / potential fill soils or cohesive and non-cohesive natural glacial till soils. These materials and the general profile are further described below.

Fill / potential fill soils were encountered in each of the borings performed at the bridge site extending to depths ranging from 4.5 ft to 7.0 ft bgs (elevations 864.0 to 868.7 ft amsl). Based on laboratory testing results and a visual review of the samples obtained, the fill at the site is generally comprised of cohesive, fine-grained materials that are classified on the boring logs as either cohesive Sandy Silt (A-4a), Silt and Clay (A-6a) and Silty Clay (A-6b). The exception being a sample of non-cohesive, coarse-grained material encountered within boring B-008-0-23 that is classified on the boring log as Stone Fragments with Sand and Silt (A-2-4). With respect to the soil strength of the fill soils encountered, these soils can be described as having a consistency of soft to hard correlating to converted SPT-N (N_{60}) values between 3 and 42 blows per foot (bpf) and unconfined compressive strengths (estimated by means of hand penetrometer) between 0.75 and 4.5 tons per square foot (tsf). The natural moisture content of these soils ranged from 9 to 24 percent. Based on Atterberg Limits tests performed on representative samples of the natural cohesive material, the liquid and plastic limits ranged from 25 to 36 percent and from 16 to 19 percent, respectively.

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With respect to the non-cohesive fill sample, these soils are described as having a relative compactness of medium dense correlating to an N_{60} of 30 bpf and was determined to have a moisture content of 1 percent.

Naturally deposited glacial till soils were encountered underlying the fill soils in each of the borings performed at the bridge site. In general, the till can be divided into an upper and lower stratum based on the characteristics. The upper till stratum generally consisted of cohesive, fine-grained soils, though relatively thin layers (2.5-ft to 12.5-ft thick) of non-cohesive, coarse- and fine-grained material were encountered in this stratum. The lower till stratum generally consisted of non-cohesive, coarse- and fine-grained soils with the exception being a sample of cohesive, fine-grained material encountered at a depth of 38.5 ft bgs (elevation 832.1 ft amsl) in boring B-008-0-23. The natural till material extended to either borehole termination depth or bedrock (B-006-0-23 & B-008-0-23) at depths ranging from 53.0 to 86.5 ft bgs (elevations 787.8 to 799.0 ft amsl). The cohesive till encountered at the site is classified on the boring logs as Sandy Silt (A-4a) and can be described as having a consistency of stiff to hard correlating to N_{60} values between 13 and 88 bpf and unconfined compressive strengths (estimated by means of hand penetrometer) between 2.0 and 4.5 tsf. Natural moisture contents of the cohesive till ranged from 9 to 14 percent. Based on Atterberg Limits tests performed on representative samples of the natural cohesive material, the liquid and plastic limits ranged from 18 to 25 percent and from 13 to 16 percent, respectively. The non-cohesive till encountered at the site classified on the boring logs as Gravel and/or Stone Fragments with Sand (A-1-b), Gravel and/or Stone Fragments with Sand and Silt (A-2-4), Fine Sand (A-3), Coarse and Fine Sand (A-3a), non-cohesive Sandy Silt (A-4a) and non-cohesive Silt (A-4b) and can be described as having a relative compactness of medium dense to very dense correlating to N_{60} values between 13 bpf and SPT-N refusal (i.e., less than 6 inches of penetration over 50 blows). Natural moisture contents of the non-cohesive till soils ranged from 6 to 20 percent.

3.2.1.2. Groundwater

Groundwater measurements were taken during the boring drilling procedures and immediately following the completion of each borehole. Groundwater was observed during and/or upon completion of drilling in each of the borings performed as part of the referenced structure foundation exploration. Based on measurements at boring location, groundwater was encountered at depths ranging from 10.0 to 40.0 ft bgs (elevations 830.3 to 860.5 ft amsl).

It should be noted that groundwater is affected by many hydrologic characteristics in the area and may vary from those measured at the time of the exploration. The specific groundwater readings are included on the boring logs located within Appendix 1B.

3.2.1.3. Bedrock

Bedrock was encountered in two of the five borings performed at the bridge site (B-006-0-23 & B-008-0-23) and was classified as limestone. The bedrock surface was present at depths between 53.0 and 80.0 ft bgs (elevations 793.2 to 817.6 ft amsl). Rock coring was not performed in the referenced project borings once auger refusal was encountered, and therefore, bedrock core samples were not obtained.

3.3. BRIDGE ANALYSIS AND RECOMMENDATIONS

3.3.1. Soil Profile for Analysis

For deep foundation analyses purposes, each boring drilled for the proposed bridge was reviewed, and a generalized material profile was developed. Utilizing the generalized soil profile, engineering properties

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for each soil stratum were estimated based on their field (i.e., SPT N Values, etc.) and laboratory test (i.e., Atterberg Limits, grain size, etc.) results using correlations provided in published engineering manuals, research reports and guidance documents. Engineering soil properties were estimated for each individual classified layer per boring location. The developed soil profiles and estimated engineering soil properties for use in analysis (with cited correlation/reference material) are summarized within Tables 1 through 5 below.

Table 1: Soil Profile and Estimated Engineering Properties - At Boring B-006-0-23

Pedestrian Bridge over IR-270: Forward Abutment and RW-2, B-006-0-23					
Soil Description	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Setup Factor (f_{su})
Sandy Silt Depth (870.3 ft - 848.3 ft)	122	5000	355	28	1.5
Gravel with Sand and Silt Depth (848.3 ft - 843.3 ft)	122	-	-	40	1.2
Sandy Silt Depth (843.3 ft - 838.3 ft)	132	5150	365	28	1.5
Gravel with Sand and Silt Depth (838.3 ft - 832 ft)	140	-	-	42	1.2
Gravel with Sand Depth (832 ft - 827 ft)	135	-	-	41	1.0
Sandy Silt Depth (827 ft - 822 ft)	135	-	-	36	1.2
Gravel with Sand and Silt Depth (822 ft - 817 ft)	140	-	-	42	1.2
Gravel with Sand Depth (817 ft - 812 ft)	140	-	-	47	1.0
Gravel with Sand and Silt Elevation (812 ft - 807 ft)	140	-	-	44	1.2
Fine Sand Elevation (807 ft - 802 ft)	140	-	-	39	1.2
Sandy Silt Elevation (802 ft - 790.3 ft)	140	-	-	36	1.2
Notes: 1. Values calculated per ODOT GDM Section 404/1304 and/or ODOT BDM Table 305-2.					

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Table 2: Soil Profile and Estimated Engineering Properties - At Boring B-007-0-23

Pedestrian Bridge over IR-270: Forward Abutment and RW-2, B-007-0-23					
Soil Description	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Setup Factor (f_{su})
Sandy Silt Depth (870.5 ft - 866 ft)	125	1850	185	25	1.5
Sandy Silt Depth (866 ft - 860.3 ft)	125	-	-	34	1.2
Silt Depth (860.3 ft - 858.5 ft)	132	-	-	39	1.5
Sandy Silt Depth (858.5 ft - 843.5 ft)	128	3650	290	27	1.5
Coarse and Fine Sand Depth (843.5 ft - 837.8 ft)	130	-	-	38	1.0
Gravel with Sand Depth (837.8 ft - 832.2 ft)	128	-	-	38	1.0
Coarse and Fine Sand Depth (832.2 ft - 827.2 ft)	130	-	-	38	1.0
Sandy Silt Depth (827.2 ft - 822.2 ft)	132	-	-	37	1.2
Gravel with Sand and Silt Elevation (822.2 ft - 817.2 ft)	132	-	-	39	1.2
Sandy Silt Elevation (817.2 ft - 812.2 ft)	132	-	-	36	1.2
Gravel with Sand Elevation (812.2 ft - 807.2 ft)	132	-	-	40	1.0
Sandy Silt Elevation (807.2 ft - 802.2 ft)	132	-	-	36	1.2
Coarse and Fine Sand Elevation (802.2 ft - 799 ft)	132	-	-	38	1.0
Notes: 1. Values calculated per ODOT GDM Section 404/1304 and/or ODOT BDM Table 305-2.					

Table 3: Soil Profile and Estimated Engineering Properties - At Boring B-008-0-23

Pedestrian Bridge over IR-270: Pier 1, B-008-0-23					
Soil Description	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Setup Factor (f_{su})
Gravel with Sand and Silt Depth (870.25 ft - 867.25 ft)	125	-	-	40	1.2
Sandy Silt Depth (867.25 ft - 863.15 ft)	112	1750	175	25	1.5
Sandy Silt Depth (863.15 ft - 840.05 ft)	130	4750	345	28	1.5
Gravel with Sand and Silt Depth (840.05 ft - 833.75 ft)	132	-	-	39	1.2
Sandy Silt Depth (833.75 ft - 828.75 ft)	132	5350	375	28	1.5
Sandy Silt Depth (828.75 ft - 823.75 ft)	132	-	-	36	1.2
Gravel with Sand and Silt Depth (823.75 ft - 817.55 ft)	140	-	-	40	1.2
Notes: 1. Values calculated per ODOT GDM Section 404/1304 and/or ODOT BDM Table 305-2.					

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Table 4: Soil Profile and Estimated Engineering Properties - At Boring B-009-0-23

Pedestrian Bridge over IR-270: Rear Abutment and RW-1, B-009-0-23					
Soil Description	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Setup Factor (f_{su})
Silt and Clay Depth (870.6 ft - 866.1 ft)	100	350	35	19	1.5
Silt and Clay Depth (866.1 ft - 863.6 ft)	108	875	85	22	1.5
Sandy Silt Depth (863.6 ft - 858.6 ft)	125	3250	270	27	1.5
Coarse and Fine Sand Depth (858.6 ft - 856.1 ft)	132	-	-	40	1.0
Sandy Silt Depth (856.1 ft - 827.3 ft)	125	2700	240	26	1.5
Gravel with Sand and Silt Depth (827.3 ft - 822.3 ft)	132	-	-	40	1.2
Gravel with Sand Depth (822.3 ft - 799.1 ft)	140	-	-	40	1.0
Notes: 1. Values calculated per ODOT GDM Section 404/1304 and/or ODOT BDM Table 305-2.					

Table 5: Soil Profile and Estimated Engineering Properties - At Boring B-010-0-23

Pedestrian Bridge over IR-270: Rear Abutment and RW-1, B-010-0-23					
Soil Description	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Setup Factor (f_{su})
Silty Clay Depth (873.6 ft - 866.6 ft)	120	1500	150	23	1.75
Sandy Silt Depth (866.6 ft - 859.1 ft)	125	2500	225	26	1.5
Coarse and Fine Sand Depth (859.1 ft - 856.6 ft)	125	-	-	39	1.0
Gravel with Sand and Silt Depth (856.6 ft - 854.1 ft)	130	-	-	40	1.2
Sandy Silt Depth (854.1 ft - 833.1 ft)	125	2900	250	26	1.5
Gravel with Sand and Silt Depth (833.1 ft - 820.3 ft)	132	-	-	40	1.2
Gravel with Sand Depth (820.3 ft - 805.3 ft)	132	-	-	40	1.0
Coarse and Fine Sand Depth (805.3 ft - 800.3 ft)	125	-	-	33	1.0
Gravel with Sand Elevation (800.3 ft - 787.1 ft)	140	-	-	40	1.0
Notes: 1. Values calculated per ODOT GDM Section 404/1304 and/or ODOT BDM Table 305-2.					

It should be noted, the soil engineering properties presented in the tables above designate each soil layer as either cohesive or non-cohesive based on the Unified Soil Classification System (USCS) (i.e., greater than 50% passing No. 200 sieve, plasticity index greater than 4 percent criterion for cohesive soils) rather than the AASHTO/ODOT classification presented on the boring logs. This USCS-based interpretation was applied specifically for pile design because of the governing methods required by AASHTO LRFD Articles 10.7.3.8.6b, 10.7.3.8.6e, and 10.7.3.8.6f. These AASHTO LRFD articles specify using the α -method and Nordlund/Thurman methods per FHWA GEC-12 (Hannigan et al., 2016) which states USCS is better suited to deep foundation design and will be the classification system utilized in FHWA GEC-12.

3.3.2. Pile Foundation Analysis

Based on the determined soil profile and our estimated engineering soil properties, a pile analysis was performed using the computer program *GRLWEAP 14* (as noted in Appendix 3A) to determine the estimated geotechnical pile length needed to achieve the UBV required to support the design load for a single pile at each substructure (GRLWEAP results included within Appendix 3A). For the purposes of this

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report and our analysis, the term 'geotechnical pile length' has been assumed to represent the length of pile from bottom of pile cap (assumed pile cap bearing elevation) to the depth at which the required ultimate bearing value (UBV) is obtained. Based on the soil profile encountered at the site, it is our opinion that pile resistances obtained during dynamic testing (driving) may be reduced due to the potential for soil disturbance (development of high pore water pressure) near the pile perimeter. This disturbance could cause piles to potentially drive easily or "run" for extended depths and initial driving resistances may not reach the indicated target UBV utilizing the estimated pile lengths. This reduced-resistance value obtained at the end of driving the estimated pile length is designated as the End of Initial Driving resistance or EOID. If the EOID is significantly different than the required UBV, it may be necessary to let the piles "set up" (reduction of pore water pressure in the soils adjacent to the pile) for an established period of time. To estimate the potential effects of this disturbance during driving, the setup factors are presented in Table 2, Table 3 and Table 4 of Section 3.3.1. of this report are used to estimate driving strength losses as well as the side resistance expected to gain following the setup period at the Rear Abutment, Pier 1 and Forward Abutment locations, respectively.

The UBV and EOID values are determined in accordance with Sections 305.3.2.4 and 305.3.5.9 of the ODOT BDM. The UBV is determined by dividing the total factored load for the highest loaded pile at each substructure by the appropriate driven pile resistance factor, while the EOID is determined by subtracting the amount of side resistance expected to gain from soil setup from the UBV value. The amount of side resistance expected to gain from soil setup is taken as the difference between the side resistance obtained in ultimate (post setup) conditions and the side resistance obtained during driving (dynamic) conditions at the determined geotechnical pile length. It is recommended that the piles for the referenced project be installed according to ODOT's Construction and Material Specifications (CMS) 507 and CMS 523, and therefore, a driven pile resistance factor of 0.7 should be used. For our analysis it is assumed that the proposed pile cap elevations will match those shown in the proposed bridge FRA-270-1262 SUP site plan developed by B&N dated May 5, 2025.

The results for our analysis including the estimated skin friction (Rs) and pile tip bearing (Rp) for ultimate and during driving conditions are summarized for the proposed structure in Table 6 below (GRLWEAP results included within Appendix 3A). The referenced table also includes 1) the required geotechnical pile length in ultimate conditions for 12-inch diameter (abutments) and 14-inch diameter (pier) CIP piles driven to the respective UBV per substructure location; 2) the length of driven pile required in driving conditions for 12-inch diameter (abutments) and 14-inch diameter (pier) CIP piles driven to the respective UBV per substructure location; and, 3) the estimated difference in pile length between a pile in ultimate and driving conditions.

Table 6: Deep Foundation Analysis Summary

Pile Type	Ultimate Conditions		Driving Conditions		Pile Length Difference Ultimate vs. Driving Conditions (ft)	End of Initial Driving Value ⁽³⁾ (kips)	Setup Factor (f_{su})
	Geotechnical Pile Length ⁽¹⁾ (ft)	Ultimate Bearing Value ⁽²⁾ (kips)	Driven Pile Length ⁽¹⁾ (ft)	Bearing Value During Driving ⁽²⁾ (kips)			
FRA-Cemetery Rd-SUP Bridge - Rear Abutment, B-009-0-23							
12-inch CIP	56.1	250	56.1	250	0.0	250	1.0
FRA-Cemetery Rd-SUP Bridge - Pier 1, B-008-0-23							
14-inch CIP	26.4	323	26.4	323	0.0	323	1.0
FRA-Cemetery Rd-SUP Bridge - Forward Abutment, B-007-0-23							
12-inch CIP	57.6	250	59.7	250	2.1	234	1.1
Notes: 1. The length of pile from bottom of pile cap (pile cap bearing elevation) to the depth at which the required UBV is obtained. 2. Resistance factor for driven piles, dynamic analysis and static load test methods (BDM Table 305-1) for piles installed according to C&MS 507 using dynamic test methods according to C&MS 523 has not been applied to values calculated. 3. EOID is based on driving resistance obtained at the indicated geotechnical pile length.							

3.3.3. Pile Drivability

NEAS's pile drivability evaluation estimated a Delmag D19-42 diesel hammer to determine if the pile type or size being considered would be either overstressed (i.e., compressive stresses experienced by pile during driving are greater than 90% of the yield strength of the steel) or encounter driving refusal (i.e., hammer blow counts higher than 100 blows per foot) at any time during pile installation. The results of the evaluation indicated that the ODOT preferred CIP pile sizes would be overstressed during the pile installation process at the each substructure location, based on: 1) a minimum wall thickness of 0.25-inches; 2) the use of ASTM A 252 Grade 3 steel piles; 3) the use of a Delmag D19-42 (minimum rated energy of 44,000 ft-lbs); and, 4) our developed model used in the computer program *GRLWEAP 14* (as noted in Appendix 3A) by GRL Engineers, Inc. Based on the results of our drivability analysis, to prevent potential overstressing of proposed pile foundations during pile installation we recommend that ASTM A 252 Grade 3 steel piles with a minimum pile wall thickness of 0.312-inches be utilized for the proposed structure foundations. *GRLWEAP* results for each substructure location are included within Appendix 3A.

It should be noted that the driving resistance of CIP piles through soils encountered at the bridge site is expected to be high. Drivability is difficult to assess quantitatively as the field test results (i.e., SPT N_{60} values, pocket penetrometer values, etc.) tend to be very high. Furthermore, pile drivability is highly reliant upon the specific equipment used in construction; therefore, it is recommended that the contractor provide an analysis to demonstrate that the equipment and pile combination planned for use is capable of obtaining the UBV without over-stressing the piles.

3.3.4. Pile Foundation Recommendations

Based on our evaluation of the subsurface conditions and our geotechnical engineering analysis for the proposed FRA-270-1262 SUP bridge, it is our opinion that the bridge foundations can be supported on driven friction CIP piles seated within the stiff to hard/medium dense to very dense natural subsurface material encountered at the site.

We recommend that a driven pile foundation be used for support for the proposed bridge abutment and pier foundations. New 12-inch diameter (abutments) or 14-inch diameter (pier) CIP piles consisting of ASTM A 252, Grade 3 steel with pile wall thicknesses of at least 0.312 inches are recommended to be installed in accordance with Sections 507 and 523 of ODOT's CMS. During driving conditions and if driven to the UBVs indicated in Table 6 of this report, it is anticipated that the newly driven CIP piles would not "run" for extended depths any of the proposed substructures (i.e., run lengths greater than 10 ft). Therefore, pile/soil setup will not be utilized during the installation process at this structure and it is recommended that the proposed piles be driven to the required UBV. It is recommended that all applicable plan notes provided in Section 606.2 be included in the plans.

When new piles are installed in accordance with referenced construction specifications utilizing the referenced method as specified in the ODOT BDM at the proposed substructure locations, 12-inch diameter CIP piles driven to the indicated UBV at the abutments may be used to support a total factored load (single pile) of 175 kips, while 14-inch diameter CIP piles driven to the indicated UBV at the pier may be used to support a total factored load (single pile) of 226 kips. For piles driven to the indicated UBVs, pile tip elevations are estimated to range from 827 to 840 ft amsl across the bridge site.

Pile lengths based on: 1) our Deep Foundation Analysis (presented in Section 3.3.2); and, 2) the "Estimated Length" and "Order Length" definitions and formulas presented in Section 305.3.5.2 of the ODOT BDM, are presented in Tables 7 below.

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Table 7: Estimated Pile Lengths

Pile Type	Bottom of Pile Cap Elevation (ft amsl)	Geotechnical Pile Length (ft)	Geotechnical Pile Tip Elevation (ft amsl)	Estimated Pile Length ⁽¹⁾ (ft)	Order Length ⁽¹⁾ (ft)
FRA-Cemetery Rd-SUP Bridge - Rear Abutment, B-009-0-23					
12-inch CIP	883.4	56.1	827.3	60	65
FRA-Cemetery Rd-SUP Bridge - Pier 1, B-008-0-23					
14-inch CIP	866.0	26.4	839.6	30	35
FRA-Cemetery Rd-SUP Bridge - Forward Abutment, B-007-0-23					
12-inch CIP	882.9	59.7	823.2	65	70
Notes: 1. Based on definitions and formulas presented in Section 303.3.5.2 of the 2020 BDM.					

3.3.5. Settlement and Downdrag

At the abutment locations, it is our understanding that new approach embankments and MSE wall abutments are planned as part of the proposed bridge construction. New embankment loading has the potential to cause long-term settlement which may cause downdrag loading on the proposed bridge pile foundations that should be considered in design. Therefore, it is recommended that all applicable plan notes provided in Section 605.1 be included in the plans. See Section 3.4.2. of this report regarding recommendations for coordination of waiting times following embankment construction with pile driving operations.

3.4. RETAINING WALL ANALYSIS AND RECOMMENDATIONS

3.4.1. Retaining Wall Design Assumptions

As the proposed abutment retaining walls (RW-1 & RW-2) are planned as MSE type, ODOT's BDM and AASHTO's LRFD BDS dictate analysis parameters and design minimums/constraints to be used in the analysis and design process. The referenced parameters and design minimums/constraints that were significant to our analyses consist of the following:

- Minimum reinforcement strap lengths of proposed MSE walls are 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 section of wall being analyzed, per ODOT's BDM section 307.4-A. The exception being at sections of the wall where the strap lengths are constrained by the wall's back-to-back configuration;
- 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 the requirements of ODOT Construction & Materials Specifications Section 204.07 (CMS 204.07);
- Maximum allowable differential settlement in the longitudinal direction is 1%. (BDM Section 307.1.6); and,
- Reinforced Zone and Retained Fill soils will meet the minimum design soil parameters per Table 840.04-1 of the ODOT Supplemental Specification 840 (SS-840) as shown in Table 8 below.

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Table 8: 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
Notes: 1. Table reproduced from Section 840.04 - A-1 of ODOT's SS 840.				

With respect to design constraints and assumptions specific to the RW-1 & RW-2 MSE walls, the geometry of the proposed walls (i.e., exposed wall heights, existing ground elevations, proposed final grade behind/at the toe of the wall, etc.) is assumed to be consistent with that shown in the proposed MSE wall plans developed by B&N provided via email on October 23, 2025.

3.4.1.1. Retaining Wall - Soil Profile for Settlement Analysis

For proposed retaining settlement analysis purposes, the generalized material profile and estimated engineering properties developed for the proposed Shared Use Path bridge FRA-270-1262 substructures, presented in Section 3.1.1. of this report, were assumed to also be representative of the subsurface conditions at the proposed RW-1 and RW-2 locations. Specifically, soil properties estimated at the proposed rear abutment were utilized for RW-1 analyses while soil properties estimated at the proposed forward abutment were utilized for RW-2 analyses. Settlement parameters (with sited correlation/reference material) developed for RW-1 (rear abutment) and RW-2 (forward abutment) are presented below within Table 9. Thicknesses of each soil layer referenced in Table 9 assumed in our settlement analysis by boring location are presented in Table 10.

Table 9: Settlement Parameters for Analysis - Retaining Walls 1 and 2

Retaining Walls 1 and 2: Settlement Analysis B-007-0-23, B-009-0-23 & B-010-0-23								
Soil Description	Unit Weight (pcf)	Elastic Modulus ⁽¹⁾ (psf)	Poissons Ratio ⁽¹⁾ , ν	Void Ratio e_o	Compression Index ⁽²⁾ , C_c	Recompression Index ⁽³⁾ , C_r	OCR ⁽⁴⁾	Coeff. of Consol. ⁽⁵⁾ , C_v (ft ² /day)
Cohesive Soil 1	120	392000	0.40	0.649	0.19	0.039	11.5	0.6
Granular Soil 1	118	321000	0.30	-	-	-	-	-
Cohesive Soil 2 ⁽⁶⁾	135	1452000	0.45	0.330	0.05	0.007	3.0	0.6
Granular Soil 2	135	891000	0.40	-	-	-	-	-
Notes: 1. Values interpreted from 2017 AASHTO LRFD BDS Table C10.4.6.3-1 2. Values calculated from Kulhawy and Mayne, 1990, Equation 6-6. 3. Values calculated from Kulhawy and Mayne, 1990, Equation 6-9. 4. Values interpreted from Mayne and Kemper, 1988, Figure 7. 5. Values interpreted from FHWA GEC No. 5, Boeckmann, et al., 2016, Figure 6-37. 6. Based on laboratory testing results of sample ST-7 in boring B-008-0-23.								

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Table 10: Settlement Profile for Analysis per Boring Locations

Retaining Walls 1 and 2: Settlement Profile for Analysis			
Soil Description	Layer Thickness per Boring Location (ft)		
	B-007-0-23	B-009-0-23	B-010-0-23
Cohesive Soil 1	5.0	7.0	7.0
Granular Soil 1	7.0	2.5	12.5
Cohesive Soil 2 ⁽⁶⁾	15.0	33.8	21.0
Granular Soil 2	44.5	28.2	46.0

3.4.2. External Stability

Based on our estimated engineering soil properties, the developed generalized profile and the retaining wall design assumptions provided in Section 3.3.1. of this report, external stability analyses of the proposed RW-1 and RW-2 were performed. Specifically, worse-case soil conditions from borings B-009-0-23 and B-010-0-23 were used for external stability at the RW-1 location, while worse case soil conditions from boring B-007-0-23 were used for global stability at the RW-2 location. Groundwater levels for external stability analyses were conservatively assumed to be at the top of leveling pad elevation. External stability was evaluated at each leveling pad step along both the proposed RW-1 and RW-2 alignments. Each section was evaluated for resistance to bearing pressure, sliding forces and overturning at the Strength Limit State in accordance with Section 11.10.5 of the AASHTO's LRFD BDS. The capacity to demand ratios (CDRs) calculated for the referenced sections with respect to bearing, sliding and overturning, as well as the calculated factored bearing pressures and resistances for RW-1 and RW-2 are presented in Tables 11 and 12, respectively. (External Stability and Bearing Resistance Calculation Results can be found in Appendix 3B).

Table 11: External Stability Analysis Summary – Retaining Wall 1

Dimensions							
Retaining Wall 1							
Design Wall Height (ft)	14.5	17.2	20.0	22.7	25.5	28.5	
Length of Reinforcement (ft)	14.0	14.0	14.0	18.0	18.0	20.0	
Length of Reinf. To Height Ratio	1.0	0.8	0.7	0.8	0.7	0.7	
Approximate Station ⁽¹⁾	0+00.00 - 0+09.17 1+52.61 - 1+61.78	0+09.17 - 0+18.33 1+43.44 - 1+52.61	0+18.33 - 0+27.50 1+34.27 - 1+43.44	0+27.50 - 0+37.50 1+24.27 - 1+34.27	0+37.50 - 0+47.50 1+14.27 - 1+24.27	0+47.50 - 1+14.27	
Additional Undercut (ft)	N/A	N/A	N/A	N/A	N/A	N/A	1.0 ⁽⁴⁾
Capacity Demand Ratio (CDR)							
Bearing Capacity	2.33	1.82	1.44	1.41	1.17	0.55	1.51
Overturning / Eccentricity	3.48	2.54	1.92	2.50	2.01	2.00	2.00
Sliding	1.62	1.39	1.21	1.39	1.25	1.19	1.44
Factored Bearing Resistance (ksf)	6.8	6.6	6.5	6.7	6.6	3.4	9.5
Service Bearing Pressure (ksf)	2.1	2.6	3.3	3.5	4.1	4.6	
Factored Bearing Pressure (ksf)	2.9	3.6	4.5	4.7	5.6	6.3	

Notes:

1. Stationing in reference to Retaining Wall 1 alignment.
2. Strap lengths are minimum per plan.
3. Values based on weakest material within 1.5B below bearing elevation.
4. Undercut value is in addition to the ODOT standard 1.0-ft foundation preparation layer.

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Table 12: External Stability Analysis Summary – Retaining Wall 2

Dimensions						
Retaining Wall 2						
Design Wall Height (ft)	13.8	16.6	19.3	22.1	24.9	28.0
Length of Reinforcement (ft)	14.0	14.0	14.0	18.0	18.0	20.0
Length of Reinf. To Height Ratio	1.0	0.8	0.7	0.8	0.7	0.7
Approximate Station ⁽¹⁾	0+00.00 - 0+09.00 1+47.78 - 1+56.78	0+09.00 - 0+18.00 1+38.78 - 1+47.78	0+18.00 - 0+27.00 1+29.78 - 1+38.78	0+27.00 - 0+36.00 1+20.78 - 1+29.78	0+36.00 - 0+45.00 1+11.78 - 1+20.78	0+45.00 - 1+11.78
Capacity Demand Ratio (CDR)						
Bearing Capacity	2.99	2.34	1.86	1.85	1.53	2.55
Overturning / Eccentricity	3.81	2.71	2.05	2.63	2.10	2.07
Sliding	1.86	1.58	1.38	1.57	1.40	2.02
Factored Bearing Resistance (ksf)	8.2	8.1	8.0	8.5	8.3	15.6
Service Bearing Pressure (ksf)	2.0	2.5	3.1	3.4	4.0	4.5
Factored Bearing Pressure (ksf)	2.7	3.5	4.3	4.6	5.4	6.1
Notes: 1. Stationing in reference to Retaining Wall 2 alignment. 2. Strap lengths are minimum per plan.						

As indicated in Table 11 above, a CDR of less than 1.0 was calculated for the tallest wall height section of RW-1 (i.e., STA.0+47.50 to 1+14.27) when bearing on the material encountered at an elevation just below the typical 1.0-ft foundation preparation layer. Therefore, it is recommended that the undercut for the foundation preparation layer extend an additional 1.0-ft below typical (i.e., 2.0-ft total undercut) to extend below the weaker soils and bear on more competent material. Assuming RW-1 will bear on the lower more competent soils, a CDR of greater than 1.0 for bearing, overturning and sliding was calculated when considering the weakest material within 1.5B below the recommended bottom of foundation preparation layer elevation.

3.4.3. Global Stability

For purposes of evaluating the stability of the proposed RW-1 and RW-2, NEAS reviewed cross-sections along the length of the proposed retaining walls to determine the subsurface conditions that posed the greatest potential for slope instability. In general, cross-sections along the proposed wall alignments were reviewed to determine the sections that 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, proposed/existing grades behind and in front of the wall, weak and/or thick soil layer, etc.). Based on our review of the available information at the referenced location and the associated soil properties, one (1) cross-section for each retaining wall was estimated to be most "critical" and were analyzed for global stability. The cross-sections analyzed for global stability for each of the proposed walls consisted of the maximum wall-height sections immediately behind the proposed stub abutments at STA. 00+61.3 (RW-1 alignment) for RW-1 and STA. 00+58.8 (RW-2 alignment) for RW-2.

For the indicated cross-sections, NEAS developed a representative cross-sectional model to use as the basis for global stability analysis. The models were developed from NEAS's interpretation of the available information which included: 1) the proposed MSE wall plans developed by B&N provided via email on October 23, 2025; 2) a live load surcharge of 250 pounds per square foot (psf) accounting for traffic induced loads; and, 3) test borings and laboratory data developed as part of this project. With respect to the soil's engineering properties, the provided generalized soil profile and estimated engineering properties presented in Section 3.3.1. of this report were used in our analysis. Specifically, worse-case soil conditions from borings B-009-0-23 and B-010-0-23 were used for global stability at the RW-1 location, while worse case

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soil conditions from boring B-007-0-23 were used for global stability at the RW-2 location. Groundwater levels in our global stability analyses were conservatively assumed to be at the top of leveling pad elevation.

The above referenced global stability models were analyzed for long-term (Effective Stress) and short-term (Total Stress) slope stability utilizing the software entitled *Slide2* by Rocscience, Inc. Specifically, the Spencer analysis method was used to calculate a factor of safety (FOS) for circular and non-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 1.5 which equates to an AASHTO resistance factor less than 0.65 (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 0.65 or lower was targeted as the MSE wall abutments contain or support a structural element of the proposed bridge. Based on our slope stability analyses for the referenced MSE wall sections, the minimum slope stability factor was estimated to be about 1.81 (0.55 resistance factor). Graphical outputs of the slope stability program (cross-sectional model, calculated safety factor, and critical failure plane) are presented within Appendix 3C.

3.4.4. Settlement Analysis

In order to estimate the maximum total and differential settlement that could result within the subsurface soils supporting the proposed MSE retaining walls (RW-1 and RW-2), NEAS reviewed: 1) the proposed MSE wall plans developed by B&N provided via email on October 23, 2025; 2) Service Limit State loading conditions; and, 3) the generalized subsurface profile and Settlement Parameters for Analysis provided in Section 3.4.1.1. of this report. Utilizing this information and the software entitled *Settle3* by Rocscience Inc., a settlement model was developed and analyzed for both elastic (immediate) and consolidation (long term) settlement. The settlement model developed for RW-1 and the rear abutment was utilized to evaluate potential settlement impact on the proposed Bridge FRA-270-1262 SUP forward abutment. It is our opinion that the settlement at the rear abutment and RW-1 location will be generally representative of the settlement conditions at the forward abutment and RW-2 locations based on: 1) the relative uniformity of the soil stratification and estimated soil compressibility characteristics across the site; 2) the similar geometry, including length of wall and proposed fill heights, of the proposed MSE wall abutments; and, 3) the similar planned construction sequence for both the rear and forward abutment. The Outputs of the settlement analysis program are presented within Appendix 3D.

Based on our analysis, the estimated maximum total settlement that could occur along the length of proposed RW-1 and RW-2 as a result of the induced MSE retaining wall and embankment loads is estimated to be about 4.5 inches with about 2.9 inches of the total settlement estimated to be long-term (consolidation). The maximum differential settlement along the length of the proposed RW-1 and RW-2 is estimated to be less than 1 percent. The exception being at locations immediately in front of and behind the backside of the proposed abutments where the proposed fill heights rapidly vary from only about 10 to 12 ft of new fill beneath the abutments to about 23 ft of new fill proposed immediately behind the abutments. Therefore, it is recommended that slip joints be utilized along both wall alignments to limit any potential impacts of differential settlement to RW-1 and RW-2. Furthermore, it is anticipated that approximately 90 percent of the estimated long-term settlement (i.e., 2.6 inches) will be complete within the first 30 days following the application of new retaining wall and approach embankment loading with less than 0.4 inches of long-term settlement anticipated following this time period.

With respect to potential downdrag effects induced by these settlement magnitudes on the proposed pile foundations at the rear and forward abutment location, it is recommended that the plan note provided in ODOT BDM Section 605.1-4 be included in the plans and that a waiting period of 30 calendar days

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following the application of new retaining wall and/or embankment loading be implemented to prevent any additional loading on the piles as a result of downdrag. Additionally, to monitor and verify the progress of anticipated settlements it is recommended that two settlement platforms be utilized at both the rear and forward abutment. Specifically, at each abutment it is recommended that one platform be located about 10 to 20 ft behind the new abutment and one be located about 100 to 150 ft behind the abutment, both in the areas of the tallest height of new fill. It is recommended that ODOT OGE note for “Item 203, Special-Settlement Platform” be included in the plan set for this work.

APPENDIX 3A

GRLWEAP ANALYSIS

REAR ABUTMENT

PILE LENGTH

Boring Used: B-009-0-23

Starting Elevation (0.0 ft) = 864.5 ft amsl =

866.0 ft amsl (Top of Leveling Pad Ele.) - 1.5 ft (6-inch leveling pad + 1.0-ft foundation prep layer)

Assumed Groundwater Depth = 0.0 ft (Ele. 864.5 ft amsl)

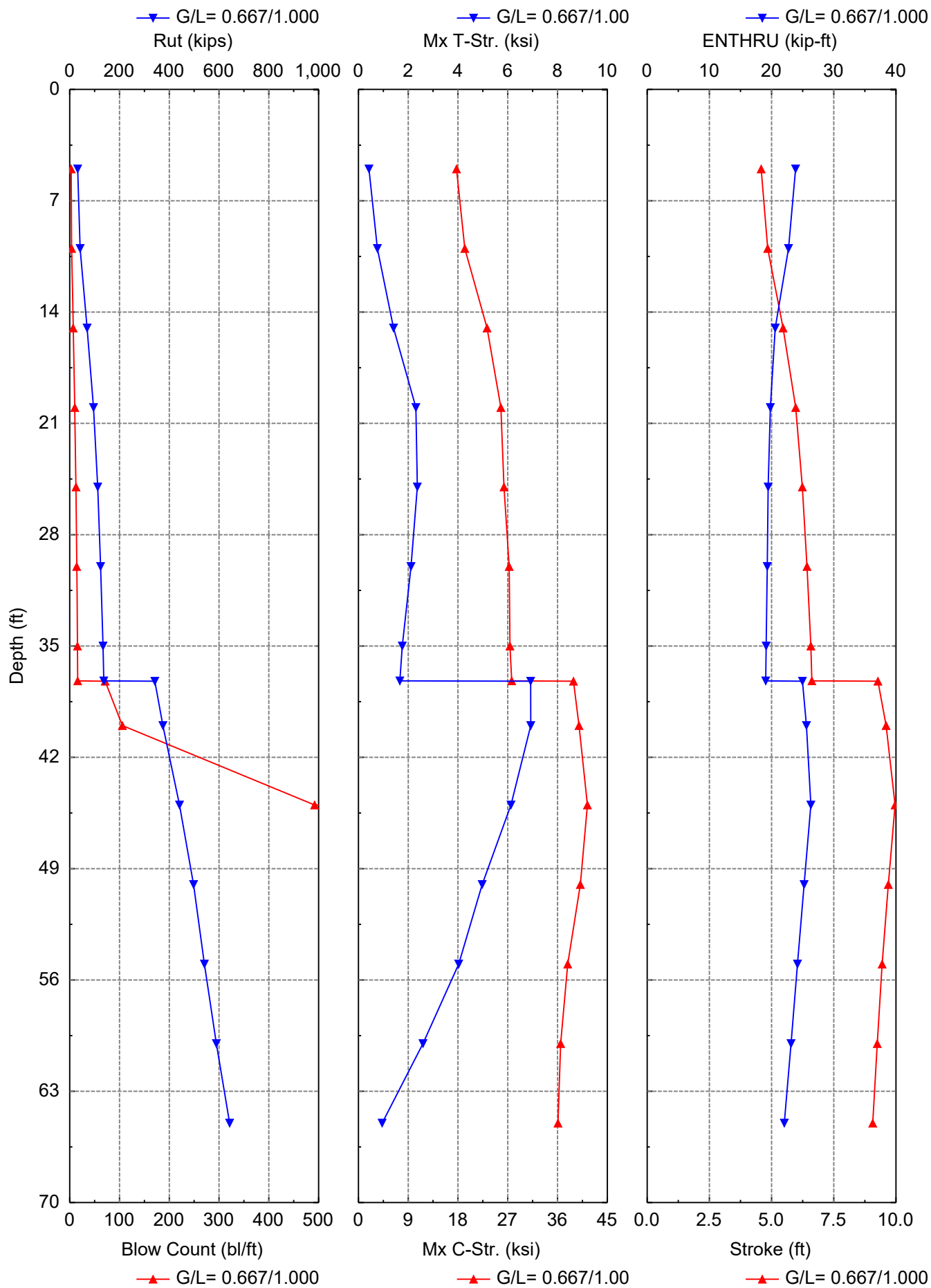
Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 0.667/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	32.2	8.8	23.3	2.8	17.724	0.422	4.58	23.9	D 19-42
10.0	41.6	22.5	19.1	3.7	19.237	0.762	4.84	22.7	D 19-42
15.0	69.9	50.8	19.1	7.0	23.250	1.410	5.46	20.6	D 19-42
20.0	95.8	76.7	19.1	10.3	25.748	2.310	5.97	19.8	D 19-42
25.0	112.3	93.3	19.1	12.6	26.322	2.366	6.23	19.4	D 19-42
30.0	124.0	104.9	19.1	14.1	27.263	2.114	6.42	19.3	D 19-42
35.0	133.9	114.9	19.1	15.5	27.396	1.760	6.58	19.1	D 19-42
37.2	137.0	117.9	19.1	16.0	27.714	1.663	6.62	19.1	D 19-42
37.2	342.0	117.9	224.1	71.2	38.886	6.919	9.27	25.0	D 19-42
40.0	374.5	132.1	242.4	105.8	39.887	6.926	9.60	25.6	D 19-42
45.0	440.9	163.7	277.3	492.0	41.398	6.128	9.96	26.3	D 19-42
50.0	497.6	202.4	295.3	9999.0	40.118	4.974	9.69	25.2	D 19-42
55.0	541.1	245.8	295.3	9999.0	37.845	4.029	9.44	24.2	D 19-42
60.0	589.3	294.1	295.3	9999.0	36.563	2.593	9.25	23.1	D 19-42
65.0	642.3	347.1	295.3	9999.0	36.079	0.952	9.06	22.0	D 19-42

Refusal occurred; no driving time output possible.

Starting Elevation (0.0 ft) = 864.5 ft amsl

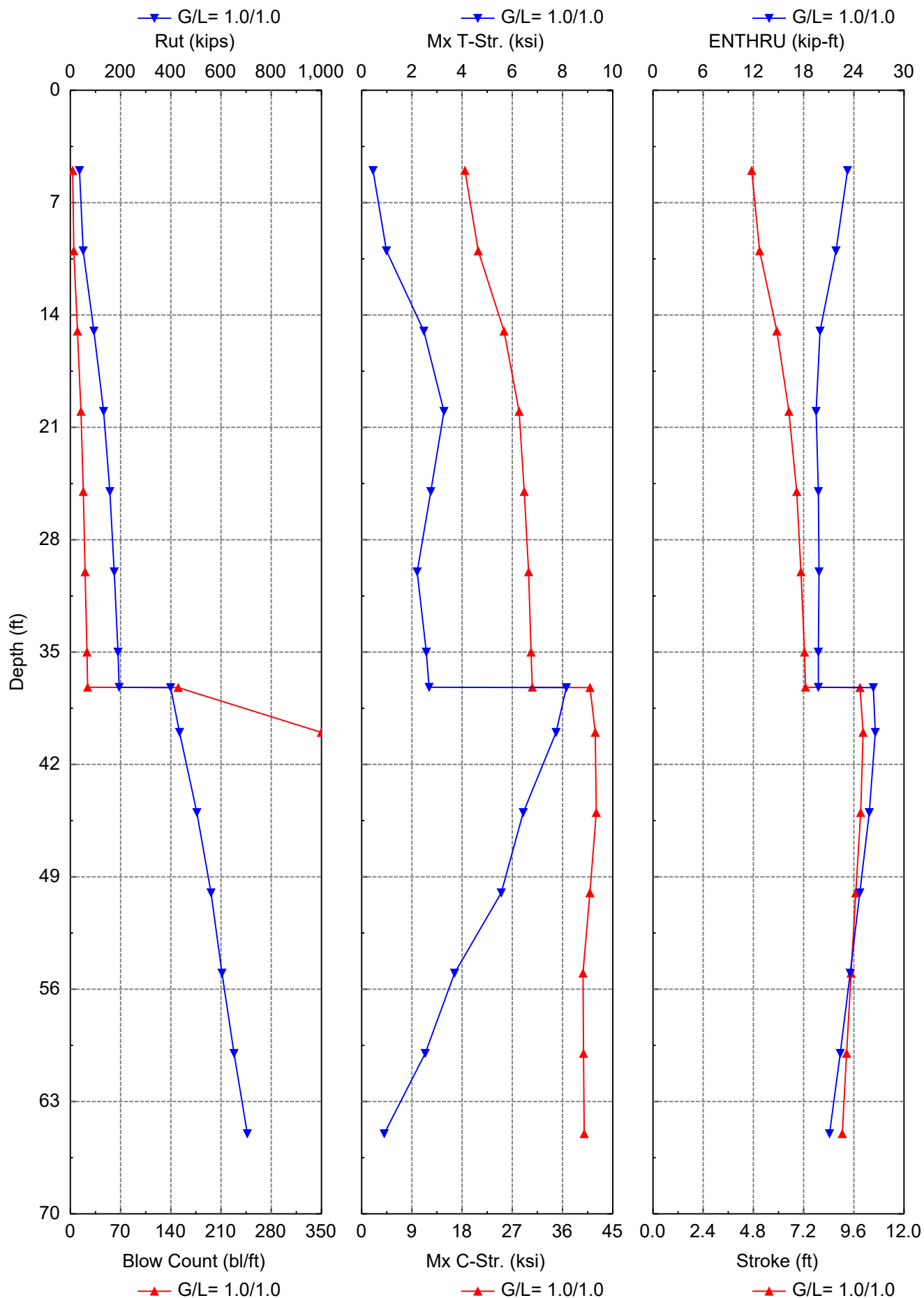
Assumed Groundwater Depth = 0.0 ft (Ele. 864.5 ft amsl)



Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	36.6	13.3	23.3	3.2	18.522	0.452	4.72	23.2	D 19-42
10.0	51.3	32.3	19.1	4.8	20.888	0.989	5.10	21.9	D 19-42
15.0	93.8	74.7	19.1	9.9	25.512	2.477	5.92	20.0	D 19-42
20.0	132.6	113.5	19.1	14.8	28.221	3.278	6.50	19.5	D 19-42
25.0	157.5	138.4	19.1	18.0	29.137	2.752	6.87	19.8	D 19-42
30.0	175.0	155.9	19.1	20.5	29.926	2.210	7.07	19.8	D 19-42
35.0	189.9	170.8	19.1	23.2	30.369	2.577	7.24	19.8	D 19-42
37.2	194.4	175.3	19.1	24.1	30.582	2.688	7.29	19.8	D 19-42
37.2	399.5	175.4	224.1	150.4	40.928	8.150	9.90	26.3	D 19-42
40.0	434.8	192.4	242.4	349.8	41.860	7.744	10.05	26.6	D 19-42
45.0	503.6	226.4	277.3	9999.0	42.043	6.435	9.93	25.8	D 19-42
50.0	560.3	265.0	295.3	9999.0	40.907	5.559	9.69	24.7	D 19-42
55.0	603.8	308.5	295.3	9999.0	39.663	3.701	9.47	23.6	D 19-42
60.0	652.0	356.7	295.3	9999.0	39.759	2.533	9.26	22.4	D 19-42
65.0	705.0	409.8	295.3	9999.0	39.884	0.901	9.04	21.1	D 19-42

Refusal occurred; no driving time output possible.



SOIL PROFILE

Depth ft	Soil Type -	Spec. Wt lb/ft ³	Su ksf	Phi °	Unit Rs ksf	Unit Rt ksf
0.0	Clay	108.0	0.9	0.0	0.66	7.87
0.9	Clay	108.0	0.9	0.0	0.66	7.87
0.9	Clay	125.0	3.2	0.0	0.70	29.25
5.9	Clay	125.0	3.2	0.0	0.70	29.25
5.9	Sand	132.0	0.0	40.0	0.28	42.45
8.4	Sand	132.0	0.0	40.0	0.41	63.32
8.4	Clay	125.0	2.7	0.0	1.40	24.30
37.2	Clay	125.0	2.7	0.0	1.40	24.30
37.2	Sand	132.0	0.0	40.0	1.83	279.49
42.2	Sand	132.0	0.0	40.0	2.10	321.22
42.2	Sand	140.0	0.0	40.0	2.10	321.22
65.4	Sand	140.0	0.0	40.0	3.51	375.94

Groundwater Assumptions:

Rear Abutment – Pile Length

General Site Information

✕

General Information

Total Pile Length: ft

Initial Pene. Depth: ft

Final Pene. Depth: ft

Water Table: ft

Eff. Overb. Pressure: ksf

☐ Additional Input

Import SPT Data

Import CPT Data

Import DIGGS Data

OK

Cancel

GRLWEAP 1980 Tomlinson Graphs:

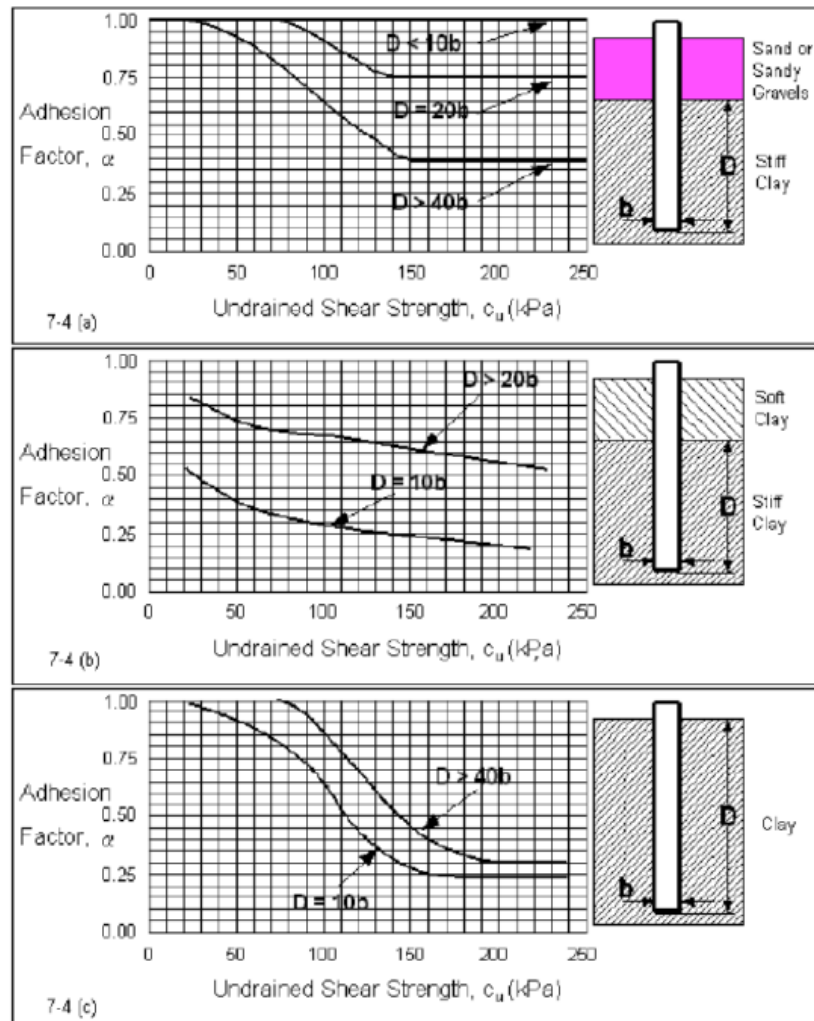


Figure 5.2: Adhesion factor for driven pile in cohesive soil (after Tomlinson 1980).

Rear Abutment:

Soil Profile Input

FHWA Method | Other Parameters (Impact)

Update=> Ru=668.3; Rs=373.1 kips

Layers Expand Graph

Depth Strength

Depth below ground surface (ft)

0.0

5.9 S=3.3 Cohesive

8.4 a=40 Sand

14

21 S=2.7 Cohesive

28

35.2

42.2 a=40 Sand

49

56 a=40 Sand

63

65.4

Site Properties

Number of Layers: 6

Final Tip Penetration: 65 ft

Water Table: 0 ft

Eff. Overburden at Grade: 0 ksf

Layer Properties

Layer: 1 of 6

Depth: Top: 0 Bottom: 0.9 ft

Thickness: 0.9 ft

Soil Type/Property

Gravel

Other

Granular: Skin Friction End Bearing

SPT N Value: 0 0 <=60

ϕ degree <= 45

Cohesive: S_u 0.875 ksf <= 5

Tomlinson 1980 (3) Click to change.

Rock: Description:

Other: Cohesive

Unit Weight: 108 lb/ft³

For selected soil, you need to specify:

Friction: End Bearing:

Top: 0.665 7.875 ksf

Bottom: 0.665 7.875 ksf

*Rock and Other are not FHWA defined soil types

OK Cancel

Soil Profile Input

FHWA Method | Other Parameters (Impact)

Update=> Ru=668.3; Rs=373.1 kips

Layers Expand Graph

Depth Strength

Depth below ground surface (ft)

0.0

5.9 S=1.5 Stiff Cohesive

8.4 a=40 Sand

14

21 S=2.7 Cohesive

28

35.2

42.2 a=40 Sand

49

56 a=40 Sand

63

65.4

Site Properties

Number of Layers: 6

Final Tip Penetration: 65 ft

Water Table: 0 ft

Eff. Overburden at Grade: 0 ksf

Layer Properties

Layer: 2 of 6

Depth: Top: 0.9 Bottom: 5.9 ft

Thickness: 5 ft

Soil Type/Property

Gravel

Other

Granular: Skin Friction End Bearing

SPT N Value: 0 0 <=60

ϕ degree <= 45

Cohesive: Stiff

S_u 1.462 ksf <= 5

Auto Adhesion (1980 - 3) Click to change.

Rock: Description:

Other: Stiff Cohesive

Unit Weight: 130.5 lb/ft³

For selected soil, you need to specify:

Friction: End Bearing:

Top: 0.703 29.25 ksf

Bottom: 0.703 29.25 ksf

*Rock and Other are not FHWA defined soil types

OK Cancel

Soil Profile Input

FHWA Method | Other Parameters (Impact)

Update=> Ru=668.3; Rs=373.1 kips

Layers

Soil Profile Graph:

- Depth (ft): 0, 5.9, 8.4, 14, 21, 28, 35, 37.2, 42.2, 49, 56, 63, 65.4
- Strength: 0.0
- Layers:
 - 0 to 5.9 ft: Stiff Cohesive
 - 5.9 to 8.4 ft: Sand
 - 8.4 to 37.2 ft: Cohesive
 - 37.2 to 42.2 ft: Sand
 - 42.2 to 65.4 ft: Sand

Site Properties

- Number of Layers: 6
- Final Tip Penetration: 65 ft
- Water Table: 0 ft
- Eff. Overburden at Grade: 0 ksf

Layer Properties

- Layer: 4 of 6
- Depth: Top: 8.4, Bottom: 37.2 ft, Thickness: 28.8 ft

Soil Type/Property

- ☐ Gravel
- ☐ Other Granular:
 - Skin Friction:
 - End Bearing:
 - SPT N Value: 0, <= 60
 - ϕ : degree <= 45
- ☒ Cohesive:
 - S_u : 2.7 ksf <= 5
 - Tomlinson 1980 (1) Click to change.

Rock/Other

- ☐ Rock*: Description:
- ☐ Other*: Cohesive
- Unit Weight: 125 lb/ft³

For selected soil, you need to specify:

- Friction: End Bearing:
- Top: 1.396, 24.3 ksf
- Bottom: 1.396, 24.3 ksf

*Rock and Other are not FHWA defined soil types

OK Cancel

GRLWEAP: Wave Equation Analysis of Pile Foundations

FRA-Cemetery Rd SUP Bridge + Rear Abutment - 12-inch CIP
NATIONAL ENGINEERING AND ARCHITECTURAL7/9/2026
GRLWEAP 14.2.2.0

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

The GRLWEAP program simulates the behavior of a preformed pile driven by either an impact hammer or a vibratory hammer. The program is based on mathematical models, which describe motion and forces of hammer, driving system, pile and soil under the hammer action. Under certain conditions, the models only crudely approximate, often complex, dynamic situations.

A wave equation analysis generally relies on input data, which represents normal situations. In particular, the hammer data file supplied with the program assumes that the hammer is in good working order. All of the input data selected by the user may be the best available information at the time when the analysis is performed. However, input data and therefore results may significantly differ from actual field conditions.

Therefore, the program authors recommend prudent use of the GRLWEAP results. Soil response and hammer performance should be verified by static and/or dynamic testing and measurements. Estimates of bending or other local stresses (e.g., helmet or clamp contact, uneven rock surfaces etc.), prestress effects and others must also be accounted for by the user.

The calculated capacity-blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

Finally, the GRLWEAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors.

HAMMER DATA

Hammer Model:	D 19-42	Made By:	DELMAG
Hammer ID:	41	Hammer Type:	OED
Hammer Database Type:	PDI		
Hammer Database Name:			PDIHammer.gwh

Hammer and Drive System Segment Data

Segment	Weight kips	Stiffness kips/in	COR -	C-Slack in	Damping kips/ft/s
-					
1	0.800	140,084.4	1.000	0.000	
2	0.800	140,084.4	1.000	0.000	
3	0.800	140,084.4	1.000	0.000	
4	0.800	140,084.4	1.000	0.000	
5	0.800	70,754.7	0.900	0.120	
Imp Block	0.753	109,976.0	0.800	0.120	
Helmet	2.500				5.3

Ram Weight: (kips)	4.00	Ram Length: (ft)	10.76
Ram Area: (in ²)	124.69		
Maximum (Eq) Stroke: (ft)	10.81	Actual (Eq) Stroke: (ft)	10.81
Efficiency:	0.800	Rated Energy: (kip-ft)	43.24
Maximum Pressure: (psi)	1,600.00	Actual Pressure: (psi)	1,600.00
Combustion Delay: (ms)	2.00	Ignition Duration: (ms)	2.00
Expansion Exponent:	1.25		

Hammer Cushion

Pile Cushion

Cross Sect. Area: (in ²)	415.00	Cross Sect. Area: (in ²)	0.00
Elastic Modulus: (ksi)	530.0	Elastic Modulus: (ksi)	0.0
Thickness: (in)	2.00	Thickness: (in)	0.00
Coeff. of Restitution:	0.800	Coeff. of Restitution:	0.500
RoundOut: (in)	0.120	RoundOut: (in)	0.120
Stiffness: (kips/in)	109,976.0	Stiffness: (kips/in)	0.0
Helmet Weight: (kips)	2.500		

PILE INPUT

Uniform Pile		Pile Type:	Closed-End Pipe
Pile Length: (ft)	70.000	Pile Penetration: (ft)	65.000
Pile Size: (ft)	1.00	Toe Area: (in ²)	113.10

Table of Depths Analyzed with Driving System Modifiers

Depth ft	Temp Length ft	Wait Time Hr	Hammer -
5.00	70.0	0.0	DELMAG D 19-42
10.00	70.0	0.0	DELMAG D 19-42
15.00	70.0	0.0	DELMAG D 19-42
20.00	70.0	0.0	DELMAG D 19-42
25.00	70.0	0.0	DELMAG D 19-42
30.00	70.0	0.0	DELMAG D 19-42
35.00	70.0	0.0	DELMAG D 19-42
37.19	70.0	0.0	DELMAG D 19-42
37.21	70.0	0.0	DELMAG D 19-42
40.00	70.0	0.0	DELMAG D 19-42
45.00	70.0	0.0	DELMAG D 19-42
50.00	70.0	0.0	DELMAG D 19-42
55.00	70.0	0.0	DELMAG D 19-42
60.00	70.0	0.0	DELMAG D 19-42
65.00	70.0	0.0	DELMAG D 19-42

Other Information for DELMAG D 19-42

Depth ft	Stroke ft	Diesel Pressure %	Efficiency -	P.C. Stiff. Fact. -	P.C. COR -
5.00	10.8	100.0	0.80	1.0	0.50
10.00	10.8	100.0	0.80	1.0	0.50
15.00	10.8	100.0	0.80	1.0	0.50
20.00	10.8	100.0	0.80	1.0	0.50
25.00	10.8	100.0	0.80	1.0	0.50
30.00	10.8	100.0	0.80	1.0	0.50
35.00	10.8	100.0	0.80	1.0	0.50
37.19	10.8	100.0	0.80	1.0	0.50
37.21	10.8	100.0	0.80	1.0	0.50
40.00	10.8	100.0	0.80	1.0	0.50
45.00	10.8	100.0	0.80	1.0	0.50
50.00	10.8	100.0	0.80	1.0	0.50
55.00	10.8	100.0	0.80	1.0	0.50
60.00	10.8	100.0	0.80	1.0	0.50
65.00	10.8	100.0	0.80	1.0	0.50

PILE, SOIL, ANALYSIS OPTIONS

Analysis type:	Driveability Analysis	Parallel Driveability Analysis:	No
Static Analysis Method:	FHWA	Soil Damping Option:	Smith
Max No Analysis Iterations:	0	Time Increment/Critical:	160
Residual Stress Analysis:	0	Analysis Time-Input(ms):	0
Output Level:	Normal	Gravitational Acceleration (ft/s ²):	32.169
Hammer Gravity (ft/s ²):	32.169	Pile Gravity (ft/s ²):	32.169

DRIVEABILITY ANALYSIS

Analysis Depth (ft)	65.00	Standard Soil Setup	
Hammer Name	DELMAG D 19-42	Hammer ID	41
Diesel Pressure: (psi)	230.40	Stroke (ft)	10.81
Efficiency	0.80		
Shaft Gain/Loss Factor	0.667	Toe Gain/Loss Factor	1.000
Shaft Gain/Loss Factor	1.000	Toe Gain/Loss Factor	1.000

SOIL RESISTANCE PARAMETERS (FHWA)

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Setup F. -	Limit D. ft	Setup T. Hours	EB Area in ²
0.00	0.8	7.9	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
0.90	0.8	7.9	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
0.90	0.9	29.7	0.100	0.108	0.150	0.149	1.5	6.00	168.0	113.10
5.90	0.9	29.7	0.100	0.108	0.150	0.149	1.5	6.00	168.0	113.10
5.90	0.3	48.2	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
7.15	0.4	58.6	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
8.40	0.5	69.0	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
8.40	1.7	24.3	0.100	0.113	0.150	0.149	1.5	6.00	168.0	113.10
37.20	1.7	24.3	0.100	0.113	0.150	0.149	1.5	6.00	168.0	113.10
37.20	1.9	285.2	0.100	0.109	0.100	0.149	1.2	6.00	24.0	113.10
38.87	2.0	299.1	0.100	0.109	0.100	0.149	1.2	6.00	24.0	113.10
40.53	2.0	313.0	0.100	0.109	0.100	0.149	1.2	6.00	24.0	113.10
42.20	2.1	327.0	0.100	0.109	0.100	0.149	1.2	6.00	24.0	113.10
42.20	2.1	327.0	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
43.86	2.2	342.4	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
45.51	2.3	357.8	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
47.17	2.4	373.2	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
48.83	2.5	375.9	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
50.49	2.6	375.9	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
52.14	2.7	375.9	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
53.80	2.8	375.9	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
55.46	2.9	375.9	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
57.11	3.0	375.9	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
58.77	3.1	375.9	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
60.43	3.2	375.9	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
62.09	3.4	375.9	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
63.74	3.5	375.9	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
65.00	3.5	375.9	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10

PILE PROFILE

Lb Top ft	X-Area in ²	E-Mod ksi	Spec. Wt lb/ft ³	Perim. ft	C-Index -	Wave Sp ft/s	Impedance kips/ft/s
0.00	11.5	30,000	492.00	3.142	0	16,806.4	20.4
70.00	11.5	30,000	492.00	3.142	0	16,806.4	20.4

PILE AND SOIL MODEL

Total Capacity Rut (kips):

642.348

Seg. -	Weight kips	Stiffn. kips/in	C-Slk in	T-Slk in	COR -	Ru kips	Js/Jt s/ft	Qs/Qt in	LbTop ft	Perim. ft	X-Area in ²
1	0.13	8,592	0.12	0.00	0.85	0.0	0.000	0.100	3.33	3.14	11.5
2	0.13	8,592	0.00	0.00	1.00	2.9	0.150	0.100	6.67	3.14	11.5

3	0.13	8,592	0.00	0.00	1.00	5.9	0.150	0.100	10.00	3.14	11.5
4	0.13	8,592	0.00	0.00	1.00	4.5	0.095	0.100	13.33	3.14	11.5
5	0.13	8,592	0.00	0.00	1.00	11.9	0.149	0.100	16.67	3.14	11.5
6	0.13	8,592	0.00	0.00	1.00	12.1	0.150	0.100	20.00	3.14	11.5
12	0.13	8,592	0.00	0.00	1.00	12.1	0.150	0.100	40.00	3.14	11.5
13	0.13	8,592	0.00	0.00	1.00	13.6	0.132	0.100	43.33	3.14	11.5
14	0.13	8,592	0.00	0.00	1.00	17.6	0.100	0.100	46.67	3.14	11.5
15	0.13	8,592	0.00	0.00	1.00	22.5	0.058	0.100	50.00	3.14	11.5
16	0.13	8,592	0.00	0.00	1.00	25.3	0.050	0.100	53.33	3.14	11.5
17	0.13	8,592	0.00	0.00	1.00	27.4	0.050	0.100	56.67	3.14	11.5
18	0.13	8,592	0.00	0.00	1.00	29.5	0.050	0.100	60.00	3.14	11.5
19	0.13	8,592	0.00	0.00	1.00	31.6	0.050	0.100	63.33	3.14	11.5
20	0.13	8,592	0.00	0.00	1.00	33.8	0.050	0.100	66.67	3.14	11.5
21	0.13	8,592	0.00	0.00	1.00	35.9	0.050	0.100	70.00	3.14	11.5
Toe						295.3	0.149	0.100	70.00		

2.740 kips total unreduced pile weight ($g = 32.169 \text{ ft/s}^2$)

2.740 kips total reduced pile weight ($g = 32.169 \text{ ft/s}^2$)

OTHER OPTIONS

Pile Damping (%):	1	Pile Damping Fact. (kips/ft/s):	0.409
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EXTREMA TABLE at 65.0 FT; HAMMER: D 19-42

Shaft/Toe Gain/Loss Factor = 0.667/1.000

Rut = 642.3 kips

Rtoe = 295.3 kips

Time Inc. = 0.049 ms

Hammer

DELMAG D 19-42

Efficiency

0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	400.4	0.00	34.95	16.65	0.796	22.05
6.7	7.4	411.2	0.65	35.89	16.20	0.755	21.24
10.0	10.9	413.3	0.95	36.08	15.70	0.712	19.99
13.3	10.1	407.7	0.88	35.59	14.90	0.668	18.69
16.7	10.8	409.0	0.94	35.70	14.12	0.623	17.08
20.0	1.1	398.9	0.10	34.82	13.35	0.580	15.15
23.3	0.0	392.6	0.00	34.27	12.60	0.537	13.36
26.7	0.0	378.9	0.00	33.07	11.86	0.494	11.68
30.0	0.0	379.1	0.00	33.09	11.12	0.450	10.08
33.3	0.0	386.4	0.00	33.72	10.40	0.406	8.58
36.7	0.0	381.0	0.00	33.25	9.68	0.363	7.24
40.0	0.0	366.0	0.00	31.95	9.15	0.321	6.05
43.3	0.0	352.1	0.00	30.73	8.67	0.282	5.00
46.7	0.0	342.0	0.00	29.85	8.19	0.244	4.05
50.0	0.0	333.0	0.00	29.06	7.73	0.209	3.18
53.3	0.0	308.9	0.00	26.97	7.30	0.175	2.43
56.7	0.0	282.1	0.00	24.63	6.86	0.143	1.78
60.0	0.0	251.6	0.00	21.96	6.45	0.115	1.28
63.3	0.0	220.4	0.00	19.24	6.14	0.091	0.91
66.7	0.0	192.2	0.00	16.78	5.64	0.070	0.65
70.0	0.0	172.6	0.00	15.07	4.11	0.051	0.48

Converged Stroke (ft)

9.06

Fixed Combustion Pressure (psi)

1,600.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81

9.20

9.06

9.06

Shaft/Toe Gain/Loss Factor = 1.000/1.000

Rut = 705.0 kips

Rtoe = 295.3 kips

Time Inc. = 0.049 ms

Hammer

DELMAG D 19-42

Efficiency

0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	442.0	0.00	38.58	16.48	0.719	21.09
6.7	8.3	455.7	0.73	39.78	15.87	0.674	20.09
10.0	10.3	456.9	0.90	39.88	15.27	0.628	18.56
13.3	5.7	447.1	0.50	39.03	14.20	0.583	17.10
16.7	5.9	449.3	0.51	39.22	13.12	0.538	15.37
20.0	0.0	422.7	0.00	36.90	12.04	0.495	13.19
23.3	0.0	396.3	0.00	34.59	10.99	0.453	11.21
26.7	0.0	371.1	0.00	32.40	9.99	0.411	9.42
30.0	0.0	363.4	0.00	31.72	9.14	0.369	7.79
33.3	0.0	362.2	0.00	31.62	8.52	0.327	6.33
36.7	0.0	350.1	0.00	30.56	7.97	0.288	5.09
40.0	0.0	327.5	0.00	28.59	7.47	0.253	4.06
43.3	0.0	304.3	0.00	26.56	7.04	0.220	3.21
46.7	0.0	291.0	0.00	25.40	6.65	0.190	2.51

50.0	0.0	276.9	0.00	24.17	6.32	0.161	1.93
53.3	0.0	249.2	0.00	21.75	5.98	0.135	1.46
56.7	0.0	218.7	0.00	19.09	5.64	0.110	1.06
60.0	0.0	191.7	0.00	16.73	5.33	0.088	0.75
63.3	0.0	167.4	0.00	14.61	5.11	0.069	0.53
66.7	0.0	144.9	0.00	12.65	4.80	0.055	0.39
70.0	0.0	129.5	0.00	11.31	3.64	0.041	0.30

Converged Stroke (ft) 9.04 Fixed Combustion Pressure (psi) 1,600.0
(Eq) Strokes Analyzed and Last Return (ft)
10.81 9.16 9.04 9.04

SUMMARY TABLE at 65.0 FT; HAMMER: D 19-42

Rut	BI Ct	Stk Dn	Stk Up	Mx T-Str	LTop	Mx C-Str	LTop	ENTHRU	BI Rt	ActRes
kips	b/ft	ft	ft	ksi	ft	ksi	ft	kip-ft	b/min	kips
642.3	9,999	9.06	9.06	0.95	10.0	36.08	10.0	22.0	39.3	466.2
705.0	9,999	9.04	9.04	0.90	10.0	39.88	10.0	21.1	39.3	480.8

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.667/1.000									
Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	32.2	8.8	23.3	2.8	17.72	0.42	4.58	23.9	D 19-42
10.0	41.6	22.5	19.1	3.7	19.24	0.76	4.84	22.7	D 19-42
15.0	69.9	50.8	19.1	7.0	23.25	1.41	5.46	20.6	D 19-42
20.0	95.8	76.7	19.1	10.3	25.75	2.31	5.97	19.8	D 19-42
25.0	112.3	93.3	19.1	12.6	26.32	2.37	6.23	19.4	D 19-42
30.0	124.0	104.9	19.1	14.1	27.26	2.11	6.42	19.3	D 19-42
35.0	133.9	114.9	19.1	15.5	27.40	1.76	6.58	19.1	D 19-42
37.2	137.0	117.9	19.1	16.0	27.71	1.66	6.62	19.1	D 19-42
37.2	342.0	117.9	224.1	71.2	38.89	6.92	9.27	25.0	D 19-42
40.0	374.5	132.1	242.4	105.8	39.89	6.93	9.60	25.6	D 19-42
45.0	440.9	163.7	277.3	492.0	41.40	6.13	9.96	26.3	D 19-42
50.0	497.6	202.4	295.3	9,999.0	40.12	4.97	9.69	25.2	D 19-42
55.0	541.1	245.8	295.3	9,999.0	37.84	4.03	9.44	24.2	D 19-42
60.0	589.3	294.1	295.3	9,999.0	36.56	2.59	9.25	23.1	D 19-42
65.0	642.3	347.1	295.3	9,999.0	36.08	0.95	9.06	22.0	D 19-42

G/L at Shaft and Toe: 1.000/1.000									
Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	36.6	13.3	23.3	3.2	18.52	0.45	4.72	23.2	D 19-42
10.0	51.3	32.3	19.1	4.8	20.89	0.99	5.10	21.9	D 19-42
15.0	93.8	74.7	19.1	9.9	25.51	2.48	5.92	20.0	D 19-42
20.0	132.6	113.5	19.1	14.8	28.22	3.28	6.50	19.5	D 19-42
25.0	157.5	138.4	19.1	18.0	29.14	2.75	6.87	19.8	D 19-42
30.0	175.0	155.9	19.1	20.5	29.93	2.21	7.07	19.8	D 19-42
35.0	189.9	170.8	19.1	23.2	30.37	2.58	7.24	19.8	D 19-42
37.2	194.4	175.3	19.1	24.1	30.58	2.69	7.29	19.8	D 19-42
37.2	399.5	175.4	224.1	150.4	40.93	8.15	9.90	26.3	D 19-42
40.0	434.8	192.4	242.4	349.8	41.86	7.74	10.05	26.6	D 19-42
45.0	503.6	226.4	277.3	9,999.0	42.04	6.43	9.93	25.8	D 19-42
50.0	560.3	265.0	295.3	9,999.0	40.91	5.56	9.69	24.7	D 19-42
55.0	603.8	308.5	295.3	9,999.0	39.66	3.70	9.47	23.6	D 19-42
60.0	652.0	356.7	295.3	9,999.0	39.76	2.53	9.26	22.4	D 19-42
65.0	705.0	409.8	295.3	9,999.0	39.88	0.90	9.04	21.1	D 19-42

REAR ABUTMENT

DRIVABILITY

Boring Used: B-009-0-23

Starting Elevation (0.0 ft) = 883.4 ft amsl (Bottom of pile cap elevation)

Assume Zero-Friction Length to 18.9 ft = 883.4 - 864.5 ft amsl (bottom of foundation prep layer)

Assumed Groundwater Depth = 18.9 ft (Ele. 864.5 ft amsl)

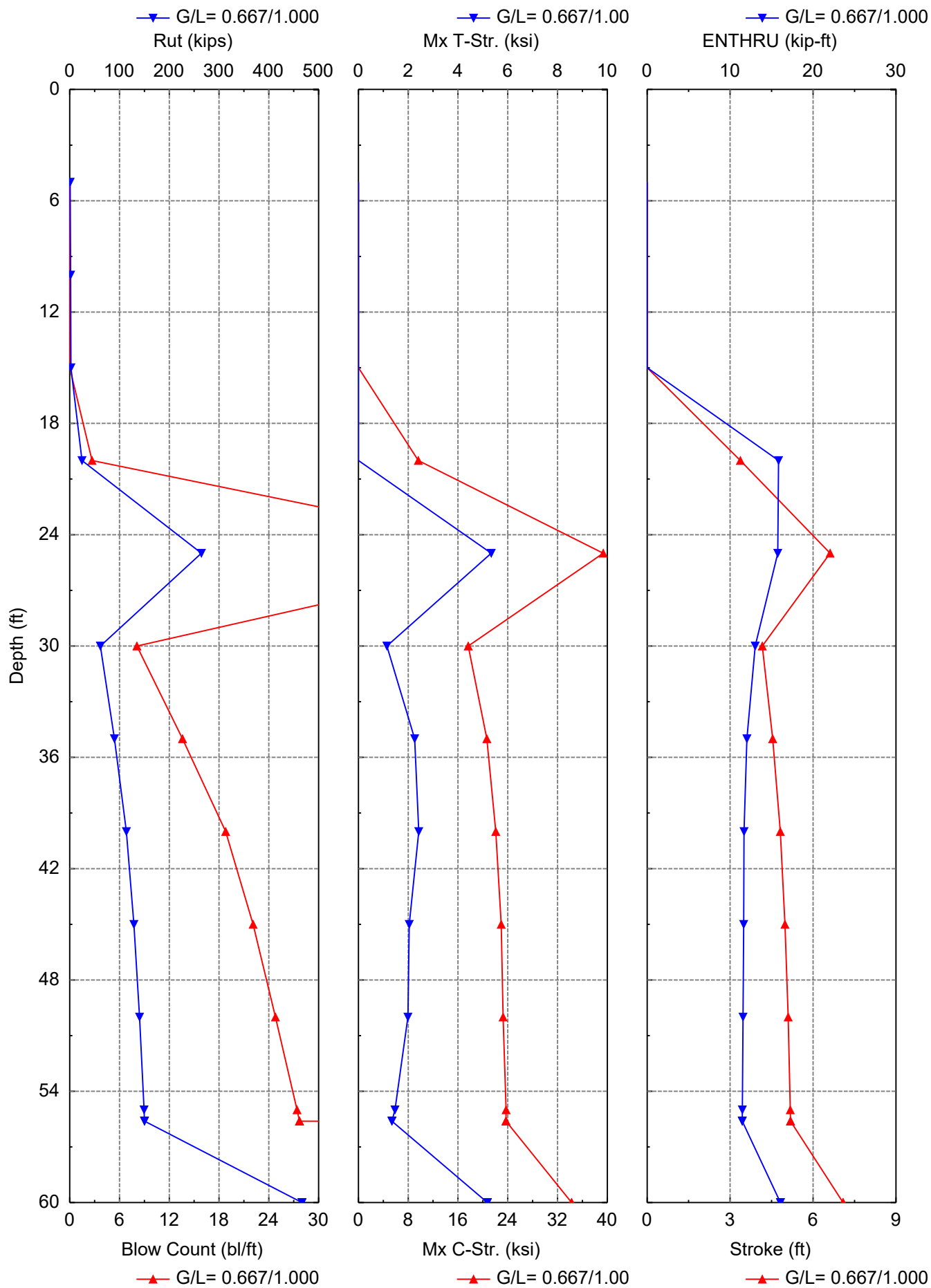
Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 0.667/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.8	0.0	0.8	0.3	0.000	0.000	10.81	0.0	D 19-42
10.0	1.7	0.0	1.7	0.3	0.000	0.000	10.81	0.0	D 19-42
15.0	2.5	0.0	2.5	0.3	0.000	0.000	10.81	0.0	D 19-42
20.0	24.7	1.7	23.0	2.7	9.626	0.000	3.37	15.8	D 19-42
25.0	264.5	11.6	252.9	57.4	39.345	5.336	6.62	15.7	D 19-42
30.0	61.6	42.6	19.1	8.1	17.640	1.142	4.16	13.0	D 19-42
35.0	89.9	70.8	19.1	13.6	20.639	2.261	4.54	12.0	D 19-42
40.0	113.8	94.7	19.1	18.8	22.076	2.423	4.82	11.7	D 19-42
45.0	129.0	109.9	19.1	22.1	22.954	2.041	4.98	11.6	D 19-42
50.0	140.4	121.3	19.1	24.8	23.249	1.988	5.10	11.6	D 19-42
55.0	149.4	130.3	19.1	27.4	23.734	1.466	5.18	11.5	D 19-42
55.6	150.2	131.1	19.1	27.7	23.718	1.347	5.18	11.5	D 19-42
55.6	150.2	131.1	19.1	27.7	23.719	1.345	5.19	11.5	D 19-42
60.0	466.6	171.3	295.3	9999.0	34.262	5.182	7.09	16.1	D 19-42
60.0	466.6	171.3	295.3	9999.0	34.262	5.182	7.09	16.1	D 19-42

Refusal occurred; no driving time output possible.

Starting Elevation (0.0 ft) = 883.4 ft amsl

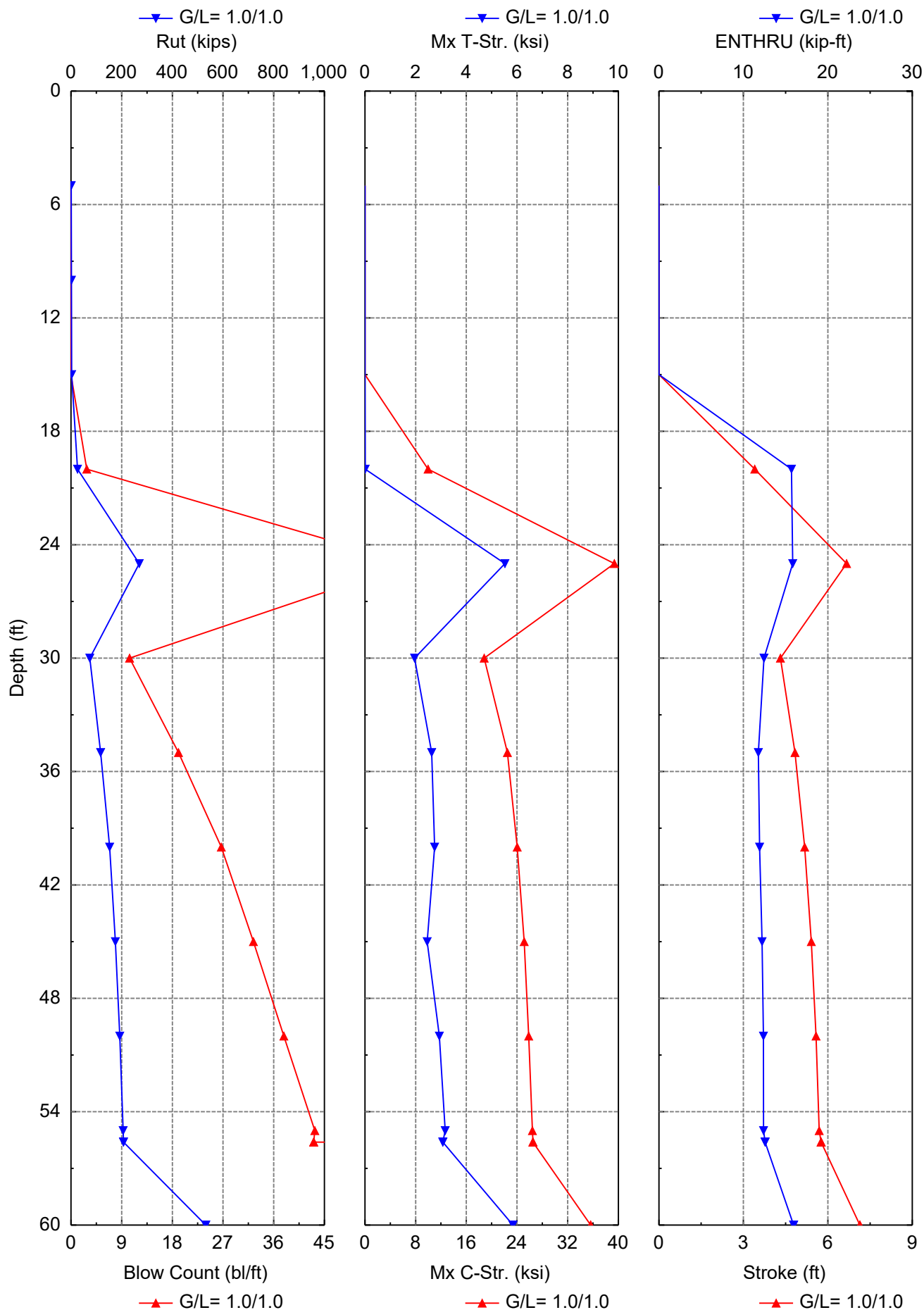
Assumed Groundwater Depth = 18.9 ft (Ele. 864.5 ft amsl)



Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.8	0.0	0.8	0.3	0.000	0.000	10.81	0.0	D 19-42
10.0	1.7	0.0	1.7	0.3	0.000	0.000	10.81	0.0	D 19-42
15.0	2.5	0.0	2.5	0.3	0.000	0.000	10.81	0.0	D 19-42
20.0	25.6	2.6	23.0	2.8	9.972	0.010	3.40	15.7	D 19-42
25.0	269.7	16.8	252.9	60.1	39.363	5.522	6.67	15.8	D 19-42
30.0	74.4	55.3	19.1	10.4	18.828	1.958	4.32	12.4	D 19-42
35.0	116.8	97.8	19.1	19.1	22.486	2.633	4.83	11.8	D 19-42
40.0	152.6	133.6	19.1	26.7	24.025	2.747	5.17	11.9	D 19-42
45.0	175.4	156.3	19.1	32.4	25.135	2.454	5.41	12.2	D 19-42
50.0	192.6	173.5	19.1	37.8	25.849	2.935	5.58	12.4	D 19-42
55.0	206.1	187.0	19.1	43.3	26.437	3.168	5.69	12.4	D 19-42
55.6	207.2	188.2	19.1	43.1	26.518	3.075	5.76	12.6	D 19-42
55.6	207.3	188.2	19.1	43.1	26.511	3.073	5.76	12.6	D 19-42
60.0	531.9	236.6	295.3	9999.0	35.628	5.841	7.14	15.9	D 19-42
60.0	531.9	236.6	295.3	9999.0	35.628	5.841	7.14	15.9	D 19-42

Refusal occurred; no driving time output possible.



SOIL PROFILE

Depth ft	Soil Type -	Spec. Wt lb/ft ³	Su ksf	Phi °	Unit Rs ksf	Unit Rt ksf
0.0	Sand	120.0	0.0	0.0	0.00	0.00
18.9	Sand	120.0	0.0	0.0	0.00	3.97
18.9	Clay	108.0	0.9	0.0	0.74	7.87
19.8	Clay	108.0	0.9	0.0	0.74	7.87
19.8	Clay	125.0	3.2	0.0	1.02	29.25
24.8	Clay	125.0	3.2	0.0	1.02	29.25
24.8	Sand	132.0	0.0	40.0	2.03	310.86
27.3	Sand	132.0	0.0	40.0	2.17	331.73
27.3	Clay	125.0	2.7	0.0	1.45	24.30
56.1	Clay	125.0	2.7	0.0	1.45	24.30
56.1	Sand	132.0	0.0	40.0	3.59	375.94
61.1	Sand	132.0	0.0	40.0	3.86	375.94
61.1	Sand	140.0	0.0	40.0	3.86	375.94
84.3	Sand	140.0	0.0	40.0	5.27	375.94

Groundwater Assumptions:

Rear Abutment - Drivability

General Site Information

×

General Information

Total Pile Length: ft

Initial Pene. Depth: ft

Final Pene. Depth: ft

Water Table: ft

Eff. Overb. Pressure: ksf

☐ Additional Input

Import SPT Data

Import CPT Data

Import DIGGS Data

OK

Cancel

GRLWEAP 1980 Tomlinson Graphs:

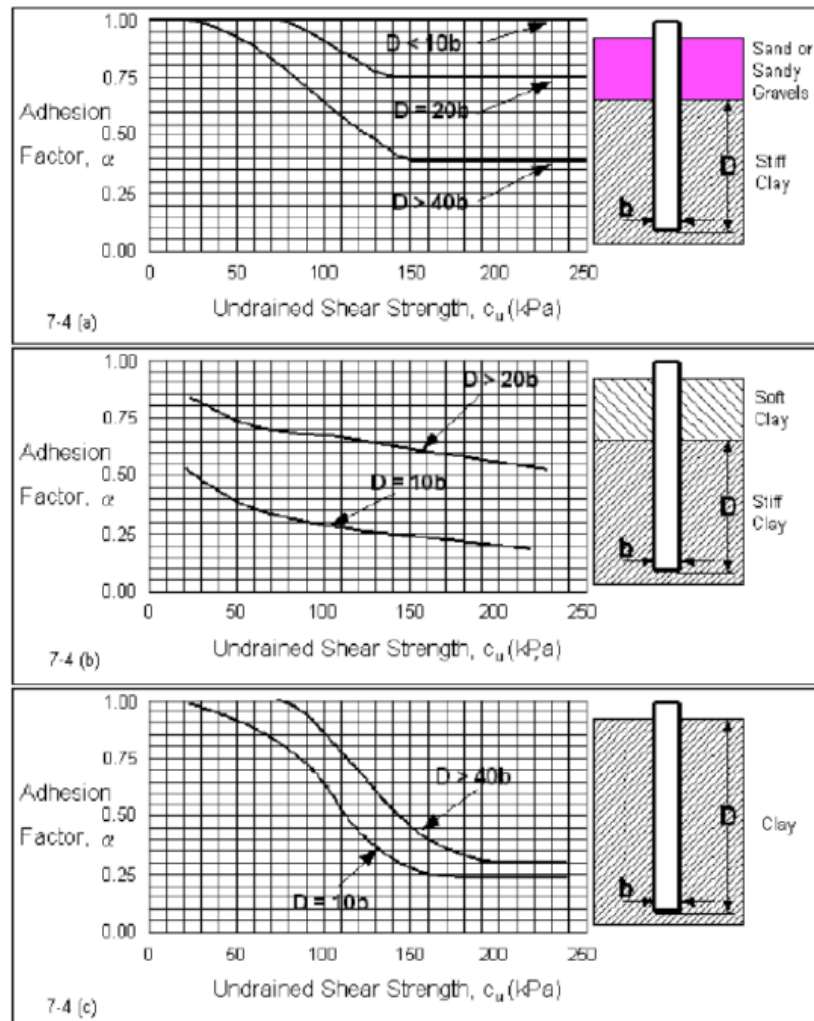


Figure 5.2: Adhesion factor for driven pile in cohesive soil (after Tomlinson 1980).

Rear Abutment:

Soil Profile Input

FHWA Method | Other Parameters (Impact)

Update=> Ru=668.3; Rs=373.1 kips

Layers Expand Graph

Depth Strength

Depth below ground surface (ft)

0.0

5.9 S=3.3 Cohesive

8.4 a=40 Sand

14

21 S=2.7 Cohesive

28

35.2

42.2 a=40 Sand

49

56 a=40 Sand

63

65.4

Site Properties

Number of Layers: 6

Final Tip Penetration: 65 ft

Water Table: 0 ft

Eff. Overburden at Grade: 0 ksf

Layer Properties

Layer: 1 of 6

Depth: Top: 0 Bottom: 0.9 ft

Thickness: 0.9 ft

Soil Type/Property

Gravel

Other

Granular: Skin Friction End Bearing

SPT N Value: 0 0 <=60

ϕ degree <= 45

Cohesive: S_u 0.875 ksf <= 5

Tomlinson 1980 (3) Click to change.

Rock*: Description:

Other*: Cohesive

Unit Weight: 108 lb/ft³

For selected soil, you need to specify:

Friction: End Bearing:

Top: 0.665 7.875 ksf

Bottom: 0.665 7.875 ksf

*Rock and Other are not FHWA defined soil types

OK Cancel

Soil Profile Input

FHWA Method | Other Parameters (Impact)

Update=> Ru=668.3; Rs=373.1 kips

Layers Expand Graph

Depth Strength

Depth below ground surface (ft)

0.0

5.9 S=1.5 Stiff Cohesive

8.4 a=40 Sand

14

21 S=2.7 Cohesive

28

35.2

42.2 a=40 Sand

49

56 a=40 Sand

63

65.4

Site Properties

Number of Layers: 6

Final Tip Penetration: 65 ft

Water Table: 0 ft

Eff. Overburden at Grade: 0 ksf

Layer Properties

Layer: 2 of 6

Depth: Top: 0.9 Bottom: 5.9 ft

Thickness: 5 ft

Soil Type/Property

Gravel

Other

Granular: Skin Friction End Bearing

SPT N Value: 0 0 <=60

ϕ degree <= 45

Cohesive: Stiff

S_u 1.462 ksf <= 5

Auto Adhesion (1980 - 3) Click to change.

Rock*: Description:

Other*: Stiff Cohesive

Unit Weight: 130.5 lb/ft³

For selected soil, you need to specify:

Friction: End Bearing:

Top: 0.703 29.25 ksf

Bottom: 0.703 29.25 ksf

*Rock and Other are not FHWA defined soil types

OK Cancel

Soil Profile Input

FHWA Method | Other Parameters (Impact)

Update=> Ru=668.3; Rs=373.1 kips

Layers

Soil Profile Graph:

- Depth (ft): 0, 5.9, 8.4, 14, 21, 28, 35, 37.2, 42.2, 49, 56, 63, 65.4
- Strength: 0.0
- Layers:
 - 0 to 5.9 ft: Stiff Cohesive
 - 5.9 to 8.4 ft: Sand
 - 8.4 to 37.2 ft: Cohesive
 - 37.2 to 42.2 ft: Sand
 - 42.2 to 65.4 ft: Sand

Site Properties

- Number of Layers: 6
- Final Tip Penetration: 65 ft
- Water Table: 0 ft
- Eff. Overburden at Grade: 0 ksf

Layer Properties

- Layer: 4 of 6
- Depth: Top: 8.4, Bottom: 37.2 ft, Thickness: 28.8 ft

Soil Type/Property

- ☐ Gravel
- ☐ Other Granular:
 - Skin Friction: []
 - End Bearing: []
 - SPT N Value: 0, 0, <=60
 - ϕ [] degree <= 45
- ☒ Cohesive:
 - S_u 2.7 ksf <= 5
 - Tomlinson 1980 (1) Click to change.

Rock/Other

- ☐ Rock*: [] Description: []
- ☐ Other*: [] Cohesive []
- Unit Weight: 125 lb/ft³

For selected soil, you need to specify:

- Friction: End Bearing:
- Top: 1.396, 24.3 ksf
- Bottom: 1.396, 24.3 ksf

*Rock and Other are not FHWA defined soil types

OK Cancel

GRLWEAP: Wave Equation Analysis of Pile Foundations

FRA-Cemetery Rd SUP Bridge + Rear Abutment - 12-inch CIP
NATIONAL ENGINEERING AND ARCHITECTURAL7/9/2026
GRLWEAP 14.2.2.0

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

The GRLWEAP program simulates the behavior of a preformed pile driven by either an impact hammer or a vibratory hammer. The program is based on mathematical models, which describe motion and forces of hammer, driving system, pile and soil under the hammer action. Under certain conditions, the models only crudely approximate, often complex, dynamic situations.

A wave equation analysis generally relies on input data, which represents normal situations. In particular, the hammer data file supplied with the program assumes that the hammer is in good working order. All of the input data selected by the user may be the best available information at the time when the analysis is performed. However, input data and therefore results may significantly differ from actual field conditions.

Therefore, the program authors recommend prudent use of the GRLWEAP results. Soil response and hammer performance should be verified by static and/or dynamic testing and measurements. Estimates of bending or other local stresses (e.g., helmet or clamp contact, uneven rock surfaces etc.), prestress effects and others must also be accounted for by the user.

The calculated capacity-blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

Finally, the GRLWEAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors.

HAMMER DATA

Hammer Model:	D 19-42	Made By:	DELMAG
Hammer ID:	41	Hammer Type:	OED
Hammer Database Type:	PDI		
Hammer Database Name:			PDIHammer.gwh

Hammer and Drive System Segment Data

Segment	Weight kips	Stiffness kips/in	COR -	C-Slack in	Damping kips/ft/s
-					
1	0.800	140,084.4	1.000	0.000	
2	0.800	140,084.4	1.000	0.000	
3	0.800	140,084.4	1.000	0.000	
4	0.800	140,084.4	1.000	0.000	
5	0.800	70,754.7	0.900	0.120	
Imp Block	0.753	109,976.0	0.800	0.120	
Helmet	2.500				5.3

Ram Weight: (kips)	4.00	Ram Length: (ft)	10.76
Ram Area: (in ²)	124.69		
Maximum (Eq) Stroke: (ft)	10.81	Actual (Eq) Stroke: (ft)	10.81
Efficiency:	0.800	Rated Energy: (kip-ft)	43.24
Maximum Pressure: (psi)	1,600.00	Actual Pressure: (psi)	1,165.00
Combustion Delay: (ms)	2.00	Ignition Duration: (ms)	2.00
Expansion Exponent:	1.25		

Hammer Cushion

Pile Cushion

Cross Sect. Area: (in ²)	415.00	Cross Sect. Area: (in ²)	0.00
Elastic Modulus: (ksi)	530.0	Elastic Modulus: (ksi)	0.0
Thickness: (in)	2.00	Thickness: (in)	0.00
Coeff. of Restitution:	0.800	Coeff. of Restitution:	0.500
RoundOut: (in)	0.120	RoundOut: (in)	0.120
Stiffness: (kips/in)	109,976.0	Stiffness: (kips/in)	0.0
Helmet Weight: (kips)	2.500		

PILE INPUT

Uniform Pile		Pile Type:	Closed-End Pipe
Pile Length: (ft)	65.000	Pile Penetration: (ft)	60.000
Pile Size: (ft)	1.00	Toe Area: (in ²)	113.10

Table of Depths Analyzed with Driving System Modifiers

Depth ft	Temp Length ft	Wait Time Hr	Hammer -
5.00	65.0	0.0	DELMAG D 19-42
10.00	65.0	0.0	DELMAG D 19-42
15.00	65.0	0.0	DELMAG D 19-42
20.00	65.0	0.0	DELMAG D 19-42
25.00	65.0	0.0	DELMAG D 19-42
30.00	65.0	0.0	DELMAG D 19-42
35.00	65.0	0.0	DELMAG D 19-42
40.00	65.0	0.0	DELMAG D 19-42
45.00	65.0	0.0	DELMAG D 19-42
50.00	65.0	0.0	DELMAG D 19-42
55.00	65.0	0.0	DELMAG D 19-42
55.60	65.0	0.0	DELMAG D 19-42
55.61	65.0	0.0	DELMAG D 19-42
60.00	65.0	0.0	DELMAG D 19-42
60.00	65.0	0.0	DELMAG D 19-42

Other Information for DELMAG D 19-42

Depth ft	Stroke ft	Diesel Pressure %	Efficiency -	P.C. Stiff. Fact. -	P.C. COR -
5.00	10.8	72.8	0.80	1.0	0.50
10.00	10.8	72.8	0.80	1.0	0.50
15.00	10.8	72.8	0.80	1.0	0.50
20.00	10.8	72.8	0.80	1.0	0.50
25.00	10.8	72.8	0.80	1.0	0.50
30.00	10.8	72.8	0.80	1.0	0.50
35.00	10.8	72.8	0.80	1.0	0.50
40.00	10.8	72.8	0.80	1.0	0.50
45.00	10.8	72.8	0.80	1.0	0.50
50.00	10.8	72.8	0.80	1.0	0.50
55.00	10.8	72.8	0.80	1.0	0.50
55.60	10.8	72.8	0.80	1.0	0.50
55.61	10.8	72.8	0.80	1.0	0.50
60.00	10.8	72.8	0.80	1.0	0.50
60.00	10.8	72.8	0.80	1.0	0.50

PILE, SOIL, ANALYSIS OPTIONS

Analysis type:	Driveability Analysis	Parallel Driveability Analysis:	No
Static Analysis Method:	FHWA	Soil Damping Option:	Smith
Max No Analysis Iterations:	0	Time Increment/Critical:	160
Residual Stress Analysis:	0	Analysis Time-Input(ms):	0
Output Level:	Normal	Gravitational Acceleration (ft/s ²):	32.169
Hammer Gravity (ft/s ²):	32.169	Pile Gravity (ft/s ²):	32.169

DRIVEABILITY ANALYSIS

Analysis Depth (ft)	60.00	Standard Soil Setup	
Hammer Name	DELMAG D 19-42	Hammer ID	41
Diesel Pressure: (psi)	167.76	Stroke (ft)	10.81
Efficiency	0.80		
Shaft Gain/Loss Factor	0.667	Toe Gain/Loss Factor	1.000
Shaft Gain/Loss Factor	1.000	Toe Gain/Loss Factor	1.000

SOIL RESISTANCE PARAMETERS (FHWA)

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Setup F. -	Limit D. ft	Setup T. Hours	EB Area in ²
0.00	0.0	0.0	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
1.72	0.0	0.4	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
3.44	0.0	0.7	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
5.15	0.0	1.1	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
6.87	0.0	1.5	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
8.59	0.0	1.8	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
10.31	0.0	2.2	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
12.03	0.0	2.6	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
13.75	0.0	2.9	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
15.46	0.0	3.3	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
17.18	0.0	3.7	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
18.90	0.0	4.0	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
18.90	0.7	7.9	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
19.80	0.7	7.9	0.100	0.147	0.150	0.149	1.5	6.00	168.0	113.10
19.80	0.9	29.2	0.100	0.108	0.150	0.149	1.5	6.00	168.0	113.10
24.80	0.9	29.2	0.100	0.108	0.150	0.149	1.5	6.00	168.0	113.10
24.80	2.1	320.3	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
26.05	2.2	330.8	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
27.30	2.2	341.2	0.100	0.100	0.050	0.149	1.0	6.00	1.0	113.10
27.30	1.7	24.3	0.100	0.113	0.150	0.149	1.5	6.00	168.0	113.10
56.10	1.7	24.3	0.100	0.113	0.150	0.149	1.5	6.00	168.0	113.10
56.10	3.8	375.9	0.100	0.109	0.100	0.149	1.2	6.00	24.0	113.10
57.77	3.9	375.9	0.100	0.109	0.100	0.149	1.2	6.00	24.0	113.10
59.43	4.0	375.9	0.100	0.109	0.100	0.149	1.2	6.00	24.0	113.10
60.00	4.0	375.9	0.100	0.109	0.100	0.149	1.2	6.00	24.0	113.10

PILE PROFILE

Lb Top ft	X-Area in ²	E-Mod ksi	Spec. Wt lb/ft ³	Perim. ft	C-Index -	Wave Sp ft/s	Impedance kips/ft/s
0.00	11.5	30,000	492.00	3.142	0	16,806.4	20.4
65.00	11.5	30,000	492.00	3.142	0	16,806.4	20.4

PILE AND SOIL MODEL

Total Capacity Rut (kips):

466.584

Seg. -	Weight kips	Stiffn. kips/in	C-Slk in	T-Slk in	COR -	Ru kips	Js/Jt s/ft	Qs/Qt in	LbTop ft	Perim. ft	X-Area in ²
1	0.13	8,813	0.12	0.00	0.85	0.0	0.000	0.100	3.25	3.14	11.5
2	0.13	8,813	0.00	0.00	1.00	0.0	0.000	0.100	6.50	3.14	11.5
7	0.13	8,813	0.00	0.00	1.00	0.0	0.000	0.100	22.75	3.14	11.5
8	0.13	8,813	0.00	0.00	1.00	3.5	0.150	0.100	26.00	3.14	11.5
9	0.13	8,813	0.00	0.00	1.00	5.8	0.150	0.100	29.25	3.14	11.5

10	0.13	8,813	0.00	0.00	1.00	18.7	0.063	0.100	32.50	3.14	11.5
11	0.13	8,813	0.00	0.00	1.00	11.8	0.150	0.100	35.75	3.14	11.5
18	0.13	8,813	0.00	0.00	1.00	11.8	0.150	0.100	58.50	3.14	11.5
19	0.13	8,813	0.00	0.00	1.00	15.9	0.132	0.100	61.75	3.14	11.5
20	0.13	8,813	0.00	0.00	1.00	33.2	0.100	0.100	65.00	3.14	11.5
Toe						295.3	0.149	0.109	65.00		

2.544 kips total unreduced pile weight ($g = 32.169 \text{ ft/s}^2$)

2.544 kips total reduced pile weight ($g = 32.169 \text{ ft/s}^2$)

OTHER OPTIONS

Pile Damping (%):	1	Pile Damping Fact. (kips/ft/s):	0.409
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EXTREMA TABLE at 60.0 FT; HAMMER: D 19-42

Shaft/Toe Gain/Loss Factor = 0.667/1.000

Rut = 466.6 kips

Rtoe = 295.3 kips

Time Inc. = 0.047 ms

Hammer

DELMAG D 19-42

Efficiency

0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	360.8	0.00	31.50	13.98	0.800	16.06
6.5	11.0	362.7	0.96	31.66	13.92	0.764	15.61
9.7	20.9	362.5	1.83	31.64	13.85	0.726	15.10
13.0	30.4	360.8	2.65	31.50	13.76	0.686	14.53
16.3	39.3	364.5	3.43	31.81	13.65	0.646	13.95
19.5	46.9	368.2	4.09	32.14	13.45	0.606	13.37
22.8	53.6	370.0	4.68	32.30	13.11	0.564	12.75
26.0	59.4	382.5	5.18	33.39	12.66	0.521	11.94
29.3	58.8	392.5	5.14	34.26	12.11	0.478	10.89
32.5	54.6	392.2	4.76	34.23	11.52	0.435	9.57
35.8	37.9	376.0	3.30	32.82	10.88	0.395	8.19
39.0	28.2	362.5	2.46	31.64	10.24	0.356	6.99
42.3	18.8	349.7	1.64	30.52	9.59	0.318	5.90
45.5	9.9	336.9	0.86	29.40	8.95	0.281	4.92
48.7	0.9	326.8	0.08	28.52	8.41	0.245	4.05
52.0	0.0	316.1	0.00	27.59	7.96	0.210	3.29
55.2	0.0	302.9	0.00	26.44	7.59	0.177	2.64
58.5	0.0	289.6	0.00	25.28	7.25	0.145	2.08
61.7	0.0	278.8	0.00	24.34	6.42	0.114	1.58
65.0	0.0	265.1	0.00	23.14	4.56	0.085	1.20

Converged Stroke (ft)

7.06

Fixed Combustion Pressure (psi)

1,165.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81

7.82

7.21

7.09

7.06

Shaft/Toe Gain/Loss Factor = 1.000/1.000

Rut = 531.9 kips

Rtoe = 295.3 kips

Time Inc. = 0.046 ms

Hammer

DELMAG D 19-42

Efficiency

0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	349.0	0.00	30.47	14.06	0.768	15.94
6.5	12.4	350.0	1.08	30.55	13.99	0.730	15.44
9.7	24.1	352.1	2.10	30.74	13.92	0.690	14.87
13.0	34.1	368.2	2.98	32.14	13.82	0.649	14.24
16.3	43.9	377.1	3.84	32.91	13.69	0.607	13.61
19.5	52.7	379.5	4.60	33.13	13.45	0.564	12.96
22.8	60.3	385.3	5.26	33.63	13.01	0.521	12.28
26.0	66.9	403.5	5.84	35.22	12.48	0.477	11.38
29.3	65.8	408.2	5.74	35.63	11.83	0.433	10.19
32.5	59.2	401.2	5.17	35.02	11.00	0.390	8.85
35.8	41.1	383.3	3.59	33.45	10.07	0.349	7.42
39.0	21.6	363.2	1.89	31.70	9.15	0.311	6.09
42.3	4.5	343.7	0.39	30.00	8.39	0.274	4.94
45.5	0.0	324.2	0.00	28.30	7.77	0.239	3.96
48.7	0.0	306.2	0.00	26.73	7.23	0.206	3.12

52.0	0.0	288.7	0.00	25.20	6.76	0.175	2.42
55.2	0.0	270.2	0.00	23.58	6.36	0.145	1.85
58.5	0.0	251.6	0.00	21.96	6.05	0.118	1.38
61.7	0.0	234.4	0.00	20.46	5.48	0.092	1.01
65.0	0.0	216.6	0.00	18.90	4.04	0.068	0.75

Converged Stroke (ft) 7.11 Fixed Combustion Pressure (psi) 1,165.0
(Eq) Strokes Analyzed and Last Return (ft)
10.81 7.84 7.25 7.14 7.11

SUMMARY TABLE at 60.0 FT; HAMMER: D 19-42

Rut kips	BI Ct b/ft	Stk Dn ft	Stk Up ft	Mx T-Str ksi	LTop ft	Mx C-Str ksi	LTop ft	ENTHRU kip-ft	BI Rt b/min	ActRes kips
466.6	9,999	7.09	7.06	5.18	26.0	34.26	29.3	16.1	44.2	396.2
531.9	9,999	7.14	7.11	5.84	26.0	35.63	29.3	15.9	44.1	405.7

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.667/1.000									
Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.8	0.0	0.8	0.0	0.00	0.00	10.81	0.0	D 19-42
10.0	1.7	0.0	1.7	0.0	0.00	0.00	10.81	0.0	D 19-42
15.0	2.5	0.0	2.5	0.0	0.00	0.00	10.81	0.0	D 19-42
20.0	24.7	1.7	23.0	2.7	9.63	0.00	3.37	15.8	D 19-42
25.0	264.5	11.6	252.9	57.4	39.35	5.34	6.62	15.7	D 19-42
30.0	61.6	42.6	19.1	8.1	17.64	1.14	4.16	13.0	D 19-42
35.0	89.9	70.8	19.1	13.6	20.64	2.26	4.54	12.0	D 19-42
40.0	113.8	94.7	19.1	18.8	22.08	2.42	4.82	11.7	D 19-42
45.0	129.0	109.9	19.1	22.1	22.95	2.04	4.98	11.6	D 19-42
50.0	140.4	121.3	19.1	24.8	23.25	1.99	5.10	11.6	D 19-42
55.0	149.4	130.3	19.1	27.4	23.73	1.47	5.18	11.5	D 19-42
55.6	150.2	131.1	19.1	27.7	23.72	1.35	5.18	11.5	D 19-42
55.6	150.2	131.1	19.1	27.7	23.72	1.34	5.19	11.5	D 19-42
60.0	466.6	171.3	295.3	9,999.0	34.26	5.18	7.09	16.1	D 19-42
60.0	466.6	171.3	295.3	9,999.0	34.26	5.18	7.09	16.1	D 19-42

G/L at Shaft and Toe: 1.000/1.000									
Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.8	0.0	0.8	0.0	0.00	0.00	10.81	0.0	D 19-42
10.0	1.7	0.0	1.7	0.0	0.00	0.00	10.81	0.0	D 19-42
15.0	2.5	0.0	2.5	0.0	0.00	0.00	10.81	0.0	D 19-42
20.0	25.6	2.6	23.0	2.8	9.97	0.01	3.40	15.7	D 19-42
25.0	269.7	16.8	252.9	60.1	39.36	5.52	6.67	15.8	D 19-42
30.0	74.4	55.3	19.1	10.4	18.83	1.96	4.32	12.4	D 19-42
35.0	116.8	97.8	19.1	19.1	22.49	2.63	4.83	11.8	D 19-42
40.0	152.6	133.6	19.1	26.7	24.03	2.75	5.17	11.9	D 19-42
45.0	175.4	156.3	19.1	32.4	25.14	2.45	5.41	12.2	D 19-42
50.0	192.6	173.5	19.1	37.8	25.85	2.94	5.58	12.4	D 19-42
55.0	206.1	187.0	19.1	43.3	26.44	3.17	5.69	12.4	D 19-42
55.6	207.2	188.2	19.1	43.1	26.52	3.07	5.76	12.6	D 19-42
55.6	207.3	188.2	19.1	43.1	26.51	3.07	5.76	12.6	D 19-42
60.0	531.9	236.6	295.3	9,999.0	35.63	5.84	7.14	15.9	D 19-42
60.0	531.9	236.6	295.3	9,999.0	35.63	5.84	7.14	15.9	D 19-42

PIER 1

PILE LENGTH & DRIVABILITY

Boring Used: B-008-0-23

Starting Elevation (0.0 ft) = 870.0 ft amsl =

866.0 ft amsl (Bottom of Pile Cap Ele.) + 4.0 ft (assumed footing excavation)

Assumed Groundwater Depth = 4.0 ft (Ele. 866.0 ft amsl)

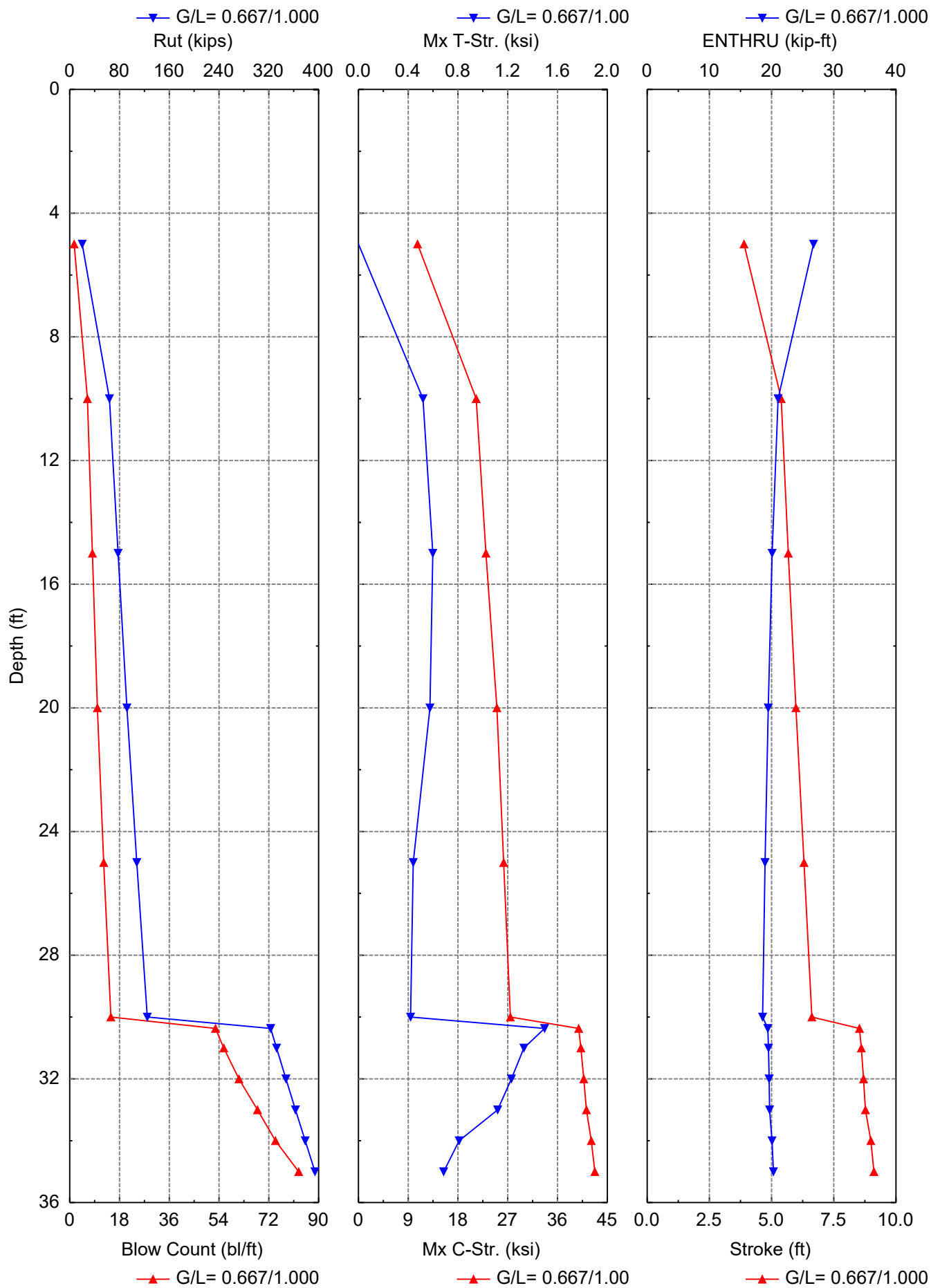
Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 0.667/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	20.2	3.3	16.8	1.6	10.695	0.000	3.89	26.7	D 19-42
10.0	63.9	18.2	45.7	6.4	21.295	0.519	5.39	21.0	D 19-42
15.0	77.8	32.1	45.7	8.2	23.057	0.598	5.67	20.1	D 19-42
20.0	92.0	46.3	45.7	10.0	25.030	0.575	5.98	19.5	D 19-42
25.0	107.9	62.2	45.7	12.3	26.255	0.440	6.30	19.0	D 19-42
30.0	124.7	79.0	45.7	14.9	27.474	0.419	6.62	18.6	D 19-42
30.4	323.0	80.8	242.2	52.7	39.815	1.496	8.53	19.4	D 19-42
31.0	332.4	85.0	247.4	55.7	40.225	1.329	8.61	19.5	D 19-42
32.0	347.5	91.8	255.7	61.2	40.754	1.229	8.69	19.6	D 19-42
33.0	362.9	98.8	264.0	67.9	41.232	1.119	8.77	19.7	D 19-42
34.0	378.4	106.1	272.3	74.4	42.107	0.809	8.99	20.1	D 19-42
35.0	394.2	113.6	280.6	82.8	42.737	0.685	9.11	20.3	D 19-42

Total driving time: 13 minutes; Total Number of Blows: 545 (starting at penetration 5.0 ft)

Starting Elevation (0.0 ft) = 870.0 ft amsl =
(Pile cap ele. 866.0 + assumed excavation 4.0 ft)

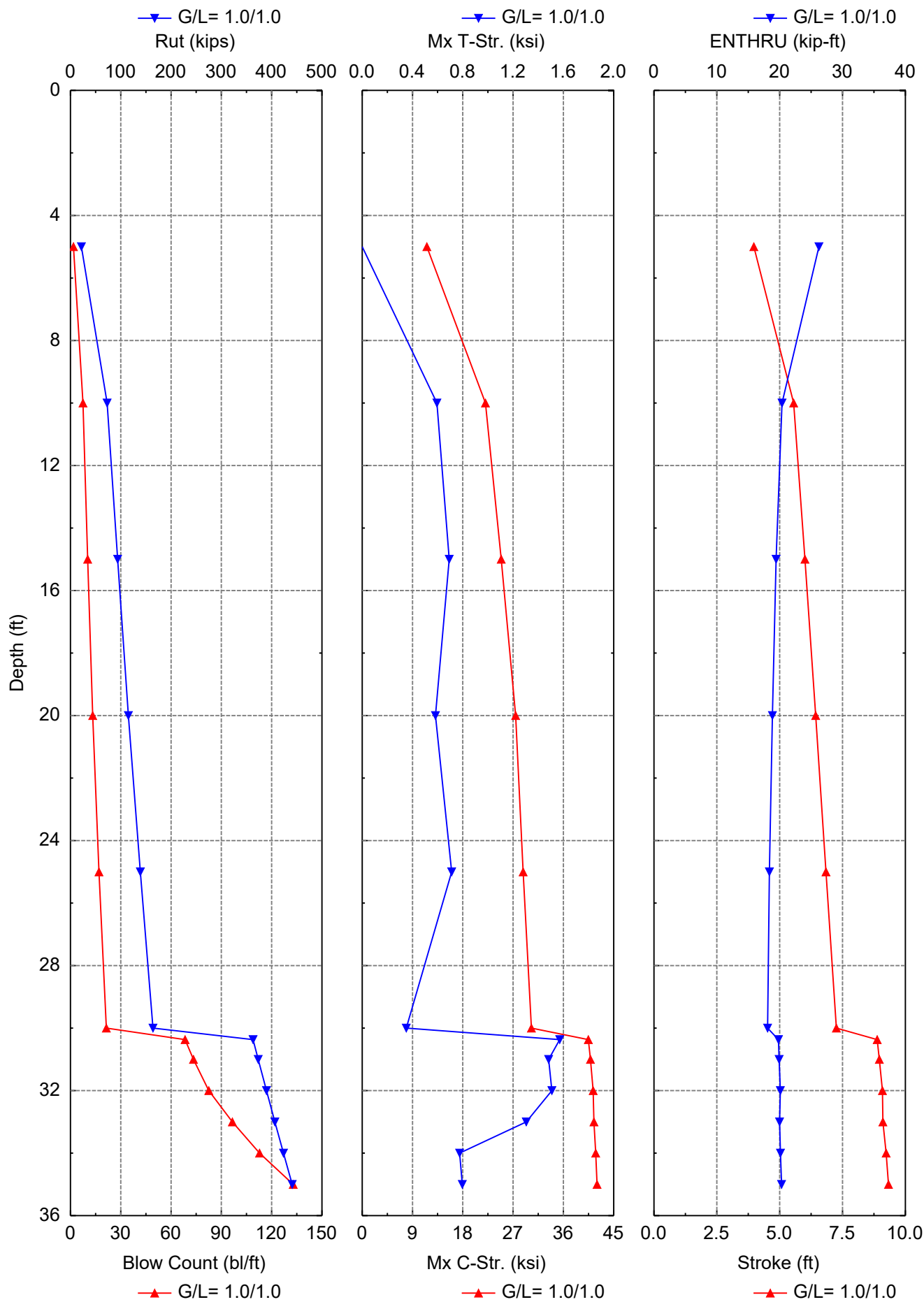
Assumed Groundwater Depth = 0.0 ft (Ele. 866.0 ft amsl)



Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	21.8	5.0	16.8	1.8	11.563	0.000	3.97	26.2	D 19-42
10.0	73.0	27.3	45.7	7.5	22.058	0.594	5.55	20.4	D 19-42
15.0	93.8	48.1	45.7	10.3	24.882	0.692	6.01	19.4	D 19-42
20.0	115.2	69.5	45.7	13.3	27.448	0.582	6.43	18.8	D 19-42
25.0	139.0	93.3	45.7	17.0	28.794	0.712	6.84	18.4	D 19-42
30.0	164.2	118.5	45.7	21.4	30.289	0.351	7.25	18.1	D 19-42
30.4	363.0	120.8	242.2	68.4	40.475	1.571	8.88	19.8	D 19-42
31.0	373.3	125.9	247.4	73.4	40.849	1.482	8.96	19.9	D 19-42
32.0	389.8	134.1	255.7	82.5	41.325	1.509	9.09	20.1	D 19-42
33.0	406.5	142.5	264.0	96.6	41.467	1.305	9.10	20.0	D 19-42
34.0	423.6	151.2	272.3	112.8	41.784	0.775	9.23	20.1	D 19-42
35.0	440.9	160.2	280.6	132.9	42.030	0.796	9.32	20.3	D 19-42

Total driving time: 18 minutes; Total Number of Blows: 754 (starting at penetration 5.0 ft)



SOIL PROFILE

Depth ft	Soil Type -	Spec. Wt lb/ft ³	Su ksf	Phi °	Unit Rs ksf	Unit Rt ksf
0.0	Sand	120.0	0.0	0.0	0.00	0.00
4.0	Sand	120.0	0.0	0.0	0.00	0.85
4.0	Clay	112.0	1.7	0.0	1.10	15.75
7.1	Clay	112.0	1.7	0.0	1.10	15.75
7.1	Clay	130.0	4.7	0.0	2.48	42.75
30.2	Clay	130.0	4.7	0.0	2.48	42.75
30.2	Sand	140.0	0.0	39.0	2.08	219.69
36.5	Sand	140.0	0.0	39.0	2.55	268.61
36.5	Clay	132.0	5.3	0.0	5.35	48.15
41.5	Clay	132.0	5.3	0.0	5.35	48.15
41.5	Sand	132.0	0.0	36.0	2.02	151.75
46.5	Sand	132.0	0.0	36.0	2.25	151.75
46.5	Sand	140.0	0.0	40.0	3.52	375.94
52.7	Sand	140.0	0.0	40.0	4.03	375.94

Groundwater Assumptions:

Pier 1 – Pile Length & Drivability

General Site Information

×

General Information

Total Pile Length: ft

Initial Pene. Depth: ft

Final Pene. Depth: ft

Water Table: ft

Eff. Overb. Pressure: ksf

☐ Additional Input

Import SPT Data

Import CPT Data

Import DIGGS Data

OK

Cancel

GRLWEAP 1980 Tomlinson Graphs:

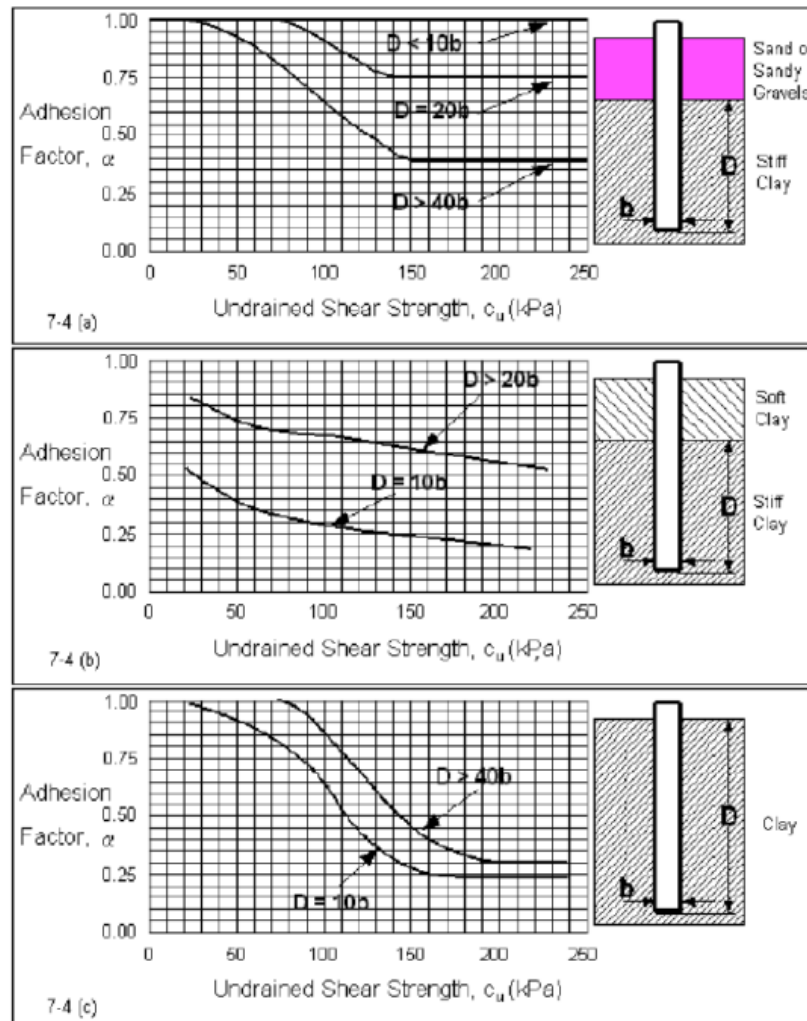


Figure 5.2: Adhesion factor for driven pile in cohesive soil (after Tomlinson 1980).

Pier 1

Soil Profile Input

FHWA Method | Other Parameters (Impact)

Update=> Ru=268.5; Rs=222.8 kips

Layers Expand Graph

Site Properties

Number of Layers: 8

Final Tip Penetration: 30.2 ft

Water Table: 4. ft

Eff. Overburden at Grade: 0. ksf

Layer Properties

Layer: 1 of 8

Depth: Top: 4.1 Bottom: 7.1 ft

Thickness: 3. ft

Soil Type/Property

Gravel

Other

Granular: Skin Friction End Bearing

SPT N Value: 0. 0. <=60

ϕ degree <= 45

Cohesive: S_u 1.75 ksf <= 5

Tomlinson 1980 (3) Click to change.

Rock*: Description:

Other*: Cohesive

Unit Weight: 112. lb/ft³

For selected soil, you need to specify:

Friction: End Bearing:

Top: 1.097 15.75 ksf

Bottom: 1.097 15.75 ksf

*Rock and Other are not FHWA defined soil types

OK Cancel

Soil Profile Input

FHWA Method | Other Parameters (Impact)

Update=> Ru=268.5; Rs=222.8 kips

Layers Expand Graph

Site Properties

Number of Layers: 8

Final Tip Penetration: 30.2 ft

Water Table: 4. ft

Eff. Overburden at Grade: 0. ksf

Layer Properties

Layer: 4 of 8

Depth: Top: 7.1 Bottom: 30.2 ft

Thickness: 23.1 ft

Soil Type/Property

Gravel

Other

Granular: Skin Friction End Bearing

SPT N Value: 0. 0. <=60

ϕ degree <= 45

Cohesive: S_u 4.75 ksf <= 5

Auto Adhesion (1980 - 3) Click to change.

Rock*: Description:

Other*: Cohesive

Unit Weight: 130. lb/ft³

For selected soil, you need to specify:

Friction: End Bearing:

Top: 2.484 42.75 ksf

Bottom: 2.484 42.75 ksf

*Rock and Other are not FHWA defined soil types

OK Cancel

GRLWEAP: Wave Equation Analysis of Pile Foundations

FRA-Cemetery Rd SUP Bridge + Pier 1 - 14-inch CIP
NATIONAL ENGINEERING AND ARCHITECTURAL

7/9/2026
GRLWEAP 14.2.2.0

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

The GRLWEAP program simulates the behavior of a preformed pile driven by either an impact hammer or a vibratory hammer. The program is based on mathematical models, which describe motion and forces of hammer, driving system, pile and soil under the hammer action. Under certain conditions, the models only crudely approximate, often complex, dynamic situations.

A wave equation analysis generally relies on input data, which represents normal situations. In particular, the hammer data file supplied with the program assumes that the hammer is in good working order. All of the input data selected by the user may be the best available information at the time when the analysis is performed. However, input data and therefore results may significantly differ from actual field conditions.

Therefore, the program authors recommend prudent use of the GRLWEAP results. Soil response and hammer performance should be verified by static and/or dynamic testing and measurements. Estimates of bending or other local stresses (e.g., helmet or clamp contact, uneven rock surfaces etc.), prestress effects and others must also be accounted for by the user.

The calculated capacity-blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

Finally, the GRLWEAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors.

HAMMER DATA

Hammer Model:	D 19-42	Made By:	DELMAG
Hammer ID:	41	Hammer Type:	OED
Hammer Database Type:	PDI		
Hammer Database Name:			PDIHammer.gwh

Hammer and Drive System Segment Data

Segment	Weight kips	Stiffness kips/in	COR -	C-Slack in	Damping kips/ft/s
-					
1	0.800	140,084.4	1.000	0.000	
2	0.800	140,084.4	1.000	0.000	
3	0.800	140,084.4	1.000	0.000	
4	0.800	140,084.4	1.000	0.000	
5	0.800	70,754.7	0.900	0.120	
Imp Block	0.753	109,976.0	0.800	0.120	
Helmet	2.500				5.3

Ram Weight: (kips)	4.00	Ram Length: (ft)	10.76
Ram Area: (in ²)	124.69		
Maximum (Eq) Stroke: (ft)	10.81	Actual (Eq) Stroke: (ft)	10.81
Efficiency:	0.800	Rated Energy: (kip-ft)	43.24
Maximum Pressure: (psi)	1,600.00	Actual Pressure: (psi)	1,600.00
Combustion Delay: (ms)	2.00	Ignition Duration: (ms)	2.00
Expansion Exponent:	1.25		

Hammer Cushion

Pile Cushion

Cross Sect. Area: (in ²)	415.00	Cross Sect. Area: (in ²)	0.00
Elastic Modulus: (ksi)	530.0	Elastic Modulus: (ksi)	0.0
Thickness: (in)	2.00	Thickness: (in)	0.00
Coeff. of Restitution:	0.800	Coeff. of Restitution:	0.500
RoundOut: (in)	0.120	RoundOut: (in)	0.120
Stiffness: (kips/in)	109,976.0	Stiffness: (kips/in)	0.0
Helmet Weight: (kips)	2.500		

PILE INPUT

Uniform Pile		Pile Type:	Closed-End Pipe
Pile Length: (ft)	35.000	Pile Penetration: (ft)	35.000
Pile Size: (ft)	1.17	Toe Area: (in ²)	153.94

Table of Depths Analyzed with Driving System Modifiers

Depth ft	Temp Length ft	Wait Time Hr	Hammer -
5.00	35.0	0.0	DELMAG D 19-42
10.00	35.0	0.0	DELMAG D 19-42
15.00	35.0	0.0	DELMAG D 19-42
20.00	35.0	0.0	DELMAG D 19-42
25.00	35.0	0.0	DELMAG D 19-42
30.00	35.0	0.0	DELMAG D 19-42
30.37	35.0	0.0	DELMAG D 19-42
31.00	35.0	0.0	DELMAG D 19-42
32.00	35.0	0.0	DELMAG D 19-42
33.00	35.0	0.0	DELMAG D 19-42
34.00	35.0	0.0	DELMAG D 19-42
35.00	35.0	0.0	DELMAG D 19-42

Other Information for DELMAG D 19-42

Depth ft	Stroke ft	Diesel Pressure %	Efficiency -	P.C. Stiff. Fact. -	P.C. COR -
5.00	10.8	100.0	0.80	1.0	0.50
10.00	10.8	100.0	0.80	1.0	0.50
15.00	10.8	100.0	0.80	1.0	0.50
20.00	10.8	100.0	0.80	1.0	0.50
25.00	10.8	100.0	0.80	1.0	0.50
30.00	10.8	100.0	0.80	1.0	0.50
30.37	10.8	100.0	0.80	1.0	0.50
31.00	10.8	100.0	0.80	1.0	0.50
32.00	10.8	100.0	0.80	1.0	0.50
33.00	10.8	100.0	0.80	1.0	0.50
34.00	10.8	100.0	0.80	1.0	0.50
35.00	10.8	100.0	0.80	1.0	0.50

PILE, SOIL, ANALYSIS OPTIONS

Analysis type:	Driveability Analysis	Parallel Driveability Analysis:	No
Static Analysis Method:	FHWA	Soil Damping Option:	Smith
Max No Analysis Iterations:	0	Time Increment/Critical:	160
Residual Stress Analysis:	0	Analysis Time-Input(ms):	0
Output Level:	Normal	Gravitational Acceleration (ft/s ²):	32.169
Hammer Gravity (ft/s ²):	32.169	Pile Gravity (ft/s ²):	32.169

DRIVEABILITY ANALYSIS

Analysis Depth (ft)	35.00	Standard Soil Setup	
Hammer Name	DELMAG D 19-42	Hammer ID	41
Diesel Pressure: (psi)	230.40	Stroke (ft)	10.81
Efficiency	0.80		
Shaft Gain/Loss Factor	0.667	Toe Gain/Loss Factor	1.000
Shaft Gain/Loss Factor	1.000	Toe Gain/Loss Factor	1.000

SOIL RESISTANCE PARAMETERS (FHWA)

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Setup F. -	Limit D. ft	Setup T. Hours	EB Area in ²
0.00	0.0	0.0	0.100	0.147	0.150	0.149	1.5	6.00	168.0	153.94
2.00	0.0	0.4	0.100	0.147	0.150	0.149	1.5	6.00	168.0	153.94
4.00	0.0	0.9	0.100	0.147	0.150	0.149	1.5	6.00	168.0	153.94
4.00	1.7	15.3	0.100	0.147	0.150	0.149	1.5	6.00	168.0	153.94
4.10	1.7	15.3	0.100	0.147	0.150	0.149	1.5	6.00	168.0	153.94
4.10	1.3	15.7	0.100	0.146	0.150	0.149	1.5	6.00	168.0	153.94
7.10	1.3	15.7	0.100	0.146	0.150	0.149	1.5	6.00	168.0	153.94
7.10	1.2	42.7	0.100	0.117	0.150	0.149	1.5	6.00	168.0	153.94
30.20	1.2	42.7	0.100	0.117	0.150	0.149	1.5	6.00	168.0	153.94
30.20	2.1	225.2	0.100	0.120	0.100	0.149	1.2	6.00	24.0	153.94
32.30	2.3	241.6	0.100	0.120	0.100	0.149	1.2	6.00	24.0	153.94
34.40	2.4	257.9	0.100	0.120	0.100	0.149	1.2	6.00	24.0	153.94
35.00	2.5	262.5	0.100	0.120	0.100	0.149	1.2	6.00	24.0	153.94

PILE PROFILE

Lb Top ft	X-Area in ²	E-Mod ksi	Spec. Wt lb/ft ³	Perim. ft	C-Index -	Wave Sp ft/s	Impedance kips/ft/s
0.00	13.4	30,000	492.00	3.665	0	16,806.4	23.9
35.00	13.4	30,000	492.00	3.665	0	16,806.4	23.9

PILE AND SOIL MODEL

Total Capacity Rut (kips):

394.227

Seg. -	Weight kips	Stiffn. kips/in	C-Slk in	T-Slk in	COR -	Ru kips	Js/Jt s/ft	Qs/Qt in	LbTop ft	Perim. ft	X-Area in ²
1	0.15	10,542	0.12	0.00	0.85	0.0	0.000	0.100	3.18	3.67	13.4
2	0.15	10,542	0.00	0.00	1.00	7.7	0.150	0.100	6.36	3.67	13.4
3	0.15	10,542	0.00	0.00	1.00	9.7	0.150	0.100	9.55	3.67	13.4
4	0.15	10,542	0.00	0.00	1.00	9.6	0.150	0.100	12.73	3.67	13.4
9	0.15	10,542	0.00	0.00	1.00	9.6	0.150	0.100	28.64	3.67	13.4
10	0.15	10,542	0.00	0.00	1.00	15.6	0.118	0.100	31.82	3.67	13.4
11	0.15	10,542	0.00	0.00	1.00	23.0	0.100	0.100	35.00	3.67	13.4
Toe						280.6	0.149	0.120	35.00		

1.604 kips total unreduced pile weight (g = 32.169 ft/s²)1.604 kips total reduced pile weight (g = 32.169 ft/s²)

OTHER OPTIONS

Pile Damping (%):	1	Pile Damping Fact. (kips/ft/s):	0.479
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EXTREMA TABLE at 35.0 FT; HAMMER: D 19-42

Shaft/Toe Gain/Loss Factor = 0.667/1.000

Rut = 394.2 kips

Rtoe = 280.6 kips

Time Inc. = 0.055 ms

Hammer

DELMAG D 19-42

Efficiency

0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.2	0.0	567.1	0.00	42.26	16.13	0.642	20.29
6.4	9.2	573.4	0.68	42.74	15.51	0.598	19.00
9.5	7.4	553.8	0.55	41.28	14.91	0.555	17.24
12.7	4.1	537.0	0.30	40.02	14.30	0.512	15.49
15.9	0.1	519.5	0.01	38.72	13.72	0.472	13.92
19.1	0.0	510.1	0.00	38.02	13.14	0.433	12.55
22.3	0.0	497.6	0.00	37.09	12.60	0.396	11.28
25.5	0.0	476.6	0.00	35.52	12.20	0.358	10.10
28.6	0.0	457.5	0.00	34.10	11.48	0.321	9.02
31.8	0.0	460.9	0.00	34.35	10.56	0.289	8.11
35.0	0.0	466.9	0.00	34.80	7.89	0.259	7.48

Converged Stroke (ft)

9.10

Fixed Combustion Pressure (psi)

1,600.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81

9.00

9.11

9.10

Shaft/Toe Gain/Loss Factor = 1.000/1.000

Rut = 440.9 kips

Rtoe = 280.6 kips

Time Inc. = 0.055 ms

Hammer

DELMAG D 19-42

Efficiency

0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.2	0.0	553.0	0.00	41.22	16.17	0.604	20.30
6.4	10.7	563.9	0.80	42.03	15.31	0.555	18.68
9.5	4.0	551.7	0.30	41.12	14.46	0.509	16.44
12.7	0.0	529.5	0.00	39.47	13.63	0.464	14.29
15.9	0.0	508.6	0.00	37.90	12.81	0.420	12.35
19.1	0.0	500.4	0.00	37.30	12.01	0.379	10.66
22.3	0.0	492.4	0.00	36.70	11.28	0.341	9.21
25.5	0.0	471.3	0.00	35.13	10.76	0.304	7.94
28.6	0.0	448.8	0.00	33.45	10.09	0.267	6.81
31.8	0.0	430.0	0.00	32.05	9.57	0.233	5.81
35.0	0.0	409.5	0.00	30.53	7.40	0.203	5.20

Converged Stroke (ft)

9.31

Fixed Combustion Pressure (psi)

1,600.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81

9.32

9.31

SUMMARY TABLE at 35.0 FT; HAMMER: D 19-42

Rut kips	Bl Ct b/ft	Stk Dn ft	Stk Up ft	Mx T-Str ksi	LTop ft	Mx C-Str ksi	LTop ft	ENTHRU kip-ft	Bl Rt b/min	ActRes kips
394.2	82.8	9.11	9.10	0.68	6.4	42.74	6.4	20.3	39.2	394.2
440.9	132.9	9.32	9.31	0.80	6.4	42.03	6.4	20.3	38.8	440.9

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.667/1.000									
Depth ft	Rut kips	Rshaft kips	Rtoe kips	BI Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	20.2	3.3	16.8	1.6	10.69	0.00	3.89	26.7	D 19-42
10.0	63.9	18.2	45.7	6.4	21.30	0.52	5.39	21.0	D 19-42
15.0	77.8	32.1	45.7	8.2	23.06	0.60	5.67	20.1	D 19-42
20.0	92.0	46.3	45.7	10.0	25.03	0.57	5.98	19.5	D 19-42
25.0	107.9	62.2	45.7	12.3	26.26	0.44	6.30	19.0	D 19-42
30.0	124.7	79.0	45.7	14.9	27.47	0.42	6.62	18.6	D 19-42
30.4	323.0	80.8	242.2	52.7	39.82	1.50	8.53	19.4	D 19-42
31.0	332.4	85.0	247.4	55.7	40.22	1.33	8.61	19.5	D 19-42
32.0	347.5	91.8	255.7	61.2	40.75	1.23	8.69	19.6	D 19-42
33.0	362.9	98.8	264.0	67.9	41.23	1.12	8.77	19.7	D 19-42
34.0	378.4	106.1	272.3	74.4	42.11	0.81	8.99	20.1	D 19-42
35.0	394.2	113.6	280.6	82.8	42.74	0.68	9.11	20.3	D 19-42

G/L at Shaft and Toe: 1.000/1.000									
Depth ft	Rut kips	Rshaft kips	Rtoe kips	BI Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	21.8	5.0	16.8	1.8	11.56	0.00	3.97	26.2	D 19-42
10.0	73.0	27.3	45.7	7.5	22.06	0.59	5.55	20.4	D 19-42
15.0	93.8	48.1	45.7	10.3	24.88	0.69	6.01	19.4	D 19-42
20.0	115.2	69.5	45.7	13.3	27.45	0.58	6.43	18.8	D 19-42
25.0	139.0	93.3	45.7	17.0	28.79	0.71	6.84	18.4	D 19-42
30.0	164.2	118.5	45.7	21.4	30.29	0.35	7.25	18.1	D 19-42
30.4	363.0	120.8	242.2	68.4	40.47	1.57	8.88	19.8	D 19-42
31.0	373.3	125.9	247.4	73.4	40.85	1.48	8.96	19.9	D 19-42
32.0	389.8	134.1	255.7	82.5	41.32	1.51	9.09	20.1	D 19-42
33.0	406.5	142.5	264.0	96.6	41.47	1.31	9.10	20.0	D 19-42
34.0	423.6	151.2	272.3	112.8	41.78	0.78	9.23	20.1	D 19-42
35.0	440.9	160.2	280.6	132.9	42.03	0.80	9.32	20.3	D 19-42

FORWARD ABUTMENT

PILE LENGTH

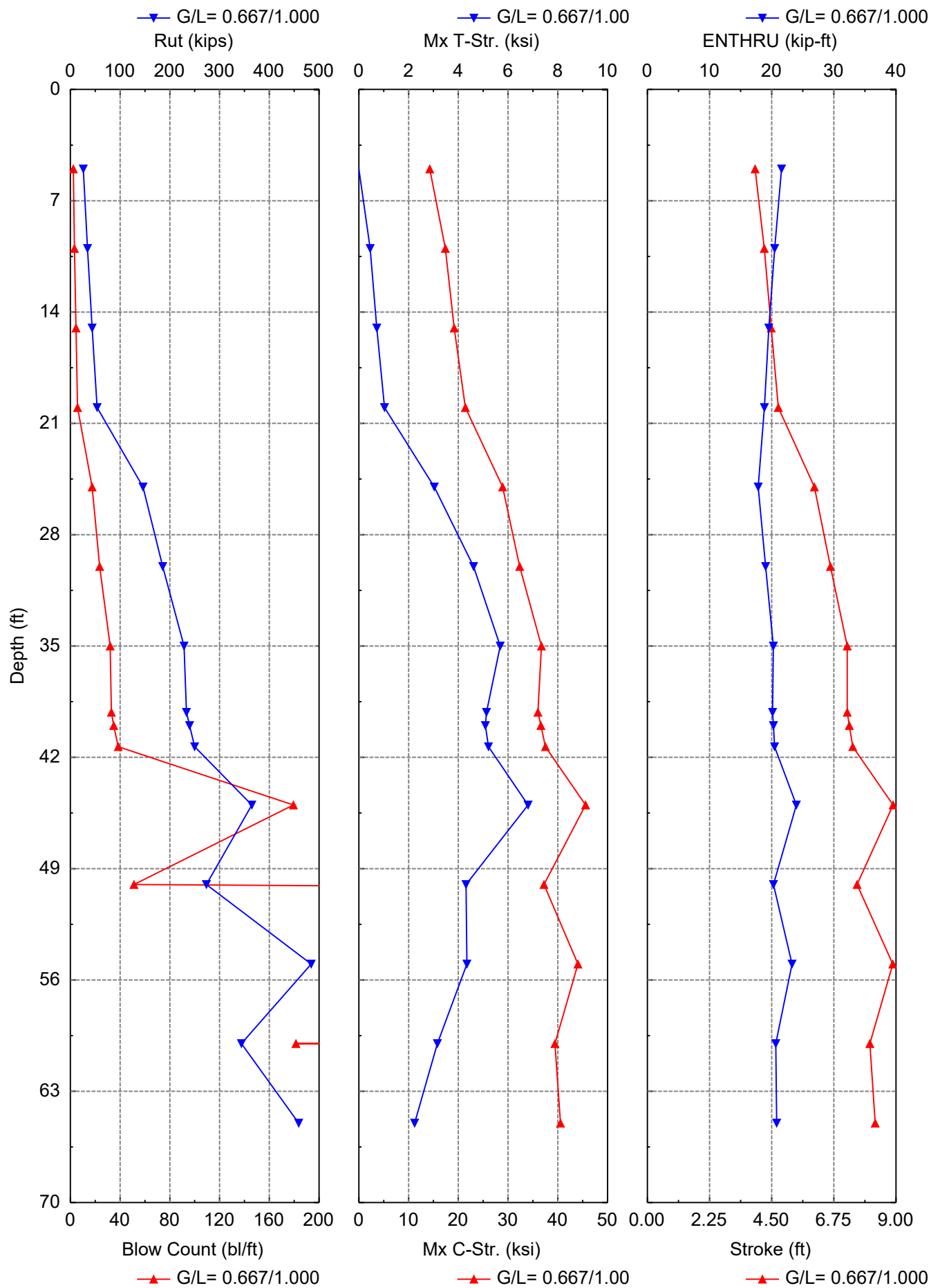
Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 0.667/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	25.9	0.9	25.0	2.4	14.254	0.000	3.90	21.6	D 19-42
10.0	34.6	8.8	25.8	3.3	17.396	0.453	4.23	20.5	D 19-42
15.0	43.8	18.0	25.8	4.5	19.173	0.722	4.48	19.5	D 19-42
20.0	53.9	28.1	25.8	5.8	21.417	1.033	4.74	18.8	D 19-42
25.0	146.2	42.6	103.6	17.5	28.917	3.033	6.04	17.8	D 19-42
30.0	185.6	61.1	124.5	23.6	32.342	4.613	6.62	19.0	D 19-42
35.0	228.6	83.1	145.5	32.0	36.703	5.682	7.23	20.3	D 19-42
39.2	233.5	101.5	132.0	33.0	36.007	5.133	7.23	20.1	D 19-42
40.0	239.8	104.8	134.9	34.9	36.598	5.088	7.31	20.3	D 19-42
41.3	250.0	110.3	139.7	38.5	37.518	5.215	7.43	20.5	D 19-42
45.0	364.8	129.2	235.6	179.4	45.581	6.803	8.89	24.0	D 19-42
50.0	273.5	154.4	119.2	51.1	37.223	4.309	7.59	20.3	D 19-42
55.0	484.2	189.0	295.3	9999.0	44.042	4.347	8.88	23.3	D 19-42
60.0	343.9	224.7	119.2	181.4	39.429	3.160	8.05	20.7	D 19-42
65.0	459.1	261.3	197.8	9999.0	40.528	2.244	8.25	20.8	D 19-42

Refusal occurred; no driving time output possible.

Starting Elevation (0.0 ft) = 882.9 ft amsl

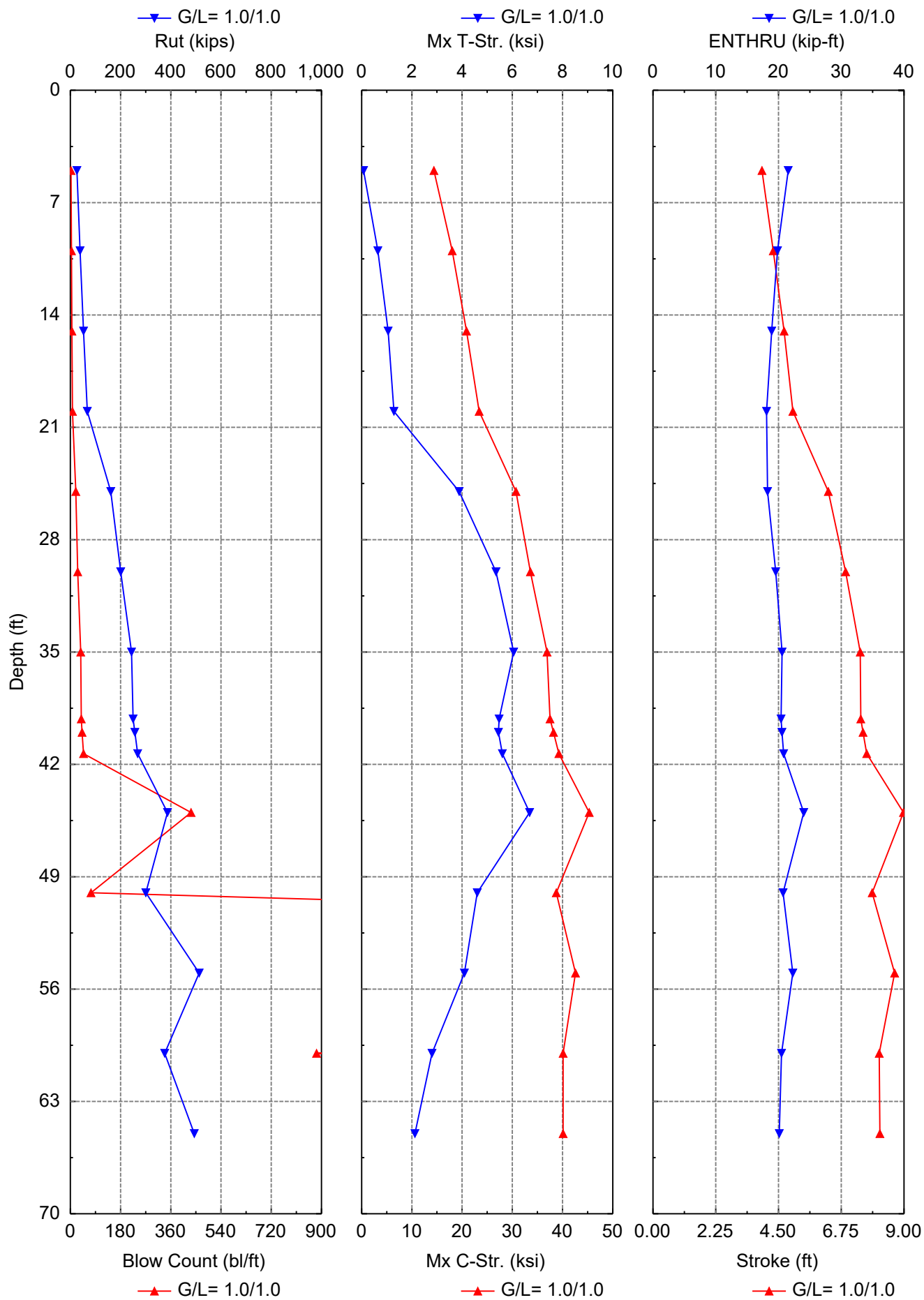
Assumed Groundwater Depth = 0.0 ft (Ele. 882.9 ft amsl)



Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	26.2	1.2	25.0	2.4	14.390	0.077	3.91	21.5	D 19-42
10.0	38.8	13.0	25.8	3.9	18.043	0.648	4.31	19.8	D 19-42
15.0	52.5	26.7	25.8	5.6	20.883	1.054	4.70	18.9	D 19-42
20.0	67.8	42.0	25.8	7.6	23.384	1.285	5.02	18.1	D 19-42
25.0	161.1	57.6	103.6	19.7	30.732	3.884	6.28	18.3	D 19-42
30.0	200.6	76.1	124.5	26.5	33.599	5.348	6.91	19.6	D 19-42
35.0	243.5	98.0	145.5	37.1	36.905	6.054	7.43	20.6	D 19-42
39.2	249.9	117.9	132.0	39.1	37.504	5.474	7.45	20.4	D 19-42
40.0	256.8	121.9	134.9	42.0	38.199	5.449	7.53	20.5	D 19-42
41.3	268.1	128.5	139.7	47.1	39.263	5.603	7.66	20.8	D 19-42
45.0	386.7	151.1	235.6	432.5	45.317	6.687	8.98	24.0	D 19-42
50.0	300.5	181.3	119.2	74.2	38.754	4.602	7.86	20.7	D 19-42
55.0	513.3	218.1	295.3	9999.0	42.588	4.094	8.66	22.3	D 19-42
60.0	375.9	256.7	119.2	882.3	40.111	2.802	8.11	20.5	D 19-42
65.0	493.6	295.8	197.8	9999.0	40.108	2.120	8.13	20.1	D 19-42

Refusal occurred; no driving time output possible.



SOIL PROFILE

Depth ft	Soil Type -	Spec. Wt lb/ft ³	Su ksf	Phi °	Unit Rs ksf	Unit Rt ksf
0.0	Sand	125.0	0.0	34.0	0.00	0.00
4.2	Sand	125.0	0.0	34.0	0.11	9.73
4.2	Sand	132.0	0.0	39.0	0.19	26.31
6.0	Sand	132.0	0.0	39.0	0.28	38.84
6.0	Clay	128.0	3.6	0.0	3.19	32.85
21.0	Clay	128.0	3.6	0.0	3.19	32.85
21.0	Sand	130.0	0.0	38.0	0.90	110.22
26.7	Sand	130.0	0.0	38.0	1.15	141.17
26.7	Sand	128.0	0.0	38.0	1.15	141.17
32.3	Sand	128.0	0.0	38.0	1.39	170.68
32.3	Sand	130.0	0.9	38.0	1.39	170.68
37.3	Sand	130.0	0.9	38.0	1.61	197.83
37.3	Sand	132.0	0.9	37.0	1.44	159.68
42.3	Sand	132.0	0.9	37.0	1.64	182.24
42.3	Sand	132.0	3.2	39.0	2.02	281.25
47.3	Sand	132.0	3.2	39.0	2.27	313.69
47.3	Sand	132.0	0.0	36.0	1.63	151.75
52.3	Sand	132.0	0.0	36.0	1.81	151.75
52.3	Sand	132.0	2.7	40.0	2.75	375.94
57.3	Sand	132.0	2.7	40.0	3.02	375.94
57.3	Sand	132.0	0.0	36.0	1.99	151.75
62.3	Sand	132.0	0.0	36.0	2.17	151.75
62.3	Sand	132.0	0.0	38.0	2.74	251.83
65.5	Sand	132.0	0.0	38.0	2.89	251.83

Groundwater Assumptions:

Forward Abutment – Pile Length

General Site Information

×

General Information

Total Pile Length: 70.000 ft

Initial Pene. Depth: 0 ft

Final Pene. Depth: 65.000 ft

Water Table: 0 ft

Eff. Overb. Pressure: 0 ksf

☐ Additional Input

Import SPT Data

Import CPT Data

Import DIGGS Data

OK

Cancel

GRLWEAP 1980 Tomlinson Graphs:

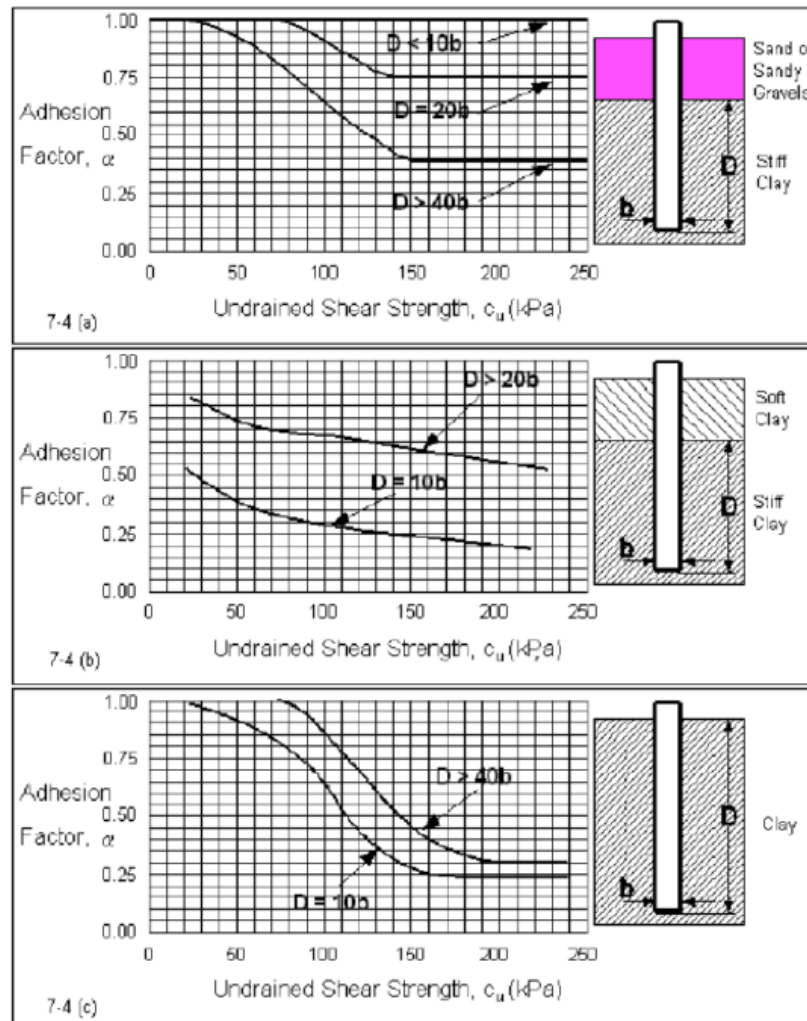


Figure 5.2: Adhesion factor for driven pile in cohesive soil (after Tomlinson 1980).

Forward Abutment:

Soil Profile Input

FHWA Method | Other Parameters (Impact)

Update=> Ru=350.8; Rs=205.3 kips

Layers Expand Graph

Site Properties

Number of Layers: 13

Final Tip Penetration: 35.0 ft

Water Table: 0.0 ft

Eff. Overburden at Grade: 0.0 ksf

Layer Properties

Layer: 4 of 13

Depth: Top: 6.1 Bottom: 21.0 ft

Thickness: 14.9 ft

Soil Type/Property

Gravel

Other Granular: Skin Friction End Bearing

SPT N Value: 0.0 0.0 <=60

ϕ degree <= 45

Cohesive: Stiff

S_u 3.65 ksf <= 5

Auto Adhesion (1980 - 3) Click to change.

Rock: Description:

Other: Stiff Cohesive

Unit Weight: 128 lb/ft³

For selected soil, you need to specify:

Friction: End Bearing:

Top: 3.194 32.85 ksf

Bottom: 3.194 32.85 ksf

*Rock and Other are not FHWA defined soil types

OK Cancel

Depth (ft)	Soil Type	Parameter
0.0 - 4.2	Granular	a=34
4.2 - 6.0	Granular	a=39
6.1 - 21.0	Stiff Cohesive	S=3.6
21.0 - 26.7	Granular	a=38
26.7 - 32.3	Granular	a=38
32.3 - 37.3	Granular	a=38
37.3 - 42.3	Granular	a=37
42.3 - 47.3	Granular	a=39
47.3 - 52.3	Sand	a=36
52.3 - 57.3	Granular	a=40
57.3 - 62.3	Sand	a=36
62.3 - 65.5	Sand	a=38

GRLWEAP: Wave Equation Analysis of Pile Foundations

FRA-Cemetery Rd SUP Bridged + Forward Abutment - 12-inch CIP
NATIONAL ENGINEERING AND ARCHITECTURAL7/9/2026
GRLWEAP 14.2.2.0

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

The GRLWEAP program simulates the behavior of a preformed pile driven by either an impact hammer or a vibratory hammer. The program is based on mathematical models, which describe motion and forces of hammer, driving system, pile and soil under the hammer action. Under certain conditions, the models only crudely approximate, often complex, dynamic situations.

A wave equation analysis generally relies on input data, which represents normal situations. In particular, the hammer data file supplied with the program assumes that the hammer is in good working order. All of the input data selected by the user may be the best available information at the time when the analysis is performed. However, input data and therefore results may significantly differ from actual field conditions.

Therefore, the program authors recommend prudent use of the GRLWEAP results. Soil response and hammer performance should be verified by static and/or dynamic testing and measurements. Estimates of bending or other local stresses (e.g., helmet or clamp contact, uneven rock surfaces etc.), prestress effects and others must also be accounted for by the user.

The calculated capacity-blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

Finally, the GRLWEAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors.

HAMMER DATA

Hammer Model:	D 19-42	Made By:	DELMAG
Hammer ID:	41	Hammer Type:	OED
Hammer Database Type:	PDI		
Hammer Database Name:			PDIHammer.gwh

Hammer and Drive System Segment Data

Segment	Weight kips	Stiffness kips/in	COR -	C-Slack in	Damping kips/ft/s
-					
1	0.800	140,084.4	1.000	0.000	
2	0.800	140,084.4	1.000	0.000	
3	0.800	140,084.4	1.000	0.000	
4	0.800	140,084.4	1.000	0.000	
5	0.800	70,754.7	0.900	0.120	
Imp Block	0.753	109,976.0	0.800	0.120	
Helmet	2.500				5.3

Ram Weight: (kips)	4.00	Ram Length: (ft)	10.76
Ram Area: (in ²)	124.69		
Maximum (Eq) Stroke: (ft)	10.81	Actual (Eq) Stroke: (ft)	10.81
Efficiency:	0.800	Rated Energy: (kip-ft)	43.24
Maximum Pressure: (psi)	1,600.00	Actual Pressure: (psi)	1,440.00
Combustion Delay: (ms)	2.00	Ignition Duration: (ms)	2.00
Expansion Exponent:	1.25		

Hammer Cushion

Pile Cushion

Cross Sect. Area: (in ²)	415.00	Cross Sect. Area: (in ²)	0.00
Elastic Modulus: (ksi)	530.0	Elastic Modulus: (ksi)	0.0
Thickness: (in)	2.00	Thickness: (in)	0.00
Coeff. of Restitution:	0.800	Coeff. of Restitution:	0.500
RoundOut: (in)	0.120	RoundOut: (in)	0.120
Stiffness: (kips/in)	109,976.0	Stiffness: (kips/in)	0.0
Helmet Weight: (kips)	2.500		

PILE INPUT

Uniform Pile		Pile Type:	Closed-End Pipe
Pile Length: (ft)	70.000	Pile Penetration: (ft)	65.000
Pile Size: (ft)	1.00	Toe Area: (in ²)	113.10

Table of Depths Analyzed with Driving System Modifiers

Depth ft	Temp Length ft	Wait Time Hr	Hammer -
5.00	70.0	0.0	DELMAG D 19-42
10.00	70.0	0.0	DELMAG D 19-42
15.00	70.0	0.0	DELMAG D 19-42
20.00	70.0	0.0	DELMAG D 19-42
25.00	70.0	0.0	DELMAG D 19-42
30.00	70.0	0.0	DELMAG D 19-42
35.00	70.0	0.0	DELMAG D 19-42
39.17	70.0	0.0	DELMAG D 19-42
40.00	70.0	0.0	DELMAG D 19-42
41.33	70.0	0.0	DELMAG D 19-42
45.00	70.0	0.0	DELMAG D 19-42
50.00	70.0	0.0	DELMAG D 19-42
55.00	70.0	0.0	DELMAG D 19-42
60.00	70.0	0.0	DELMAG D 19-42
65.00	70.0	0.0	DELMAG D 19-42

Other Information for DELMAG D 19-42

Depth ft	Stroke ft	Diesel Pressure %	Efficiency -	P.C. Stiff. Fact. -	P.C. COR -
5.00	10.8	90.0	0.80	1.0	0.50
10.00	10.8	90.0	0.80	1.0	0.50
15.00	10.8	90.0	0.80	1.0	0.50
20.00	10.8	90.0	0.80	1.0	0.50
25.00	10.8	90.0	0.80	1.0	0.50
30.00	10.8	90.0	0.80	1.0	0.50
35.00	10.8	90.0	0.80	1.0	0.50
39.17	10.8	90.0	0.80	1.0	0.50
40.00	10.8	90.0	0.80	1.0	0.50
41.33	10.8	90.0	0.80	1.0	0.50
45.00	10.8	90.0	0.80	1.0	0.50
50.00	10.8	90.0	0.80	1.0	0.50
55.00	10.8	90.0	0.80	1.0	0.50
60.00	10.8	90.0	0.80	1.0	0.50
65.00	10.8	90.0	0.80	1.0	0.50

PILE, SOIL, ANALYSIS OPTIONS

Analysis type:	Driveability Analysis	Parallel Driveability Analysis:	No
Static Analysis Method:	FHWA	Soil Damping Option:	Smith
Max No Analysis Iterations:	0	Time Increment/Critical:	160
Residual Stress Analysis:	0	Analysis Time-Input(ms):	0
Output Level:	Normal	Gravitational Acceleration (ft/s ²):	32.169
Hammer Gravity (ft/s ²):	32.169	Pile Gravity (ft/s ²):	32.169

DRIVEABILITY ANALYSIS

Analysis Depth (ft)	65.00	Standard Soil Setup	
Hammer Name	DELMAG D 19-42	Hammer ID	41
Diesel Pressure: (psi)	207.36	Stroke (ft)	10.81
Efficiency	0.80		
Shaft Gain/Loss Factor	0.667	Toe Gain/Loss Factor	1.000
Shaft Gain/Loss Factor	1.000	Toe Gain/Loss Factor	1.000

SOIL RESISTANCE PARAMETERS (FHWA)

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Setup F. -	Limit D. ft	Setup T. Hours	EB Area in ²
0.00	0.0	0.0	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
2.10	0.1	4.9	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
4.20	0.1	9.7	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
4.20	0.2	26.2	0.100	0.100	0.100	0.149	1.5	6.00	168.0	113.10
5.10	0.2	32.5	0.100	0.100	0.100	0.149	1.5	6.00	168.0	113.10
6.00	0.3	38.8	0.100	0.100	0.100	0.149	1.5	6.00	168.0	113.10
6.00	0.7	32.8	0.100	0.105	0.150	0.149	1.5	6.00	168.0	113.10
6.10	0.7	32.8	0.100	0.105	0.150	0.149	1.5	6.00	168.0	113.10
6.10	0.9	32.8	0.100	0.105	0.150	0.149	1.5	6.00	168.0	113.10
21.00	0.9	32.8	0.100	0.105	0.150	0.149	1.5	6.00	168.0	113.10
21.00	0.9	110.2	0.100	0.113	0.050	0.140	1.0	6.00	1.0	113.10
22.90	1.0	120.5	0.100	0.113	0.050	0.140	1.0	6.00	1.0	113.10
24.80	1.1	130.8	0.100	0.113	0.050	0.140	1.0	6.00	1.0	113.10
26.70	1.1	141.1	0.100	0.113	0.050	0.140	1.0	6.00	1.0	113.10
26.70	1.1	141.1	0.100	0.128	0.050	0.149	1.0	6.00	1.0	113.10
28.57	1.2	150.9	0.100	0.128	0.050	0.149	1.0	6.00	1.0	113.10
30.43	1.3	160.8	0.100	0.128	0.050	0.149	1.0	6.00	1.0	113.10
32.30	1.4	170.6	0.100	0.128	0.050	0.149	1.0	6.00	1.0	113.10
32.30	1.4	170.6	0.100	0.117	0.050	0.149	1.0	6.00	1.0	113.10
33.97	1.5	179.7	0.100	0.117	0.050	0.149	1.0	6.00	1.0	113.10
35.63	1.5	188.7	0.100	0.117	0.050	0.149	1.0	6.00	1.0	113.10
37.30	1.6	197.8	0.100	0.117	0.050	0.149	1.0	6.00	1.0	113.10
37.30	1.4	159.6	0.100	0.110	0.100	0.149	1.2	6.00	24.0	113.10
38.97	1.5	167.1	0.100	0.110	0.100	0.149	1.2	6.00	24.0	113.10
40.63	1.6	174.7	0.100	0.110	0.100	0.149	1.2	6.00	24.0	113.10
42.30	1.6	182.2	0.100	0.110	0.100	0.149	1.2	6.00	24.0	113.10
42.30	2.0	281.2	0.100	0.112	0.100	0.149	1.2	6.00	24.0	113.10
43.97	2.1	292.8	0.100	0.112	0.100	0.149	1.2	6.00	24.0	113.10
45.63	2.2	304.4	0.100	0.112	0.100	0.149	1.2	6.00	24.0	113.10
47.30	2.3	313.7	0.100	0.112	0.100	0.149	1.2	6.00	24.0	113.10
47.30	1.6	151.8	0.100	0.113	0.100	0.149	1.2	6.00	24.0	113.10
48.97	1.7	151.8	0.100	0.113	0.100	0.149	1.2	6.00	24.0	113.10
50.63	1.7	151.8	0.100	0.113	0.100	0.149	1.2	6.00	24.0	113.10
52.30	1.8	151.8	0.100	0.113	0.100	0.149	1.2	6.00	24.0	113.10
52.30	2.8	375.9	0.100	0.114	0.050	0.149	1.0	6.00	1.0	113.10
53.97	2.8	375.9	0.100	0.114	0.050	0.149	1.0	6.00	1.0	113.10
55.63	2.9	375.9	0.100	0.114	0.050	0.149	1.0	6.00	1.0	113.10
57.30	3.0	375.9	0.100	0.114	0.050	0.149	1.0	6.00	1.0	113.10
57.30	2.0	151.8	0.100	0.115	0.100	0.149	1.2	6.00	24.0	113.10
58.97	2.0	151.8	0.100	0.115	0.100	0.149	1.2	6.00	24.0	113.10

60.63	2.1	151.8	0.100	0.115	0.100	0.149	1.2	6.00	24.0	113.10
62.30	2.2	151.8	0.100	0.115	0.100	0.149	1.2	6.00	24.0	113.10
62.30	2.7	251.8	0.100	0.116	0.050	0.149	1.0	6.00	1.0	113.10
63.90	2.8	251.8	0.100	0.116	0.050	0.149	1.0	6.00	1.0	113.10
65.00	2.9	251.8	0.100	0.116	0.050	0.149	1.0	6.00	1.0	113.10

PILE PROFILE

Lb Top ft	X-Area in ²	E-Mod ksi	Spec. Wt lb/ft ³	Perim. ft	C-Index -	Wave Sp ft/s	Impedance kips/ft/s
0.00	9.2	30,000	492.00	3.142	0	16,806.4	16.5
70.00	9.2	30,000	492.00	3.142	0	16,806.4	16.5

PILE AND SOIL MODEL

Total Capacity Rut (kips):

459.103

Seg. -	Weight kips	Stiffn. kips/in	C-Slk in	T-Slk in	COR -	Ru kips	Js/Jt s/ft	Qs/Qt in	LbTop ft	Perim. ft	X-Area in ²
1	0.11	6,921	0.12	0.00	0.85	0.0	0.000	0.100	3.33	3.14	9.2
2	0.11	6,921	0.00	0.00	1.00	0.1	0.100	0.100	6.67	3.14	9.2
3	0.11	6,921	0.00	0.00	1.00	0.9	0.100	0.100	10.00	3.14	9.2
4	0.11	6,921	0.00	0.00	1.00	5.0	0.145	0.100	13.33	3.14	9.2
5	0.11	6,921	0.00	0.00	1.00	6.4	0.150	0.100	16.67	3.14	9.2
7	0.11	6,921	0.00	0.00	1.00	6.4	0.150	0.100	23.33	3.14	9.2
8	0.11	6,921	0.00	0.00	1.00	7.0	0.130	0.100	26.67	3.14	9.2
9	0.11	6,921	0.00	0.00	1.00	10.4	0.050	0.100	30.00	3.14	9.2
10	0.11	6,921	0.00	0.00	1.00	12.0	0.050	0.100	33.33	3.14	9.2
11	0.11	6,921	0.00	0.00	1.00	13.5	0.050	0.100	36.67	3.14	9.2
12	0.11	6,921	0.00	0.00	1.00	15.0	0.050	0.100	40.00	3.14	9.2
13	0.11	6,921	0.00	0.00	1.00	15.2	0.065	0.100	43.33	3.14	9.2
14	0.11	6,921	0.00	0.00	1.00	13.5	0.100	0.100	46.67	3.14	9.2
15	0.11	6,921	0.00	0.00	1.00	17.5	0.100	0.100	50.00	3.14	9.2
16	0.11	6,921	0.00	0.00	1.00	17.8	0.100	0.100	53.33	3.14	9.2
17	0.11	6,921	0.00	0.00	1.00	15.1	0.100	0.100	56.67	3.14	9.2
18	0.11	6,921	0.00	0.00	1.00	26.9	0.056	0.100	60.00	3.14	9.2
19	0.11	6,921	0.00	0.00	1.00	26.8	0.062	0.100	63.33	3.14	9.2
20	0.11	6,921	0.00	0.00	1.00	18.2	0.100	0.100	66.67	3.14	9.2
21	0.11	6,921	0.00	0.00	1.00	27.3	0.058	0.100	70.00	3.14	9.2
Toe						197.8	0.149	0.116	70.00		

2.207 kips total unreduced pile weight ($g = 32.169 \text{ ft/s}^2$)2.207 kips total reduced pile weight ($g = 32.169 \text{ ft/s}^2$)

OTHER OPTIONS

Pile Damping (%):	1	Pile Damping Fact. (kips/ft/s):	0.329
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EXTREMA TABLE at 65.0 FT; HAMMER: D 19-42

Shaft/Toe Gain/Loss Factor = 0.667/1.000

Rut = 459.1 kips

Rtoe = 197.8 kips

Time Inc. = 0.056 ms

Hammer

DELMAG D 19-42

Efficiency

0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	364.3	0.00	39.47	16.04	0.947	20.80
6.7	7.8	366.1	0.84	39.67	15.77	0.898	20.14
10.0	15.2	365.5	1.65	39.61	15.27	0.848	19.38
13.3	20.7	364.3	2.24	39.48	14.69	0.796	18.19
16.7	18.3	372.3	1.99	40.34	14.13	0.743	16.58
20.0	13.7	374.0	1.48	40.53	13.62	0.690	14.96
23.3	10.2	368.6	1.11	39.94	13.14	0.638	13.43
26.7	6.8	363.2	0.74	39.36	12.68	0.587	11.99
30.0	2.6	355.6	0.28	38.53	12.20	0.536	10.65
33.3	0.0	352.1	0.00	38.16	11.65	0.487	9.37
36.7	0.0	343.4	0.00	37.21	11.04	0.439	8.12
40.0	0.0	331.2	0.00	35.89	10.34	0.393	6.92
43.3	0.0	315.6	0.00	34.20	9.59	0.349	5.83
46.7	0.0	300.3	0.00	32.54	8.96	0.308	4.85
50.0	0.0	287.5	0.00	31.15	8.33	0.268	3.92
53.3	0.0	271.3	0.00	29.40	7.77	0.230	3.07
56.7	0.0	254.4	0.00	27.57	7.28	0.195	2.39
60.0	0.0	239.6	0.00	25.97	6.79	0.161	1.79
63.3	0.0	212.0	0.00	22.98	6.51	0.132	1.28
66.7	0.0	185.2	0.00	20.06	6.19	0.106	0.93
70.0	0.0	167.7	0.00	18.17	4.90	0.082	0.71

Converged Stroke (ft)

8.21

Fixed Combustion Pressure (psi)

1,440.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81

8.55

8.25

8.21

Shaft/Toe Gain/Loss Factor = 1.000/1.000

Rut = 493.6 kips

Rtoe = 197.8 kips

Time Inc. = 0.056 ms

Hammer

DELMAG D 19-42

Efficiency

0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	355.5	0.00	38.52	15.85	0.888	20.12
6.7	7.6	357.8	0.82	38.77	15.51	0.840	19.47
10.0	14.7	355.4	1.59	38.51	14.81	0.790	18.69
13.3	19.6	363.5	2.12	39.39	14.03	0.738	17.36
16.7	13.2	370.1	1.43	40.11	13.31	0.684	15.48
20.0	7.5	368.8	0.81	39.96	12.64	0.632	13.59
23.3	1.5	360.3	0.16	39.04	12.05	0.581	11.86
26.7	0.0	350.0	0.00	37.93	11.57	0.531	10.31
30.0	0.0	339.6	0.00	36.80	11.07	0.483	9.01
33.3	0.0	338.1	0.00	36.64	10.50	0.436	7.87
36.7	0.0	329.6	0.00	35.72	9.86	0.390	6.76
40.0	0.0	315.8	0.00	34.22	9.17	0.346	5.72
43.3	0.0	299.9	0.00	32.50	8.61	0.306	4.77
46.7	0.0	283.2	0.00	30.68	8.02	0.266	3.89

50.0	0.0	273.0	0.00	29.58	7.40	0.229	3.05
53.3	0.0	251.1	0.00	27.21	6.87	0.195	2.31
56.7	0.0	227.8	0.00	24.68	6.41	0.163	1.72
60.0	0.0	210.3	0.00	22.79	5.99	0.133	1.25
63.3	0.0	182.7	0.00	19.79	5.76	0.108	0.88
66.7	0.0	154.2	0.00	16.71	5.53	0.086	0.62
70.0	0.0	136.3	0.00	14.77	4.49	0.067	0.48

Converged Stroke (ft) 8.11 Fixed Combustion Pressure (psi) 1,440.0
(Eq) Strokes Analyzed and Last Return (ft)
10.81 8.39 8.13 8.11

SUMMARY TABLE at 65.0 FT; HAMMER: D 19-42

Rut	BI Ct	Stk Dn	Stk Up	Mx T-Str	LTop	Mx C-Str	LTop	ENTHRU	BI Rt	ActRes
kips	b/ft	ft	ft	ksi	ft	ksi	ft	kip-ft	b/min	kips
459.1	9,999	8.25	8.21	2.24	13.3	40.53	20.0	20.8	41.1	396.9
493.6	9,999	8.13	8.11	2.12	13.3	40.11	16.7	20.1	41.4	397.7

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.667/1.000									
Depth ft	Rut kips	Rshaft kips	Rtoe kips	BI Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	25.9	0.9	25.0	2.4	14.25	0.00	3.90	21.6	D 19-42
10.0	34.6	8.8	25.8	3.3	17.40	0.45	4.23	20.5	D 19-42
15.0	43.8	18.0	25.8	4.5	19.17	0.72	4.48	19.5	D 19-42
20.0	53.9	28.1	25.8	5.8	21.42	1.03	4.74	18.8	D 19-42
25.0	146.2	42.6	103.6	17.5	28.92	3.03	6.04	17.8	D 19-42
30.0	185.6	61.1	124.5	23.6	32.34	4.61	6.62	19.0	D 19-42
35.0	228.6	83.1	145.5	32.0	36.70	5.68	7.23	20.3	D 19-42
39.2	233.5	101.5	132.0	33.0	36.01	5.13	7.23	20.1	D 19-42
40.0	239.8	104.8	134.9	34.9	36.60	5.09	7.31	20.3	D 19-42
41.3	250.0	110.3	139.7	38.5	37.52	5.21	7.43	20.5	D 19-42
45.0	364.8	129.2	235.6	179.4	45.58	6.80	8.89	24.0	D 19-42
50.0	273.5	154.4	119.2	51.1	37.22	4.31	7.59	20.3	D 19-42
55.0	484.2	189.0	295.3	9,999.0	44.04	4.35	8.88	23.3	D 19-42
60.0	343.9	224.7	119.2	181.4	39.43	3.16	8.05	20.7	D 19-42
65.0	459.1	261.3	197.8	9,999.0	40.53	2.24	8.25	20.8	D 19-42

G/L at Shaft and Toe: 1.000/1.000									
Depth ft	Rut kips	Rshaft kips	Rtoe kips	BI Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	26.2	1.2	25.0	2.4	14.39	0.08	3.91	21.5	D 19-42
10.0	38.8	13.0	25.8	3.9	18.04	0.65	4.31	19.8	D 19-42
15.0	52.5	26.7	25.8	5.6	20.88	1.05	4.70	18.9	D 19-42
20.0	67.8	42.0	25.8	7.6	23.38	1.29	5.02	18.1	D 19-42
25.0	161.1	57.6	103.6	19.7	30.73	3.88	6.28	18.3	D 19-42
30.0	200.6	76.1	124.5	26.5	33.60	5.35	6.91	19.6	D 19-42
35.0	243.5	98.0	145.5	37.1	36.90	6.05	7.43	20.6	D 19-42
39.2	249.9	117.9	132.0	39.1	37.50	5.47	7.45	20.4	D 19-42
40.0	256.8	121.9	134.9	42.0	38.20	5.45	7.53	20.5	D 19-42
41.3	268.1	128.5	139.7	47.1	39.26	5.60	7.66	20.8	D 19-42
45.0	386.7	151.1	235.6	432.5	45.32	6.69	8.98	24.0	D 19-42
50.0	300.5	181.3	119.2	74.2	38.75	4.60	7.86	20.7	D 19-42
55.0	513.3	218.1	295.3	9,999.0	42.59	4.09	8.66	22.3	D 19-42
60.0	375.9	256.7	119.2	882.3	40.11	2.80	8.11	20.5	D 19-42
65.0	493.6	295.8	197.8	9,999.0	40.11	2.12	8.13	20.1	D 19-42

GRLWEAP: Wave Equation Analysis of Pile Foundations

FRA-Cemetery Rd SUP Bridge + Forward Abutment - 12-inch CIP

1/13/2026

NATIONAL ENGINEERING AND ARCHITECTURAL

GRLWEAP 14.1.20.1

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

The GRLWEAP program simulates the behavior of a preformed pile driven by either an impact hammer or a vibratory hammer. The program is based on mathematical models, which describe motion and forces of hammer, driving system, pile and soil under the hammer action. Under certain conditions, the models only crudely approximate, often complex, dynamic situations.

A wave equation analysis generally relies on input data, which represents normal situations. In particular, the hammer data file supplied with the program assumes that the hammer is in good working order. All of the input data selected by the user may be the best available information at the time when the analysis is performed. However, input data and therefore results may significantly differ from actual field conditions.

Therefore, the program authors recommend prudent use of the GRLWEAP results. Soil response and hammer performance should be verified by static and/or dynamic testing and measurements. Estimates of bending or other local stresses (e.g., helmet or clamp contact, uneven rock surfaces etc.), prestress effects and others must also be accounted for by the user.

The calculated capacity-blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

Finally, the GRLWEAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors.

HAMMER DATA

Hammer Model:	D 19-42	Made By:	DELMAG
Hammer ID:	41	Hammer Type:	OED
Hammer Database Type:	PDI		
Hammer Database Name:			PDIHammer.gwh

Hammer and Drive System Segment Data

Segment	Weight kips	Stiffness kips/in	COR -	C-Slack in	Damping kips/ft/s
-					
1	0.800	140,084.4	1.000	0.000	
2	0.800	140,084.4	1.000	0.000	
3	0.800	140,084.4	1.000	0.000	
4	0.800	140,084.4	1.000	0.000	
5	0.800	70,754.7	0.900	0.120	
Imp Block	0.753	109,976.0	0.800	0.120	
Helmet	2.500				5.3

Ram Weight: (kips)	4.00	Ram Length: (ft)	10.76
Ram Area: (in ²)	124.69		
Maximum (Eq) Stroke: (ft)	10.81	Actual (Eq) Stroke: (ft)	10.81
Efficiency:	0.800	Rated Energy: (kip-ft)	43.24
Maximum Pressure: (psi)	1,600.00	Actual Pressure: (psi)	1,440.00
Combustion Delay: (ms)	2.00	Ignition Duration: (ms)	2.00
Expansion Exponent:	1.25		

Hammer Cushion

Pile Cushion

Cross Sect. Area: (in ²)	415.00	Cross Sect. Area: (in ²)	0.00
Elastic Modulus: (ksi)	530.0	Elastic Modulus: (ksi)	0.0
Thickness: (in)	2.00	Thickness: (in)	0.00
Coeff. of Restitution:	0.800	Coeff. of Restitution:	0.500
RoundOut: (in)	0.120	RoundOut: (in)	0.120
Stiffness: (kips/in)	109,976.0	Stiffness: (kips/in)	0.0
Helmet Weight: (kips)	2.500		

PILE INPUT

Uniform Pile		Pile Type:	Closed-End Pipe
Pile Length: (ft)	70.000	Pile Penetration: (ft)	65.000
Pile Size: (ft)	1.00	Toe Area: (in ²)	113.10

Table of Depths Analyzed with Driving System Modifiers

Depth ft	Temp Length ft	Wait Time Hr	Hammer -
5.00	70.0	0.0	DELMAG D 19-42
10.00	70.0	0.0	DELMAG D 19-42
15.00	70.0	0.0	DELMAG D 19-42
20.00	70.0	0.0	DELMAG D 19-42
25.00	70.0	0.0	DELMAG D 19-42
29.11	70.0	0.0	DELMAG D 19-42
30.00	70.0	0.0	DELMAG D 19-42
35.00	70.0	0.0	DELMAG D 19-42
40.00	70.0	0.0	DELMAG D 19-42
45.00	70.0	0.0	DELMAG D 19-42
50.00	70.0	0.0	DELMAG D 19-42
55.00	70.0	0.0	DELMAG D 19-42
60.00	70.0	0.0	DELMAG D 19-42
65.00	70.0	0.0	DELMAG D 19-42

Other Information for DELMAG D 19-42

Depth ft	Stroke ft	Diesel Pressure %	Efficiency -	P.C. Stiff. Fact. -	P.C. COR -
5.00	10.8	90.0	0.80	1.0	0.50
10.00	10.8	90.0	0.80	1.0	0.50
15.00	10.8	90.0	0.80	1.0	0.50
20.00	10.8	90.0	0.80	1.0	0.50
25.00	10.8	90.0	0.80	1.0	0.50
29.11	10.8	90.0	0.80	1.0	0.50
30.00	10.8	90.0	0.80	1.0	0.50
35.00	10.8	90.0	0.80	1.0	0.50
40.00	10.8	90.0	0.80	1.0	0.50
45.00	10.8	90.0	0.80	1.0	0.50
50.00	10.8	90.0	0.80	1.0	0.50
55.00	10.8	90.0	0.80	1.0	0.50
60.00	10.8	90.0	0.80	1.0	0.50
65.00	10.8	90.0	0.80	1.0	0.50

PILE, SOIL, ANALYSIS OPTIONS

Analysis type:	Driveability Analysis	Soil Damping Option:	Smith
Max No Analysis Iterations:	0	Time Increment/Critical:	160
Residual Stress Analysis:	0	Analysis Time-Input(ms):	0
Output Level:	Normal	Gravitational Acceleration (ft/s ²):	32.169
Hammer Gravity (ft/s ²):	32.169	Pile Gravity (ft/s ²):	32.169

DRIVEABILITY ANALYSIS

Analysis Depth (ft)	65.00	Standard Soil Setup	
Hammer Name	DELMAG D 19-42	Hammer ID	41
Diesel Pressure: (psi)	207.36	Stroke (ft)	10.81
Efficiency	0.80		
Shaft Gain/Loss Factor	0.667	Toe Gain/Loss Factor	1.000
Shaft Gain/Loss Factor	1.000	Toe Gain/Loss Factor	1.000

SOIL RESISTANCE PARAMETERS

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Setup F.Limit -	D.Setup ft	TEB Area Hours	
0.00	0.0	0.0	0.10	0.124	0.100	0.1	1.2	6.00	24.0	113.10
2.10	0.1	4.9	0.10	0.124	0.100	0.1	1.2	6.00	24.0	113.10
4.20	0.1	9.7	0.10	0.124	0.100	0.1	1.2	6.00	24.0	113.10
4.20	0.2	26.3	0.10	0.100	0.100	0.1	1.5	6.00	168.0	113.10
5.10	0.2	32.6	0.10	0.100	0.100	0.1	1.5	6.00	168.0	113.10
6.00	0.3	38.8	0.10	0.100	0.100	0.1	1.5	6.00	168.0	113.10
6.00	3.2	32.8	0.10	0.105	0.150	0.1	1.5	6.00	168.0	113.10
21.00	3.2	32.8	0.10	0.105	0.150	0.1	1.5	6.00	168.0	113.10
21.00	0.9	110.2	0.10	0.113	0.050	0.1	1.0	6.00	1.0	113.10
22.90	1.0	120.5	0.10	0.113	0.050	0.1	1.0	6.00	1.0	113.10
24.80	1.1	130.9	0.10	0.113	0.050	0.1	1.0	6.00	1.0	113.10
26.70	1.1	141.2	0.10	0.113	0.050	0.1	1.0	6.00	1.0	113.10
26.70	1.1	141.2	0.10	0.128	0.050	0.1	1.0	6.00	1.0	113.10
28.57	1.2	151.0	0.10	0.128	0.050	0.1	1.0	6.00	1.0	113.10
30.43	1.3	160.8	0.10	0.128	0.050	0.1	1.0	6.00	1.0	113.10
32.30	1.4	170.7	0.10	0.128	0.050	0.1	1.0	6.00	1.0	113.10
32.30	1.4	170.7	0.10	0.117	0.050	0.1	1.0	6.00	1.0	113.10
33.97	1.5	179.7	0.10	0.117	0.050	0.1	1.0	6.00	1.0	113.10
35.63	1.5	188.8	0.10	0.117	0.050	0.1	1.0	6.00	1.0	113.10
37.30	1.6	197.8	0.10	0.117	0.050	0.1	1.0	6.00	1.0	113.10
37.30	1.4	159.7	0.10	0.110	0.100	0.1	1.2	6.00	24.0	113.10
38.97	1.5	167.2	0.10	0.110	0.100	0.1	1.2	6.00	24.0	113.10
40.63	1.6	174.7	0.10	0.110	0.100	0.1	1.2	6.00	24.0	113.10
42.30	1.6	182.2	0.10	0.110	0.100	0.1	1.2	6.00	24.0	113.10
42.30	2.0	281.2	0.10	0.112	0.100	0.1	1.2	6.00	24.0	113.10
43.97	2.1	292.9	0.10	0.112	0.100	0.1	1.2	6.00	24.0	113.10
45.63	2.2	304.5	0.10	0.112	0.100	0.1	1.2	6.00	24.0	113.10
47.30	2.3	313.7	0.10	0.112	0.100	0.1	1.2	6.00	24.0	113.10
47.30	1.6	151.8	0.10	0.113	0.100	0.1	1.2	6.00	24.0	113.10
48.97	1.7	151.8	0.10	0.113	0.100	0.1	1.2	6.00	24.0	113.10
50.63	1.7	151.8	0.10	0.113	0.100	0.1	1.2	6.00	24.0	113.10

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52.30	1.8	151.8	0.10	0.113	0.100	0.1	1.2	6.00	24.0	113.10
52.30	2.8	375.9	0.10	0.114	0.050	0.1	1.0	6.00	1.0	113.10
53.97	2.8	375.9	0.10	0.114	0.050	0.1	1.0	6.00	1.0	113.10
55.63	2.9	375.9	0.10	0.114	0.050	0.1	1.0	6.00	1.0	113.10
57.30	3.0	375.9	0.10	0.114	0.050	0.1	1.0	6.00	1.0	113.10
57.30	2.0	151.8	0.10	0.115	0.100	0.1	1.2	6.00	24.0	113.10
58.97	2.0	151.8	0.10	0.115	0.100	0.1	1.2	6.00	24.0	113.10
60.63	2.1	151.8	0.10	0.115	0.100	0.1	1.2	6.00	24.0	113.10
62.30	2.2	151.8	0.10	0.115	0.100	0.1	1.2	6.00	24.0	113.10
62.30	2.7	251.8	0.10	0.116	0.050	0.1	1.0	6.00	1.0	113.10
63.90	2.8	251.8	0.10	0.116	0.050	0.1	1.0	6.00	1.0	113.10
65.00	2.9	251.8	0.10	0.116	0.050	0.1	1.0	6.00	1.0	113.10

PILE PROFILE

Lb Top ft	X-Area in ²	E-Mod ksi	Spec. Wt lb/ft ³	Perim. ft	C-Index -	Wave Sp ft/s	Impedance kips/ft/s
0.00	9.2	30,000	492.00	3.142	0	16,806.4	16.5
70.00	9.2	30,000	492.00	3.142	0	16,806.4	16.5

PILE AND SOIL MODEL Total Capacity Rut (kips): 530.718

Seg. -	Weight kips	Stiffn. kips/in	C-Slk in	T-Slk in	COR -	Ru kips	Js/Jt s/ft	Qs/Qt in	LbTop ft	Perim. ft	X-Area in ²
1	0.11	6,921	0.12	0.00	0.85	0.0	0.000	0.10	3.33	3.14	9.2
2	0.11	6,921	0.00	0.00	1.00	0.1	0.100	0.10	6.67	3.14	9.2
3	0.11	6,921	0.00	0.00	1.00	0.9	0.100	0.10	10.00	3.14	9.2
4	0.11	6,921	0.00	0.00	1.00	16.1	0.148	0.10	13.33	3.14	9.2
5	0.11	6,921	0.00	0.00	1.00	22.3	0.150	0.10	16.67	3.14	9.2
7	0.11	6,921	0.00	0.00	1.00	22.3	0.150	0.10	23.33	3.14	9.2
8	0.11	6,921	0.00	0.00	1.00	19.7	0.143	0.10	26.67	3.14	9.2
9	0.11	6,921	0.00	0.00	1.00	10.5	0.050	0.10	30.00	3.14	9.2
10	0.11	6,921	0.00	0.00	1.00	12.0	0.050	0.10	33.33	3.14	9.2
11	0.11	6,921	0.00	0.00	1.00	13.5	0.050	0.10	36.67	3.14	9.2
12	0.11	6,921	0.00	0.00	1.00	15.0	0.050	0.10	40.00	3.14	9.2
13	0.11	6,921	0.00	0.00	1.00	15.2	0.065	0.10	43.33	3.14	9.2
14	0.11	6,921	0.00	0.00	1.00	13.5	0.100	0.10	46.67	3.14	9.2
15	0.11	6,921	0.00	0.00	1.00	17.5	0.100	0.10	50.00	3.14	9.2
16	0.11	6,921	0.00	0.00	1.00	17.8	0.100	0.10	53.33	3.14	9.2
17	0.11	6,921	0.00	0.00	1.00	15.1	0.100	0.10	56.67	3.14	9.2
18	0.11	6,921	0.00	0.00	1.00	26.9	0.056	0.10	60.00	3.14	9.2
19	0.11	6,921	0.00	0.00	1.00	26.8	0.062	0.10	63.33	3.14	9.2
20	0.11	6,921	0.00	0.00	1.00	18.2	0.100	0.10	66.67	3.14	9.2
21	0.11	6,921	0.00	0.00	1.00	27.3	0.058	0.10	70.00	3.14	9.2

Toe	197.8	0.149	0.12	70.00
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2.207 kips total unreduced pile weight ($g = 32.169 \text{ ft/s}^2$)

2.207 kips total reduced pile weight ($g = 32.169 \text{ ft/s}^2$)

OTHER OPTIONS

Pile Damping (%):	1	Pile Damping Fact. (kips/ft/s):	0.329
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EXTREMA TABLE at 65.0 FT; HAMMER: D 19-42

Shaft/Toe Gain/Loss Factor = 0.667/1.000

Rut = 530.7 kips

Rtoe = 197.8 kips

Time Inc. = 0.056 ms

Hammer DELMAG D 19-42 Efficiency 0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	401.4	0.00	43.50	15.44	0.743	18.27
6.7	8.1	407.3	0.88	44.14	14.91	0.695	17.61
10.0	14.8	411.0	1.61	44.54	13.68	0.646	16.90
13.3	20.0	415.9	2.17	45.07	12.34	0.596	15.37
16.7	2.3	387.6	0.25	42.00	10.95	0.547	12.93
20.0	0.0	347.4	0.00	37.64	9.65	0.499	10.51
23.3	0.0	310.9	0.00	33.69	8.67	0.456	8.42
26.7	0.0	288.6	0.00	31.27	8.09	0.415	6.66
30.0	0.0	272.9	0.00	29.57	7.57	0.376	5.49
33.3	0.0	274.4	0.00	29.73	7.32	0.337	4.71
36.7	0.0	267.9	0.00	29.04	7.04	0.301	3.99
40.0	0.0	253.4	0.00	27.46	6.75	0.267	3.33
43.3	0.0	236.1	0.00	25.58	6.44	0.236	2.73
46.7	0.0	224.0	0.00	24.27	6.12	0.206	2.22
50.0	0.0	218.2	0.00	23.65	5.78	0.177	1.75
53.3	0.0	199.3	0.00	21.59	5.46	0.150	1.34
56.7	0.0	177.4	0.00	19.23	5.16	0.125	1.00
60.0	0.0	163.9	0.00	17.76	4.86	0.102	0.72
63.3	0.0	137.9	0.00	14.95	4.71	0.082	0.51
66.7	0.0	115.5	0.00	12.52	4.68	0.067	0.38
70.0	0.0	105.6	0.00	11.44	3.93	0.052	0.29

Converged Stroke (ft) 7.89 Fixed Combustion Pressure (psi) 1,440.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81 8.02 7.90 7.89

Shaft/Toe Gain/Loss Factor = 1.000/1.000

Rut = 601.0 kips

Rtoe = 197.8 kips

Time Inc. = 0.056 ms

Hammer DELMAG D 19-42 Efficiency 0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	453.5	0.00	49.14	15.68	0.674	17.83
6.7	11.0	460.8	1.20	49.93	15.01	0.620	16.98
10.0	19.7	466.8	2.14	50.58	13.49	0.565	16.08
13.3	25.4	473.5	2.75	51.31	11.57	0.511	14.25
16.7	0.0	430.7	0.00	46.67	9.56	0.461	11.46

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20.0	0.0	370.5	0.00	40.14	8.19	0.416	8.81
23.3	0.0	314.0	0.00	34.02	7.18	0.375	6.59
26.7	0.0	266.6	0.00	28.89	6.58	0.337	4.82
30.0	0.0	243.2	0.00	26.35	6.39	0.302	3.76
33.3	0.0	244.5	0.00	26.50	6.21	0.267	3.15
36.7	0.0	238.3	0.00	25.82	5.98	0.235	2.61
40.0	0.0	223.4	0.00	24.21	5.76	0.206	2.14
43.3	0.0	204.1	0.00	22.11	5.49	0.179	1.72
46.7	0.0	194.0	0.00	21.02	5.20	0.155	1.36
50.0	0.0	183.5	0.00	19.89	4.89	0.132	1.04
53.3	0.0	160.1	0.00	17.35	4.59	0.110	0.77
56.7	0.0	134.8	0.00	14.60	4.34	0.091	0.56
60.0	0.0	120.7	0.00	13.08	4.09	0.074	0.41
63.3	0.0	102.2	0.00	11.08	3.96	0.061	0.30
66.7	0.0	85.1	0.00	9.22	3.98	0.050	0.22
70.0	0.0	75.8	0.00	8.21	3.46	0.039	0.18

Converged Stroke (ft) 8.07 Fixed Combustion Pressure (psi) 1,440.0
(Eq) Strokes Analyzed and Last Return (ft)
10.81 8.18 8.08 8.07

SUMMARY TABLE at 65.0 FT; HAMMER: D 19-42

Rut	Bl Ct	Stk Dn	Stk Up	Mx T-Str	LTop	Mx C-Str	LTop	ENTHRU	Bl Rt	ActRes
kips	b/ft	ft	ft	ksi	ft	ksi	ft	kip-ft	b/min	kips
530.7	9,999	7.89	0.00	2.17	13.3	45.07	13.3	18.3	42.0	398.7
601.0	9,999	8.07	0.00	2.75	13.3	51.31	13.3	17.8	41.5	422.7

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.667/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	BI Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	26.0	0.9	25.0	2.4	14.30	0.02	3.92	21.6	D 19-42
10.0	57.9	32.1	25.8	6.3	21.40	1.29	4.79	18.6	D 19-42
15.0	96.1	70.3	25.8	11.5	26.23	2.38	5.47	17.3	D 19-42
20.0	123.6	97.8	25.8	15.2	28.91	2.50	5.83	17.3	D 19-42
25.0	217.8	114.2	103.6	30.3	36.67	7.63	7.17	20.2	D 19-42
29.1	250.0	129.2	120.9	40.9	39.66	8.33	7.63	20.9	D 19-42
30.0	257.2	132.7	124.5	43.5	40.27	8.30	7.71	21.2	D 19-42
35.0	300.2	154.6	145.6	69.0	43.33	8.53	8.21	22.2	D 19-42
40.0	311.4	176.4	135.0	85.6	43.98	7.49	8.24	21.8	D 19-42
45.0	436.4	200.8	235.7	9,999.0	48.48	6.78	8.70	22.7	D 19-42
50.0	345.1	226.0	119.2	213.3	43.62	5.51	8.15	20.7	D 19-42
55.0	555.8	260.6	295.3	9,999.0	44.36	5.06	8.30	20.6	D 19-42
60.0	415.5	296.4	119.2	9,999.0	43.46	3.08	7.99	19.1	D 19-42
65.0	530.7	332.9	197.8	9,999.0	45.07	2.17	7.89	18.3	D 19-42

G/L at Shaft and Toe: 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	BI Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	26.3	1.2	25.0	2.4	14.40	0.10	3.93	21.5	D 19-42
10.0	73.7	47.9	25.8	8.4	22.96	1.67	5.09	17.9	D 19-42
15.0	131.0	105.2	25.8	16.1	29.93	3.08	5.91	17.5	D 19-42
20.0	172.3	146.5	25.8	21.9	33.65	5.27	6.56	18.9	D 19-42
25.0	268.5	164.9	103.6	46.1	42.02	10.65	7.96	21.8	D 19-42
29.1	300.7	179.9	120.9	65.1	45.52	10.84	8.35	22.8	D 19-42
30.0	308.0	183.4	124.5	71.2	45.89	10.81	8.43	23.0	D 19-42
35.0	350.9	205.4	145.6	143.5	48.46	10.44	8.80	23.4	D 19-42
40.0	364.2	229.3	135.0	219.9	48.61	9.03	8.72	22.8	D 19-42
45.0	494.1	258.5	235.7	9,999.0	52.40	8.39	8.86	22.8	D 19-42
50.0	407.9	288.7	119.2	9,999.0	49.17	6.68	8.52	21.2	D 19-42
55.0	620.7	325.4	295.3	9,999.0	49.27	5.95	8.49	20.5	D 19-42
60.0	483.3	364.1	119.2	9,999.0	49.50	4.05	8.24	19.0	D 19-42
65.0	601.0	403.2	197.8	9,999.0	51.31	2.75	8.07	17.8	D 19-42

FORWARD ABUTMENT

DRIVABILITY

Boring Used: B-007-0-23

Starting Elevation (0.0 ft) = 882.9 ft amsl (Bottom of pile cap elevation)

Assume Zero-Friction Length to 18.4 ft = 882.9 - 864.5 ft amsl (bottom of foundation prep layer)

Assumed Groundwater Depth = 18.4 ft (Ele. 864.5 ft amsl)

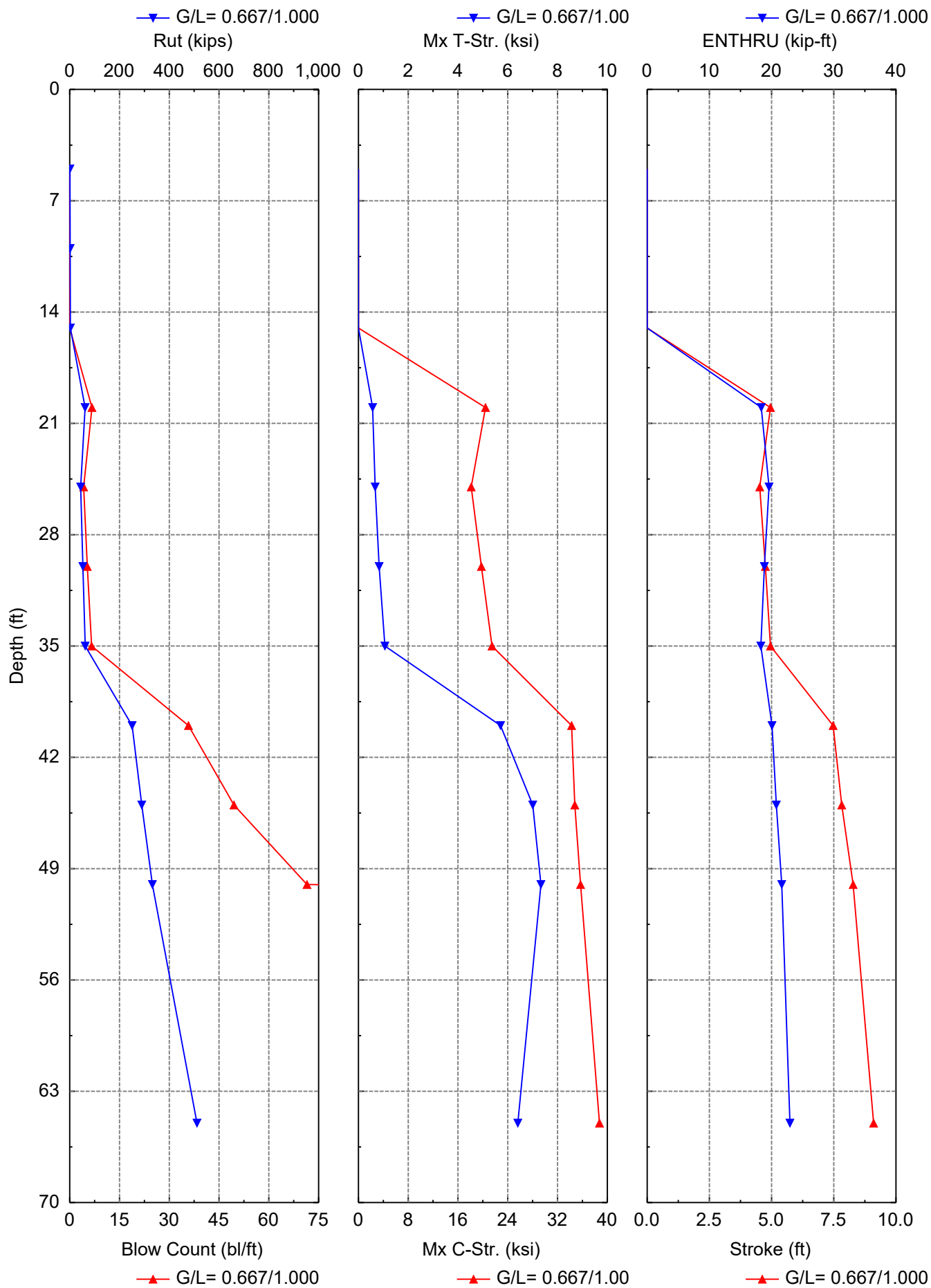
Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 0.667/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.8	0.0	0.8	0.3	0.000	0.000	10.81	0.0	D 19-42
10.0	1.7	0.0	1.7	0.3	0.000	0.000	10.81	0.0	D 19-42
15.0	2.5	0.0	2.5	0.3	0.000	0.000	10.81	0.0	D 19-42
20.0	61.8	3.9	57.8	6.7	20.426	0.572	4.96	18.3	D 19-42
25.0	43.9	18.1	25.8	4.2	18.134	0.672	4.52	19.6	D 19-42
30.0	53.1	27.3	25.8	5.3	19.771	0.834	4.75	18.8	D 19-42
35.0	62.5	36.7	25.8	6.6	21.464	1.063	4.96	18.3	D 19-42
40.0	250.8	53.0	197.8	35.8	34.265	5.711	7.47	20.1	D 19-42
45.0	289.7	91.9	197.8	49.5	34.769	7.004	7.82	20.8	D 19-42
50.0	332.0	134.2	197.8	71.5	35.709	7.335	8.28	21.6	D 19-42
65.0	511.5	265.1	246.4	9999.0	38.739	6.406	9.10	23.0	D 19-42

Refusal occurred; no driving time output possible.

Starting Elevation (0.0 ft) = 882.9 ft amsl

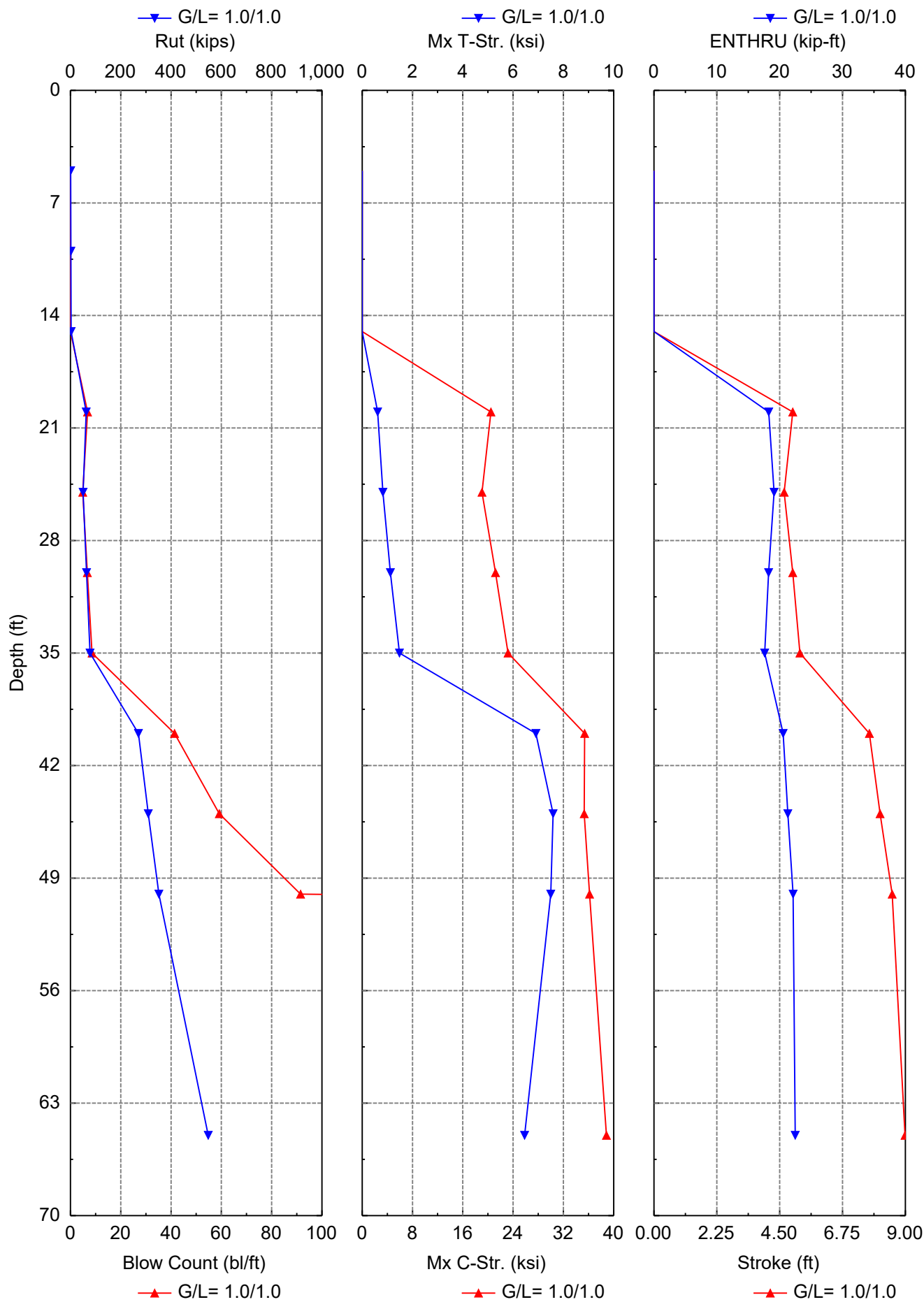
Assumed Groundwater Depth = 18.4 ft (Ele. 864.5 ft amsl)



Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.8	0.0	0.8	0.3	0.000	0.000	10.81	0.0	D 19-42
10.0	1.7	0.0	1.7	0.3	0.000	0.000	10.81	0.0	D 19-42
15.0	2.5	0.0	2.5	0.3	0.000	0.000	10.81	0.0	D 19-42
20.0	62.5	4.7	57.8	6.8	20.473	0.613	4.97	18.3	D 19-42
25.0	50.2	24.4	25.8	4.9	19.047	0.823	4.66	19.1	D 19-42
30.0	64.0	38.2	25.8	6.7	21.204	1.123	4.97	18.2	D 19-42
35.0	78.0	52.2	25.8	8.6	23.193	1.480	5.22	17.6	D 19-42
40.0	270.5	72.7	197.8	41.4	35.390	6.906	7.71	20.6	D 19-42
45.0	309.4	111.6	197.8	59.2	35.319	7.590	8.09	21.3	D 19-42
50.0	351.7	153.9	197.8	91.5	36.165	7.497	8.52	22.1	D 19-42
65.0	547.9	301.5	246.4	9999.0	38.842	6.458	8.99	22.5	D 19-42

Refusal occurred; no driving time output possible.



SOIL PROFILE

Depth ft	Soil Type -	Spec. Wt lb/ft ³	Su ksf	Phi °	Unit Rs ksf	Unit Rt ksf
0.0	Sand	120.0	0.0	0.0	0.00	0.00
18.4	Sand	120.0	0.0	0.0	0.00	3.92
18.4	Sand	125.0	0.0	34.0	0.91	73.64
22.1	Sand	125.0	0.0	34.0	1.01	73.64
22.1	Sand	132.0	0.0	39.0	1.75	244.26
23.9	Sand	132.0	0.0	39.0	1.84	256.80
23.9	Clay	130.5	3.6	0.0	1.00	32.85
24.0	Clay	130.5	3.6	0.0	1.00	32.85
24.0	Clay	128.0	3.6	0.0	0.92	32.85
38.9	Clay	128.0	3.6	0.0	0.92	32.85
38.9	Sand	130.0	0.0	38.0	2.32	251.83
44.6	Sand	130.0	0.0	38.0	2.57	251.83
44.6	Sand	128.0	0.0	38.0	2.57	251.83
50.2	Sand	128.0	0.0	38.0	2.81	251.83
50.2	Sand	130.0	0.0	38.0	2.81	251.83
55.2	Sand	130.0	0.0	38.0	3.03	251.83
55.2	Sand	132.0	0.0	37.0	2.71	203.27
60.2	Sand	132.0	0.0	37.0	2.91	203.27
60.2	Sand	132.0	0.0	39.0	3.59	313.69
65.2	Sand	132.0	0.0	39.0	3.84	313.69
65.2	Sand	132.0	0.0	36.0	2.75	151.75
70.2	Sand	132.0	0.0	36.0	2.93	151.75
70.2	Sand	132.0	0.0	40.0	4.46	375.94
75.2	Sand	132.0	0.0	40.0	4.73	375.94
75.2	Sand	132.0	0.0	36.0	3.11	151.75
80.2	Sand	132.0	0.0	36.0	3.29	151.75
80.2	Sand	132.0	0.0	38.0	4.16	251.83
83.4	Sand	132.0	0.0	38.0	4.31	251.83

Groundwater Assumptions:

Forward Abutment - Drivability

General Site Information

×

General Information

Total Pile Length: ft

Initial Pene. Depth: ft

Final Pene. Depth: ft

Water Table: ft

Eff. Overb. Pressure: ksf

☐ Additional Input

Import SPT Data

Import CPT Data

Import DIGGS Data

OK

Cancel

GRLWEAP 1980 Tomlinson Graphs:

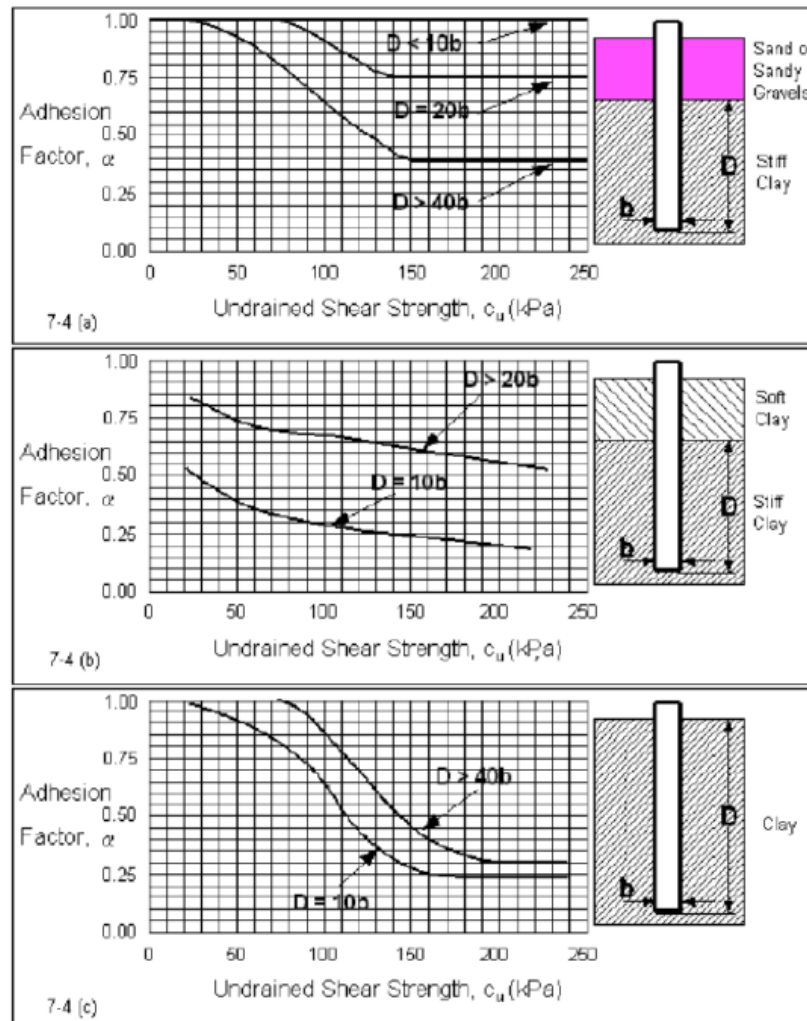


Figure 5.2: Adhesion factor for driven pile in cohesive soil (after Tomlinson 1980).

Forward Abutment:

Soil Profile Input

FHWA Method | Other Parameters (Impact)

Update=> Ru=350.8; Rs=205.3 kips

Layers

Site Properties

Number of Layers: 13

Final Tip Penetration: 35.0 ft

Water Table: 0.0 ft

Eff. Overburden at Grade: 0.0 ksf

Layer Properties

Layer: 4 of 13

Depth: Top: 6.1 Bottom: 21.0 ft

Thickness: 14.9 ft

Soil Type/Property

☐ Gravel

☐ Other Granular: Skin Friction End Bearing

SPT N Value: 0.0 0.0 <=60

ϕ ☐ degree <= 45

☒ Cohesive: Stiff

S_u 3.65 ksf <= 5

Auto Adhesion (1980 - 3) Click to change.

☐ Rock*: Description:

☐ Other*: Stiff Cohesive

Unit Weight: 128 lb/ft³

For selected soil, you need to specify:

Friction: End Bearing:

Top: 3.194 32.85 ksf

Bottom: 3.194 32.85 ksf

*Rock and Other are not FHWA defined soil types

OK Cancel

Depth (ft)	Soil Type	Parameter
0.0	Granular	a=34
4.2	Granular	a=39
6.1	Stiff Cohesive	S=3.6
21.0	Granular	a=38
26.7	Granular	a=38
32.3	Granular	a=38
37.3	Granular	a=37
42.3	Granular	a=39
47.3	Sand	a=36
52.3	Granular	a=40
57.3	Sand	a=36
62.3	Sand	a=38
65.5		

GRLWEAP: Wave Equation Analysis of Pile Foundations

FRA-Cemetery Rd SUP Bridge + Forward Abutment - 12-inch CIP
NATIONAL ENGINEERING AND ARCHITECTURAL7/9/2026
GRLWEAP 14.2.2.0

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

The GRLWEAP program simulates the behavior of a preformed pile driven by either an impact hammer or a vibratory hammer. The program is based on mathematical models, which describe motion and forces of hammer, driving system, pile and soil under the hammer action. Under certain conditions, the models only crudely approximate, often complex, dynamic situations.

A wave equation analysis generally relies on input data, which represents normal situations. In particular, the hammer data file supplied with the program assumes that the hammer is in good working order. All of the input data selected by the user may be the best available information at the time when the analysis is performed. However, input data and therefore results may significantly differ from actual field conditions.

Therefore, the program authors recommend prudent use of the GRLWEAP results. Soil response and hammer performance should be verified by static and/or dynamic testing and measurements. Estimates of bending or other local stresses (e.g., helmet or clamp contact, uneven rock surfaces etc.), prestress effects and others must also be accounted for by the user.

The calculated capacity-blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

Finally, the GRLWEAP capacities are ultimate values. They MUST be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors.

HAMMER DATA

Hammer Model:	D 19-42	Made By:	DELMAG
Hammer ID:	41	Hammer Type:	OED
Hammer Database Type:	PDI		
Hammer Database Name:			PDIHammer.gwh

Hammer and Drive System Segment Data

Segment	Weight kips	Stiffness kips/in	COR -	C-Slack in	Damping kips/ft/s
-					
1	0.800	140,084.4	1.000	0.000	
2	0.800	140,084.4	1.000	0.000	
3	0.800	140,084.4	1.000	0.000	
4	0.800	140,084.4	1.000	0.000	
5	0.800	70,754.7	0.900	0.120	
Imp Block	0.753	109,976.0	0.800	0.120	
Helmet	2.500				5.3

Ram Weight: (kips)	4.00	Ram Length: (ft)	10.76
Ram Area: (in ²)	124.69		
Maximum (Eq) Stroke: (ft)	10.81	Actual (Eq) Stroke: (ft)	10.81
Efficiency:	0.800	Rated Energy: (kip-ft)	43.24
Maximum Pressure: (psi)	1,600.00	Actual Pressure: (psi)	1,440.00
Combustion Delay: (ms)	2.00	Ignition Duration: (ms)	2.00
Expansion Exponent:	1.25		

Hammer Cushion

Pile Cushion

Cross Sect. Area: (in ²)	415.00	Cross Sect. Area: (in ²)	0.00
Elastic Modulus: (ksi)	530.0	Elastic Modulus: (ksi)	0.0
Thickness: (in)	2.00	Thickness: (in)	0.00
Coeff. of Restitution:	0.800	Coeff. of Restitution:	0.500
RoundOut: (in)	0.120	RoundOut: (in)	0.120
Stiffness: (kips/in)	109,976.0	Stiffness: (kips/in)	0.0
Helmet Weight: (kips)	2.500		

PILE INPUT

Uniform Pile		Pile Type:	Closed-End Pipe
Pile Length: (ft)	70.000	Pile Penetration: (ft)	65.000
Pile Size: (ft)	1.00	Toe Area: (in ²)	113.10

Table of Depths Analyzed with Driving System Modifiers

Depth ft	Temp Length ft	Wait Time Hr	Hammer -
5.00	70.0	0.0	DELMAG D 19-42
10.00	70.0	0.0	DELMAG D 19-42
15.00	70.0	0.0	DELMAG D 19-42
20.00	70.0	0.0	DELMAG D 19-42
25.00	70.0	0.0	DELMAG D 19-42
30.00	70.0	0.0	DELMAG D 19-42
35.00	70.0	0.0	DELMAG D 19-42
40.00	70.0	0.0	DELMAG D 19-42
45.00	70.0	0.0	DELMAG D 19-42
50.00	70.0	0.0	DELMAG D 19-42
65.00	70.0	0.0	DELMAG D 19-42

Other Information for DELMAG D 19-42

Depth ft	Stroke ft	Diesel Pressure %	Efficiency -	P.C. Stiff. Fact. -	P.C. COR -
5.00	10.8	90.0	0.80	1.0	0.50
10.00	10.8	90.0	0.80	1.0	0.50
15.00	10.8	90.0	0.80	1.0	0.50
20.00	10.8	90.0	0.80	1.0	0.50
25.00	10.8	90.0	0.80	1.0	0.50
30.00	10.8	90.0	0.80	1.0	0.50
35.00	10.8	90.0	0.80	1.0	0.50
40.00	10.8	90.0	0.80	1.0	0.50
45.00	10.8	90.0	0.80	1.0	0.50
50.00	10.8	90.0	0.80	1.0	0.50
65.00	10.8	90.0	0.80	1.0	0.50

PILE, SOIL, ANALYSIS OPTIONS

Analysis type:	Driveability Analysis	Parallel Driveability Analysis:	No
Static Analysis Method:	FHWA	Soil Damping Option:	Smith
Max No Analysis Iterations:	0	Time Increment/Critical:	160
Residual Stress Analysis:	0	Analysis Time-Input(ms):	0
Output Level:	Normal	Gravitational Acceleration (ft/s ²):	32.169
Hammer Gravity (ft/s ²):	32.169	Pile Gravity (ft/s ²):	32.169

DRIVEABILITY ANALYSIS

Analysis Depth (ft)	65.00	Standard Soil Setup	
Hammer Name	DELMAG D 19-42	Hammer ID	41
Diesel Pressure: (psi)	207.36	Stroke (ft)	10.81
Efficiency	0.80		
Shaft Gain/Loss Factor	0.667	Toe Gain/Loss Factor	1.000
Shaft Gain/Loss Factor	1.000	Toe Gain/Loss Factor	1.000

SOIL RESISTANCE PARAMETERS (FHWA)

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Setup F. -	Limit D. ft	Setup T. Hours	EB Area in ²
0.00	0.0	0.0	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
1.67	0.0	0.4	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
3.35	0.0	0.7	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
5.02	0.0	1.1	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
6.69	0.0	1.4	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
8.36	0.0	1.8	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
10.04	0.0	2.1	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
11.71	0.0	2.5	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
13.38	0.0	2.9	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
15.05	0.0	3.2	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
16.73	0.0	3.6	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
18.40	0.0	3.9	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
18.40	0.9	73.6	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
20.25	1.0	73.6	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
22.10	1.0	73.6	0.100	0.124	0.100	0.149	1.2	6.00	24.0	113.10
22.10	1.8	244.3	0.100	0.100	0.100	0.149	1.5	6.00	168.0	113.10
23.00	1.8	250.5	0.100	0.100	0.100	0.149	1.5	6.00	168.0	113.10
23.90	1.8	256.8	0.100	0.100	0.100	0.149	1.5	6.00	168.0	113.10
23.90	1.0	32.8	0.100	0.105	0.150	0.149	1.5	6.00	168.0	113.10
24.00	1.0	32.8	0.100	0.105	0.150	0.149	1.5	6.00	168.0	113.10
24.00	0.9	32.8	0.100	0.105	0.150	0.149	1.5	6.00	168.0	113.10
38.90	0.9	32.8	0.100	0.105	0.150	0.149	1.5	6.00	168.0	113.10
38.90	2.3	251.8	0.100	0.113	0.050	0.140	1.0	6.00	1.0	113.10
40.80	2.4	251.8	0.100	0.113	0.050	0.140	1.0	6.00	1.0	113.10
42.70	2.5	251.8	0.100	0.113	0.050	0.140	1.0	6.00	1.0	113.10
44.60	2.6	251.8	0.100	0.113	0.050	0.140	1.0	6.00	1.0	113.10
44.60	2.6	251.8	0.100	0.128	0.050	0.149	1.0	6.00	1.0	113.10
46.47	2.6	251.8	0.100	0.128	0.050	0.149	1.0	6.00	1.0	113.10
48.33	2.7	251.8	0.100	0.128	0.050	0.149	1.0	6.00	1.0	113.10
50.20	2.8	251.8	0.100	0.128	0.050	0.149	1.0	6.00	1.0	113.10
50.20	2.8	251.8	0.100	0.117	0.050	0.149	1.0	6.00	1.0	113.10
51.87	2.9	251.8	0.100	0.117	0.050	0.149	1.0	6.00	1.0	113.10
53.53	3.0	251.8	0.100	0.117	0.050	0.149	1.0	6.00	1.0	113.10
55.20	3.0	251.8	0.100	0.117	0.050	0.149	1.0	6.00	1.0	113.10
55.20	2.7	203.3	0.100	0.110	0.100	0.149	1.2	6.00	24.0	113.10
56.87	2.8	203.3	0.100	0.110	0.100	0.149	1.2	6.00	24.0	113.10
58.53	2.8	203.3	0.100	0.110	0.100	0.149	1.2	6.00	24.0	113.10
60.20	2.9	203.3	0.100	0.110	0.100	0.149	1.2	6.00	24.0	113.10
60.20	3.6	313.7	0.100	0.112	0.100	0.149	1.2	6.00	24.0	113.10
61.87	3.7	313.7	0.100	0.112	0.100	0.149	1.2	6.00	24.0	113.10

63.53	3.8	313.7	0.100	0.112	0.100	0.149	1.2	6.00	24.0	113.10
65.00	3.8	313.7	0.100	0.112	0.100	0.149	1.2	6.00	24.0	113.10

PILE PROFILE

Lb Top ft	X-Area in ²	E-Mod ksi	Spec. Wt lb/ft ³	Perim. ft	C-Index -	Wave Sp ft/s	Impedance kips/ft/s
0.00	11.5	30,000	492.00	3.142	0	16,806.4	20.4
70.00	11.5	30,000	492.00	3.142	0	16,806.4	20.4

PILE AND SOIL MODEL

Total Capacity Rut (kips):

511.521

Seg. -	Weight kips	Stiffn. kips/in	C-Slk in	T-Slk in	COR -	Ru kips	Js/Jt s/ft	Qs/Qt in	LbTop ft	Perim. ft	X-Area in ²
1	0.13	8,592	0.12	0.00	0.85	0.0	0.000	0.100	3.33	3.14	11.5
2	0.13	8,592	0.00	0.00	1.00	0.0	0.000	0.100	6.67	3.14	11.5
7	0.13	8,592	0.00	0.00	1.00	0.0	0.000	0.100	23.33	3.14	11.5
8	0.13	8,592	0.00	0.00	1.00	8.2	0.100	0.100	26.67	3.14	11.5
9	0.13	8,592	0.00	0.00	1.00	10.1	0.111	0.100	30.00	3.14	11.5
10	0.13	8,592	0.00	0.00	1.00	6.4	0.150	0.100	33.33	3.14	11.5
13	0.13	8,592	0.00	0.00	1.00	6.4	0.150	0.100	43.33	3.14	11.5
14	0.13	8,592	0.00	0.00	1.00	21.8	0.057	0.100	46.67	3.14	11.5
15	0.13	8,592	0.00	0.00	1.00	26.3	0.050	0.100	50.00	3.14	11.5
16	0.13	8,592	0.00	0.00	1.00	27.8	0.050	0.100	53.33	3.14	11.5
17	0.13	8,592	0.00	0.00	1.00	29.3	0.050	0.100	56.67	3.14	11.5
18	0.13	8,592	0.00	0.00	1.00	30.8	0.050	0.100	60.00	3.14	11.5
19	0.13	8,592	0.00	0.00	1.00	24.6	0.097	0.100	63.33	3.14	11.5
20	0.13	8,592	0.00	0.00	1.00	28.0	0.100	0.100	66.67	3.14	11.5
21	0.13	8,592	0.00	0.00	1.00	32.7	0.100	0.100	70.00	3.14	11.5
Toe						246.4	0.149	0.112	70.00		

2.740 kips total unreduced pile weight ($g = 32.169 \text{ ft/s}^2$)2.740 kips total reduced pile weight ($g = 32.169 \text{ ft/s}^2$)

OTHER OPTIONS

Pile Damping (%):	1	Pile Damping Fact. (kips/ft/s):	0.409
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EXTREMA TABLE at 65.0 FT; HAMMER: D 19-42

Shaft/Toe Gain/Loss Factor = 0.667/1.000

Rut = 511.5 kips

Rtoe = 246.4 kips

Time Inc. = 0.055 ms

Hammer

DELMAG D 19-42

Efficiency

0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	430.0	0.00	37.54	17.00	0.973	22.95
6.7	13.6	431.6	1.19	37.67	16.92	0.931	22.33
10.0	26.4	430.3	2.30	37.56	16.83	0.886	21.63
13.3	38.4	427.6	3.35	37.33	16.72	0.838	20.83
16.7	48.9	433.0	4.27	37.79	16.58	0.788	19.95
20.0	57.9	443.4	5.05	38.70	16.34	0.740	19.12
23.3	65.9	443.8	5.75	38.74	15.89	0.693	18.33
26.7	73.4	443.3	6.41	38.69	15.41	0.644	17.08
30.0	69.4	435.5	6.06	38.02	14.98	0.593	15.31
33.3	62.8	442.8	5.48	38.65	14.55	0.542	13.60
36.7	61.4	441.6	5.36	38.55	14.09	0.491	12.09
40.0	60.3	436.8	5.27	38.13	13.57	0.442	10.69
43.3	59.4	427.9	5.18	37.35	12.87	0.395	9.43
46.7	57.7	420.4	5.03	36.70	12.00	0.347	8.01
50.0	40.4	406.9	3.53	35.52	11.05	0.301	6.48
53.3	18.7	384.7	1.63	33.58	10.16	0.258	5.12
56.7	0.0	355.4	0.00	31.02	9.41	0.218	3.97
60.0	0.0	324.3	0.00	28.31	8.77	0.183	3.02
63.3	0.0	291.3	0.00	25.43	8.29	0.150	2.26
66.7	0.0	272.9	0.00	23.82	7.52	0.120	1.65
70.0	0.0	255.9	0.00	22.34	5.61	0.092	1.27

Converged Stroke (ft)

9.04

Fixed Combustion Pressure (psi)

1,440.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81

9.38

9.10

9.04

Shaft/Toe Gain/Loss Factor = 1.000/1.000

Rut = 547.9 kips

Rtoe = 246.4 kips

Time Inc. = 0.054 ms

Hammer

DELMAG D 19-42

Efficiency

0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	418.4	0.00	36.52	16.84	0.936	22.47
6.7	14.4	420.7	1.26	36.72	16.76	0.894	21.86
10.0	26.7	418.3	2.33	36.51	16.67	0.850	21.16
13.3	38.4	416.0	3.35	36.32	16.56	0.802	20.37
16.7	48.8	430.1	4.26	37.54	16.41	0.753	19.50
20.0	57.8	441.2	5.04	38.51	16.14	0.705	18.68
23.3	66.3	441.5	5.79	38.54	15.60	0.657	17.87
26.7	74.0	440.2	6.46	38.42	14.96	0.607	16.55
30.0	66.8	443.1	5.83	38.68	14.32	0.556	14.56
33.3	52.3	445.0	4.56	38.84	13.69	0.505	12.62
36.7	45.9	439.5	4.01	38.37	13.05	0.456	10.96
40.0	41.0	430.3	3.58	37.56	12.39	0.408	9.46
43.3	36.0	417.9	3.14	36.48	11.62	0.361	8.10
46.7	31.3	406.9	2.73	35.51	10.75	0.316	6.74

50.0	14.0	383.8	1.22	33.50	9.94	0.272	5.40
53.3	0.0	359.3	0.00	31.36	9.23	0.232	4.22
56.7	0.0	332.2	0.00	28.99	8.57	0.194	3.21
60.0	0.0	302.2	0.00	26.37	7.99	0.160	2.40
63.3	0.0	270.6	0.00	23.62	7.50	0.130	1.74
66.7	0.0	242.3	0.00	21.15	6.84	0.102	1.23
70.0	0.0	218.2	0.00	19.04	5.20	0.078	0.93

Converged Stroke (ft) 8.94 Fixed Combustion Pressure (psi) 1,440.0
(Eq) Strokes Analyzed and Last Return (ft)
10.81 9.27 8.99 8.94

SUMMARY TABLE at 65.0 FT; HAMMER: D 19-42

Rut	BI Ct	Stk Dn	Stk Up	Mx T-Str	LTop	Mx C-Str	LTop	ENTHRU	BI Rt	ActRes
kips	b/ft	ft	ft	ksi	ft	ksi	ft	kip-ft	b/min	kips
511.5	9,999	9.10	9.04	6.41	26.7	38.74	23.3	23.0	39.2	465.5
547.9	9,999	8.99	8.94	6.46	26.7	38.84	33.3	22.5	39.4	465.7

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.667/1.000									
Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.8	0.0	0.8	0.0	0.00	0.00	10.81	0.0	D 19-42
10.0	1.7	0.0	1.7	0.0	0.00	0.00	10.81	0.0	D 19-42
15.0	2.5	0.0	2.5	0.0	0.00	0.00	10.81	0.0	D 19-42
20.0	61.8	3.9	57.8	6.7	20.43	0.57	4.96	18.3	D 19-42
25.0	43.9	18.1	25.8	4.2	18.13	0.67	4.52	19.6	D 19-42
30.0	53.1	27.3	25.8	5.3	19.77	0.83	4.75	18.8	D 19-42
35.0	62.5	36.7	25.8	6.6	21.46	1.06	4.96	18.3	D 19-42
40.0	250.8	53.0	197.8	35.8	34.26	5.71	7.47	20.1	D 19-42
45.0	289.7	91.9	197.8	49.5	34.77	7.00	7.82	20.8	D 19-42
50.0	332.0	134.2	197.8	71.5	35.71	7.33	8.28	21.6	D 19-42
65.0	511.5	265.1	246.4	9,999.0	38.74	6.41	9.10	23.0	D 19-42

G/L at Shaft and Toe: 1.000/1.000									
Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.8	0.0	0.8	0.0	0.00	0.00	10.81	0.0	D 19-42
10.0	1.7	0.0	1.7	0.0	0.00	0.00	10.81	0.0	D 19-42
15.0	2.5	0.0	2.5	0.0	0.00	0.00	10.81	0.0	D 19-42
20.0	62.5	4.7	57.8	6.8	20.47	0.61	4.97	18.3	D 19-42
25.0	50.2	24.4	25.8	4.9	19.05	0.82	4.66	19.1	D 19-42
30.0	64.0	38.2	25.8	6.7	21.20	1.12	4.97	18.2	D 19-42
35.0	78.0	52.2	25.8	8.6	23.19	1.48	5.22	17.6	D 19-42
40.0	270.5	72.7	197.8	41.4	35.39	6.91	7.71	20.6	D 19-42
45.0	309.4	111.6	197.8	59.2	35.32	7.59	8.09	21.3	D 19-42
50.0	351.7	153.9	197.8	91.5	36.16	7.50	8.52	22.1	D 19-42
65.0	547.9	301.5	246.4	9,999.0	38.84	6.46	8.99	22.5	D 19-42

APPENDIX 3B

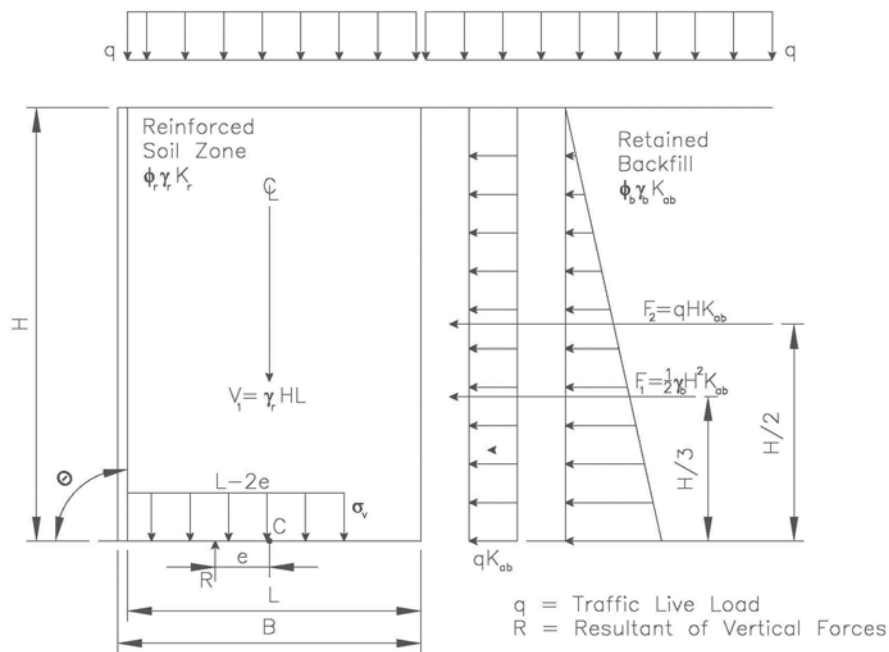
EXTERNAL STABILITY ANALYSIS

RW-1 – STA. 00+09.17

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.
Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- Horizontal backfill behind MSE wall on granular (drained) soils.
- 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
- Load combinations and wall configuration are as shown below:



Givens:

Wall Geometry:

$$H_e := 5.5 \cdot ft$$

Exposed wall height

$$\theta := 90 \cdot deg$$

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = $80 \text{ deg} < \theta < 100 \text{ deg}$)

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_r := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_b := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 23 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 120 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

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

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 1500 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use S_u if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 9 \text{ ft}$$

$$d_{user} := 0 \text{ ft}$$

Local Frost Depth

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 9 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 14.5 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 0.7 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 9 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 14.5 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 14 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot \text{ft}, L_{user})$$

$$L = 14 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb} \cdot \text{f}}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 3749.9 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 431 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 24360 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 1400 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 3749.9 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 431 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 4.8 \text{ ft}$$

Moment:

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 18124.7 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 7.3 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 3124.9 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 24360 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 1400 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 3749.9 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 431 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 18124.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 3124.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 24360 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 35336 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 6379.2 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 6379.2 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 32655.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 32655.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$$\Sigma M_R := MV_{lb}$$

$$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength lb)

$$\Sigma M_O := MH_{lb}$$

$$\Sigma M_O = 32655.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength lb)

$$\Sigma V := V_{lb}$$

$$\Sigma V = 35336 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength lb)

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$$

$$e_{wall} = 0.9 \text{ ft}$$

Wall Eccentricity

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$$

$$B' = 12.2 \text{ ft}$$

Effective Bearing Width

Foundation Layout:

$$L_{wall} := 9.2 \cdot \text{ft}$$

Assumed Footing Length (Wall Section Length)

$$H' := H_{lb}$$

$$H' = 6379.2 \frac{\text{lb}}{\text{ft}}$$

Summation of Horizontal Loads (Strength lb)

$$V' := V_{lb}$$

$$V' = 35336 \frac{\text{lb}}{\text{ft}}$$

Summation of Vertical Loads (Strength lb)

$$D_f := d_e$$

$$D_f = 9 \text{ ft}$$

Footing embedment

$$d_w := -0.5 \cdot \text{ft}$$

Depth of Groundwater below Bearing Grade

$$\theta' := 90 \cdot \text{deg}$$

Direction of H' and V' resultant measured from wall back face **LRFD [Figure C10.6.3.1.2a-1]**

Drained Conditions (Effective Stress):

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

$$N_q = 8.66$$

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

$$N_c = 18.05$$

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

$$N_\gamma = 8.2$$

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

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

$$s_c = 1.634$$

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

$$s_q = 1.561$$

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

$$s_\gamma = 0.472$$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.2$$

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

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

$$N_{cm} = 29.489$$

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

$$N_{qm} = 13.517$$

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

$$N_{\gamma m} = 3.869$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 14.8 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 9.6 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

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

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.264$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 6.498$$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 10.4 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 6.8 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 9.6 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 6.8 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 6.8 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 32655.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 35336 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 0.9 \text{ ft} \quad \text{Wall Eccentricity}$$

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L) \quad B' = 12.2 \text{ ft} \quad \text{Effective Bearing Width}$$

Compute the bearing stress:

$$\sigma := \frac{V_1 + V_2}{B'} \quad \sigma = 2.1 \text{ ksf} \quad \text{Nominal Bearing Stress}$$

$$\sigma_v := \frac{\Sigma V}{B'} \quad \sigma_v = 2.9 \text{ ksf} \quad \text{Factored Bearing Stress}$$

Bearing Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} := \frac{q_R}{\sigma_v} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Bearing} = 2.33$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3} \quad e_{max} = 4.7 \text{ ft} \quad \text{Maximum Eccentricity LRFD [C11.6.3.3.]}$$

$$\Sigma M_R := MV_{Ia} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ia)}$$

$$\Sigma M_O := MH_{Ia} \quad \Sigma M_O = 32655.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ia)}$$

$$\Sigma V := V_{Ia} \quad \Sigma V = 24360 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 1.3 \text{ ft}$$

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Eccentricity} = 3.48$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 6379.2 \frac{\text{lb}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 24360 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{\tau d} := \Sigma V \cdot \tan(\phi'_f) \quad R_{\tau d} = 10340.2 \frac{\text{lb}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{\tau d} = 10.34 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{\tau d}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 10.3 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{\text{Sliding}} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

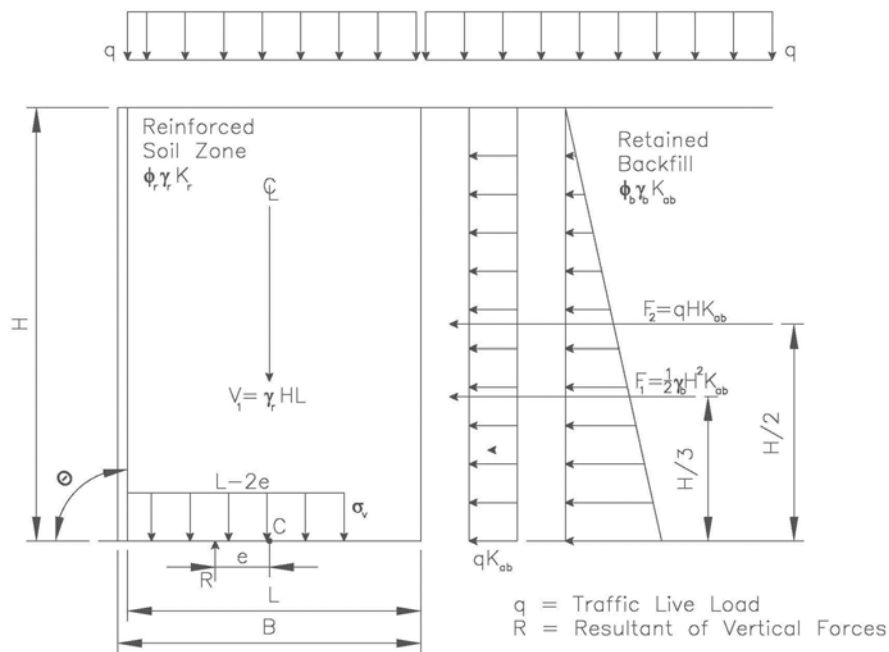
$$CDR_{\text{Sliding}} = 1.62$$

RW-1 – STA. 00+18.33

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.
Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- Horizontal backfill behind MSE wall on granular (drained) soils.
- 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
- Load combinations and wall configuration are as shown below:



Givens:

Wall Geometry:

$$H_e := 10.2 \cdot ft$$

Exposed wall height

$$\theta := 90 \cdot deg$$

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = $80 \text{ deg} < \theta < 100 \text{ deg}$)

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_r := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_b := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 23 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 120 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

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

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 1500 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use Su if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 7 \text{ ft}$$

$$d_{user} := 0 \text{ ft}$$

Local Frost Depth

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 7 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 17.2 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 0.9 \text{ ft}$$

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.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 17.2 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 14 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot \text{ft}, L_{user})$$

$$L = 14 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb} \cdot \text{f}}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 5276.5 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 511.3 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 28896 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 1400 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 5276.5 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 511.3 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 5.7 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 30251.8 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 8.6 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 4397.1 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Moment:

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 28896 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 1400 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 5276.5 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 511.3 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 30251.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 4397.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 28896 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 41459.6 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 8809.5 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 8809.5 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 53072.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 53072.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 53072.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 41459.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Vertical Loads (Strength lb)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 1.3 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 11.4 \text{ ft}$	Effective Bearing Width

Foundation Layout:

$L_{wall} := 9.2 \cdot \text{ft}$		Assumed Footing Length (Wall Section Length)
$H' := H_{lb}$	$H' = 8809.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 41459.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Summation of Vertical Loads (Strength lb)
$D_f := d_e$	$D_f = 7 \text{ ft}$	Footing embedment
$d_w := -0.5 \cdot \text{ft}$		Depth of Groundwater below Bearing Grade
$\theta' := 90 \cdot \text{deg}$		Direction of H' and V' resultant measured from wall back face LRFD [Figure C10.6.3.1.2a-1]

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \text{ deg} + \frac{\phi'_f}{2}\right), 1.0\right)$	$N_q = 8.66$
$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$	$N_c = 18.05$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$	$N_\gamma = 8.2$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{wall}}\right)\right)$	$s_c = 1.597$
$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \tan(\phi'_f), 1\right)$	$s_q = 1.528$
$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{wall}}\right), 1\right)$	$s_\gamma = 0.503$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.2$$

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

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

$$N_{cm} = 28.818$$

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

$$N_{qm} = 13.233$$

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

$$N_{\gamma m} = 4.122$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 12.4 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 8 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

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

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.249$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 6.418$$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 10.1 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 6.6 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 8 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 6.6 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 6.6 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 53072.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 41459.6 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 1.3 \text{ ft} \quad \text{Wall Eccentricity}$$

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L) \quad B' = 11.4 \text{ ft} \quad \text{Effective Bearing Width}$$

Compute the bearing stress:

$$\sigma := \frac{V_1 + V_2}{B'} \quad \sigma = 2.6 \text{ ksf} \quad \text{Nominal Bearing Stress}$$

$$\sigma_v := \frac{\Sigma V}{B'} \quad \sigma_v = 3.6 \text{ ksf} \quad \text{Factored Bearing Stress}$$

Bearing Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} := \frac{q_R}{\sigma_v} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0? \quad CDR_{Bearing} = 1.82$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3} \quad e_{max} = 4.7 \text{ ft} \quad \text{Maximum Eccentricity LRFD [C11.6.3.3.]}$$

$$\Sigma M_R := MV_{Ia} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ia)}$$

$$\Sigma M_O := MH_{Ia} \quad \Sigma M_O = 53072.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ia)}$$

$$\Sigma V := V_{Ia} \quad \Sigma V = 28896 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 1.8 \text{ ft}$$

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0? \quad CDR_{Eccentricity} = 2.54$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 8809.5 \frac{\text{lb}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 28896 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{td} := \Sigma V \cdot \tan(\phi'_f) \quad R_{td} = 12265.6 \frac{\text{lb}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{td} = 12.266 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{td}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 12.3 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

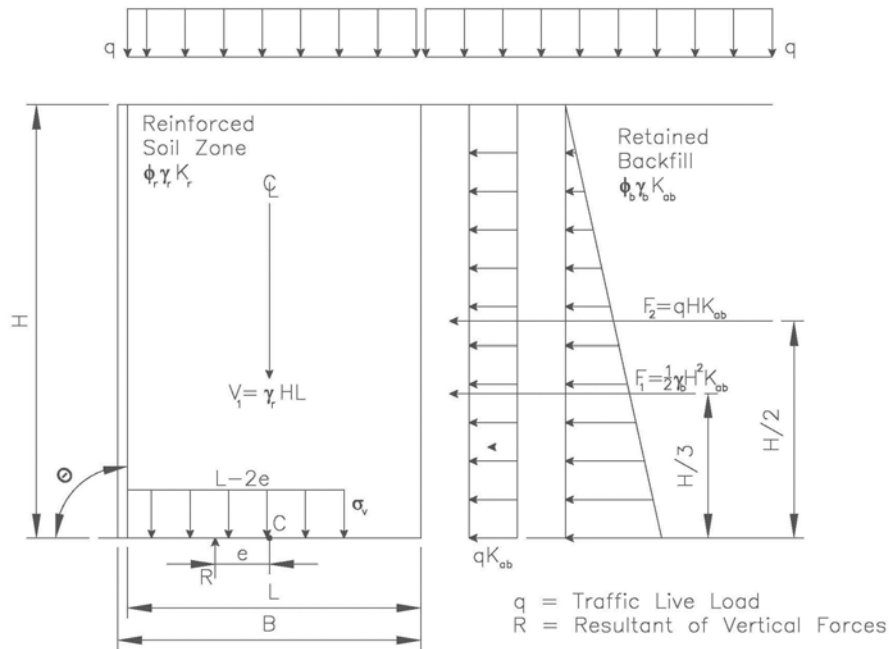
$$CDR_{Sliding} = 1.39$$

RW-1 – STA. 00+27.50

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.
Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- Horizontal backfill behind MSE wall on granular (drained) soils.
- 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
- Load combinations and wall configuration are as shown below:



Givens:

Wall Geometry:

$$H_e := 13 \cdot ft$$

Exposed wall height

$$\theta := 90 \cdot deg$$

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = $80 \text{ deg} < \theta < 100 \text{ deg}$)

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_r := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_b := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 23 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 120 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

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

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 1500 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use Su if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 7 \text{ ft}$$

$$d_{user} := 0 \text{ ft}$$

Local Frost Depth

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 7 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 20 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1 \text{ ft}$$

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.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 20 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 14.0 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot \text{ft}, L_{user})$$

$$L = 14 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb} \cdot \text{f}}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$K := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$K_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\sin(\theta))^2 \cdot \sin(\theta - \delta)} \right) K_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot K_{af}$$

$$F_T = 7134.2 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot K_{af}$$

$$F_{SUR} = 594.5 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 33600 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 1400 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 7134.2 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 594.5 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 6.7 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 47561.6 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 10 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 5945.2 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Moment:

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 33600 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 1400 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 7134.2 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 594.5 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 47561.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 5945.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 33600 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 47810 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 11741.8 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 11741.8 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 81746.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 81746.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 81746.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 47810 \frac{\text{lb}}{\text{ft}}$	Sum of Vertical Loads (Strength lb)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 1.7 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 10.6 \text{ ft}$	Effective Bearing Width

Foundation Layout:

$L_{wall} := 9.2 \cdot \text{ft}$		Assumed Footing Length (Wall Section Length)
$H' := H_{lb}$	$H' = 11741.8 \frac{\text{lb}}{\text{ft}}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 47810 \frac{\text{lb}}{\text{ft}}$	Summation of Vertical Loads (Strength lb)
$D_f := d_e$	$D_f = 7 \text{ ft}$	Footing embedment
$d_w := -0.5 \cdot \text{ft}$		Depth of Groundwater below Bearing Grade
$\theta' := 90 \cdot \text{deg}$		Direction of H' and V' resultant measured from wall back face LRFD [Figure C10.6.3.1.2a-1]

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \text{ deg} + \frac{\phi'_f}{2}\right), 1.0\right)$	$N_q = 8.66$
$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$	$N_c = 18.05$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$	$N_\gamma = 8.2$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{wall}}\right)\right)$	$s_c = 1.552$
$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \tan(\phi'_f), 1\right)$	$s_q = 1.488$
$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{wall}}\right), 1\right)$	$s_\gamma = 0.54$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.2$$

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

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

$$N_{cm} = 28.009$$

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

$$N_{qm} = 12.889$$

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

$$N_{\gamma m} = 4.429$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 12.1 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 7.9 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

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

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.23$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 6.322$$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 10 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 6.5 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 7.9 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 6.5 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 6.5 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 81746.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 47810 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 1.7 \text{ ft} \quad \text{Wall Eccentricity}$$

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L) \quad B' = 10.6 \text{ ft} \quad \text{Effective Bearing Width}$$

Compute the bearing stress:

$$\sigma := \frac{V_1 + V_2}{B'} \quad \sigma = 3.3 \text{ ksf} \quad \text{Nominal Bearing Stress}$$

$$\sigma_v := \frac{\Sigma V}{B'} \quad \sigma_v = 4.5 \text{ ksf} \quad \text{Factored Bearing Stress}$$

Bearing Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} := \frac{q_R}{\sigma_v} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0?$$

$$CDR_{Bearing} = 1.44$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3} \quad e_{max} = 4.7 \text{ ft} \quad \text{Maximum Eccentricity LRFD [C11.6.3.3.]}$$

$$\Sigma M_R := MV_{Ia} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ia)}$$

$$\Sigma M_O := MH_{Ia} \quad \Sigma M_O = 81746.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ia)}$$

$$\Sigma V := V_{Ia} \quad \Sigma V = 33600 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 2.4 \text{ ft}$$

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0?$$

$$CDR_{Eccentricity} = 1.92$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 11741.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 33600 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{\tau d} := \Sigma V \cdot \tan(\phi'_f) \quad R_{\tau d} = 14262.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{\tau d} = 14.262 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{\tau d}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 14.3 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{\text{Sliding}} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

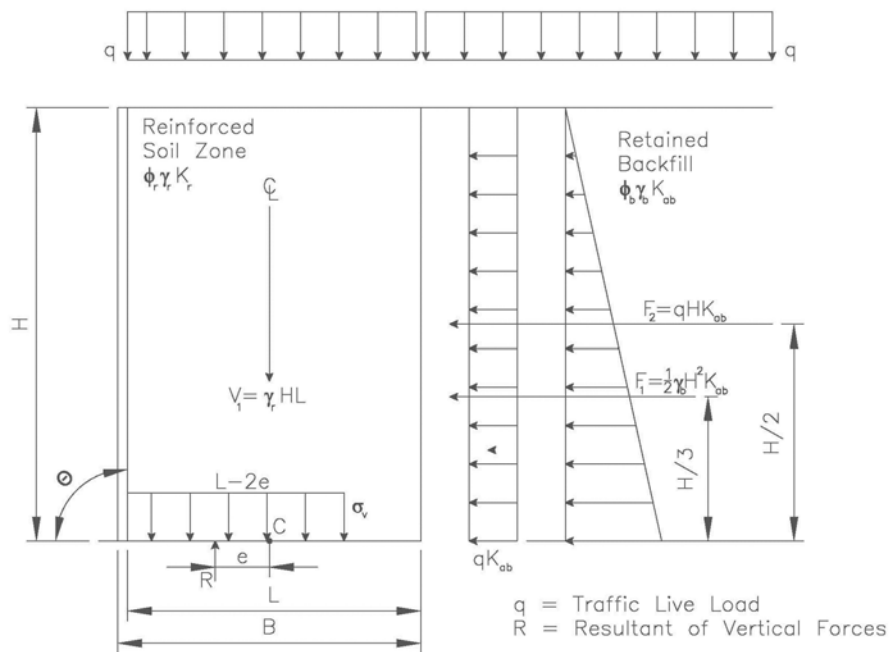
$$CDR_{\text{Sliding}} = 1.21$$

RW-1 – STA. 00+37.50

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.
Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- Horizontal backfill behind MSE wall on granular (drained) soils.
- 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
- Load combinations and wall configuration are as shown below:



Givens:

Wall Geometry:

$$H_e := 17.7 \cdot ft$$

Exposed wall height

$$\theta := 90 \cdot deg$$

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = $80 \text{ deg} < \theta < 100 \text{ deg}$)

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_r := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_b := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 23 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 120 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

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

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 1500 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use Su if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 5 \text{ ft}$$

$$d_{user} := 0 \text{ ft}$$

Local Frost Depth

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 5 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 22.7 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1.1 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 5 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 22.7 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 18 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot \text{ft}, L_{user})$$

$$L = 18 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb} \cdot \text{f}}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 9190.5 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 674.8 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 49032 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 1800 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 9190.5 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 674.8 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 7.6 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 69541.5 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 11.4 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 7658.8 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Moment:

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 49032 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 1800 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 9190.5 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 674.8 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 69541.5 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 7658.8 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 49032 \frac{\text{lbf}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 69343.2 \frac{\text{lbf}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 14966.6 \frac{\text{lbf}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 14966.6 \frac{\text{lbf}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 117715.1 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 117715.1 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 117715.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 69343.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Vertical Loads (Strength lb)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 1.7 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 14.6 \text{ ft}$	Effective Bearing Width

Foundation Layout:

$L_{wall} := 10 \cdot \text{ft}$		Assumed Footing Length (Wall Section Length)
$H' := H_{lb}$	$H' = 14966.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 69343.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Summation of Vertical Loads (Strength lb)
$D_f := d_e$	$D_f = 5 \text{ ft}$	Footing embedment
$d_w := -0.5 \cdot \text{ft}$		Depth of Groundwater below Bearing Grade
$\theta' := 90 \cdot \text{deg}$		Direction of H' and V' resultant measured from wall back face LRFD [Figure C10.6.3.1.2a-1]

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \text{ deg} + \frac{\phi'_f}{2}\right), 1.0\right)$	$N_q = 8.66$
$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$	$N_c = 18.05$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$	$N_\gamma = 8.2$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{wall}}\right)\right)$	$s_c = 1.701$
$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \tan(\phi'_f), 1\right)$	$s_q = 1.62$
$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{wall}}\right), 1\right)$	$s_\gamma = 0.416$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 30.698$$

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

$$N_{qm} = 14.031$$

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

$$N_{\gamma m} = 3.41$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 10.8 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 7 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

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

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.292$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 6.641$$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 10.3 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 6.7 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 7 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 6.7 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 6.7 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 117715.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 69343.2 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 1.7 \text{ ft} \quad \text{Wall Eccentricity}$$

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L) \quad B' = 14.6 \text{ ft} \quad \text{Effective Bearing Width}$$

Compute the bearing stress:

$$\sigma := \frac{V_1 + V_2}{B'} \quad \sigma = 3.5 \text{ ksf} \quad \text{Nominal Bearing Stress}$$

$$\sigma_v := \frac{\Sigma V}{B'} \quad \sigma_v = 4.7 \text{ ksf} \quad \text{Factored Bearing Stress}$$

Bearing Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} := \frac{q_R}{\sigma_v} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0?$$

$$CDR_{Bearing} = 1.41$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3} \quad e_{max} = 6 \text{ ft} \quad \text{Maximum Eccentricity LRFD [C11.6.3.3.]}$$

$$\Sigma M_R := MV_{Ia} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ia)}$$

$$\Sigma M_O := MH_{Ia} \quad \Sigma M_O = 117715.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ia)}$$

$$\Sigma V := V_{Ia} \quad \Sigma V = 49032 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 2.4 \text{ ft}$$

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0?$$

$$CDR_{Eccentricity} = 2.50$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 14966.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 49032 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{\tau d} := \Sigma V \cdot \tan(\phi'_f) \quad R_{\tau d} = 20812.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{\tau d} = 20.813 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{\tau d}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 20.8 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{\text{Sliding}} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

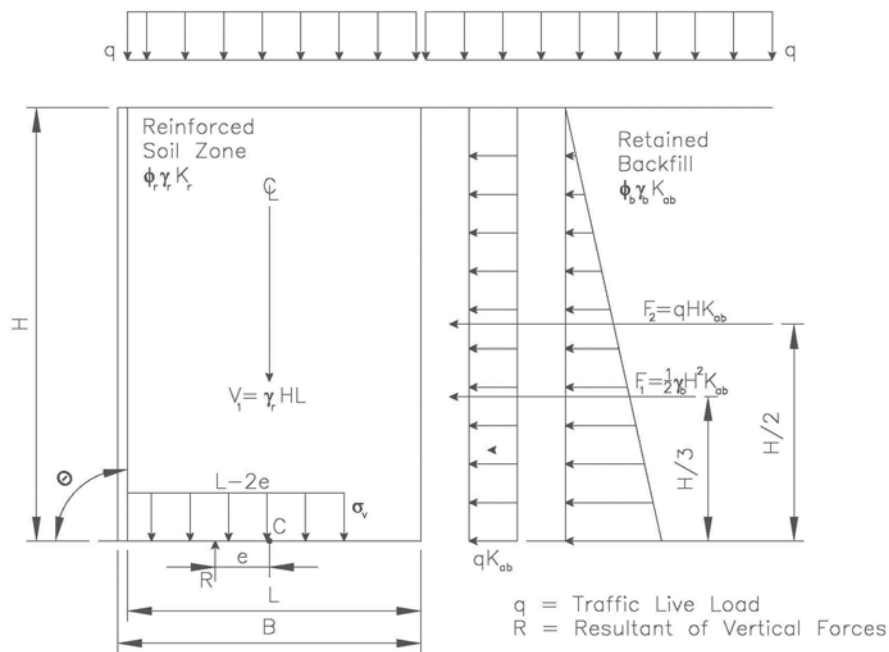
$$CDR_{\text{Sliding}} = 1.39$$

RW-1 – STA. 00+47.50

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.
Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- Horizontal backfill behind MSE wall on granular (drained) soils.
- 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
- Load combinations and wall configuration are as shown below:



Givens:

Wall Geometry:

$$H_e := 20.5 \cdot ft$$

Exposed wall height

$$\theta := 90 \cdot deg$$

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)

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_r := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_b := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 23 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 120 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

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

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 120 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 1500 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use Su if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 5 \text{ ft}$$

$$d_{user} := 0 \text{ ft}$$

Local Frost Depth

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 5 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 25.5 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1.3 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 5 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 25.5 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 18 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot \text{ft}, L_{user})$$

$$L = 18 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb} \cdot \text{ft}}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 11597.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 758 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 55080 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 1800 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb} \cdot \text{ft}^2}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{ft}^2}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 11597.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 758 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 8.5 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 98579.6 \frac{\text{lb} \cdot \text{ft}^2}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 12.8 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 9664.7 \frac{\text{lb} \cdot \text{ft}^2}{\text{ft}}$$

Moment:

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 55080 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 1800 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 11597.6 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 758 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 98579.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 9664.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 55080 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 77508 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 18722.9 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 18722.9 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 164782.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 164782.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 0 \frac{lb \cdot ft}{ft}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 164782.6 \frac{lb \cdot ft}{ft}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 77508 \frac{lb}{ft}$	Sum of Vertical Loads (Strength lb)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 2.1 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 13.7 \text{ ft}$	Effective Bearing Width

Foundation Layout:

$L_{wall} := 10 \cdot \text{ft}$		Assumed Footing Length (Wall Section Length)
$H' := H_{lb}$	$H' = 18722.9 \frac{lb}{ft}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 77508 \frac{lb}{ft}$	Summation of Vertical Loads (Strength lb)
$D_f := d_e$	$D_f = 5 \text{ ft}$	Footing embedment
$d_w := -0.5 \cdot \text{ft}$		Depth of Groundwater below Bearing Grade
$\theta' := 90 \cdot \text{deg}$		Direction of H' and V' resultant measured from wall back face LRFD [Figure C10.6.3.1.2a-1]

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \text{ deg} + \frac{\phi'_f}{2}\right), 1.0\right)$	$N_q = 8.66$
$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$	$N_c = 18.05$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$	$N_\gamma = 8.2$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{wall}}\right)\right)$	$s_c = 1.66$
$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \tan(\phi'_f), 1\right)$	$s_q = 1.584$
$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{wall}}\right), 1\right)$	$s_\gamma = 0.45$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 29.956$$

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

$$N_{qm} = 13.716$$

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

$$N_{\gamma m} = 3.691$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 10.6 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 6.9 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

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

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.275$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 6.553$$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 10.2 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 6.6 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 6.9 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 6.6 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 6.6 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 164782.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 77508 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 2.1 \text{ ft} \quad \text{Wall Eccentricity}$$

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L) \quad B' = 13.7 \text{ ft} \quad \text{Effective Bearing Width}$$

Compute the bearing stress:

$$\sigma := \frac{V_1 + V_2}{B'} \quad \sigma = 4.1 \text{ ksf} \quad \text{Nominal Bearing Stress}$$

$$\sigma_v := \frac{\Sigma V}{B'} \quad \sigma_v = 5.6 \text{ ksf} \quad \text{Factored Bearing Stress}$$

Bearing Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} := \frac{q_R}{\sigma_v} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0?$$

$$CDR_{Bearing} = 1.17$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3} \quad e_{max} = 6 \text{ ft} \quad \text{Maximum Eccentricity LRFD [C11.6.3.3.]}$$

$$\Sigma M_R := MV_{Ia} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ia)}$$

$$\Sigma M_O := MH_{Ia} \quad \Sigma M_O = 164782.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ia)}$$

$$\Sigma V := V_{Ia} \quad \Sigma V = 55080 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 3 \text{ ft}$$

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0?$$

$$CDR_{Eccentricity} = 2.01$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 18722.9 \frac{\text{lb}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 55080 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{td} := \Sigma V \cdot \tan(\phi'_f) \quad R_{td} = 23380.1 \frac{\text{lb}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{td} = 23.38 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{td}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 23.4 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding} = 1.25$$

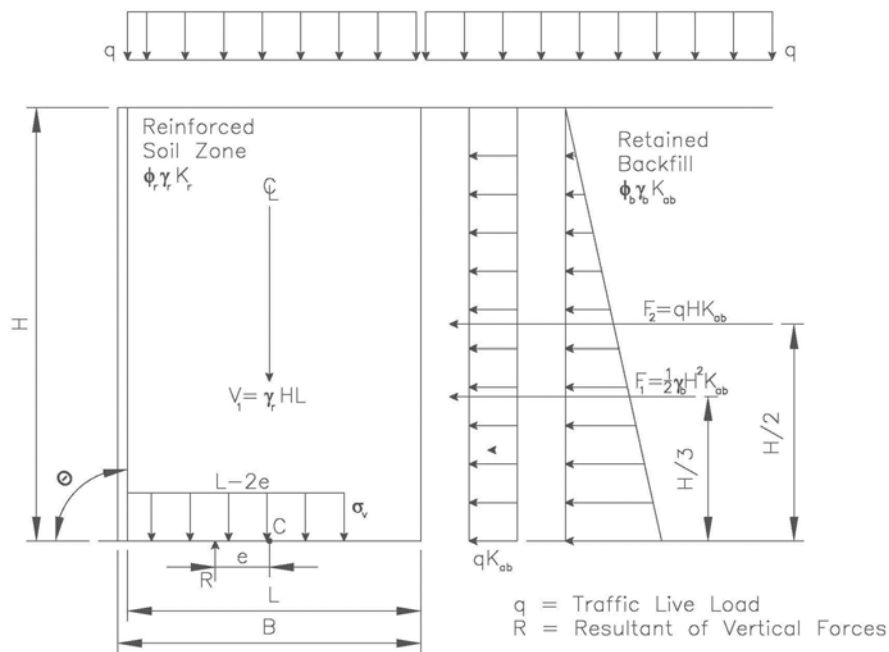
RW-1 – STA. 00+70.78

NO UNDERCUT – BOTTOM OF FOUNDATION PREP LAYER

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.
Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- Horizontal backfill behind MSE wall on granular (drained) soils.
- 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
- Load combinations and wall configuration are as shown below:



Givens:

Wall Geometry:

$$H_e := 25 \cdot ft$$

Exposed wall height

$$\theta := 90 \cdot deg$$

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = $80 \text{ deg} < \theta < 100 \text{ deg}$)

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_r := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_b := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 22 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 108 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 85 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

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

Unit weight

$$c_f := 875 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use S_u if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 3.5 \text{ ft}$$

$$d_{user} := 0 \text{ ft}$$

Local Frost Depth

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 3.5 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 28.5 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1.4 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 3.5 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 28.5 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 20 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot \text{ft}, L_{user})$$

$$L = 20 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb}f}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 14487 \frac{\text{lb}f}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 847.2 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 68400 \frac{\text{lb}f}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 2000 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 14487 \frac{\text{lb}f}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 847.2 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 9.5 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 137626.2 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 14.3 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 12072.5 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 68400 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 2000 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 14487 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 847.2 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 137626.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 12072.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 68400 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 95840 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 23213 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 23213 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 0 \frac{lb \cdot ft}{ft}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 227566.1 \frac{lb \cdot ft}{ft}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 95840 \frac{lb}{ft}$	Sum of Vertical Loads (Strength lb)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 2.4 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 15.3 \text{ ft}$	Effective Bearing Width

Foundation Layout:

$L_{wall} := 25 \cdot \text{ft}$		Assumed Footing Length (Wall Section Length)
$H' := H_{lb}$	$H' = 23213 \frac{lb}{ft}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 95840 \frac{lb}{ft}$	Summation of Vertical Loads (Strength lb)
$D_f := d_e$	$D_f = 3.5 \text{ ft}$	Footing embedment
$d_w := -0.5 \cdot \text{ft}$		Depth of Groundwater below Bearing Grade
$\theta' := 90 \cdot \text{deg}$		Direction of H' and V' resultant measured from wall back face LRFD [Figure C10.6.3.1.2a-1]

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \text{ deg} + \frac{\phi'_f}{2}\right), 1.0\right)$	$N_q = 7.82$
$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$	$N_c = 16.88$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$	$N_\gamma = 7.1$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{wall}}\right)\right)$	$s_c = 1.283$
$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \tan(\phi'_f), 1\right)$	$s_q = 1.246$
$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{wall}}\right), 1\right)$	$s_\gamma = 0.756$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 21.654$$

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

$$N_{qm} = 9.749$$

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

$$N_{\gamma m} = 5.389$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 6.3 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 4.1 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

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

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.122$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 5.767$$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 5.3 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 3.4 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 4.1 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 3.4 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 3.4 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ib)

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ib)

$$\Sigma V := V_{Ib} \quad \Sigma V = 95840 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ib)

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 2.4 \text{ ft}$$

Wall Eccentricity

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L) \quad B' = 15.3 \text{ ft}$$

Effective Bearing Width

Compute the bearing stress:

$$\sigma := \frac{V_1 + V_2}{B'} \quad \sigma = 4.6 \text{ ksf}$$

Nominal Bearing Stress

$$\sigma_v := \frac{\Sigma V}{B'} \quad \sigma_v = 6.3 \text{ ksf}$$

Factored Bearing Stress

Bearing Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} := \frac{q_R}{\sigma_v} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0?$$

$$CDR_{Bearing} = 0.55$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3} \quad e_{max} = 6.7 \text{ ft}$$

Maximum Eccentricity **LRFD [C11.6.3.3.]**

$$\Sigma M_R := MV_{Ia} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia} \quad \Sigma M_O = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia} \quad \Sigma V = 68400 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 3.3 \text{ ft}$$

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0?$$

$$CDR_{Eccentricity} = 2.00$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 23213 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 68400 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{\tau d} := \Sigma V \cdot \tan(\phi'_f) \quad R_{\tau d} = 27635.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{\tau d} = 27.635 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{\tau d}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 27.6 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{\text{Sliding}} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

$$CDR_{\text{Sliding}} = 1.19$$

RW-1 – STA. 00+70.78

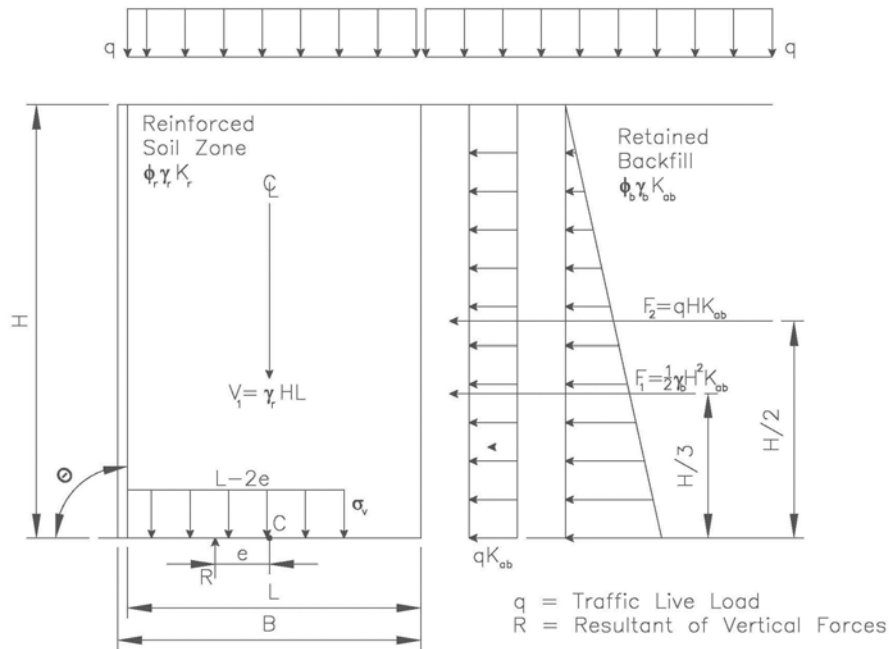
WITH UNDERCUT – TOP OF FOUNDATION PREP LAYER

Assuming Foundation Prep Layer Parameters (i.e., $\phi = 34$ degrees)

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.
Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- Horizontal backfill behind MSE wall on granular (drained) soils.
- 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
- Load combinations and wall configuration are as shown below:



Givens:

Wall Geometry:

$$H_e := 25 \cdot ft$$

Exposed wall height

$$\theta := 90 \cdot deg$$

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = $80 \text{ deg} < \theta < 100 \text{ deg}$)

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_r := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_b := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 34 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 120 \cdot \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight

$$c'_f := 0 \cdot \frac{\text{lb}_f}{\text{ft}^2}$$

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 34 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 120 \frac{\text{lb}_f}{\text{ft}^3}$$

Unit weight

$$c_f := 0 \cdot \frac{\text{lb}_f}{\text{ft}^2}$$

Cohesion (Use S_u if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 3.5 \text{ ft}$$

$$d_{user} := 0 \text{ ft}$$

Local Frost Depth

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 3.5 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 28.5 \text{ ft}$$

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1.4 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 3.5 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 28.5 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 20 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot \text{ft}, L_{user})$$

$$L = 20 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb}f}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right)$$

$$k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 14487 \frac{\text{lb}f}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 847.2 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 68400 \frac{\text{lb}f}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 2000 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 14487 \frac{\text{lb}f}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 847.2 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 9.5 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 137626.2 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 14.3 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 12072.5 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

Moment:

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 68400 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 2000 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 14487 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 847.2 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 137626.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 12072.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 68400 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 95840 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 23213 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 23213 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 95840 \frac{\text{lb}}{\text{ft}}$	Sum of Vertical Loads (Strength lb)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 2.4 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 15.3 \text{ ft}$	Effective Bearing Width

Foundation Layout:

$L_{Wall} := 25 \cdot \text{ft}$		Assumed Footing Length (Wall Section Length)
$H' := H_{lb}$	$H' = 23213 \frac{\text{lb}}{\text{ft}}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 95840 \frac{\text{lb}}{\text{ft}}$	Summation of Vertical Loads (Strength lb)
$D_f := d_e$	$D_f = 3.5 \text{ ft}$	Footing embedment
$d_w := -0.5 \cdot \text{ft}$		Depth of Groundwater below Bearing Grade
$\theta' := 90 \cdot \text{deg}$		Direction of H' and V' resultant measured from wall back face LRFD [Figure C10.6.3.1.2a-1]

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \text{ deg} + \frac{\phi'_f}{2}\right), 1.0\right)$	$N_q = 29.44$
$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$	$N_c = 42.16$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$	$N_\gamma = 41.1$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right)$	$s_c = 1.426$
$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \tan(\phi'_f), 1\right)$	$s_q = 1.411$
$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{Wall}}\right), 1\right)$	$s_\gamma = 0.756$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 60.123$$

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

$$N_{qm} = 41.554$$

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

$$N_{\gamma m} = 31.043$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 23.5 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 15.2 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 29.44$$

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

$$N_c = 42.16$$

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

$$N_\gamma = 41.1$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.426$$

$$s_q := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \cdot \tan(\phi_f) \right), 1 \right) \quad s_q = 1.411$$

$$s_\gamma := \text{if} \left(\phi_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{Wall}} \right), 1 \right) \quad s_\gamma = 0.756$$

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 60.123$$

$$N_{qm} := N_q \cdot s_q \cdot i_q \quad N_{qm} = 41.554$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 31.043$$

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 23.5 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 15.2 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 15.2 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 15.2 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 15.2 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 95840 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 2.4 \text{ ft} \quad \text{Wall Eccentricity}$$

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L) \quad B' = 15.3 \text{ ft} \quad \text{Effective Bearing Width}$$

Compute the bearing stress:

$$\sigma := \frac{V_1 + V_2}{B'} \quad \sigma = 4.6 \text{ ksf} \quad \text{Nominal Bearing Stress}$$

$$\sigma_v := \frac{\Sigma V}{B'} \quad \sigma_v = 6.3 \text{ ksf} \quad \text{Factored Bearing Stress}$$

Bearing Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} := \frac{q_R}{\sigma_v} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0?$$

$$CDR_{Bearing} = 2.43$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3} \quad e_{max} = 6.7 \text{ ft} \quad \text{Maximum Eccentricity LRFD [C11.6.3.3.]}$$

$$\Sigma M_R := MV_{Ia} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ia)}$$

$$\Sigma M_O := MH_{Ia} \quad \Sigma M_O = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ia)}$$

$$\Sigma V := V_{Ia} \quad \Sigma V = 68400 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 3.3 \text{ ft}$$

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0?$$

$$CDR_{Eccentricity} = 2.00$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 23213 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 68400 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{td} := \Sigma V \cdot \tan(\phi'_f) \quad R_{td} = 46136.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{td} = 46.136 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{td}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 46.1 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding} = 1.99$$

RW-1 – STA. 00+70.78

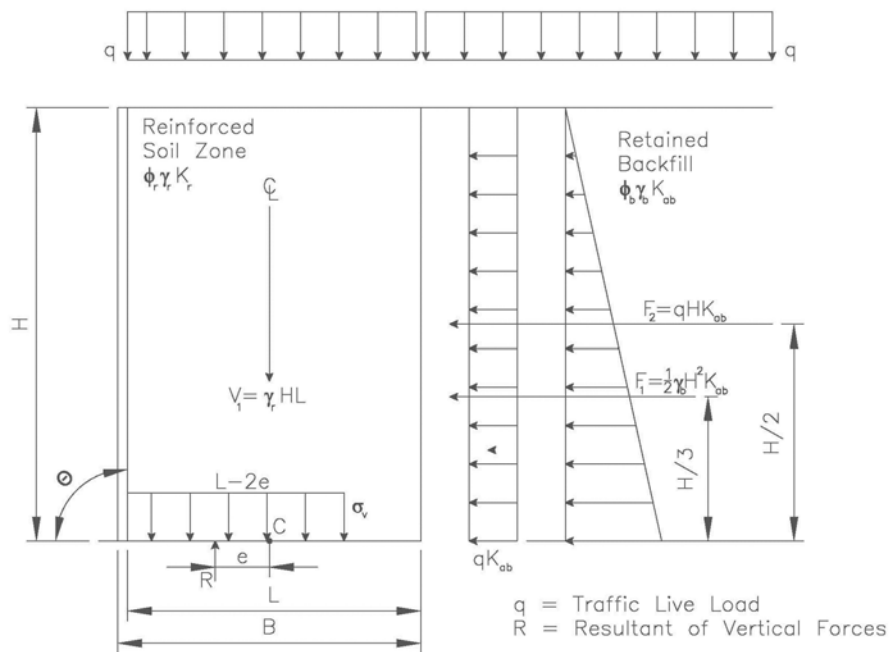
WITH UNDERCUT – BOTTOM OF FOUNDATION PREP LAYER

Assuming Weakest Soil Parameters within 1.5B Below Bearing Elevation
(i.e., eff. ϕ = 26 degrees, eff. cohesion = 225 psf, undrained shear strength = 2,500 psf)

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.
Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- Horizontal backfill behind MSE wall on granular (drained) soils.
- 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
- Load combinations and wall configuration are as shown below:



Givens:

Wall Geometry:

$$H_e := 25 \cdot ft$$

Exposed wall height

$$\theta := 90 \cdot deg$$

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)

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_r := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_b := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 26 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 125 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 225 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 125 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 2500 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use S_u if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 3.5 \text{ ft} \quad d_{user} := 0 \text{ ft}$$

$$Slope_{fw} := 0 \text{ deg}$$

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 3.5 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 28.5 \text{ ft}$$

Local Frost Depth

Inclination of ground slope in front of wall :

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1.4 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 3.5 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 28.5 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 20 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot \text{ft}, L_{user})$$

$$L = 20 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb}f}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 14487 \frac{\text{lb}f}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 847.2 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 68400 \frac{\text{lb}f}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 2000 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

Moment:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 14487 \frac{\text{lb}f}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 847.2 \frac{\text{lb}f}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

Moment:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 9.5 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 137626.2 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 14.3 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 12072.5 \frac{\text{lb}f \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 68400 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 2000 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 14487 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 847.2 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 137626.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 12072.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 68400 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 95840 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 23213 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 23213 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 0 \frac{lb \cdot ft}{ft}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 227566.1 \frac{lb \cdot ft}{ft}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 95840 \frac{lb}{ft}$	Sum of Vertical Loads (Strength lb)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 2.4 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 15.3 \text{ ft}$	Effective Bearing Width

Foundation Layout:

$L_{Wall} := 25 \cdot \text{ft}$		Assumed Footing Length (Wall Section Length)
$H' := H_{lb}$	$H' = 23213 \frac{lb}{ft}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 95840 \frac{lb}{ft}$	Summation of Vertical Loads (Strength lb)
$D_f := d_e$	$D_f = 3.5 \text{ ft}$	Footing embedment
$d_w := -0.5 \cdot \text{ft}$		Depth of Groundwater below Bearing Grade
$\theta' := 90 \cdot \text{deg}$		Direction of H' and V' resultant measured from wall back face LRFD [Figure C10.6.3.1.2a-1]

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \text{ deg} + \frac{\phi'_f}{2}\right), 1.0\right)$	$N_q = 11.85$
$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$	$N_c = 22.25$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$	$N_\gamma = 12.5$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right)$	$s_c = 1.325$
$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \tan(\phi'_f), 1\right)$	$s_q = 1.298$
$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{Wall}}\right), 1\right)$	$s_\gamma = 0.756$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 29.486$$

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

$$N_{qm} = 15.381$$

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

$$N_{\gamma m} = 9.479$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 14.6 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 9.5 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

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

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.122$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 5.767$$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 14.6 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 9.5 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 9.5 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 9.5 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 9.5 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 95840 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 2.4 \text{ ft} \quad \text{Wall Eccentricity}$$

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L) \quad B' = 15.3 \text{ ft} \quad \text{Effective Bearing Width}$$

Compute the bearing stress:

$$\sigma := \frac{V_1 + V_2}{B'} \quad \sigma = 4.6 \text{ ksf} \quad \text{Nominal Bearing Stress}$$

$$\sigma_v := \frac{\Sigma V}{B'} \quad \sigma_v = 6.3 \text{ ksf} \quad \text{Factored Bearing Stress}$$

Bearing Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} := \frac{q_R}{\sigma_v} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Bearing} = 1.51$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3} \quad e_{max} = 6.7 \text{ ft} \quad \text{Maximum Eccentricity LRFD [C11.6.3.3.]}$$

$$\Sigma M_R := MV_{Ia} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ia)}$$

$$\Sigma M_O := MH_{Ia} \quad \Sigma M_O = 227566.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ia)}$$

$$\Sigma V := V_{Ia} \quad \Sigma V = 68400 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 3.3 \text{ ft}$$

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Eccentricity} = 2.00$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 23213 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 68400 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{\tau d} := \Sigma V \cdot \tan(\phi'_f) \quad R_{\tau d} = 33360.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{\tau d} = 33.361 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{\tau d}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 33.4 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{\text{Sliding}} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

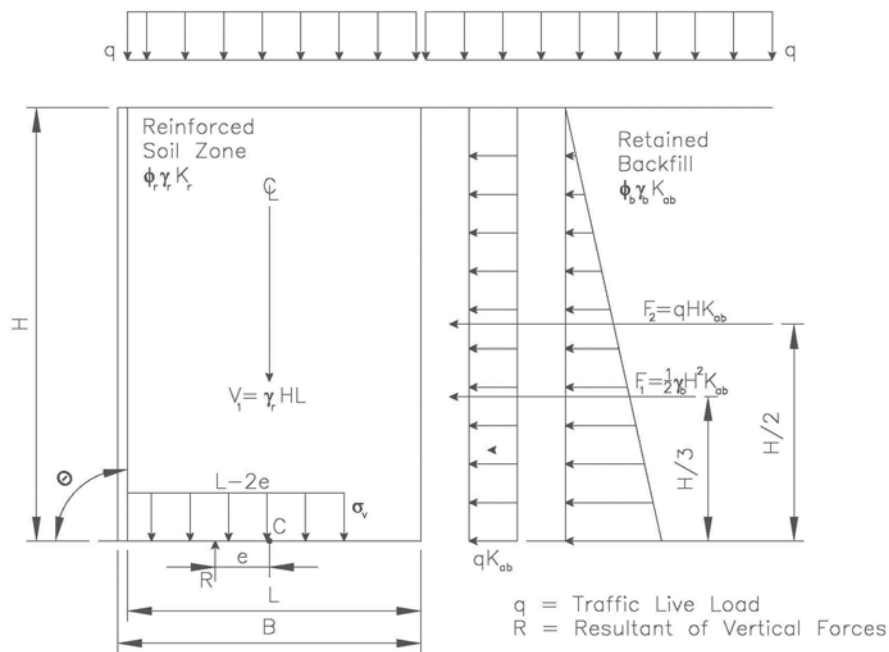
$$CDR_{\text{Sliding}} = 1.44$$

RW-2 – STA. 00+09.00

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.
Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- Horizontal backfill behind MSE wall on granular (drained) soils.
- 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
- Load combinations and wall configuration are as shown below:



Givens:

Wall Geometry:

$$H_e := 6.8 \cdot ft$$

Exposed wall height

$$\theta := 90 \cdot deg$$

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)

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_r := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_b := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 25 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 125 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 185 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 125 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 1850 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use Su if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 7 \text{ ft} \quad d_{user} := 0 \text{ ft}$$

$$Slope_{fw} := 0 \text{ deg}$$

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 7 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 13.8 \text{ ft}$$

Local Frost Depth

Inclination of ground slope in front of wall :

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 0.7 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 7 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 13.8 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 14 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := L_{user}$$

$$L = 14 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb} \cdot \text{f}}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 3396.6 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 410.2 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 23184 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 1400 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 3396.6 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 410.2 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 4.6 \text{ ft}$$

Moment:

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 15624.4 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 6.9 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 2830.5 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 23184 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 1400 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 3396.6 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 410.2 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 15624.4 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 2830.5 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 23184 \frac{\text{lbf}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 33748.4 \frac{\text{lbf}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 5812.8 \frac{\text{lbf}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 5812.8 \frac{\text{lbf}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 28390 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 28390 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 28390 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 33748.4 \frac{\text{lb}}{\text{ft}}$	Sum of Vertical Loads (Strength lb)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 0.8 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 12.3 \text{ ft}$	Effective Bearing Width

Foundation Layout:

$L_{wall} := 9 \cdot \text{ft}$		Assumed Footing Length (Wall Section Length)
$H' := H_{lb}$	$H' = 5812.8 \frac{\text{lb}}{\text{ft}}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 33748.4 \frac{\text{lb}}{\text{ft}}$	Summation of Vertical Loads (Strength lb)
$D_f := d_e$	$D_f = 7 \text{ ft}$	Footing embedment
$d_w := -0.5 \cdot \text{ft}$		Depth of Groundwater below Bearing Grade
$\theta' := 90 \cdot \text{deg}$		Direction of H' and V' resultant measured from wall back face LRFD [Figure C10.6.3.1.2a-1]

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \text{ deg} + \frac{\phi'_f}{2}\right), 1.0\right)$	$N_q = 10.66$
$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$	$N_c = 20.72$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$	$N_\gamma = 10.9$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{wall}}\right)\right)$	$s_c = 1.704$
$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \tan(\phi'_f), 1\right)$	$s_q = 1.638$
$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{wall}}\right), 1\right)$	$s_\gamma = 0.453$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot (1 - \sin(\phi'_f))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.2$$

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

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

$$N_{cm} = 35.313$$

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

$$N_{qm} = 17.467$$

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

$$N_{\gamma m} = 4.922$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 17059.7 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 11.1 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

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

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.274$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 6.547$$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 12606 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 8.2 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 11.1 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 8.2 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 8.2 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ib)

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 28390 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ib)

$$\Sigma V := V_{Ib} \quad \Sigma V = 33748.4 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ib)

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 0.8 \text{ ft}$$

Wall Eccentricity

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L) \quad B' = 12.3 \text{ ft}$$

Effective Bearing Width

Compute the ultimate bearing stress:

$$\sigma := \frac{V_1 + V_2}{B'} \quad \sigma = 2 \text{ ksf}$$

Nominal Bearing Stress

$$\sigma_v := \frac{\Sigma V}{B'} \quad \sigma_v = 2.7 \text{ ksf}$$

Factored Bearing Stress

Bearing Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} := \frac{q_R}{\sigma_v} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0?$$

$$CDR_{Bearing} = 2.99$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3} \quad e_{max} = 4.7 \text{ ft}$$

Maximum Eccentricity LRFD [C11.6.3.3.]

$$\Sigma M_R := MV_{Ia} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia} \quad \Sigma M_O = 28390 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia} \quad \Sigma V = 23184 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 1.2 \text{ ft}$$

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0?$$

$$CDR_{Eccentricity} = 3.81$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 5812.8 \frac{\text{lb}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 23184 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{td} := \Sigma V \cdot \tan(\phi'_f) \quad R_{td} = 10810.9 \frac{\text{lb}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{td} = 10.811 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{td}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 10.8 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding} = 1.86$$

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Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 25 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 125 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 185 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 125 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 1850 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use Su if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 7 \text{ ft} \quad d_{user} := 0 \text{ ft}$$

$$Slope_{fw} := 0 \text{ deg}$$

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 7 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 16.6 \text{ ft}$$

Local Frost Depth

Inclination of ground slope in front of wall :

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 0.8 \text{ ft}$$

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.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 16.6 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 14 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := L_{user}$$

$$L = 14 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb} \cdot \text{f}}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 4914.8 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 493.5 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 27888 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 1400 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 4914.8 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 493.5 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 5.5 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 27195.1 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 8.3 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 4095.6 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Moment:

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 27888 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 1400 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 4914.8 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 493.5 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 27195.1 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 4095.6 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 27888 \frac{\text{lbf}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 40098.8 \frac{\text{lbf}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 8235.7 \frac{\text{lbf}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 8235.7 \frac{\text{lbf}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 47960 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 47960 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$$\Sigma M_R := MV_{lb}$$

$$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength lb)

$$\Sigma M_O := MH_{lb}$$

$$\Sigma M_O = 47960 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength lb)

$$\Sigma V := V_{lb}$$

$$\Sigma V = 40098.8 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength lb)

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$$

$$e_{wall} = 1.2 \text{ ft}$$

Wall Eccentricity

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$$

$$B' = 11.6 \text{ ft}$$

Effective Bearing Width

Foundation Layout:

$$L_{wall} := 9 \cdot \text{ft}$$

Assumed Footing Length (Wall Section Length)

$$H' := H_{lb}$$

$$H' = 8235.7 \frac{\text{lb}}{\text{ft}}$$

Summation of Horizontal Loads (Strength lb)

$$V' := V_{lb}$$

$$V' = 40098.8 \frac{\text{lb}}{\text{ft}}$$

Summation of Vertical Loads (Strength lb)

$$D_f := d_e$$

$$D_f = 7 \text{ ft}$$

Footing embedment

$$d_w := -0.5 \cdot \text{ft}$$

Depth of Groundwater below Bearing Grade

$$\theta' := 90 \cdot \text{deg}$$

Direction of H' and V' resultant measured from wall back face **LRFD [Figure C10.6.3.1.2a-1]**

Drained Conditions (Effective Stress):

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

$$N_q = 10.66$$

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

$$N_c = 20.72$$

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

$$N_\gamma = 10.9$$

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

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

$$s_c = 1.664$$

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

$$s_q = 1.601$$

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

$$s_\gamma = 0.484$$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.2$$

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

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

$$N_{cm} = 34.472$$

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

$$N_{qm} = 17.075$$

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

$$N_{\gamma m} = 5.265$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 16803.2 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 10.9 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

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

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.258$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 6.466$$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 12460.6 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 8.1 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 10.9 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 8.1 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 8.1 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$\Sigma M_R := MV_{Ib}$	$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength Ib)
$\Sigma M_O := MH_{Ib}$	$\Sigma M_O = 47960 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength Ib)
$\Sigma V := V_{Ib}$	$\Sigma V = 40098.8 \frac{\text{lb}}{\text{ft}}$	Sum of Vertical Loads (Strength Ib)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 1.2 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 11.6 \text{ ft}$	Effective Bearing Width

Compute the ultimate bearing stress:

$\sigma := \frac{V_1 + V_2}{B'}$	$\sigma = 2.5 \text{ ksf}$	Nominal Bearing Stress
$\sigma_v := \frac{\Sigma V}{B'}$	$\sigma_v = 3.5 \text{ ksf}$	Factored Bearing Stress

Bearing Capacity:Demand Ratio (CDR)

$CDR_{Bearing} := \frac{q_R}{\sigma_v}$	Is the CDR > or = to 1.0?	$CDR_{Bearing} = 2.34$
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Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$e_{max} := \frac{L}{3}$	$e_{max} = 4.7 \text{ ft}$	Maximum Eccentricity LRFD [C11.6.3.3.]
$\Sigma M_R := MV_{Ia}$	$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength Ia)
$\Sigma M_O := MH_{Ia}$	$\Sigma M_O = 47960 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength Ia)
$\Sigma V := V_{Ia}$	$\Sigma V = 27888 \frac{\text{lb}}{\text{ft}}$	Sum of Vertical Loads (Strength Ia)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 1.7 \text{ ft}$	

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}}$	Is the CDR > or = to 1.0?	$CDR_{Eccentricity} = 2.71$
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Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 8235.7 \frac{\text{lb}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 27888 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{td} := \Sigma V \cdot \tan(\phi'_f) \quad R_{td} = 13004.4 \frac{\text{lb}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{td} = 13.004 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{td}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 13 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

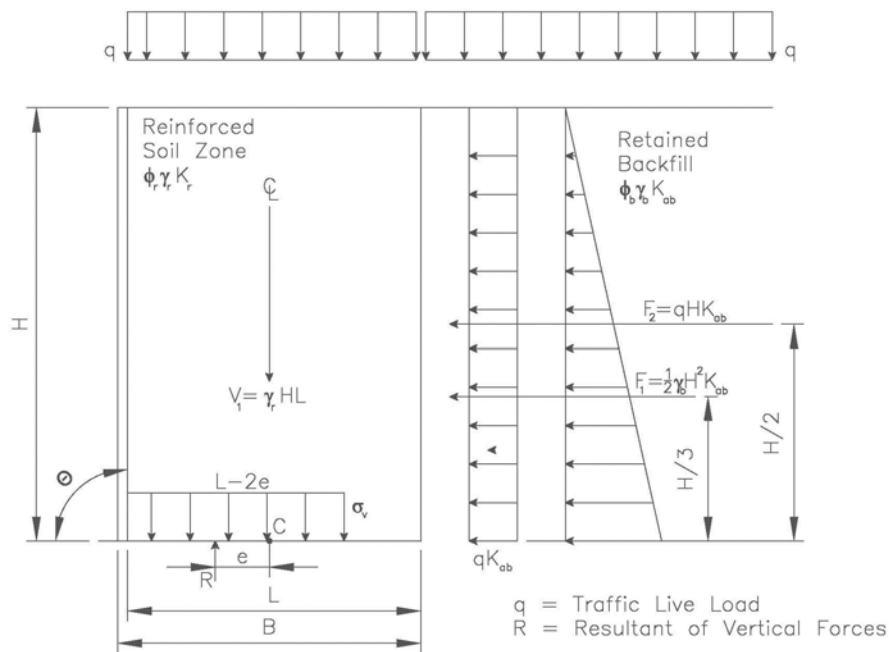
$$CDR_{Sliding} = 1.58$$

RW-2 – STA. 00+27.00

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.
Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- Horizontal backfill behind MSE wall on granular (drained) soils.
- 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
- Load combinations and wall configuration are as shown below:



Givens:

Wall Geometry:

$$H_e := 13 \cdot ft$$

Exposed wall height

$$\theta := 90 \cdot deg$$

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)

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_r := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_b := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 25 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 125 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 185 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 125 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 1850 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use S_u if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 6.3 \text{ ft} \quad d_{user} := 0 \text{ ft}$$

$$Slope_{fw} := 0 \text{ deg}$$

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 6.3 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 19.3 \text{ ft}$$

Local Frost Depth

Inclination of ground slope in front of wall :

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 6.3 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 19.3 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 14 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := L_{user}$$

$$L = 14 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb} \cdot \text{f}}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 6643.6 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 573.7 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 32424 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 1400 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 6643.6 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 573.7 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 6.4 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 42740.4 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 9.7 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 5536.3 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Moment:

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 32424 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 1400 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 6643.6 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 573.7 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 42740.4 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 5536.3 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 32424 \frac{\text{lbf}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 46222.4 \frac{\text{lbf}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 10969.4 \frac{\text{lbf}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 10969.4 \frac{\text{lbf}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 73799.1 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 73799.1 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 0 \frac{lb \cdot ft}{ft}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 73799.1 \frac{lb \cdot ft}{ft}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 46222.4 \frac{lb \cdot ft}{ft}$	Sum of Vertical Loads (Strength lb)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 1.6 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 10.8 \text{ ft}$	Effective Bearing Width

Foundation Layout:

$L_{wall} := 9 \cdot \text{ft}$		Assumed Footing Length (Wall Section Length)
$H' := H_{lb}$	$H' = 10969.4 \frac{lb \cdot ft}{ft}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 46222.4 \frac{lb \cdot ft}{ft}$	Summation of Vertical Loads (Strength lb)
$D_f := d_e$	$D_f = 6.3 \text{ ft}$	Footing embedment
$d_w := -0.5 \cdot \text{ft}$		Depth of Groundwater below Bearing Grade
$\theta' := 90 \cdot \text{deg}$		Direction of H' and V' resultant measured from wall back face LRFD [Figure C10.6.3.1.2a-1]

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \text{ deg} + \frac{\phi'_f}{2}\right)^2, 1.0\right)$	$N_q = 10.66$
$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$	$N_c = 20.72$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$	$N_\gamma = 10.9$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{wall}}\right)\right)$	$s_c = 1.618$
$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \tan(\phi'_f), 1\right)$	$s_q = 1.56$
$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{wall}}\right), 1\right)$	$s_\gamma = 0.52$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.2$$

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

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

$$N_{cm} = 33.523$$

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

$$N_{qm} = 16.632$$

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

$$N_{\gamma m} = 5.652$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 15537.1 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 10.1 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

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

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.24$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 6.374$$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 12239.1 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 8 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 10.1 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 8 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 8 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$\Sigma M_R := MV_{Ib}$	$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength Ib)
$\Sigma M_O := MH_{Ib}$	$\Sigma M_O = 73799.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength Ib)
$\Sigma V := V_{Ib}$	$\Sigma V = 46222.4 \frac{\text{lb}}{\text{ft}}$	Sum of Vertical Loads (Strength Ib)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 1.6 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 10.8 \text{ ft}$	Effective Bearing Width

Compute the ultimate bearing stress:

$\sigma := \frac{V_1 + V_2}{B'}$	$\sigma = 3.1 \text{ ksf}$	Nominal Bearing Stress
$\sigma_v := \frac{\Sigma V}{B'}$	$\sigma_v = 4.3 \text{ ksf}$	Factored Bearing Stress

Bearing Capacity:Demand Ratio (CDR)

$CDR_{Bearing} := \frac{q_R}{\sigma_v}$	Is the CDR > or = to 1.0?	$CDR_{Bearing} = 1.86$
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Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$e_{max} := \frac{L}{3}$	$e_{max} = 4.7 \text{ ft}$	Maximum Eccentricity LRFD [C11.6.3.3.]
$\Sigma M_R := MV_{Ia}$	$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength Ia)
$\Sigma M_O := MH_{Ia}$	$\Sigma M_O = 73799.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength Ia)
$\Sigma V := V_{Ia}$	$\Sigma V = 32424 \frac{\text{lb}}{\text{ft}}$	Sum of Vertical Loads (Strength Ia)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 2.3 \text{ ft}$	

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}}$	Is the CDR > or = to 1.0?	$CDR_{Eccentricity} = 2.05$
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Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 10969.4 \frac{\text{lb}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 32424 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{td} := \Sigma V \cdot \tan(\phi'_f) \quad R_{td} = 15119.6 \frac{\text{lb}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{td} = 15.12 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{td}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 15.1 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding} = 1.38$$

RW-2 – STA. 00+36.00

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 25 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 125 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 185 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 125 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 1850 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use Su if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 6 \text{ ft} \quad d_{user} := 0 \text{ ft}$$

$$Slope_{fw} := 0 \text{ deg}$$

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 6 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 22.1 \text{ ft}$$

Local Frost Depth

Inclination of ground slope in front of wall :

- Horizontal: **0**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$$d_{eSlope} := \text{if} \left(Slope_{fw} < 1 \text{ deg}, \frac{H_{est}}{20}, \text{if} \left(Slope_{fw} < 26.565 \text{ deg}, \frac{H_{est}}{10}, \text{if} \left(Slope_{fw} < 33.69 \text{ deg}, \frac{H_{est}}{7}, \frac{H_{est}}{5} \right) \right) \right)$$

$$d_{eSlope} = 1.1 \text{ ft}$$

Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 6 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 22.1 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 18 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := L_{user}$$

$$L = 18 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb} \cdot \text{f}}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 8711.1 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 656.9 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 47736 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 1800 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 8711.1 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 656.9 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 7.4 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 64171.7 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 11.1 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 7259.2 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Moment:

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 47736 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 1800 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 8711.1 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 656.9 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 64171.7 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 7259.2 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 47736 \frac{\text{lbf}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 67593.6 \frac{\text{lbf}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 14216.3 \frac{\text{lbf}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 14216.3 \frac{\text{lbf}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 108961.2 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 108961.2 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 108961.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 67593.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Vertical Loads (Strength lb)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 1.6 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 14.8 \text{ ft}$	Effective Bearing Width

Foundation Layout:

$L_{wall} := 9 \cdot \text{ft}$		Assumed Footing Length (Wall Section Length)
$H' := H_{lb}$	$H' = 14216.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 67593.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Summation of Vertical Loads (Strength lb)
$D_f := d_e$	$D_f = 6 \text{ ft}$	Footing embedment
$d_w := -0.5 \cdot \text{ft}$		Depth of Groundwater below Bearing Grade
$\theta' := 90 \cdot \text{deg}$		Direction of H' and V' resultant measured from wall back face LRFD [Figure C10.6.3.1.2a-1]

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \text{ deg} + \frac{\phi'_f}{2}\right), 1.0\right)$	$N_q = 10.66$
$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$	$N_c = 20.72$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$	$N_\gamma = 10.9$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{wall}}\right)\right)$	$s_c = 1.845$
$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \tan(\phi'_f), 1\right)$	$s_q = 1.766$
$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{wall}}\right), 1\right)$	$s_\gamma = 0.343$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 38.225$$

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

$$N_{qm} = 18.825$$

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

$$N_{\gamma m} = 3.734$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 16428.2 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 10.7 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

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

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.328$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 6.828$$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 13036.8 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 8.5 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 10.7 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 8.5 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 8.5 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 108961.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 67593.6 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 1.6 \text{ ft} \quad \text{Wall Eccentricity}$$

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L) \quad B' = 14.8 \text{ ft} \quad \text{Effective Bearing Width}$$

Compute the ultimate bearing stress:

$$\sigma := \frac{V_1 + V_2}{B'} \quad \sigma = 3.4 \text{ ksf} \quad \text{Nominal Bearing Stress}$$

$$\sigma_v := \frac{\Sigma V}{B'} \quad \sigma_v = 4.6 \text{ ksf} \quad \text{Factored Bearing Stress}$$

Bearing Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} := \frac{q_R}{\sigma_v} \quad \text{Is the CDR > or = to 1.0?} \quad CDR_{Bearing} = 1.85$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3} \quad e_{max} = 6 \text{ ft} \quad \text{Maximum Eccentricity LRFD [C11.6.3.3.]}$$

$$\Sigma M_R := MV_{Ia} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ia)}$$

$$\Sigma M_O := MH_{Ia} \quad \Sigma M_O = 108961.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ia)}$$

$$\Sigma V := V_{Ia} \quad \Sigma V = 47736 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 2.3 \text{ ft}$$

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}} \quad \text{Is the CDR > or = to 1.0?} \quad CDR_{Eccentricity} = 2.63$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 14216.3 \frac{\text{lb}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 47736 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{td} := \Sigma V \cdot \tan(\phi'_f) \quad R_{td} = 22259.7 \frac{\text{lb}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{td} = 22.26 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{td}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 22.3 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

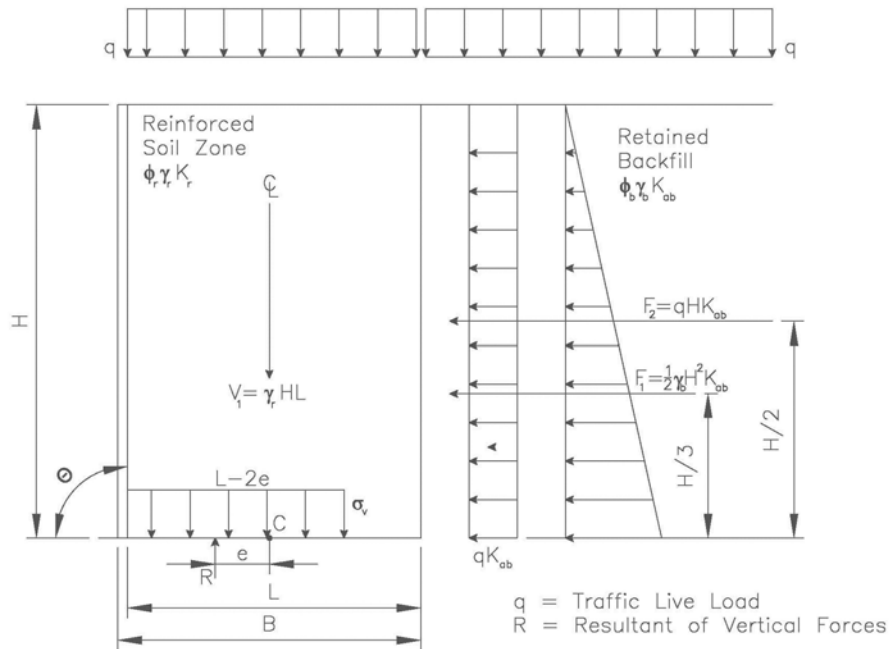
$$CDR_{Sliding} = 1.57$$

RW-2 – STA. 00+45.00

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.
Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- Horizontal backfill behind MSE wall on granular (drained) soils.
- 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
- Load combinations and wall configuration are as shown below:



Givens:

Wall Geometry:

$$H_e := 20 \cdot ft$$

Exposed wall height

$$\theta := 90 \cdot deg$$

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = $80 \text{ deg} < \theta < 100 \text{ deg}$)

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_r := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_b := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 25 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 125 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 185 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 125 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 1850 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use S_u if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 4.9 \text{ ft}$$

$$d_{user} := 0 \text{ ft}$$

Local Frost Depth

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 4.9 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 24.9 \text{ ft}$$

- 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

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 4.9 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 24.9 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 18 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := L_{user}$$

$$L = 18 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb} \cdot \text{f}}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right) \quad k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 11058.3 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 740.2 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 53784 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 1800 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 11058.3 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 740.2 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 8.3 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 91783.5 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 12.5 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 9215.2 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Moment:

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 53784 \frac{\text{lb}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 1800 \frac{\text{lb}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 11058.3 \frac{\text{lb}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 740.2 \frac{\text{lb}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 91783.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 9215.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 53784 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 75758.4 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 17882.7 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 17882.7 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 153801.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 153801.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 0 \frac{lb \cdot ft}{ft}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 153801.8 \frac{lb \cdot ft}{ft}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 75758.4 \frac{lb \cdot ft}{ft}$	Sum of Vertical Loads (Strength lb)
$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$	$e_{wall} = 2 \text{ ft}$	Wall Eccentricity
$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$	$B' = 13.9 \text{ ft}$	Effective Bearing Width

Foundation Layout:

$L_{wall} := 9 \cdot \text{ft}$		Assumed Footing Length (Wall Section Length)
$H' := H_{lb}$	$H' = 17882.7 \frac{lb \cdot ft}{ft}$	Summation of Horizontal Loads (Strength lb)
$V' := V_{lb}$	$V' = 75758.4 \frac{lb \cdot ft}{ft}$	Summation of Vertical Loads (Strength lb)
$D_f := d_e$	$D_f = 4.9 \text{ ft}$	Footing embedment
$d_w := -0.5 \cdot \text{ft}$		Depth of Groundwater below Bearing Grade
$\theta' := 90 \cdot \text{deg}$		Direction of H' and V' resultant measured from wall back face LRFD [Figure C10.6.3.1.2a-1]

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \text{ deg} + \frac{\phi'_f}{2}\right)^2, 1.0\right)$	$N_q = 10.66$
$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$	$N_c = 20.72$
$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$	$N_\gamma = 10.9$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{wall}}\right)\right)$	$s_c = 1.797$
$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{wall}}\right) \cdot \tan(\phi'_f), 1\right)$	$s_q = 1.722$
$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{wall}}\right), 1\right)$	$s_\gamma = 0.38$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 37.235$$

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

$$N_{qm} = 18.363$$

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

$$N_{\gamma m} = 4.138$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 14679.6 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 9.5 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 1$$

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

$$N_c = 5.14$$

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

$$N_\gamma = 0$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.31$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 6.732$$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 12780.7 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 8.3 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 9.5 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 8.3 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 8.3 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 153801.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 75758.4 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 2 \text{ ft} \quad \text{Wall Eccentricity}$$

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L) \quad B' = 13.9 \text{ ft} \quad \text{Effective Bearing Width}$$

Compute the ultimate bearing stress:

$$\sigma := \frac{V_1 + V_2}{B'} \quad \sigma = 4 \text{ ksf} \quad \text{Nominal Bearing Stress}$$

$$\sigma_v := \frac{\Sigma V}{B'} \quad \sigma_v = 5.4 \text{ ksf} \quad \text{Factored Bearing Stress}$$

Bearing Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} := \frac{q_R}{\sigma_v} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0? \quad CDR_{Bearing} = 1.53$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3} \quad e_{max} = 6 \text{ ft} \quad \text{Maximum Eccentricity LRFD [C11.6.3.3.]}$$

$$\Sigma M_R := MV_{Ia} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ia)}$$

$$\Sigma M_O := MH_{Ia} \quad \Sigma M_O = 153801.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ia)}$$

$$\Sigma V := V_{Ia} \quad \Sigma V = 53784 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 2.9 \text{ ft}$$

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}} \quad \text{Is the CDR} > \text{ or } = \text{ to } 1.0? \quad CDR_{Eccentricity} = 2.10$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 17882.7 \frac{\text{lb}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 53784 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{td} := \Sigma V \cdot \tan(\phi'_f) \quad R_{td} = 25079.9 \frac{\text{lb}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{td} = 25.08 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{td}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 25.1 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{F_{\tau}}$$

Is the CDR > or = to 1.0?

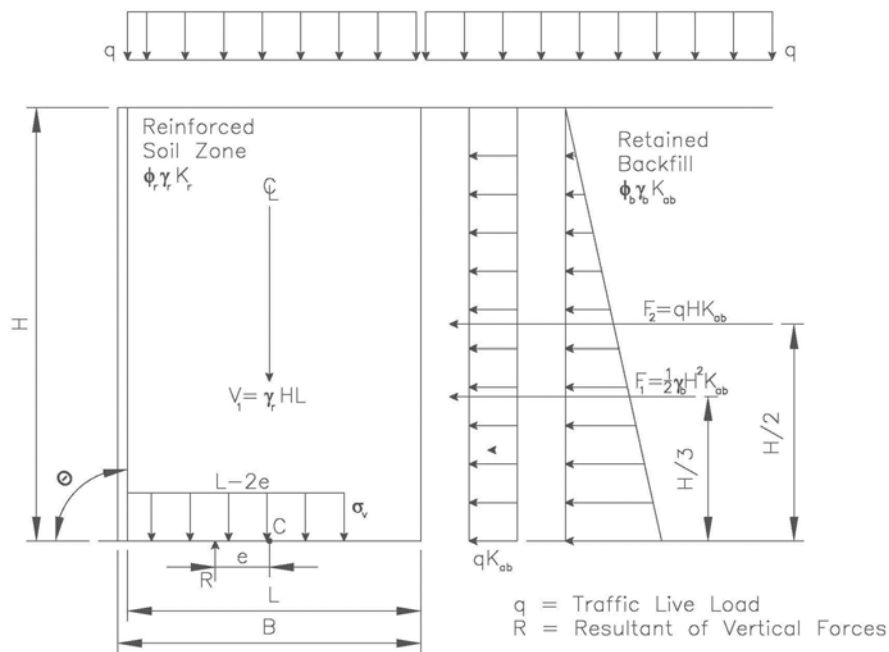
$$CDR_{Sliding} = 1.40$$

RW-2 – STA. 00+68.28

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.
Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- Horizontal backfill behind MSE wall on granular (drained) soils.
- 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
- Load combinations and wall configuration are as shown below:



Givens:

Wall Geometry:

$$H_e := 24.5 \cdot ft$$

Exposed wall height

$$\theta := 90 \cdot deg$$

Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = $80 \text{ deg} < \theta < 100 \text{ deg}$)

Reinforced Backfill Soil Design Parameters:

$$\phi'_r := 34 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_r := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Retained Backfill Soil Design Parameters:

$$\phi'_b := 30 \cdot deg$$

Effective angle of internal friction (Per BDM [Table 307-1])

$$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$$

Unit weight (Per BDM [Table 307-1])

$$c'_b := 0 \cdot \frac{lbf}{ft^2}$$

Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 34 \cdot \text{deg}$$

Effective angle of internal friction

$$\gamma_f := 125 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 0 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 34 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 125 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 0 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use S_u if Angle of internal friction = 0 deg)

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:

$$d_{frost} := 3.5 \text{ ft}$$

$$d_{user} := 0 \text{ ft}$$

Local Frost Depth

$$Slope_{fw} := 0 \text{ deg}$$

Inclination of ground slope in front of wall :

$$d_{est} := \max(d_{frost}, 3 \text{ ft}, d_{user})$$

$$d_{est} = 3.5 \text{ ft}$$

$$H_{est} := d_{est} + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H_{est} = 28 \text{ ft}$$

- 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

$$d_e := \max(d_{est}, d_{eSlope})$$

$$d_e = 3.5 \text{ ft}$$

Minimum Required Embedment Depth used in analysis.

$$H := d_e + (4 \text{ ft} \cdot \tan(Slope_{fw})) + H_e$$

$$H = 28 \text{ ft}$$

Design Wall Height

Estimate Length of Reinforcement:

$$L_{user} := 20 \cdot \text{ft}$$

User inputted value (if changes need to be made to satisfy other requirements)

$$L := \max(8 \cdot \text{ft}, L_{user})$$

$$L = 20 \text{ ft}$$

Length of Reinforcement

Live Load Surcharge Parameters:

$$SUR := 100 \cdot \frac{\text{lb} \cdot \text{f}}{\text{ft}^2}$$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)

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 traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$$\beta := 0 \text{ deg}$$

$$\delta := 0.67 \cdot \phi'_b$$

Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

$$\Gamma := \left(1 + \sqrt{\frac{(\sin(\phi'_b + \delta) \cdot \sin(\phi'_b - \beta))}{(\sin(\theta - \delta) \cdot \sin(\theta + \beta))}} \right)^2$$

$$k_{af} := \left(\frac{(\sin(\theta + \phi'_b))^2}{(\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta))} \right)$$

$$k_{af} = 0.2973$$

Active Earth Pressure Coefficient

$$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$$

$$F_T = 13983.1 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (EH)

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

$$F_{SUR} = 832.3 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge (LS)

Vertical Loads:

$$V_1 := \gamma_r \cdot H \cdot L$$

$$V_1 = 67200 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Soil backfill - reinforced soil (EV)

$$V_2 := SUR \cdot L$$

$$V_2 = 2000 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge - (LS)

Moment Arm:

$$d_{v1} := 0 \cdot \text{ft}$$

$$d_{v1} = 0 \text{ ft}$$

$$MV_1 := V_1 \cdot d_{v1}$$

$$MV_1 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{v2} := 0 \text{ ft}$$

$$d_{v2} = 0 \text{ ft}$$

$$MV_2 := V_2 \cdot d_{v2}$$

$$MV_2 = 0 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Horizontal Loads:

$$H_1 := F_T = 13983.1 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Active Earth Force Resultant (horizontal comp. - EH)

$$H_2 := F_{SUR} = 832.3 \frac{\text{lb} \cdot \text{f}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm:

$$d_{h1} := \frac{H}{3}$$

$$d_{h1} = 9.3 \text{ ft}$$

$$MH_1 := H_1 \cdot d_{h1}$$

$$MH_1 = 130509 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

$$d_{h2} := \frac{H}{2}$$

$$d_{h2} = 14 \text{ ft}$$

$$MH_2 := H_2 \cdot d_{h2}$$

$$MH_2 = 11652.6 \frac{\text{lb} \cdot \text{f} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type

$$V_{EV} := V_1$$

$$V_{EV} = 67200 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS} := V_2$$

$$V_{LS} = 2000 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_1$$

$$H_{EH} = 13983.1 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_2$$

$$H_{LS} = 832.3 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{EV} := MV_1$$

$$M_{EV} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS} := MV_2$$

$$M_{LS} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_1$$

$$M_{EH2} = 130509 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$M_{LS2} := MH_2$$

$$M_{LS2} = 11652.6 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot (Ia_{EV} \cdot V_{EV})$$

$$V_{Ia} = 67200 \frac{\text{lbf}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{EV} \cdot V_{EV}) + (Ib_{LS} \cdot V_{LS}))$$

$$V_{Ib} = 94220 \frac{\text{lbf}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}))$$

$$H_{Ia} = 22431.2 \frac{\text{lbf}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}))$$

$$H_{Ib} = 22431.2 \frac{\text{lbf}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot (Ia_{EV} \cdot M_{EV})$$

$$MV_{Ia} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{EV} \cdot M_{EV}) + (Ib_{LS} \cdot M_{LS}))$$

$$MV_{Ib} = 0 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LS2}) + (Ia_{EH} \cdot M_{EH2}))$$

$$MH_{Ia} = 216155.6 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LS2}) + (Ib_{EH} \cdot M_{EH2}))$$

$$MH_{Ib} = 216155.6 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength lb):

$$\Sigma M_R := MV_{lb}$$

$$\Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength lb)

$$\Sigma M_O := MH_{lb}$$

$$\Sigma M_O = 216155.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength lb)

$$\Sigma V := V_{lb}$$

$$\Sigma V = 94220 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength lb)

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$$

$$e_{wall} = 2.3 \text{ ft}$$

Wall Eccentricity

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$$

$$B' = 15.4 \text{ ft}$$

Effective Bearing Width

Foundation Layout:

$$L_{wall} := 23.3 \cdot \text{ft}$$

Assumed Footing Length (Wall Section Length)

$$H' := H_{lb}$$

$$H' = 22431.2 \frac{\text{lb}}{\text{ft}}$$

Summation of Horizontal Loads (Strength lb)

$$V' := V_{lb}$$

$$V' = 94220 \frac{\text{lb}}{\text{ft}}$$

Summation of Vertical Loads (Strength lb)

$$D_f := d_e$$

$$D_f = 3.5 \text{ ft}$$

Footing embedment

$$d_w := -0.5 \cdot \text{ft}$$

Depth of Groundwater below Bearing Grade

$$\theta' := 90 \cdot \text{deg}$$

Direction of H' and V' resultant measured from wall back face **LRFD [Figure C10.6.3.1.2a-1]**

Drained Conditions (Effective Stress):

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

$$N_q = 29.44$$

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

$$N_c = 42.16$$

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

$$N_\gamma = 41.1$$

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

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

$$s_c = 1.462$$

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

$$s_q = 1.446$$

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

$$s_\gamma = 0.735$$

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]:

$$C_{wq} := \text{if}(d_w \geq 0, 1, 0.5)$$

$$C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w > 1.5 \cdot B', 1, 0.5)$$

$$C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B'} \leq 1, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \frac{D_f}{B'}, 1 + 2 \cdot \tan(\phi'_f) \cdot \left(1 - \sin(\phi'_f)\right)^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)\right)$$

$$d_q = 1.1$$

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

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

$$N_{cm} = 61.637$$

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

$$N_{qm} = 42.574$$

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

$$N_{\gamma m} = 30.199$$

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

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{nd} = 24 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 15.6 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$$N_q = 29.44$$

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

$$N_c = 42.16$$

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

$$N_\gamma = 41.1$$

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}} \right) \right) \quad s_c = 1.462$$

$$s_q := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \cdot \tan(\phi_f) \right), 1 \right) \quad s_q = 1.446$$

$$s_\gamma := \text{if} \left(\phi_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{Wall}} \right), 1 \right) \quad s_\gamma = 0.735$$

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 61.637$$

$$N_{qm} := N_q \cdot s_q \cdot i_q \quad N_{qm} = 42.574$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 30.199$$

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 24 \text{ ksf}$$

Compute factored bearing resistance. LRFD [Eq 10.6.3.1.1]:

$$\phi_b := 0.65$$

Bearing resistance factor LRFD Table 11.5.7-1.

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 15.6 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 15.6 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 15.6 \text{ ksf}$$

Factored Bearing Resistance to be used in CDR Calculations:

$$q_R := q_{Ru}$$

$$q_R = 15.6 \text{ ksf}$$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

Compute the resultant location (distance from Point 'O'):

$$\Sigma M_R := MV_{Ib} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ib)}$$

$$\Sigma M_O := MH_{Ib} \quad \Sigma M_O = 216155.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ib)}$$

$$\Sigma V := V_{Ib} \quad \Sigma V = 94220 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ib)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 2.3 \text{ ft} \quad \text{Wall Eccentricity}$$

$$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L) \quad B' = 15.4 \text{ ft} \quad \text{Effective Bearing Width}$$

Compute the bearing stress:

$$\sigma := \frac{V_1 + V_2}{B'} \quad \sigma = 4.5 \text{ ksf} \quad \text{Nominal Bearing Stress}$$

$$\sigma_v := \frac{\Sigma V}{B'} \quad \sigma_v = 6.1 \text{ ksf} \quad \text{Factored Bearing Stress}$$

Bearing Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} := \frac{q_R}{\sigma_v} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Bearing} = 2.55$$

Limiting Eccentricity at Base of MSE Wall (Strength Ia):

$$e_{max} := \frac{L}{3} \quad e_{max} = 6.7 \text{ ft} \quad \text{Maximum Eccentricity LRFD [C11.6.3.3.]}$$

$$\Sigma M_R := MV_{Ia} \quad \Sigma M_R = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Resisting Moments (Strength Ia)}$$

$$\Sigma M_O := MH_{Ia} \quad \Sigma M_O = 216155.6 \frac{\text{lb} \cdot \text{ft}}{\text{ft}} \quad \text{Sum of Overturning Moments (Strength Ia)}$$

$$\Sigma V := V_{Ia} \quad \Sigma V = 67200 \frac{\text{lb}}{\text{ft}} \quad \text{Sum of Vertical Loads (Strength Ia)}$$

$$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V} \quad e_{wall} = 3.2 \text{ ft}$$

Eccentricity Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}} \quad \text{Is the CDR > or = to 1.0?}$$

$$CDR_{Eccentricity} = 2.07$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$F_{\tau} := H_{Ia} \quad F_{\tau} = 22431.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$$\Sigma V := V_{Ia} \quad \Sigma V = 67200 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{td} := \Sigma V \cdot \tan(\phi'_f) \quad R_{td} = 45327 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

$$\text{Drained Conditions: } R_{td} = 45.327 \frac{\text{kip}}{\text{ft}}$$

Nominal Sliding Resistance to be used in CDR Calculations: $R_{\tau} := R_{td}$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau}$$

$$R_R := \phi R_n$$

$$R_R = 45.3 \frac{\text{kip}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{F_{\tau}}$$

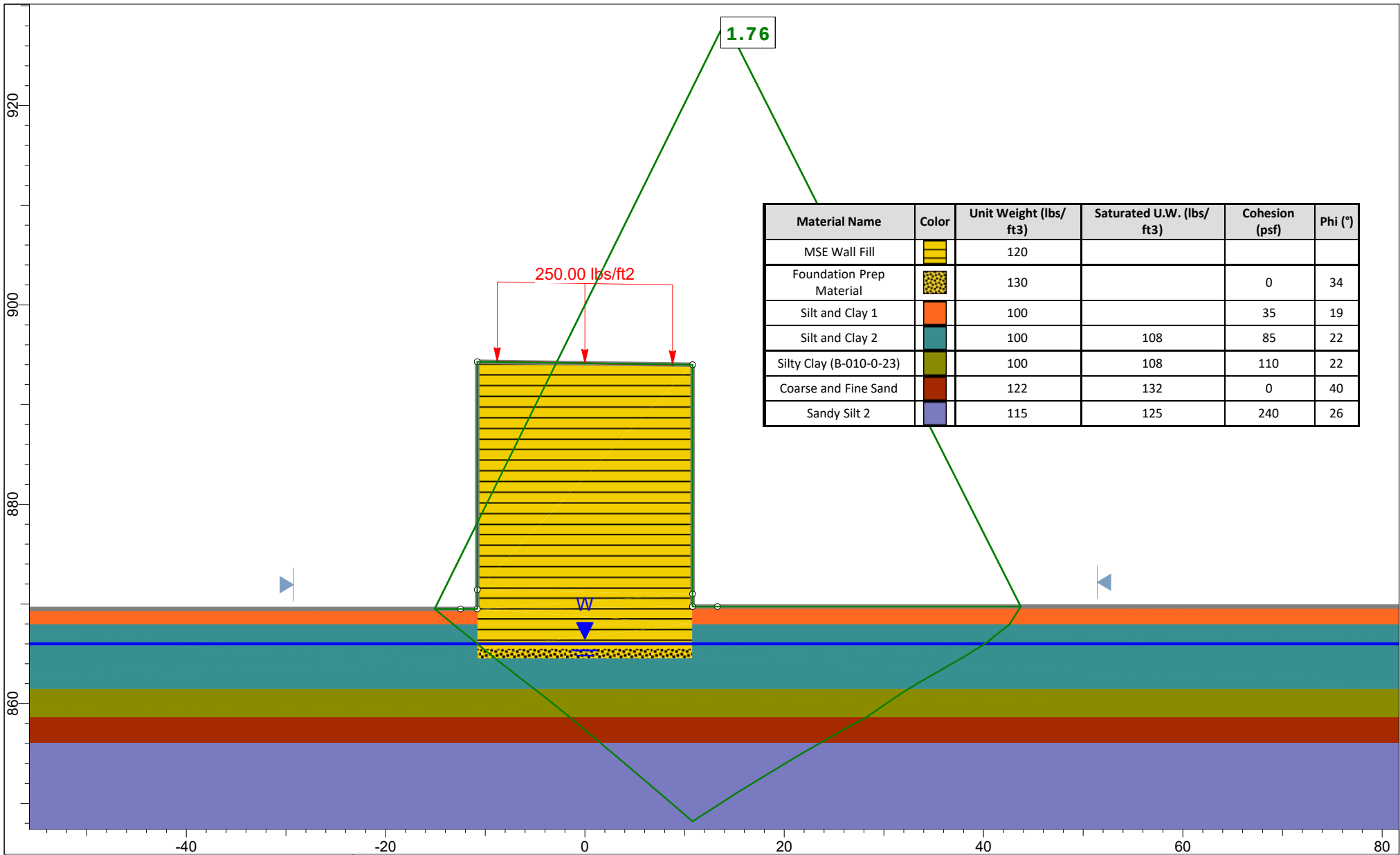
Is the CDR > or = to 1.0?

$$CDR_{Sliding} = 2.02$$

APPENDIX 3C

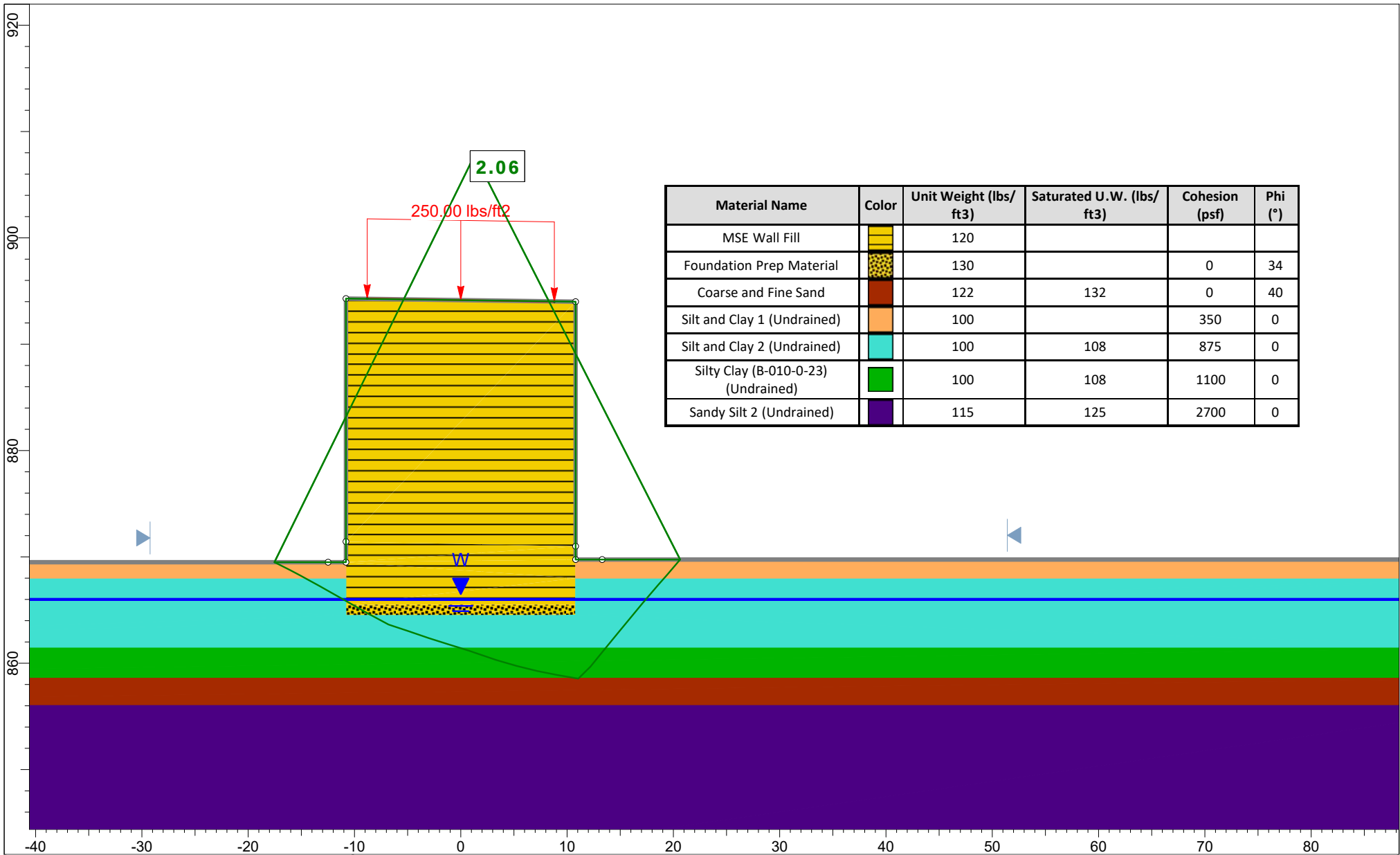
GLOBAL STABILITY ANALYSIS

RW-1 – STA. 00+61.3

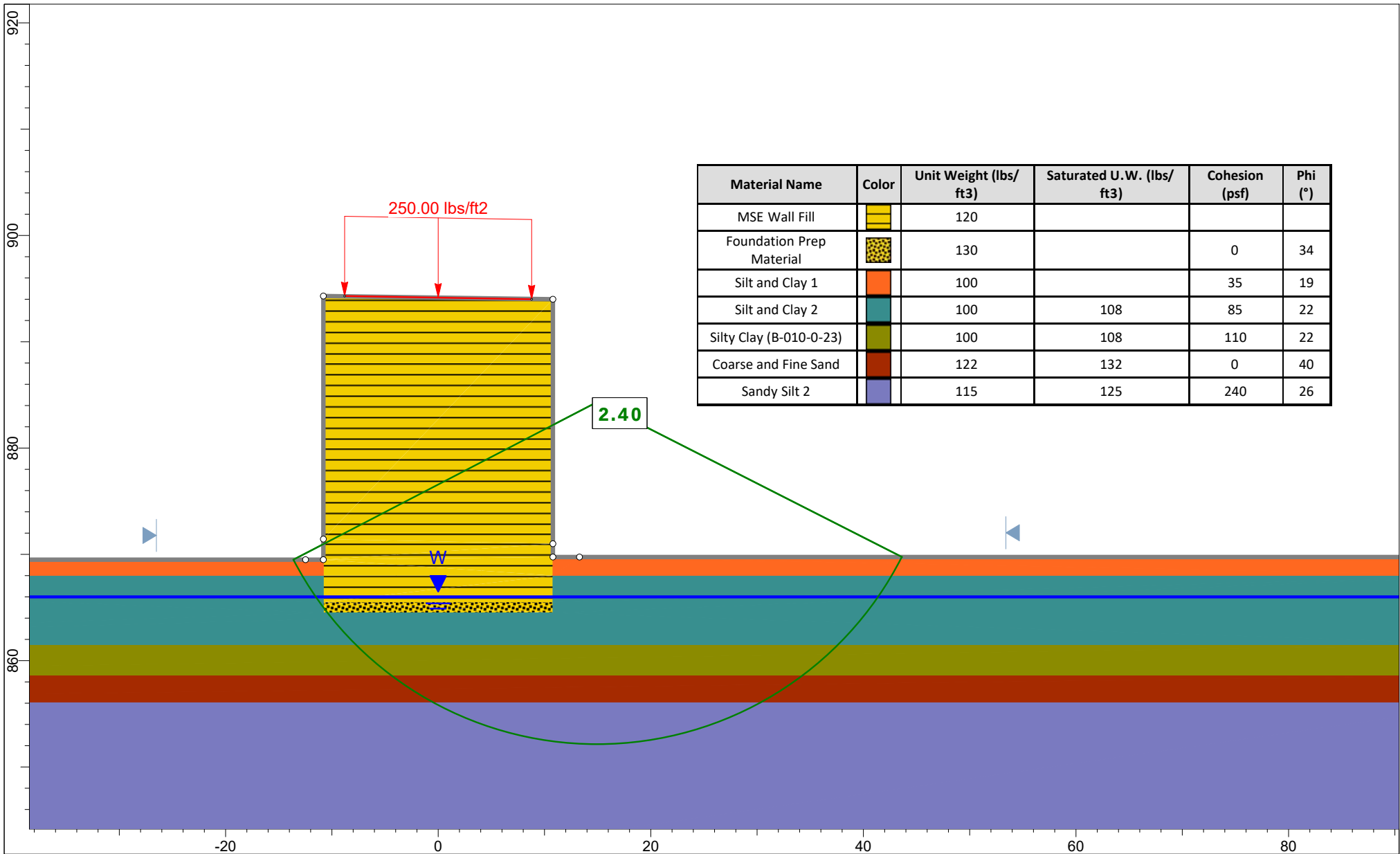


Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Cohesion (psf)	Phi (°)
MSE Wall Fill		120			
Foundation Prep Material		130		0	34
Silt and Clay 1		100		35	19
Silt and Clay 2		100	108	85	22
Silty Clay (B-010-0-23)		100	108	110	22
Coarse and Fine Sand		122	132	0	40
Sandy Silt 2		115	125	240	26

	Project		FRA-Cemetery Rd SUP, PID 120103	
	Group	Effective Stress - Non-circular Failure	Scenario	Master Scenario
	Drawn By	KCA	Company	NEAS Inc.
	Date	1/16/2026, 4:43:08 PM	File Name	RW1_STA00+61.3_061726.slmd
	SLIDEINTERPRET 9.038			



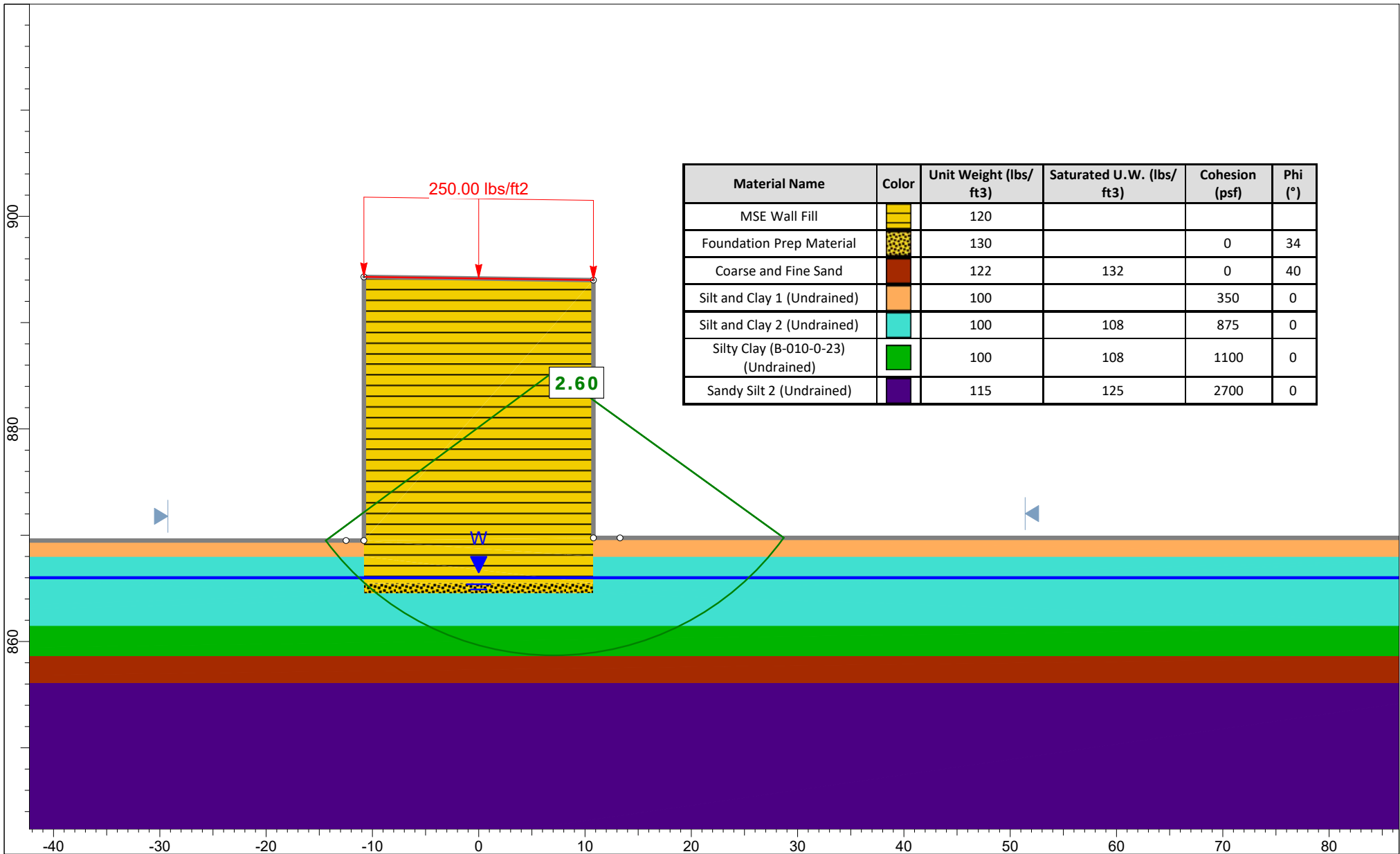
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	Group	Total Stress - Non-circular Failure	Scenario	Master Scenario
	Drawn By	KCA	Company	NEAS Inc.
	Date	1/16/2026, 4:43:08 PM	File Name	RW1_STA00+61.3_061726.slmd
	SLIDEINTERPRET 9.038			



Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Cohesion (psf)	Phi (°)
MSE Wall Fill		120			
Foundation Prep Material		130		0	34
Silt and Clay 1		100		35	19
Silt and Clay 2		100	108	85	22
Silty Clay (B-010-0-23)		100	108	110	22
Coarse and Fine Sand		122	132	0	40
Sandy Silt 2		115	125	240	26



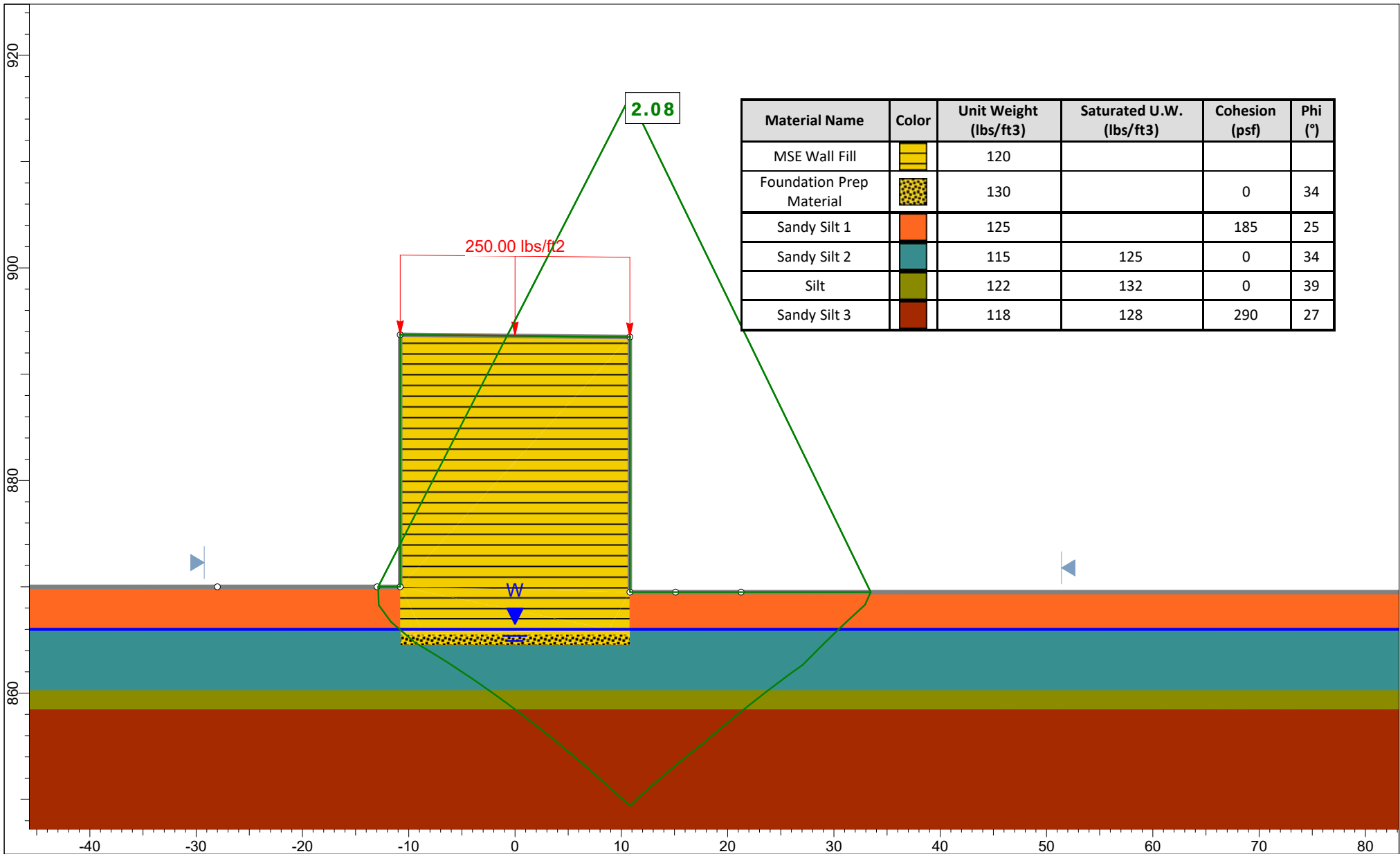
Project	FRA-Cemetery Rd SUP, PID 120103		
Group	Effective Stress - Circular Failure	Scenario	Master Scenario
Drawn By	KCA	Company	NEAS Inc.
Date	1/16/2026, 4:43:08 PM	File Name	RW1_STA00+61.3_061726.slmd



Material Name	Color	Unit Weight (lbs/ft³)	Saturated U.W. (lbs/ft³)	Cohesion (psf)	Phi (°)
MSE Wall Fill		120			
Foundation Prep Material		130		0	34
Coarse and Fine Sand		122	132	0	40
Silt and Clay 1 (Undrained)		100		350	0
Silt and Clay 2 (Undrained)		100	108	875	0
Silty Clay (B-010-0-23) (Undrained)		100	108	1100	0
Sandy Silt 2 (Undrained)		115	125	2700	0

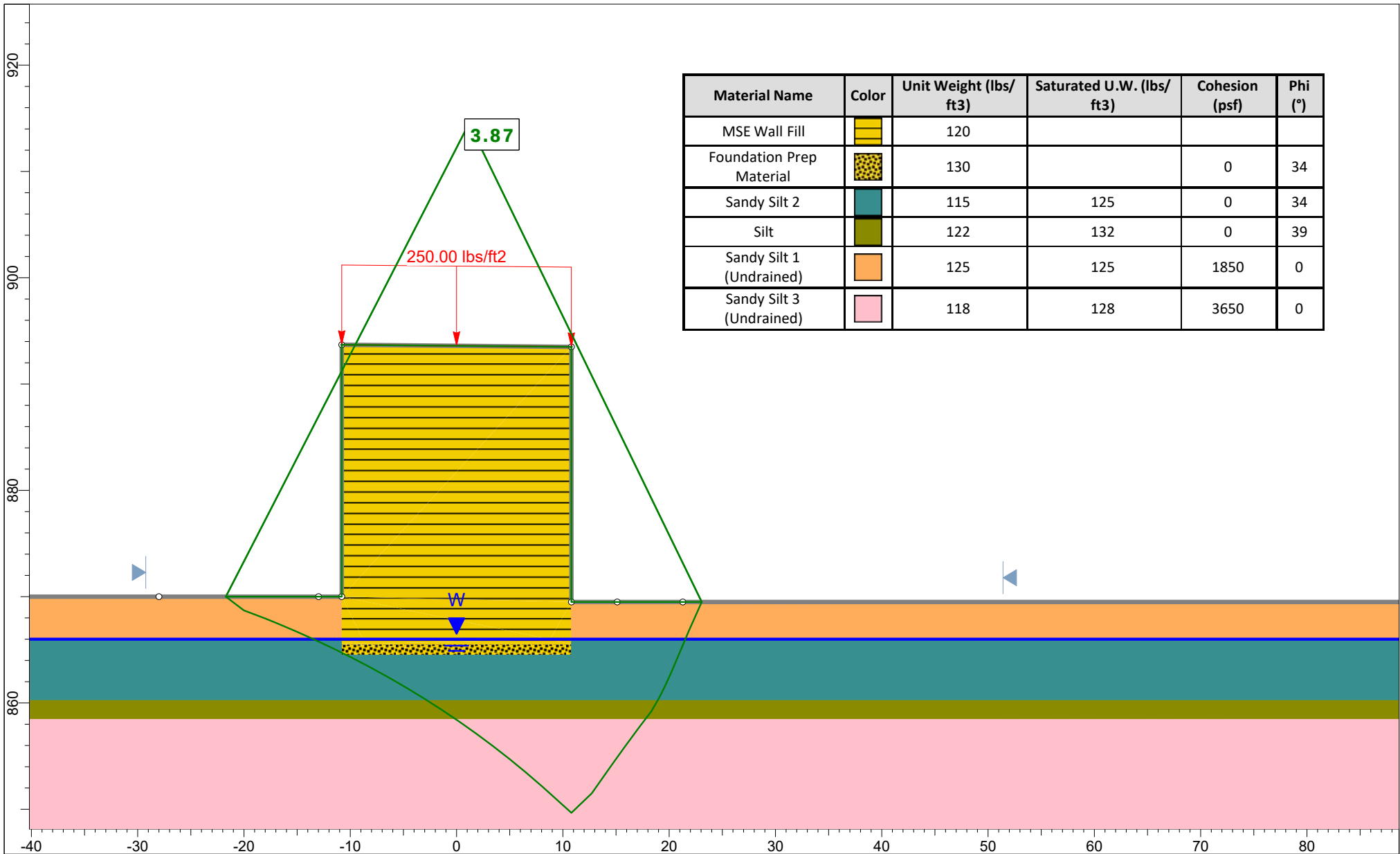
	Project	FRA-Cemetery Rd SUP, PID 120103	
	Group	Total Stress - Circular Failure	Scenario Master Scenario
	Drawn By	KCA	Company NEAS Inc.
	Date	1/16/2026, 4:43:08 PM	File Name RW1_STA00+61.3_061726.slmd

RW-2 – STA. 00+58.8



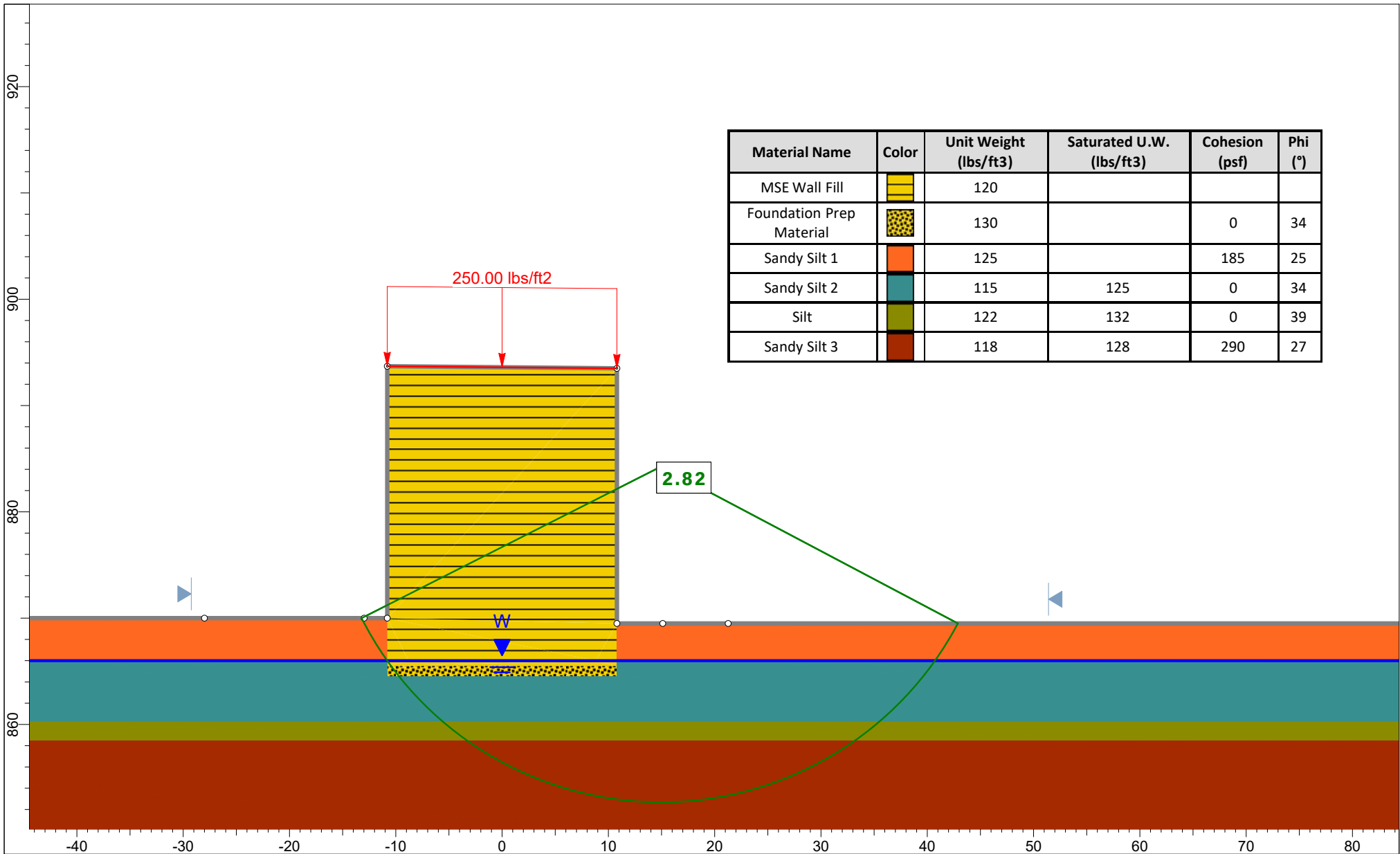
Material Name	Color	Unit Weight (lbs/ft ³)	Saturated U.W. (lbs/ft ³)	Cohesion (psf)	Phi (°)
MSE Wall Fill		120			
Foundation Prep Material		130		0	34
Sandy Silt 1		125		185	25
Sandy Silt 2		115	125	0	34
Silt		122	132	0	39
Sandy Silt 3		118	128	290	27

	Project		FRA-Cemetery Rd SUP, PID 120103	
	Group	Effective Stress - Non-circular Failure	Scenario	Master Scenario
	Drawn By	KCA	Company	NEAS Inc.
	Date	1/16/2026, 4:43:08 PM	File Name	RW2_STA00+58.8_061726.slmd
	SLIDEINTERPRET 9.038			



Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Cohesion (psf)	Phi (°)
MSE Wall Fill		120			
Foundation Prep Material		130		0	34
Sandy Silt 2		115	125	0	34
Silt		122	132	0	39
Sandy Silt 1 (Undrained)		125	125	1850	0
Sandy Silt 3 (Undrained)		118	128	3650	0

	Project		FRA-Cemetery Rd SUP, PID 120103	
	Group	Total Stress - Non-circular Failure	Scenario	Master Scenario
	Drawn By	KCA	Company	NEAS Inc.
	Date	1/16/2026, 4:43:08 PM	File Name	RW2_STA00+58.8_061726.slmd
	SLIDEINTERPRET 9.038			



Material Name	Color	Unit Weight (lbs/ft ³)	Saturated U.W. (lbs/ft ³)	Cohesion (psf)	Phi (°)
MSE Wall Fill		120			
Foundation Prep Material		130		0	34
Sandy Silt 1		125		185	25
Sandy Silt 2		115	125	0	34
Silt		122	132	0	39
Sandy Silt 3		118	128	290	27



Project

FRA-Cemetery Rd SUP, PID 120103

Group

Effective Stress - Circular Failure

Scenario

Master Scenario

Drawn By

KCA

Company

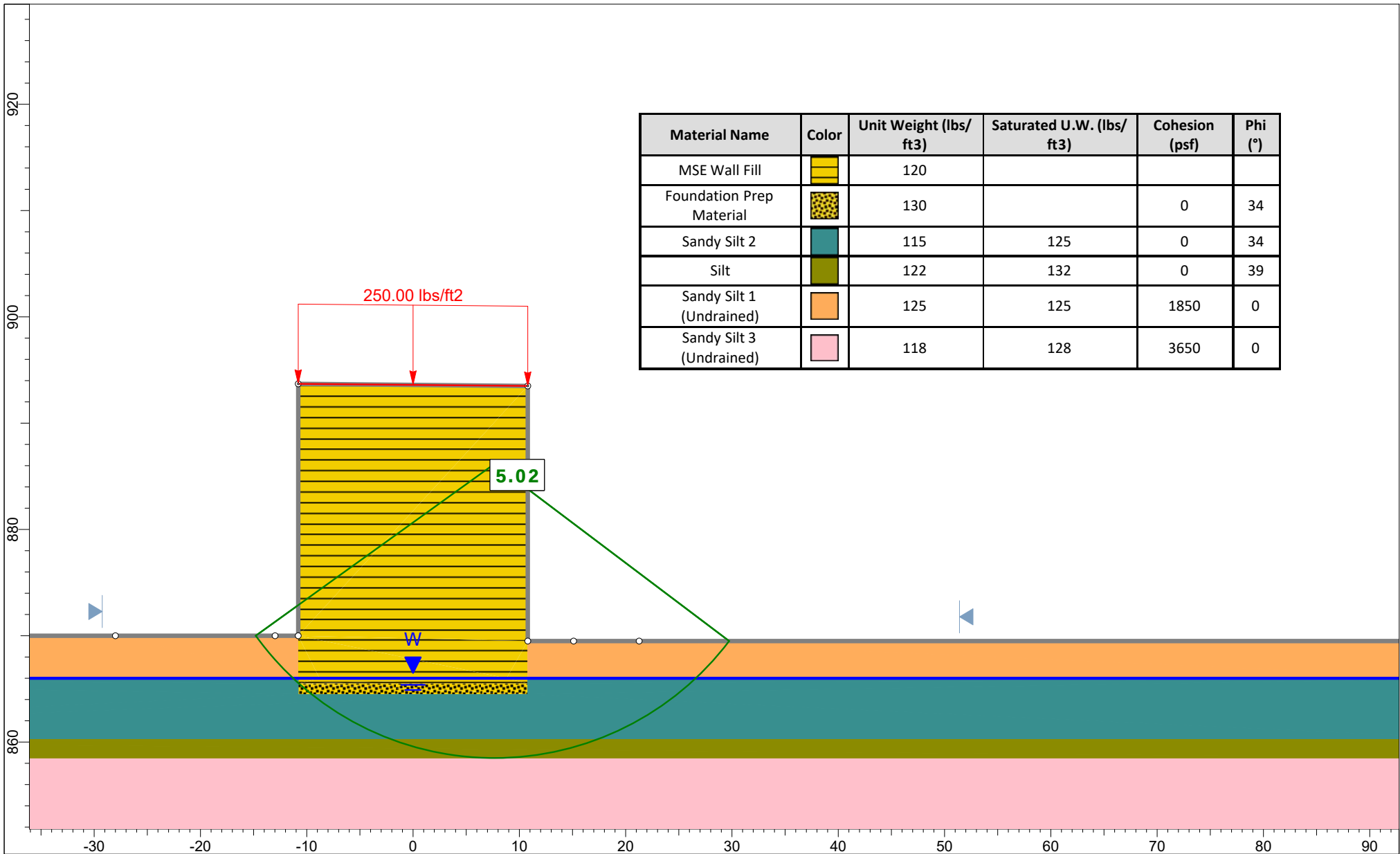
NEAS Inc.

Date

1/16/2026, 4:43:08 PM

File Name

RW2_STA00+58.8_061726.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Cohesion (psf)	Phi (°)
MSE Wall Fill		120			
Foundation Prep Material		130		0	34
Sandy Silt 2		115	125	0	34
Silt		122	132	0	39
Sandy Silt 1 (Undrained)		125	125	1850	0
Sandy Silt 3 (Undrained)		118	128	3650	0



Project	FRA-Cemetery Rd SUP, PID 120103		
Group	Total Stress - Circular Failure	Scenario	Master Scenario
Drawn By	KCA	Company	NEAS Inc.
Date	1/16/2026, 4:43:08 PM	File Name	RW2_STA00+58.8_061726.slmd

APPENDIX 3D

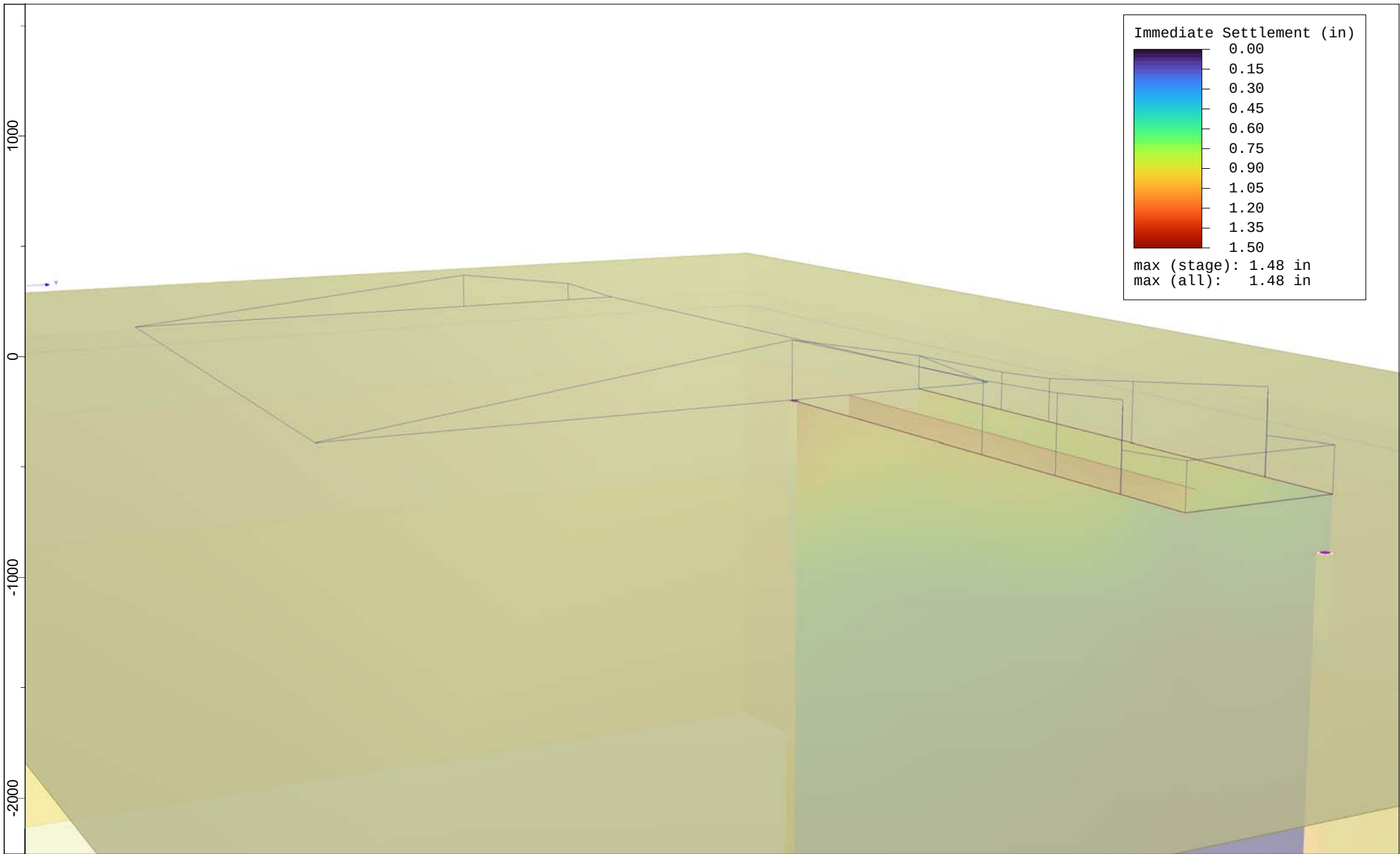
SETTLEMENT ANALYSIS



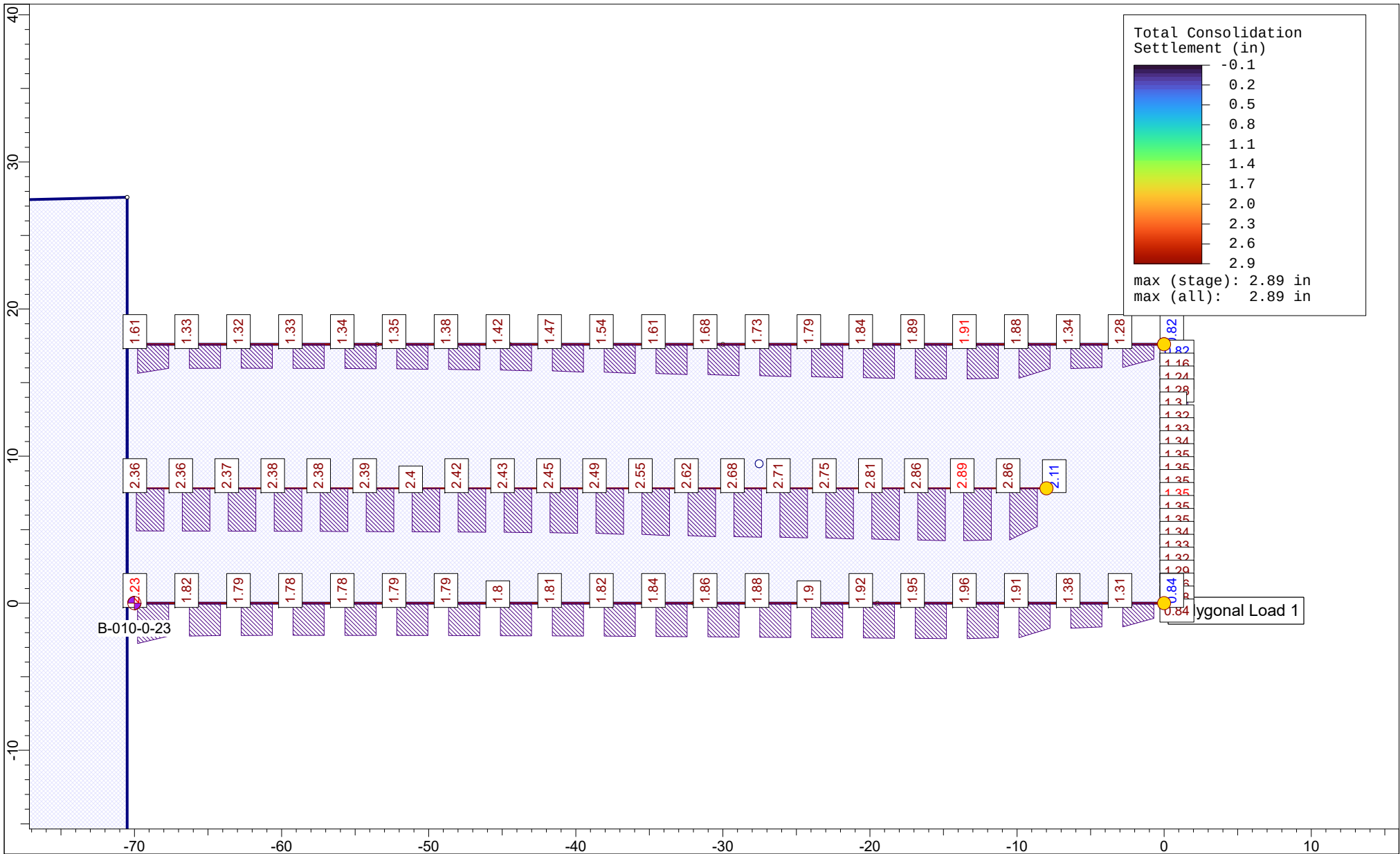
FRA-CEMETERY RD-SUP

NEAS Inc.

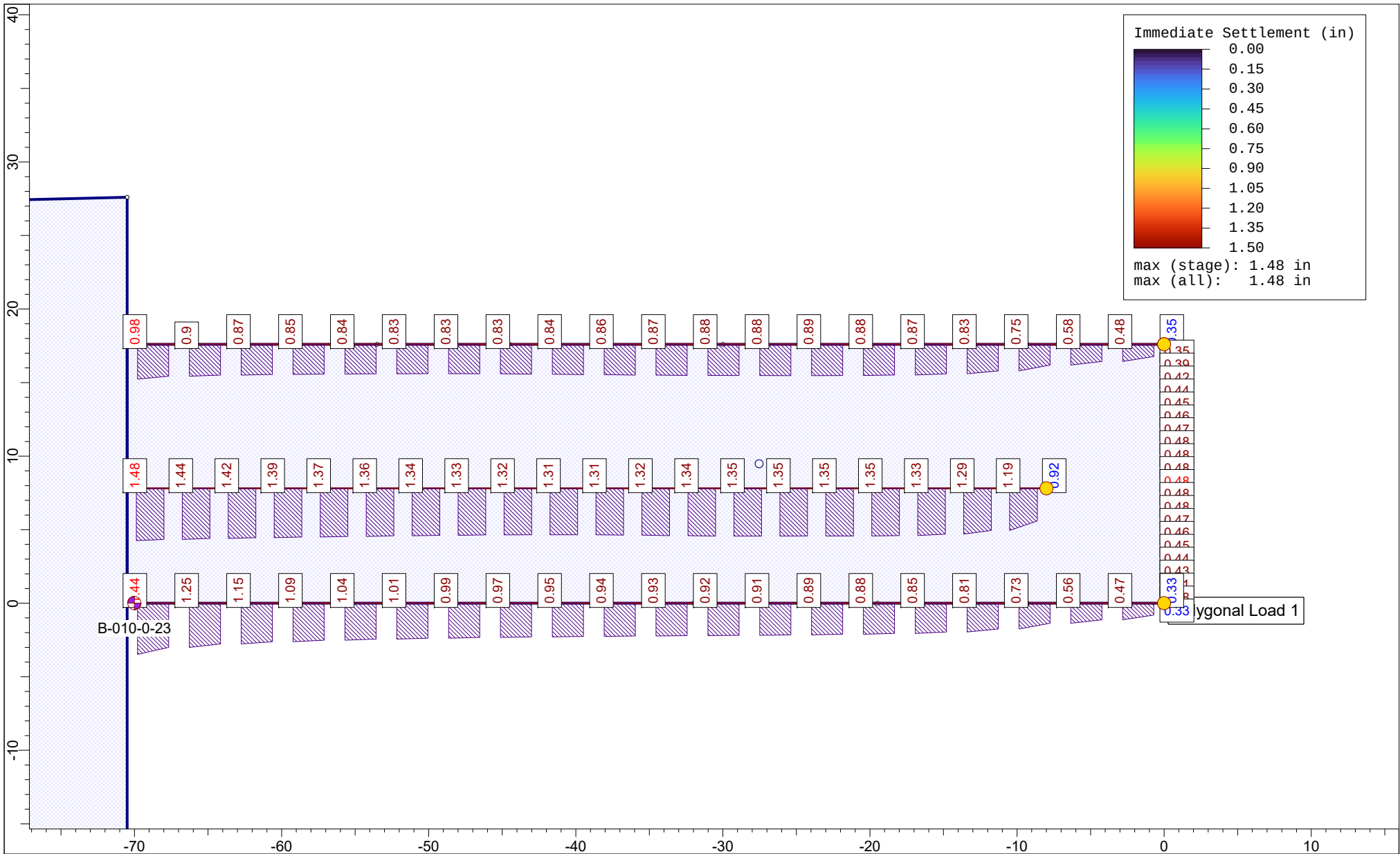
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Project	FRA-CEMETERY RD-SUP		
Analysis Description	Settlement at RW-1 & RW-2		
Drawn By	KCA	Company	NEAS Inc.
Date	1/22/2026, 1:31:40 PM	File Name	RW-1_Settle_012226.s3z



	Project		FRA-CEMETERY RD-SUP
	Analysis Description		Settlement at RW-1 & RW-2
	Drawn By	KCA	Company NEAS Inc.
	Date	1/22/2026, 1:31:40 PM	File Name RW-1_Settle_012226.s3z



Project	FRA-CEMETERY RD-SUP		
Analysis Description	Settlement at RW-1 & RW-2		
Drawn By	KCA	Company	NEAS Inc.
Date	1/22/2026, 1:31:40 PM	File Name	RW-1_Settle_012226.s3z

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Settle3 Analysis Information

FRA-CEMETERY RD-SUP

Project Settings

Document Name	RW-1_Settle_012226.s3z
Project Title	FRA-CEMETERY RD-SUP
Analysis	Settlement at RW-1 & RW-2
Author	KCA
Company	NEAS Inc.
Date Created	1/22/2026, 1:31:40 PM
Stress Computation Method	Boussinesq
Stress Units	Imperial, stress as ksf
Settlement Units	inches
Time-dependent Consolidation Analysis	
Time Units	days
Permeability Units	feet/day

Advanced Settings

Start of secondary consolidation (% of primary)	95
Min. stress for secondary consolidation (% of initial)	1
Reset time when load changes for secondary consolidation	No
Minimum settlement ratio for subgrade modulus	0.9
Use average poisson's ratio to calculate layered stresses	
Update Cv in each time step (improves consolidation accuracy)	
Ignore negative effective stresses in settlement calculations	
Add field points to load edges	

Soil Profile

Layer Option	Non-Horizontal Layers
Interpolation Method	TIN Triangulation
Use Non-Horizontal Ground Surface	No
Vertical Axis	Depth below ground surface

Stage Settings

Stage #	Name	Time [days]
1	Stage 1	0
2	Stage 2	30
3	Stage 3	18250

Results

Time taken to compute: 0.664233 seconds

Stage: Stage 1 = 0 d

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.47596
Total Consolidation Settlement [in]	0	0
Virgin Consolidation Settlement [in]	0	0
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	1.47596
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	0.157908	2.72399
Loading Stress XX [ksf]	-0.523287	2.01256
Loading Stress YY [ksf]	-0.573643	2.21417
Effective Stress ZZ [ksf]	0	6.5752
Effective Stress XX [ksf]	0.0559987	8.00027
Effective Stress YY [ksf]	0.402532	8.38456
Total Stress ZZ [ksf]	0.365	11.7515
Total Stress XX [ksf]	0.428499	13.0561
Total Stress YY [ksf]	0.810466	13.5609
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	29.4122
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	29.4122
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	0.000177289	0.0069221
Pore Water Pressure [ksf]	0.359409	5.17633
Excess Pore Water Pressure [ksf]	0.157908	2.72399
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.483	9.69651
Over-consolidation Ratio	1	11.5
Void Ratio	0	0.647481
Permeability [ft/d]	0	0.00916648
Coefficient of Consolidation [ft ² /d]	0	0.6
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	-2.22045e-16	1.11022e-16

Stage: Stage 2 = 30 d

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.91266
Total Consolidation Settlement [in]	-0.0154528	2.6148
Virgin Consolidation Settlement [in]	0	1.00706
Recompression Consolidation Settlement [in]	-0.0154528	1.62267
Immediate Settlement [in]	0	1.47596
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	0.157908	2.72399
Loading Stress XX [ksf]	-0.523287	2.01256
Loading Stress YY [ksf]	-0.573643	2.21417
Effective Stress ZZ [ksf]	0.365	6.21994
Effective Stress XX [ksf]	0.428499	7.70813
Effective Stress YY [ksf]	0.810466	7.91444
Total Stress ZZ [ksf]	0.365	11.7515
Total Stress XX [ksf]	0.428499	13.0561
Total Stress YY [ksf]	0.810466	13.5609
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	8.64338
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	29.4122
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	12.609
Total Strain	0.000164425	0.119127
Pore Water Pressure [ksf]	0	5.64645
Excess Pore Water Pressure [ksf]	0	1.0427
Degree of Consolidation [%]	0	90.6182
Pre-consolidation Stress [ksf]	0.483	9.69651
Over-consolidation Ratio	1	8.48385
Void Ratio	0	0.642592
Permeability [ft/d]	0	0.0446572
Coefficient of Consolidation [ft ² /d]	0	0.6
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	93.4325
Undrained Shear Strength	-2.22045e-16	0.178885

Stage: Stage 3 = 18250 d

Data Type	Minimum	Maximum
Total Settlement [in]	0	4.19334
Total Consolidation Settlement [in]	0	2.89382
Virgin Consolidation Settlement [in]	0	1.0079
Recompression Consolidation Settlement [in]	0	1.90077
Immediate Settlement [in]	0	1.47596
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	0.157908	2.72399
Loading Stress XX [ksf]	-0.523287	2.01256
Loading Stress YY [ksf]	-0.573643	2.21417
Effective Stress ZZ [ksf]	0.365	6.94673
Effective Stress XX [ksf]	0.428499	8.26184
Effective Stress YY [ksf]	0.810466	8.75609
Total Stress ZZ [ksf]	0.365	11.7515
Total Stress XX [ksf]	0.428499	13.0561
Total Stress YY [ksf]	0.810466	13.5609
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	8.07062
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	29.4122
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	11.4261
Total Strain	0.000177289	0.119144
Pore Water Pressure [ksf]	-5.72711e-17	4.8048
Excess Pore Water Pressure [ksf]	-4.49063e-15	2.03956e-15
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.483	9.69651
Over-consolidation Ratio	1	7.73524
Void Ratio	0	0.641018
Permeability [ft/d]	0	0.0446572
Coefficient of Consolidation [ft ² /d]	0	0.6
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	100
Undrained Shear Strength	0	0.210144

Loads

1. Polygonal Load: "Polygonal Load 1"

Label	Polygonal Load 1
Load Type	Flexible
Area of Load	1240.8 ft2
Depth	0 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

	X [ft]	Y [ft]	Load Magnitude [ksf]
0	17.6		1.46
-9.3	17.6		1.26
-9.4	17.6		2.75
-30	17.6		2.02
-44.5	17.6		1.49
-53.5	17.6		1.34
-70.5	17.6		1.24
-70.5	0		2.2
-32	0		2.38
-19.5	0		2.53
-9.4	0		2.78
-9.3	0		1.31
0	0		1.49

2. Polygonal Load: "Polygonal Load 2"

Label	Polygonal Load 2
Load Type	Flexible
Area of Load	8070 ft2
Depth	0 ft
Installation Stage	Stage 1 = 0 d

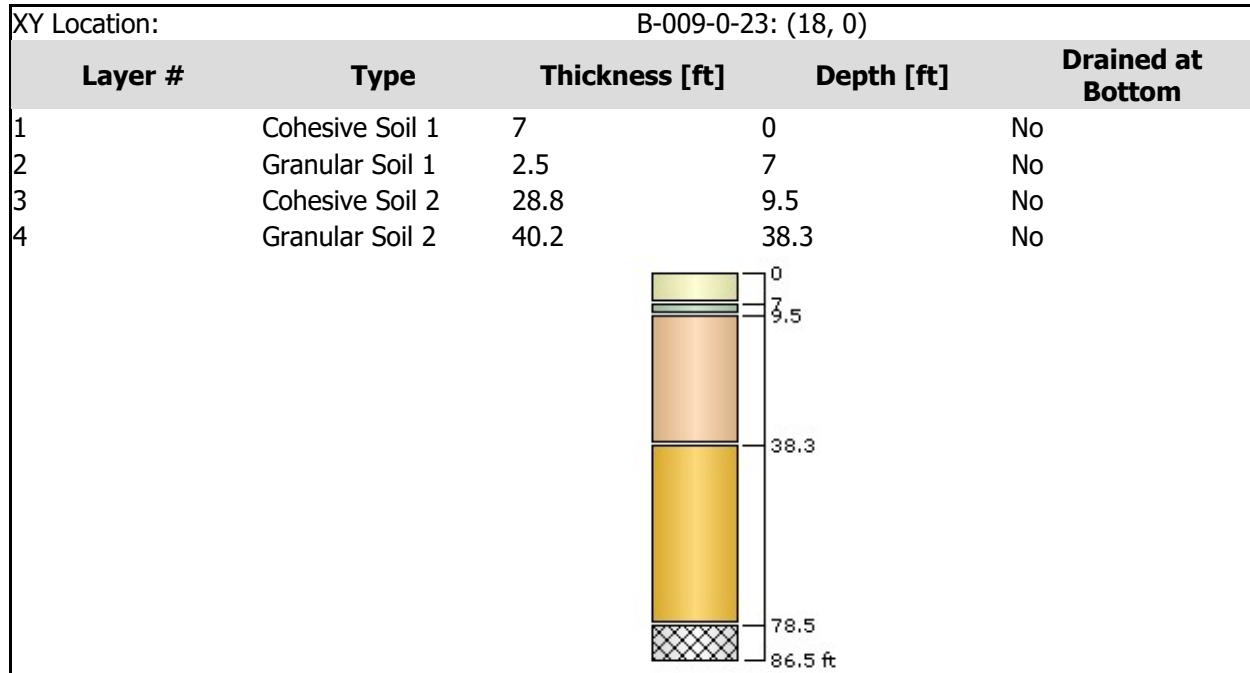
Coordinates and Load

	X [ft]	Y [ft]	Load Magnitude [ksf]
-70.5	17.6		1.24
-70.5	27.6		0
-170.5	25.2		0
-170.5	17.6		0.8
-170.5	0		1.5
-170.5	-50.7		0
-70.5	-57.9		0
-70.5	0		2.2

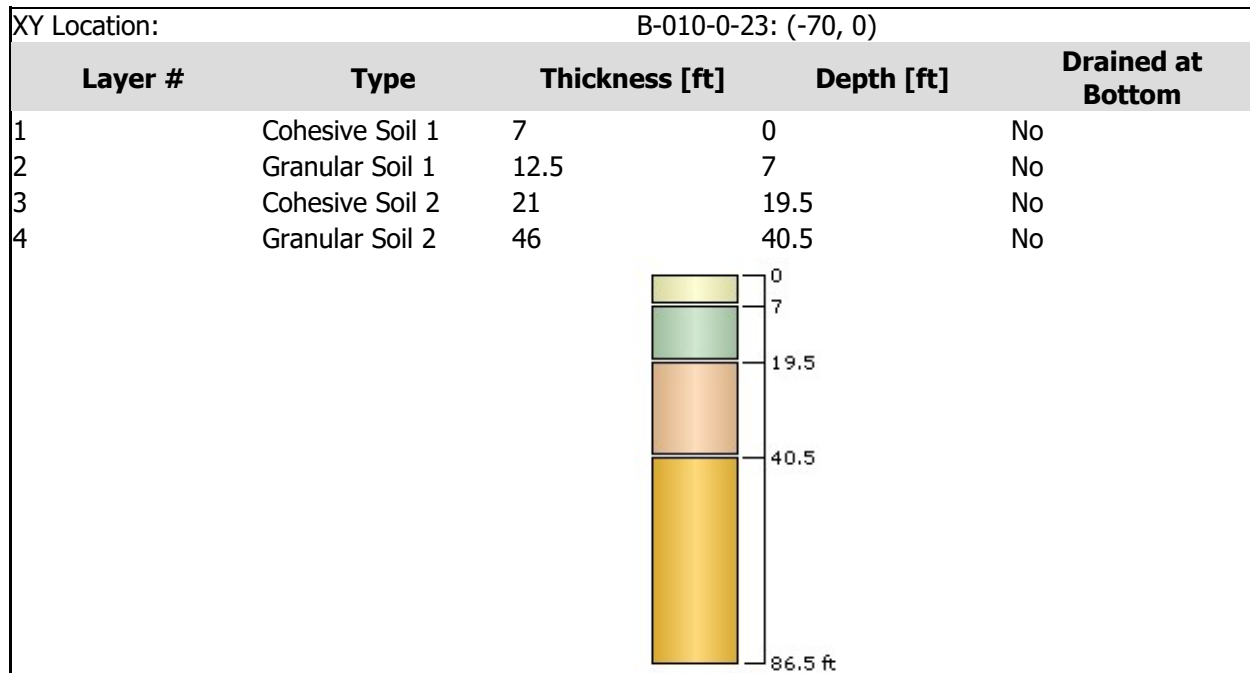
Soil Layers

Ground Surface Drained: Yes

B-009-0-23



B-010-0-23



Soil Properties

Property	Cohesive Soil 1	Granular Soil 1	Cohesive Soil 2	Granular Soil 2
Color				
Unit Weight [kips/ft3]	0.12	0.118	0.125	0.13
Saturated Unit Weight [kips/ft3]	0.13	0.12	0.135	0.135
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	392	321	1452	891
Esur [ksf]	392	321	1452	891
Primary Consolidation	Enabled	Disabled	Enabled	Disabled
Material Type	Non-Linear		Non-Linear	
Cc	0.19	-	0.05	-
Cr	0.039	-	0.007	-
e0	0.649	-	0.33	-
OCR	11.5	-	3	-
Cv [ft2/d]	0.6	-	0.6	-
Cvr [ft2/d]	0.6	-	0.6	-
B-bar	1	-	1	-
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1

Groundwater

Groundwater method

Piezometric Lines

Water Unit Weight

0.0624 kips/ft3

Generating excess pore pressure above water table

Piezometric Line Entities

ID	Depth (ft)
1	9.5 ft

Query

Query Lines

Line #	Query Line Name	Start Location	End Location	Horizontal Divisions	Vertical Divisions
1	Query Line 1	-8, 7.81	-70.5, 7.81	20	Auto: 55
2	Query Line 2	0, 17.6	-70.5, 17.6	20	Auto: 55
3	Query Line 3	0, 17.6	0, 0	20	Auto: 55
4	Query Line 4	0, 0	-70.5, 0	20	Auto: 55

SECTION 4

PEDESTRIAN TUNNEL

SECTION 4: PEDESTRIAN TUNNEL

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4.1. INTRODUCTION

4.1.1. Proposed Construction

NEAS understands that B&N is developing construction plans for the FRA-Cemetery Road Shared-Use Path (SUP) project. As part of the project, a new tunnel structure is proposed along a portion of the referenced path construction to direct the SUP movement under an existing interstate entrance ramp that is planned to be realigned as well as separate segment of the SUP further down-station. The interstate ramp that is planned to be realigned and will go over the proposed tunnel structure is the entrance ramp from Cemetery Rd eastbound (EB) to IR-270 southbound (SB).

It is our understanding that the proposed tunnel structure will consist of a 16-ft wide by 10-ft tall precast concrete box culvert with cast-in-place (CIP) reinforced concrete wingwalls and footings atop which the culvert will be supported. The tunnel structure will be approximately 61 ft in length which will encompass the 25-ft wide roadway and 11-ft wide SUP. The width of the SUP inside the tunnel structure will be about 14-ft (i.e., 7-ft in each direction). For analysis purposes, the bearing elevations of the culvert footings are anticipated to range from about 870.1 to 871.3 ft amsl.

4.2. GEOTECHNICAL FINDINGS

The subsurface conditions encountered during NEAS's explorations are described in the following subsections and on each boring log presented in Appendix 1B. 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, results of historical explorations, and consideration of the geological history of the site.

4.2.1. Subsurface Conditions.

4.2.1.1. Overburden Soil

At the proposed tunnel site, two different materials were encountered within the borings underlying the topsoil. Those materials consisted of: 1) "man-made" fill / potential fill soils; and, 2) both cohesive and non-cohesive natural glacial till soils. These materials and the general profile are further described below.

Fill soils were encountered in each of the borings performed at the tunnel site extending to depths ranging from 5.5 ft to 9.5 ft bgs (elevations 862.9 to 866.0 ft amsl). Based on laboratory testing results and a visual review of the samples obtained, the fill at the site is comprised of cohesive, fine-grained material that is classified on the boring logs as cohesive Sandy Silt (A-4a), Silt and Clay (A-6a) and Silty Clay (A-6b). The exception being a sample of coarse-grained soils encountered at a depth of 5.0 ft bgs (elevation 867.4 ft amsl) in boring B-012-0-23 that classified as Coarse and Fine Sand (A-3a). With respect to the soil strength of the cohesive fill soils encountered, these soils can be described as having a consistency of medium stiff to hard correlating to converted SPT-N (N_{60}) values between 7 and 22 blows per foot (bpf) and unconfined compressive strengths (estimated by means of hand penetrometer) between 2.0 and 4.5 tons

per square foot (tsf). The natural moisture content of the cohesive soil ranged from 13 to 21 percent. Based on Atterberg Limits tests performed on representative samples of the cohesive fill material, the liquid and plastic limits ranged from 21 to 40 percent and from 16 to 21 percent, respectively. With respect to the granular fill soil sample encountered, this material can be described as having a relative compactness of medium dense correlating to an N_{60} value of 22 bpf. Natural moisture content of this sample was determined to be 14 percent.

Natural glacial till soils were encountered underlying the fill soils in both borings performed at the tunnel site. The natural material extended to a borehole termination depth between 50 and 51.5 ft bgs (elevations 820.9 and 821.5 ft amsl). In general, the till can be divided into an upper and lower stratum based on the characteristics. The upper till stratum generally consisted of cohesive, fine-grained soils extending to depths between 43.3 and 46.8 ft bgs (elevations 824.7 and 829.1 ft amsl) and classified on the logs as cohesive Sandy Silt (A-4a). The exceptions being a sample of coarse-grained material in boring B-012-0-23 that is classified on the log as Coarse and Fine Sand (A-3a) as well as a sample of non-cohesive, fine-grained soil in boring B-013-0-23 that classified as non-cohesive Sandy Silt (A-4a). The lower stratum consisted of coarse-grained soils that classified on the logs as Gravel with Sand and Silt (A-2-4), or Gravel with Sand (A-1-b). With respect to the soil strength of the cohesive natural soils, these soils can be described as having a consistency of stiff to hard correlating to N_{60} values between 16 and 88 bpf and unconfined compressive strengths (estimated by means of hand penetrometer) between 1.75 and 4.5 tsf. Natural moisture contents of the cohesive till ranged from 10 to 13 percent. Based on Atterberg Limits tests performed on representative samples of the natural cohesive material, the liquid limits ranged from 24 to 25 percent and plastic limits of 15 percent. The relative compactness of the non-cohesive till soils encountered can generally be described as medium dense to very dense correlating to N_{60} values between 13 and 64 bpf. Natural moisture contents of these soils ranged from 10 to 15 percent.

4.2.1.2. Groundwater

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

It should be noted that groundwater is affected by many hydrologic characteristics in the area and may vary from those measured at the time of the exploration. The specific groundwater readings are included on the boring logs located within Appendix 1B.

4.3. ANALYSIS AND RECOMMENDATIONS

We understand that a new 16 ft wide by 10 ft tall precast concrete box culvert is planned to be constructed to act as a tunnel structure that will direct the proposed shared use path (SUP) traffic beneath the Cemetery Rd entrance ramp to IR-270 SB as well as a separate segment down-station of the SUP. For analysis purposes, it is assumed that the tunnel structure dimensions, configuration and footing bearing elevations are consistent with the tunnel structure plans prepared by B&N via email on February 21, 2025.

A shallow foundation bearing analysis was performed according to the LRFD BDS and BDM criteria. The analysis was completed for the proposed culvert footings based on the soil and bedrock characteristics gathered during the geotechnical exploration (i.e., SPT results, laboratory test results, etc.) and our geotechnical experience. Foundation design parameters and factored bearing resistances for the evaluated foundation system is presented in the subsections below.

4.3.1. Soil Profile for Analysis

For analysis purposes, each culvert 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 the 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 properties for use in analysis (with sited correlation/reference material) is summarized within Tables 1 and 2 below.

Table 1: Soil Properties and Estimated Engineering Properties - At Boring B-012-0-23

Tunnel Structure: Northeastern Wingwall, B-012-0-23				
Soil Description	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Silt and Clay Depth (872.4 ft - 867.9 ft)	115	2600	235	25
Coarse and Fine Sand Depth (867.9 ft - 865.4 ft)	125	-	-	37
Sandy Silt Depth (865.4 ft - 862.9 ft)	120	1250	125	24
Sandy Silt Depth (862.9 ft - 860.4 ft)	125	3000	255	26
Coarse and Fine Sand Depth (860.4 ft - 857.9 ft)	130	-	-	40
Sandy Silt Depth (857.9 ft - 829.1 ft)	128	3750	295	27
Gravel with Sand Depth (829.1 ft - 820.9 ft)	135	-	-	42
1. Values calculated per ODOT GDM Section 404/1304 and/or ODOT BDM Table 305-2.				

Table 2: Soil Properties and Estimated Engineering Properties - At Boring B-013-0-23

Tunnel Structure: Southwestern Wingwall, B-013-0-23				
Soil Description	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Silty Clay Depth (871.5 ft - 866 ft)	122	2350	220	25
Sandy Silt Depth (866 ft - 858 ft)	122	-	-	32
Sandy Silt Depth (858 ft - 811.3 ft)	125	3000	255	26
Sandy Silt Depth (811.3 ft - 764.6 ft)	135	6400	425	29
Gravel with Sand and Silt Depth (764.6 ft - 714.6 ft)	135	-	-	38
1. Values calculated per ODOT GDM Section 404/1304 and/or ODOT BDM Table 305-2.				

For settlement analysis purposes, the above referenced boring logs were reviewed and generalized material profile was developed by grouping soil layers from the borings with similar behavior with respect to compressibility (e.g., cohesive or non-cohesive/granular, moisture content, plasticity, relative density/consistency, etc.) into generalized soil units. Settlement parameters were estimated for each soil unit based on field and laboratory test results from the referenced borings as well as consolidation testing results from project boring B-008-0-23. The summary of the generalized soil profile including settlement parameters (with sited correlation/reference material) estimated for each of the referenced soil units developed for the culvert settlement analysis are presented within Table 3 below.

Table 3: Settlement Parameters for Analysis

Bridge FRA-TUNNEL Under Southbound IR-270 On-Ramp: Settlement Analysis B-012-0-23 & B-013-0-23								
Soil Description	Unit Weight (pcf)	Elastic Modulus ⁽¹⁾ (psf)	Poissons Ratio ⁽¹⁾ , ν	Void Ratio e_o	Compression Index ⁽²⁾ , C_c	Recompression Index ⁽³⁾ , C_r	OCR ⁽⁴⁾	Coeff. of Consol. ⁽⁵⁾ , C_v (ft ² /day)
Cohesive Soil 1	120	1032000	0.45	0.621	0.22	0.045	48	0.5
Granular Soil 1	118	261000	0.30	-	-	-	-	-
Cohesive Soil 2 ⁽⁶⁾	135	1866000	0.45	0.33	0.05	0.007	3.0	0.6
Granular Soil 2	140	416000	0.40	-	-	-	-	-
Notes: 1. Values interpreted from 2017 AASHTO LRFD BDS Table C10.4.6.3-1 2. Values calculated from Kulhawy and Mayne, 1990, Equation 6-6. 3. Values calculated from Kulhawy and Mayne, 1990, Equation 6-9. 4. Values interpreted from Mayne and Kemper, 1988, Figure 7. 5. Values interpreted from FHWA GEC No. 5, Boeckmann, et al., 2016, Figure 6-37. 6. Based on laboratory testing results of sample ST-7 in boring B-008-0-23.								

4.3.2. Shallow Foundation Analysis

A preliminary shallow foundation bearing analysis was performed at the tunnel structure / culvert location in accordance with the LRFD BDS, Section 10.6.3.1.2a, utilizing the engineering soil properties presented in Tables 1 and 2 of this report. The proposed shallow foundation system was assumed to consist of a rectangular spread footing bearing at an elevation of 870.1 ft amsl and 871.3 ft amsl at the southwestern and northeastern headwall, respectively. Additionally, for analysis purposes it is assumed that the groundwater elevation is equal to indicated bottom of footing elevations.

Based on the foundation configuration shown in the referenced preliminary site plans (i.e., depth of footing, bearing elevation, footing width, etc.) and our developed generalized material profile, nominal and factored bearing resistances were estimated for rectangular shallow foundation assuming a width of 9.5 ft and a length of 17 ft. The results of our analysis indicated that a shallow foundation bearing at the indicated elevations will have a nominal bearing resistance of about 13.2 kips per square foot (ksf). When multiplied by a LRFD resistance factor of 0.5 (LRFD Table 10.5.5.2.2-1), the indicated nominal bearing resistance results in a factored bearing resistance of 6.6 ksf. Bearing resistance calculations are included within Appendix 4A.

4.3.3. Settlement

In order to estimate the maximum total and differential settlement that could result within the subsurface soils supporting the proposed culvert tunnel, NEAS reviewed: 1) the proposed culvert tunnel plans provided by B&N via email on February 21, 2025; 2) Service Limit State loading conditions; and, 3) the generalized subsurface profile and Settlement Parameters for Analysis provided in Section 4.3.1. of this report. Utilizing this information and the software entitled *Settle3* by Rocscience Inc., a settlement model was developed and analyzed to for both elastic (immediate) and consolidation (long term) settlement.

Based on our analyses, the estimated maximum total settlement that could occur along the length of the proposed culvert and headwalls as a result of the induced culvert and embankment loads is estimated to range from about 2.6 to 3.6 inches with about 1.6 to 2.4 inches of the total settlement estimated to be long-term (consolidation). The maximum differential settlement along the length the proposed precast box culvert is estimated to be about 0.2%.

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Based on the results of our analysis, it is our opinion that the estimated settlement magnitudes are not anticipated to be a concern as the elastic settlement is anticipated to occur immediately and approximately 80 to 90 percent of consolidation settlement is expected to be complete within the first 30 days following construction. Additionally, differential settlement between substructures supporting the precast box culvert is estimated to be within the limits specified in ODOT BDM Section 305.1.3, while differential settlement along the length of the proposed cast-in-place headwalls is estimated to be less than the limit for rigid semi-gravity walls per ODOT BDM Section 307.1.6. The output of the settlement analysis program is included within Appendix 4B.

APPENDIX 4A

BEARING RESISTANCE ANALYSIS

SOUTHWESTERN HEADWALL

Objective: To determine the nominal bearing capacity of foundation soil for shallow foundation design.
Method: In accordance with LRFD Bridge Design Specifications, 8th Ed., 2017, [Sect. 10.6.3.1.2].

Givens:

Footing Geometry:

$$D_f := 3 \text{ ft}$$

Depth to base of footing below exterior grade

$$L := 17 \text{ ft}$$

Assumed Footing Length

$$B := 9.5 \text{ ft}$$

Width/Breadth of footing

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 25 \text{ deg}$$

Effective angle of internal friction

$$\gamma_f := 122 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 220 \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 122 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 2350 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use S_u if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

$$\gamma_q := 122 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation **LRFD BDS Section 10.6.3.1.2a-1**)

Groundwater Conditions:

$$d_w := 3 \text{ ft}$$

Depth of Groundwater below Ground Surface

$$d_{w'} := d_w - D_f$$

$$d_{w'} = 0 \text{ ft}$$

Depth of Groundwater below Bearing Grade

Drained Conditions (Effective Stress):

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

$$N_q = 10.66$$

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

$$N_c = 20.72$$

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

$$N_\gamma = 10.9$$

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

$$s_c := \text{if} \left(\phi'_f > 0, 1 + \left(\frac{B}{L} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B}{5 \cdot L} \right) \right) \quad s_c = 1.288$$

$$s_q := \text{if} \left(\phi'_f > 0, 1 + \left(\frac{B}{L} \right) \cdot \tan(\phi'_f), 1 \right) \quad s_q = 1.261$$

$$s_\gamma := \text{if} \left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B}{L} \right), 1 \right) \quad s_\gamma = 0.776$$

Load inclination factors using LRFD BDS [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 BDS [Table 10.6.3.1.2a-2]:

$$C_{wq} := \text{if} (d_w \geq D_f, 1, 0.5) \quad C_{wq} = 1$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1, 0.5) \quad C_{w\gamma} = 0.5$$

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 26.679$$

$$N_{qm} := N_q \cdot s_q \cdot i_q \quad N_{qm} = 13.441$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 8.445$$

Compute nominal bearing resistance. LRFD BDS [Eq 10.6.3.1.2a-1]:

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nd} = 13235.5 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD BDS [Eq 10.6.3.1.1]:

$$\phi_b := 0.5$$

Bearing resistance factor LRFD BDS Section 10.5.5.2.2-1

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 6.6 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

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

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B}{L} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B}{5 \cdot L} \right) \right) \quad s_c = 1.112$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 5.714$$

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

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

Compute nominal bearing resistance. LRFD BDS [Eq 10.6.3.1.2a-1]:

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 13795 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD BDS [Eq 10.6.3.1.1]:

$$\phi_b := 0.5$$

Bearing resistance factor LRFD BDS Section 10.5.5.2.2-1

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 6.9 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 6.6 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 6.9 \text{ ksf}$$

Drained Conditions Control

NORTHEASTERN HEADWALL

Objective: To determine the nominal bearing capacity of foundation soil for shallow foundation design.
Method: In accordance with LRFD Bridge Design Specifications, 8th Ed., 2017, [Sect. 10.6.3.1.2].

Givens:

Footing Geometry:

$$D_f := 3 \text{ ft}$$

Depth to base of footing below exterior grade

$$L := 17 \text{ ft}$$

Assumed Footing Length

$$B := 9.5 \text{ ft}$$

Width/Breadth of footing

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_f := 25 \text{ deg}$$

Effective angle of internal friction

$$\gamma_f := 115 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c'_f := 235 \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion

Undrained Conditions (Total Stress):

$$\phi_f := 0 \cdot \text{deg}$$

Angle of internal friction (Same as Drained Conditions if Sand)

$$\gamma_f = 115 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$c_f := 2600 \cdot \frac{\text{lbf}}{\text{ft}^2}$$

Cohesion (Use S_u if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

$$\gamma_q := 115 \cdot \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation **LRFD BDS Section 10.6.3.1.2a-1**)

Groundwater Conditions:

$$d_w := 3 \text{ ft}$$

Depth of Groundwater below Ground Surface

$$d_{w'} := d_w - D_f$$

$$d_{w'} = 0 \text{ ft}$$

Depth of Groundwater below Bearing Grade

Drained Conditions (Effective Stress):

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

$$N_q = 10.66$$

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

$$N_c = 20.72$$

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

$$N_\gamma = 10.9$$

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

$$s_c := \text{if} \left(\phi'_f > 0, 1 + \left(\frac{B}{L} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B}{5 \cdot L} \right) \right) \quad s_c = 1.288$$

$$s_q := \text{if} \left(\phi'_f > 0, 1 + \left(\frac{B}{L} \right) \cdot \tan(\phi'_f), 1 \right) \quad s_q = 1.261$$

$$s_\gamma := \text{if} \left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B}{L} \right), 1 \right) \quad s_\gamma = 0.776$$

Load inclination factors using LRFD BDS [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 BDS [Table 10.6.3.1.2a-2]:

$$C_{wq} := \text{if} (d_w \geq D_f, 1, 0.5) \quad C_{wq} = 1$$

$$C_{w\gamma} := \text{if} (d_w \geq (1.5 \cdot B) + D_f, 1, 0.5) \quad C_{w\gamma} = 0.5$$

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 26.679$$

$$N_{qm} := N_q \cdot s_q \cdot i_q \quad N_{qm} = 13.441$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 8.445$$

Compute nominal bearing resistance. LRFD BDS [Eq 10.6.3.1.2a-1]:

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nd} = 13213.1 \frac{\text{lbf}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD BDS [Eq 10.6.3.1.1]:

$$\phi_b := 0.5$$

Bearing resistance factor LRFD BDS Section 10.5.5.2.2-1

$$q_{Rd} := \phi_b \cdot q_{nd}$$

$$q_{Rd} = 6.6 \text{ ksf}$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

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

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

$$s_c := \text{if} \left(\phi_f > 0, 1 + \left(\frac{B}{L} \right) \cdot \left(\frac{N_q}{N_c} \right), 1 + \left(\frac{B}{5 \cdot L} \right) \right) \quad s_c = 1.112$$

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

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

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

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

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

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

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \quad N_{cm} = 5.714$$

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

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

Compute nominal bearing resistance. LRFD BDS [Eq 10.6.3.1.2a-1]:

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma} \quad q_{nu} = 15202.6 \frac{\text{lb}}{\text{ft}^2}$$

Compute factored bearing resistance. LRFD BDS [Eq 10.6.3.1.1]:

$$\phi_b := 0.5$$

Bearing resistance factor LRFD BDS Section 10.5.5.2.2-1

$$q_{Ru} := \phi_b \cdot q_{nu} \quad q_{Ru} = 7.6 \text{ ksf}$$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

$$\text{Drained Conditions: } q_{Rd} = 6.6 \text{ ksf}$$

$$\text{Undrained Conditions: } q_{Ru} = 7.6 \text{ ksf}$$

Drained Conditions Control

APPENDIX 4B

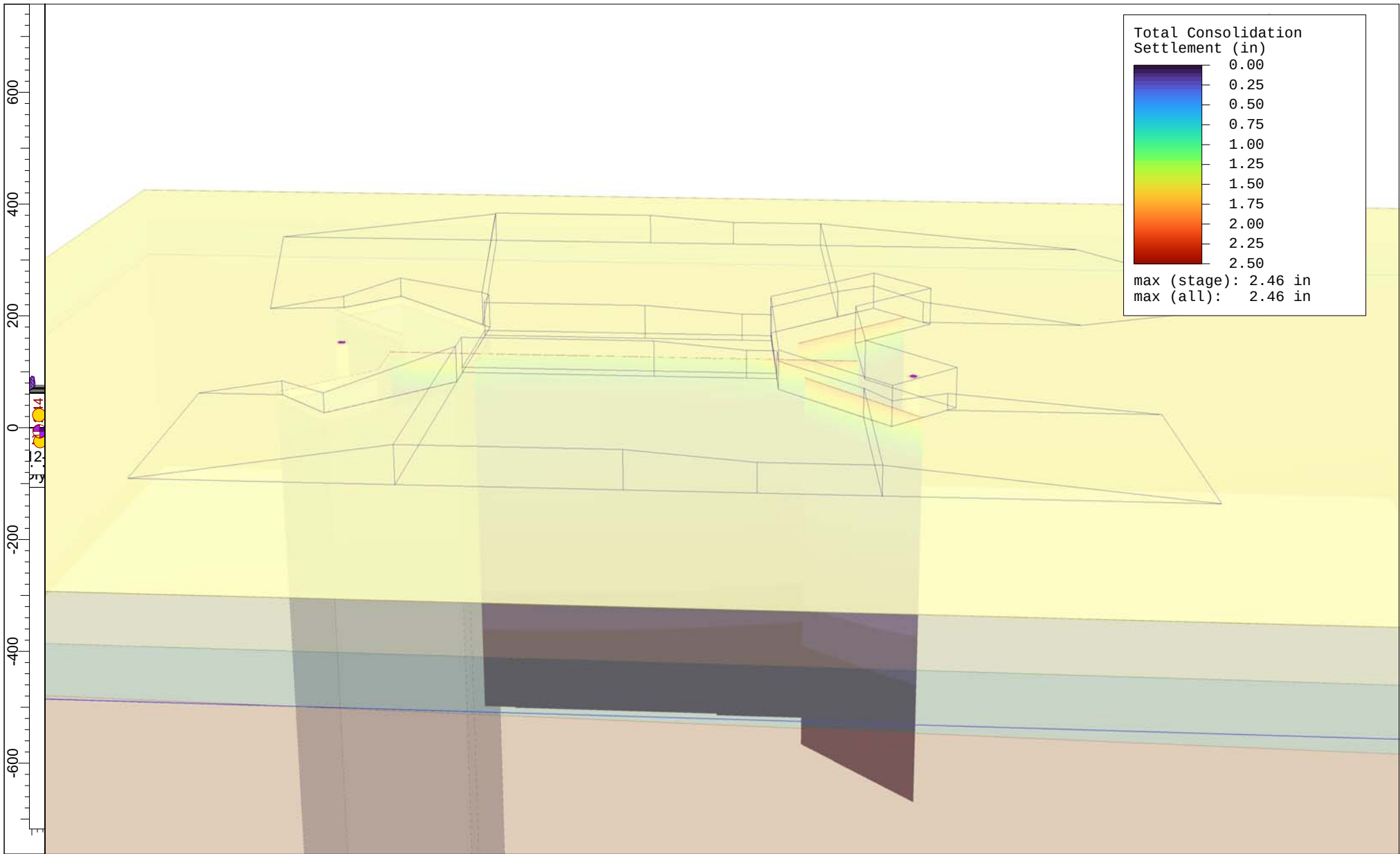
SETTLEMENT ANALYSIS



FRA-CEMETERY RS-SUP

NEAS Inc.

Report Creation Date: 2025/03/03, 10:34:03



Project		FRA-CEMETERY RS-SUP	
Analysis Description		Settlement at Culvert FRA-TUNNEL Under Southbound IR-270 On-Ramp	
Drawn By		KCA	Company NEAS Inc.
Date		2/28/2025, 12:12:02 PM	File Name CemeteryRd_Tunnel_Settle022825.s3z

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Settle3 Analysis Information

FRA-CEMETERY RS-SUP

Project Settings

Document Name	CemeteryRd_Tunnel_Settle022825.s3z
Project Title	FRA-CEMETERY RS-SUP
Analysis	Settlement at Culvert FRA-TUNNEL Under Southbound IR-270 On-Ramp
Author	KCA
Company	NEAS Inc.
Date Created	2/28/2025, 12:12:02 PM
Last saved with Settle3 version	5.024
Stress Computation Method	Boussinesq
Stress Units	Imperial, stress as ksf
Settlement Units	inches
Time-dependent Consolidation Analysis	
Time Units	days
Permeability Units	feet/day

Advanced Settings

Start of secondary consolidation (% of primary)	95
Min. stress for secondary consolidation (% of initial)	1
Reset time when load changes for secondary consolidation	No
Minimum settlement ratio for subgrade modulus	0.9
Use average poisson's ratio to calculate layered stresses	
Update Cv in each time step (improves consolidation accuracy)	
Ignore negative effective stresses in settlement calculations	
Add field points to load edges	

Soil Profile

Layer Option	Non-Horizontal Layers
Interpolation Method	TIN Triangulation
Use Non-Horizontal Ground Surface	No
Vertical Axis	Elevation
Ground Elevation (ft)	871.3

Stage Settings

	Stage #	Name	Time [days]
1		Stage 1	0
2		Stage 2	30
3		Stage 3	20000

Results

Time taken to compute: 0.0012579 seconds

Stage: Stage 1 = 0 d

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.19933
Total Consolidation Settlement [in]	0	0
Virgin Consolidation Settlement [in]	0	0
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	1.19933
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	2.86218e-11	2.30006
Loading Stress XX [ksf]	0.0719013	2.34079
Loading Stress YY [ksf]	-0.325441	1.80704
Effective Stress ZZ [ksf]	0	4.18821
Effective Stress XX [ksf]	0.0719013	5.21789
Effective Stress YY [ksf]	-0.325441	5.17775
Total Stress ZZ [ksf]	2.86218e-11	7.70281
Total Stress XX [ksf]	0.0719013	8.73782
Total Stress YY [ksf]	-0.0719013	8.69599
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	31.2332
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	31.2332
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	0	0.00815042
Pore Water Pressure [ksf]	2.86218e-11	3.52173
Excess Pore Water Pressure [ksf]	2.86218e-11	2.30006
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.309376	33.0252
Over-consolidation Ratio	1	48
Void Ratio	0	0.621
Permeability [ft/d]	0	0.0584266
Coefficient of Consolidation [ft ² /d]	0	0.6
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	-1.11022e-16	2.22045e-16

Stage: Stage 2 = 30 d

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.44967
Total Consolidation Settlement [in]	0	2.25034
Virgin Consolidation Settlement [in]	0	0.243467
Recompression Consolidation Settlement [in]	0	2.00696
Immediate Settlement [in]	0	1.19933
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	2.86218e-11	2.30006
Loading Stress XX [ksf]	0.0719013	2.34079
Loading Stress YY [ksf]	-0.325441	1.80704
Effective Stress ZZ [ksf]	2.86218e-11	5.28123
Effective Stress XX [ksf]	0.0719013	6.31624
Effective Stress YY [ksf]	-0.0719013	6.27606
Total Stress ZZ [ksf]	2.86218e-11	7.70281
Total Stress XX [ksf]	0.0719013	8.73782
Total Stress YY [ksf]	-0.0719013	8.69599
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	10.5683
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	31.2332
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	14.7487
Total Strain	-6.46981e-06	0.162864
Pore Water Pressure [ksf]	0	2.62487
Excess Pore Water Pressure [ksf]	0	1.29269
Degree of Consolidation [%]	0	99.7946
Pre-consolidation Stress [ksf]	0.309376	33.0252
Over-consolidation Ratio	1	48.0258
Void Ratio	0	0.62101
Permeability [ft/d]	0	0.280956
Coefficient of Consolidation [ft ² /d]	0	0.6
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.570646

Stage: Stage 3 = 20000 d

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.65639
Total Consolidation Settlement [in]	0	2.45748
Virgin Consolidation Settlement [in]	0	0.243468
Recompression Consolidation Settlement [in]	0	2.2142
Immediate Settlement [in]	0	1.19933
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	2.86218e-11	2.30006
Loading Stress XX [ksf]	0.0719013	2.34079
Loading Stress YY [ksf]	-0.325441	1.80704
Effective Stress ZZ [ksf]	2.86218e-11	5.28123
Effective Stress XX [ksf]	0.0719013	6.31624
Effective Stress YY [ksf]	-0.0719013	6.27606
Total Stress ZZ [ksf]	2.86218e-11	7.70281
Total Stress XX [ksf]	0.0719013	8.73782
Total Stress YY [ksf]	-0.0719013	8.69599
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	9.87167
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	31.2332
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	13.4649
Total Strain	1.86778e-07	0.162864
Pore Water Pressure [ksf]	-5.61483e-17	2.43769
Excess Pore Water Pressure [ksf]	-6.93625e-17	7.64612e-17
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.309376	33.0252
Over-consolidation Ratio	1	47.9993
Void Ratio	0	0.621
Permeability [ft/d]	0	0.280956
Coefficient of Consolidation [ft ² /d]	0	0.6
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.570646

Loads

1. Polygonal Load: "Polygonal Load 2"

Label	Polygonal Load 2
Load Type	Flexible
Area of Load	665.685 ft2
Load	2.3 ksf
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates

X [ft]	Y [ft]
58.7	27.1
44.4	12.9
46.7	-13.7
61.3	-28.2
69.4	-20
57.8	-8.4
56.3	8.5
66.8	19

2. Polygonal Load: "Polygonal Load 3"

Label	Polygonal Load 3
Load Type	Flexible
Area of Load	736.2 ft2
Load	0.31 ksf
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates

X [ft]	Y [ft]
4.6	-9
46.3	-9
44.7	9
4.6	9

3. Polygonal Load: "Polygonal Load 1"

Label	Polygonal Load 1
Load Type	Flexible
Area of Load	1427.33 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-11.4	29.2	1.21
3.4	14.4	2.06
-3	65.3	1.91
-38.3	65.3	0
-29	21	0
-18.4	22.2	0.74

4. Polygonal Load: "Polygonal Load 4"

Label	Polygonal Load 4
Load Type	Flexible
Area of Load	2825.84 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
4.6	13.2	2.06
4.6	9	2.06
27.1	9	2.06
40.6	9	1.63
44.7	9	1.63
44.4	12.9	1.63
53.6	22.1	1.63
49.9	65.3	1.56
35.8	65.3	1.56
22.3	65.3	1.98
-3	65.3	1.98
3.4	14.4	2.06

5. Polygonal Load: "Polygonal Load 5"

Label	Polygonal Load 5
Load Type	Flexible
Area of Load	2319.01 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
53.6	22.1	1.63
58.7	27.1	1.46
65.8	20	1.3
88.3	19.9	0
117.9	33.6	0
91	65.3	0
49.9	65.3	1.56

6. Polygonal Load: "Polygonal Load 6"

Label	Polygonal Load 6
Load Type	Flexible
Area of Load	2280.35 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
4.6	-13.2	2.1
4.7	-53.4	2.1
30.9	-53.4	2.1
46.1	-53.4	1.59
60.2	-53.4	1.59
57.8	-24.7	1.64
46.7	-13.7	1.65
46.3	-9	1.65
42.3	-9	1.65
30.1	-9	2.1
4.6	-9	2.1

7. Polygonal Load: "Polygonal Load 7"

Label	Polygonal Load 7
Load Type	Flexible
Area of Load	1020.31 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-16.7	-20.4	0.81
-27.6	-20.4	0
-26.4	-53.4	0
4.7	-53.4	2.1
4.6	-13.2	2.1
-9.6	-27.5	1.13

8. Polygonal Load: "Polygonal Load 8"

Label	Polygonal Load 8
Load Type	Flexible
Area of Load	1171.99 ft2
Elevation	871.3 ft
Installation Stage	Stage 1 = 0 d

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
60.2	-53.4	1.59
98	-53.4	0
95	-21.1	0
68.3	-21.1	0.96
61.3	-28.2	1.44
57.8	-24.7	1.64

Excavations

1. Excavation: "Excavation 1"

Elevation	870.1 ft
Installation Stage	Stage 1 = 0 d
Load at Bottom of Excavation	2.3 ksf
Load Installation Stage	Stage 1 = 0 d

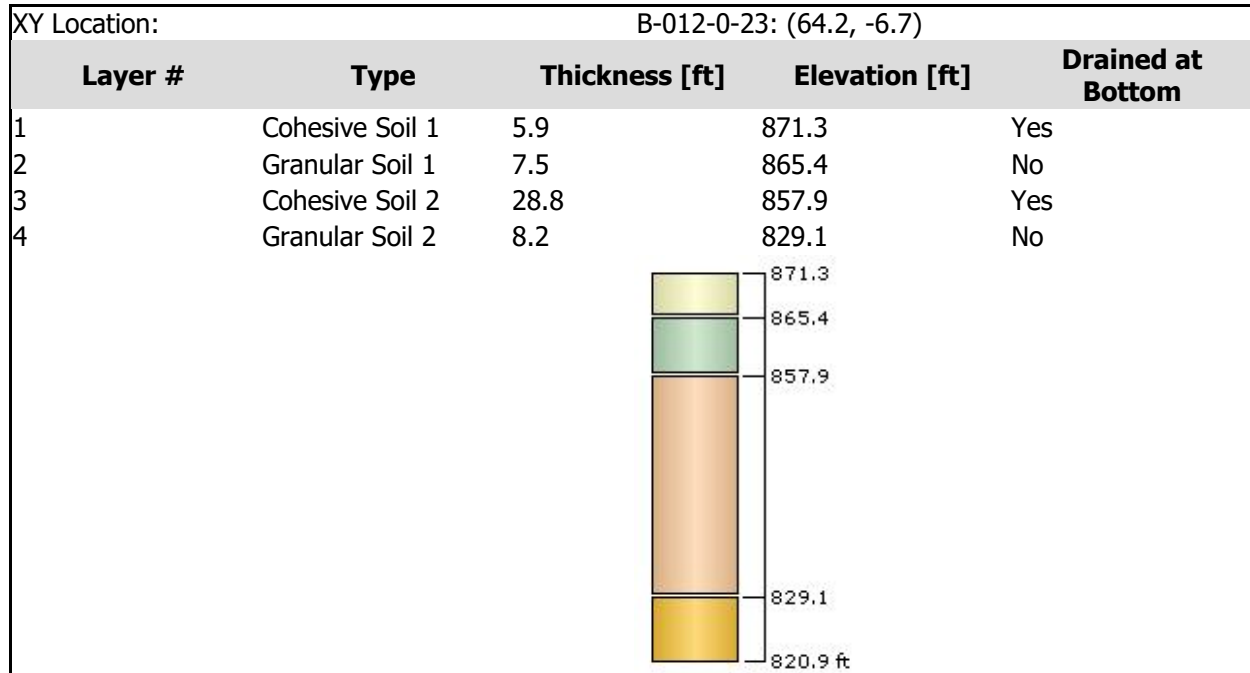
Coordinates

X [ft]		Y [ft]	
-6.88		8.5	
-6.9		-8.5	
-17.8		-19.3	
-9.6		-27.5	
4.6		-13.2	
4.6		-9	
4.6		13.2	
-11.4		29.2	
-19.5		21.1	

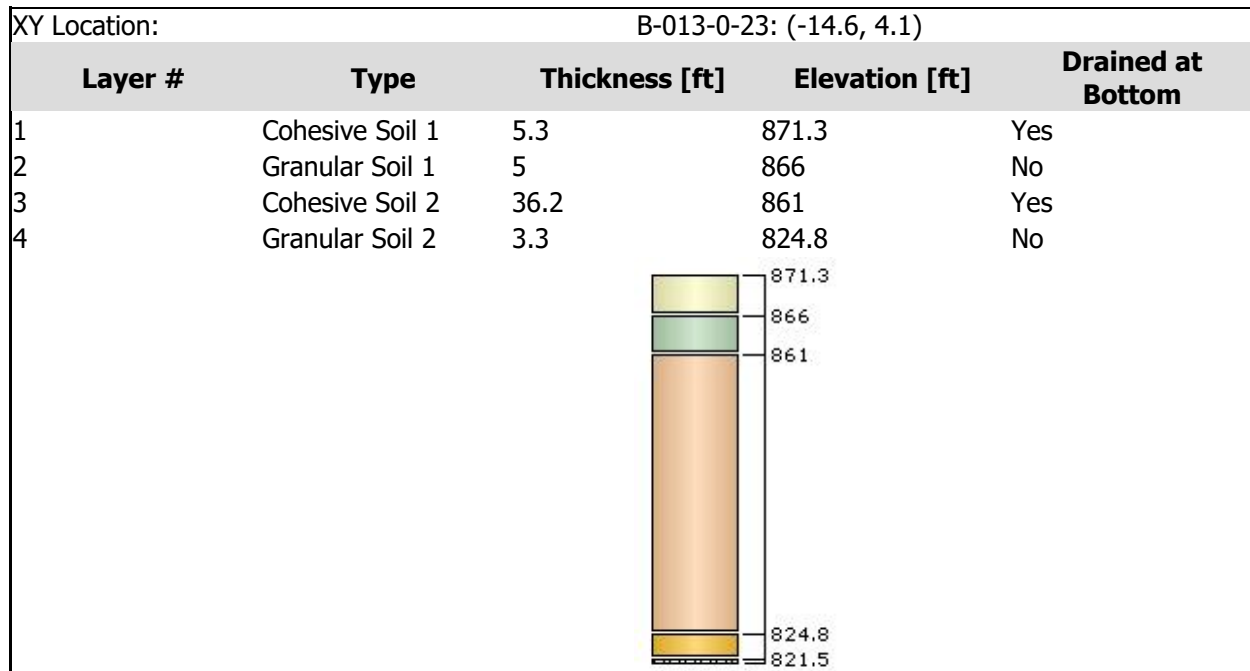
Soil Layers

Ground Surface Drained: Yes

B-012-0-23



B-013-0-23



Soil Properties

Property	Cohesive Soil 1	Granular Soil 1	Cohesive Soil 2	Granular Soil 2
Color				
Unit Weight [kips/ft ³]	0.12	0.118	0.125	0.135
Saturated Unit Weight [kips/ft ³]	0.12	0.118	0.135	0.14
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	1032	261	1866	416
Esur [ksf]	1032	261	1866	416
Primary Consolidation	Enabled	Disabled	Enabled	Disabled
Material Type	Non-Linear		Non-Linear	
Cc	0.22	-	0.05	-
Cr	0.045	-	0.007	-
e0	0.621	-	0.33	-
OCR	48	-	3	-
Cv [ft ² /d]	0.5	-	0.6	-
Cvr [ft ² /d]	0.5	-	0.6	-
B-bar	1	-	1	-
Undrained Su A [kips/ft ²]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1

Groundwater

Groundwater method

Piezometric Lines

Water Unit Weight

0.0624 kips/ft3

Generating excess pore pressure above water table

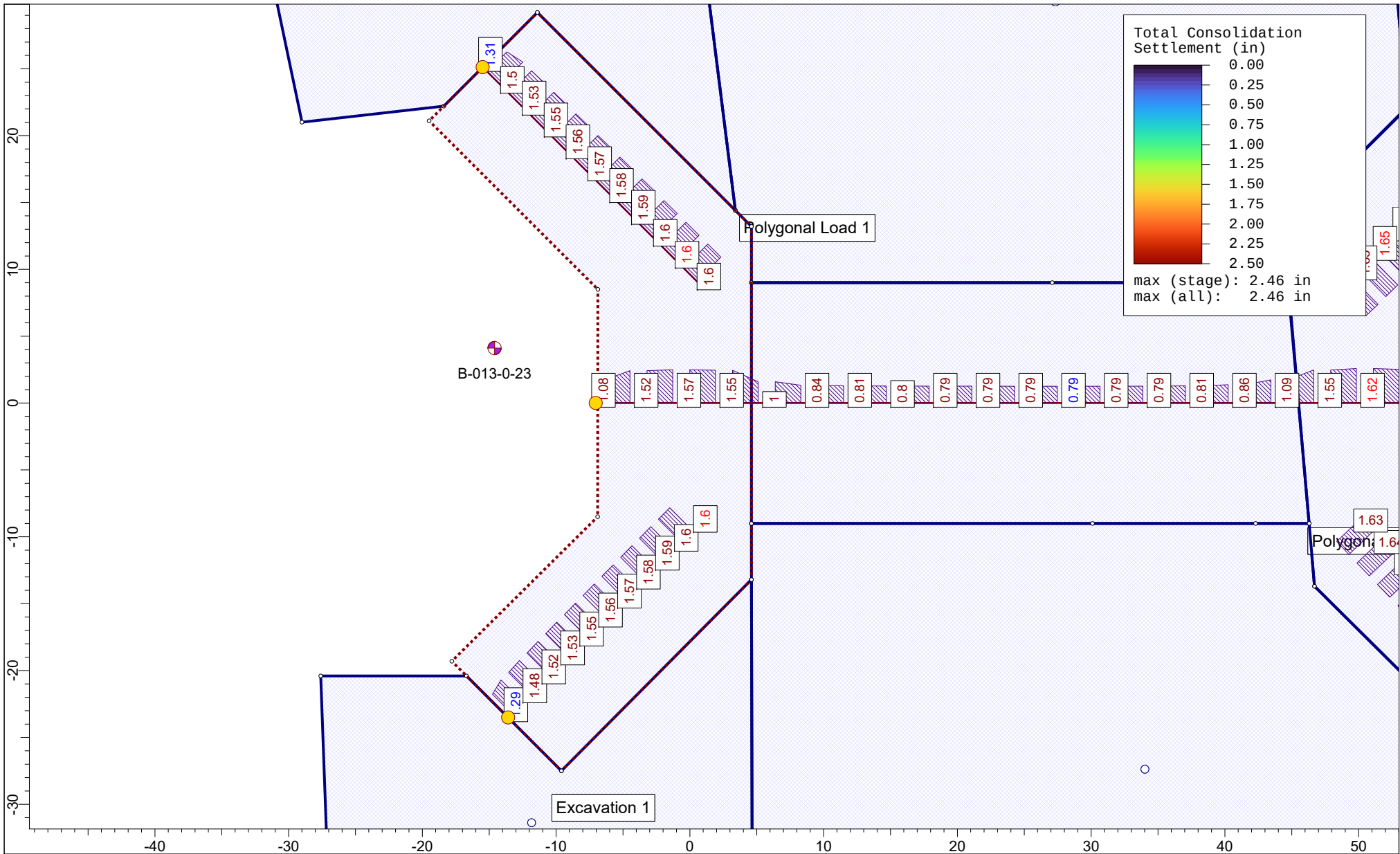
Piezometric Line Entities

ID	Elevation (ft)
1	860 ft

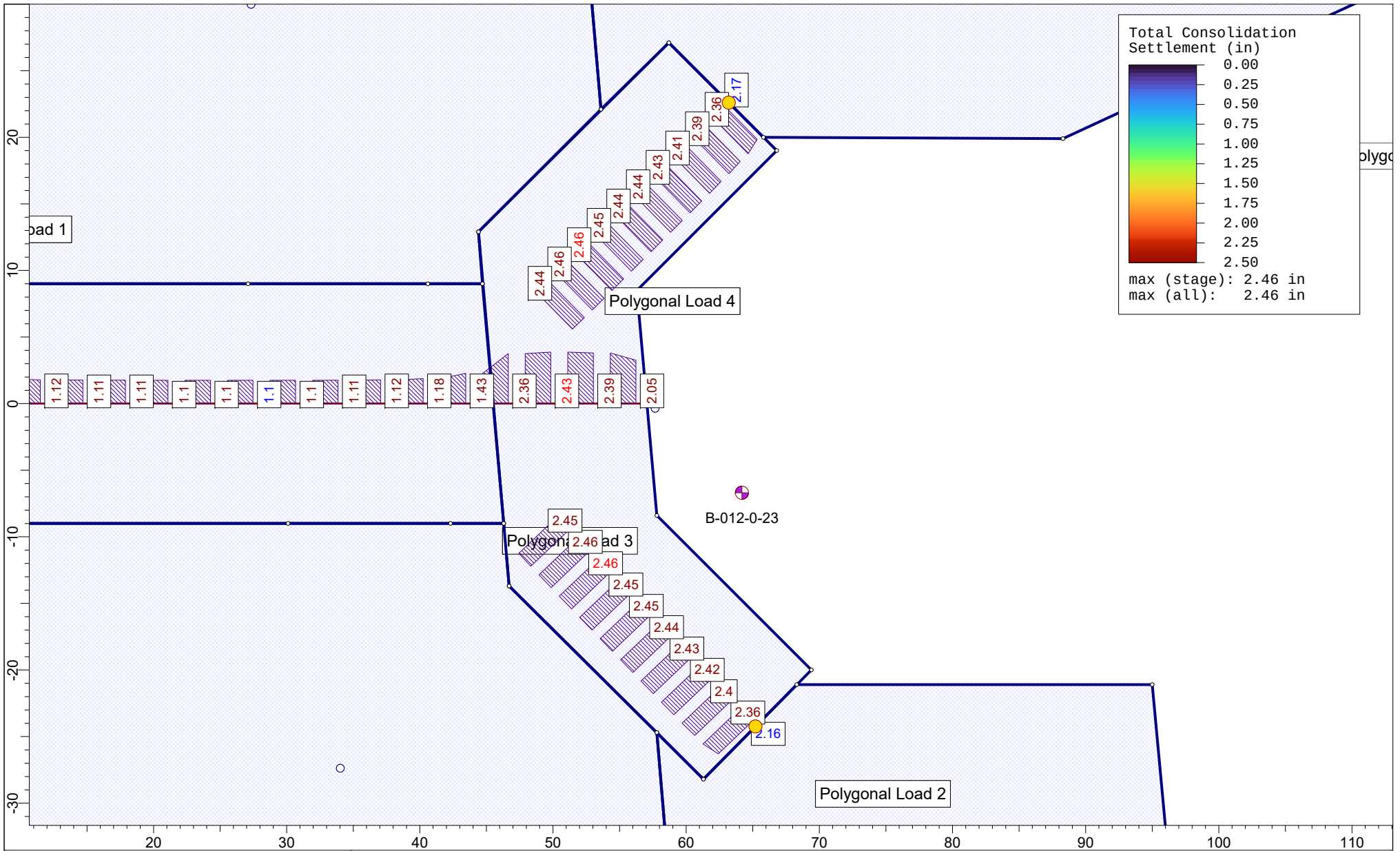
Query

Query Lines

Line #	Query Line Name	Start Location	End Location	Horizontal Divisions	Vertical Divisions
1	Query Line 1	-7.03794, - 2.22045e-16	56.8591, 2.22045e-16	20	Auto: 63
2	Query Line 2	65.2023, - 24.2495	49.944, -8.235	10	Auto: 55
3	Query Line 3	63.2048, 22.5952	48.438, 8.08	10	Auto: 55
4	Query Line 4	-15.4876, 25.1124	0.846555, 8.77855	10	Auto: 64
5	Query Line 5	-13.59, -23.51	0.575038, - 9.34504	10	Auto: 64



	Project			FRA-CEMETERY RS-SUP
	Analysis Description			Settlement at Culvert FRA-TUNNEL Under Southbound IR-270 On-Ramp
	Drawn By		Company	NEAS Inc.
	Date		File Name	CemeteryRd_Tunnel_Settle022825.s3z
KCA		2/28/2025, 12:12:02 PM		



	Project		FRA-CEMETERY RS-SUP	
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	Date		File Name	CemeteryRd_Tunnel_Settle022825.s3z
		KCA		
		2/28/2025, 12:12:02 PM		

SECTION 5
RETAINING WALL

SECTION 5: RETAINING WALL

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5.1. INTRODUCTION

5.1.1. Proposed Construction

NEAS understands that B&N is developing construction plans for the FRA-Cemetery Road SUP project. As part of the project, a new retaining wall (RW) is proposed along a portion of existing Cemetery Rd embankment slope to support the referenced path construction. The retaining wall is proposed at this location to provide grade separation between the new shared use path and the existing drainage swale that is currently downslope of Cemetery Rd.

It is our understanding that the proposed wall will be approximately 182 ft in length and about 7.3 to 10.6 ft in height. The proposed RW will be located along the southwestern edge of the proposed path alignment from approximate STA. 96+15 to approximate STA. 98+10. For analysis purposes, the proposed wall bearing elevations are anticipated to range from about 849 to 854 ft amsl. The proposed retaining wall will be constructed following a bottom-up construction sequence, and the likely wall type will be a semi-gravity cantilever cast-in-place (CIP) retaining wall, bearing on the existing natural cohesive soils encountered at the site.

5.2. GEOTECHNICAL FINDINGS

The subsurface conditions encountered during NEAS's explorations are described in the following subsections and on each boring log presented in Appendix 5A. 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, results of historical explorations, and consideration of the geological history of the site.

5.2.1. Subsurface Conditions.

5.2.1.1. Overburden Soil

At the proposed retaining wall site, two different materials were encountered within the borings underlying the existing pavement section. Those materials consisted of: 1) "man-made" fill / potential fill soils; and, 2) both cohesive and non-cohesive natural glacial till soils. These materials and the general profile are further described below.

Fill / potential fill soils were encountered in each of the borings performed at the retaining wall site extending to depths ranging from 5.5 ft to 8.0 ft bgs (elevations 851.2 to 856.2 ft amsl). Based on laboratory testing results and a visual review of the samples obtained, the fill at the site is comprised of both cohesive, fine-grained material (B-001-0-23) that is classified on the boring log as Silty Clay (A-6b), and coarse-grained material (B-002-0-23) classified as Stone Fragments (A-1-a). With respect to the soil strength of the cohesive fill soils encountered, these soils can be described as having a consistency of stiff to very stiff correlating to converted SPT-N (N_{60}) values between 11 and 17 blows per foot (bpf) and unconfined compressive strengths (estimated by means of hand penetrometer) between 2.5 and 3.25 tons per square

foot (tsf). The natural moisture content of the cohesive fill ranged from 16 to 19 percent. Based on Atterberg Limits tests performed on representative samples of the cohesive fill material, the liquid and plastic limits ranged from 38 to 40 percent and from 18 to 19 percent, respectively. With respect to the coarse-grained fill, these soils are described as having a relative compactness as medium dense correlating to N_{60} values between 22 and 28 bpf. Natural moisture content of these soils was determined to be 4 percent.

Naturally deposited glacial till soils were encountered underlying the fill soils in both borings performed at the retaining wall site and extended to the borehole termination depths of 35 ft bgs (elevations 824.2 ft amsl at B-001-0-23 and 826.7 ft amsl at B-002-0-23). The natural glacial till encountered at the site is generally comprised of cohesive, fine-grained material that is classified on the boring logs as either cohesive Sandy Silt (A-4a) or Silt and Clay (A-6a). The exception being the lower portion of the stratum encountered in boring B-001-0-23 in which the material consisted of non-cohesive, fine-and coarse-grained soil that is classified as non-cohesive Sandy Silt (A-4a) and Gravel with Sand and Silt (A-2-4). With respect to the soil strength of the cohesive natural till soils, these soils can be described as having a consistency of very stiff to hard correlating to N_{60} values between 17 and 88 bpf and unconfined compressive strengths (estimated by means of hand penetrometer) between 4.0 and 4.5 tsf. Natural moisture contents of the cohesive materials ranged from 7 to 14 percent. Based on Atterberg Limits tests performed on representative samples of the natural cohesive material, the liquid and plastic limits ranged from 21 to 28 percent and from 13 to 17 percent, respectively. With respect to the non-cohesive till, these soils are described as having a relative compactness of very dense correlating to N_{60} values between 54 and 74 bpf. Natural moisture contents of these soils ranged from 7 to 11 percent.

5.2.1.2. Groundwater

Groundwater measurements were taken during the boring drilling procedures and immediately following the completion of each borehole. Groundwater was observed during and/or upon completion of drilling in one of the borings performed as part of the referenced retaining wall exploration (B-001-0-23). Based on measurements at boring B-001-0-23 location, groundwater was encountered at depth of approximately 26 ft bgs (elevation 833.2 ft amsl).

It should be noted that groundwater is affected by many hydrologic characteristics in the area and may vary from those measured at the time of the exploration. The specific groundwater readings are included on the boring logs located within Appendix 1B.

5.3. ANALYSIS AND RECOMMENDATIONS

5.3.1. Retaining Wall Design Assumptions

As the proposed retaining wall is planned as a CIP type wall founded on the existing soil at the site, ODOT's BDM, AASHTO's LRFD BDS, and the project conditions dictate analysis parameters and design minimums/constraints to be used in the analysis and design process. The referenced parameters and design minimums/constraints that were significant to our analyses consist of the following:

- Porous backfill is to be placed from back of the wall extending from top of footing elevation to top of earth backfill with a width not less than 2 feet.
- Retained soils behind the porous backfill are to consist of material placed and compacted in accordance with Item 203, Roadway Excavation and Embankment, of the ODOT Construction and Material Specifications (CMS);

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FRA-Cemetery Road SUP
Franklin County, Ohio
PID: 120103

- Retained fill soils will meet the minimum design soil parameters per Table 307-1 of ODOT's BDM as shown in Table 1 below;

Table 1: Design Soil Parameters for Fill Materials

Fill Zone	Type of Soil	Soil Unit Weight (pcf)	Friction Angle (°)	Cohesion (psf)
Retained Soil (Soil behind the wall heel)	On-site soil varying from sandy lean clay to silty sand, per 703.16.A	120	30	0
CIP Wall Infill	Granular Embankment, per 703.16.B	120	32	0
Notes: 1. From Table 307-1 of ODOT's BDM.				

With respect to retaining wall specific design constraints and assumptions, the geometry of the proposed wall (i.e., exposed wall heights, existing ground elevations, proposed final grade behind/at the toe of the wall, etc.) are based on discussions with the design team during the preliminary design process. As the design of the proposed project progresses (i.e., alignment and elevations of the shared use path, development of cross-sections, refinement of wall alignment, bearing elevations, etc.), it is anticipated that additional analyses will be performed and recommendations for the proposed wall may need to be updated.

5.3.2. Generalized 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 was estimated based on their field (i.e., SPT N_{60} Values, hand penetrometer values, etc.) and laboratory (i.e., Atterberg Limits, grain size, etc.) test results using correlations provided in published engineering manuals, research reports and guidance documents. Engineering soil properties were estimated for each individual classified layer per boring location. Soil layers from each boring with similar behavior (i.e., cohesive or non-cohesive/granular) and characteristics (i.e., relative compactness/consistency, moisture content, etc.) were grouped into generalized soil units (i.e., Soil Types) and weighted average values of the estimated engineering soil properties were assigned to each Soil Type to develop a generalized soil profile for analysis. The summary of the generalized soil profile including designated Soil Types, elevations, average engineering soil properties per boring location are presented in Tables 2 and 3 below.

Table 2: Soil Profile and Estimated Engineering Properties - At Boring B-001-0-23

Retaining Wall at Cemetery Road: Soil Profile for Analysis, B-001-0-23					
Soil Description	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Setup Factor (f_{su})
Soil Type 2 - Cohesive Depth (859.2 ft - 851.2 ft)	122	2000	200	24	1.75
Soil Type 3 - Cohesive Depth (851.2 ft - 831.7 ft)	135	5750	395	29	1.5
Soil Type 4 - Non-cohesive Depth (831.7 ft - 824.2 ft)	135	-	-	38	1.2
1. Values calculated per Ohio Department of Transportation (ODOT) GDM Section 404/1304 and/or ODOT BDM Table 305-2.					

Table 3: Soil Profile and Estimated Engineering Properties - At Boring B-002-0-23

Retaining Wall at Cemetery Road: Soil Profile for Analysis, B-002-0-23					
Soil Description	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Setup Factor (f_{su})
Soil Type 1 - Non-cohesive Depth (861.6 ft - 854.8 ft)	128	-	-	41	1.0
Soil Type 2 - Cohesive Depth (854.8 ft - 849.8 ft)	122	2000	200	24	1.75
Soil Type 3 - Cohesive Depth (849.8 ft - 825.3 ft)	135	5750	395	29	1.5
Notes: 1. Values calculated per Ohio Department of Transportation (ODOT) GDM Section 404/1304 and/or ODOT BDM Table 305-2.					

In addition to the Soil Type parameters presented above, a graphical depiction of the generalized subsurface profile is located within Appendix 5A. The generalized subsurface profile includes: a color-coded general interpretation of the Soil Types between borings, a graphical interpretation of the soil strata identified by the project soil borings along the referenced wall profiles, representative boring data (N_{60} -values, moisture contents, and groundwater levels), current ground surface elevation and approximate proposed grades of the new shared use path.

5.3.3. Shallow Foundation Analysis

A shallow foundation bearing resistance analysis was performed at the RW-1 site in accordance with LRFD BDS Section 10.6.3.1.2c, utilizing soil properties presented above. LRFD BDS Section 10.6.3.1.2c (limit equilibrium method) was utilized as the wall segments being evaluated bear on/within the existing embankment slopes with composite layers of soil. The proposed shallow foundation system was assumed to consist of typical CIP wall bearing on the existing embankment fill soils. The analysis was performed based on preliminary wall design information provided by B&N.

For the indicated preliminary design information, NEAS developed representative cross-sectional models for various "critical" wall cross-sections. The models of proposed conditions (wall heights, proposed/existing slopes, subsurface conditions, etc.) were then inputted into slope stability software for analysis. In order to evaluate nominal bearing resistances of the proposed CIP retaining wall, specific assumptions and techniques were applied for analysis purposes. These project specific assumptions/techniques are: 1) a bearing stress is modeled to be applied over the project calculated effective footing width; 2) a maximum bearing stress is determined when a factor of safety (FOS) of 1.0 is obtained for the specific analyzed model under the specific soil conditions and slope failure type; 3) limit equilibrium failure surfaces are not allowed to develop beyond the limits of the effective footing width; and, 4) each model developed for limit equilibrium analysis was evaluated considering long-term (Effective Stress) and short-term (Total Stress) soil parameters for non-circular type failures. Utilizing the above assumptions, "back-calculation" techniques were applied to the inputted software model and analyzed to determine the maximum bearing stress (applied over the effective footing width) that would generate a FOS value of 1.0 for the referenced conditions. The "back-calculated" maximum bearing stress determined in our limit equilibrium analysis for each analyzed section equates to the nominal bearing resistance for that section. The lowest nominal bearing resistance value obtained utilizing the indicated method is the controlling nominal bearing resistance value recommended for use in design.

Limit equilibrium analysis was performed utilizing the slope stability software Slide2 by Rocscience Inc., and nominal bearing resistances were determined as indicated for non-circular type failures utilizing the Spencer analysis method. The nominal bearing resistances determined in our analyses were then multiplied by an LRFD resistance factor of 0.55 (per LRFD BDS Table 11.5.7-1) to calculate the factored bearing resistance. A summary of the bearing resistance analyses performed for the proposed wall is presented in

Table 4 located in the External Stability Section of this report. Bearing resistance calculations are presented within Appendix 5B.

5.3.3.1. External Stability

Based on our estimated engineering soil properties, the developed generalized profile and the retaining wall design assumptions provided in the above sections, an external stability analysis of the proposed wall was performed. External stability was evaluated at two (2) cross-section along the length of the proposed wall with the sections evaluated consisting of the maximum total wall height section at approximate STA. 31+81.6 (RW-1 alignment) and the tallest wall height section bearing on Soil Type 2 (as characterized in Section 5.3.2. of this report) at approximate STA. 30+00 (RW-1 alignment), based on discussions with the design regarding preliminary wall design. The referenced cross-sections were evaluated for resistance to bearing, sliding, and overturning at the Strength Limit State in accordance with Section 11.10.5 of 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 resistance are presented in Table 4 below. External Stability calculation results are included within Appendix 5C.

Table 4: External Stability Analysis Summary

Dimensions		
Design Wall Height (feet)	8.4	10.6
Exposed Wall Height (feet)	4.4	6.6
Approximate Station (Cemetery Rd)	30+00	31+81.6
Capacity Demand Ratio (CDR)		
Sliding	1.16	1.15
Overturning / Eccentricity	3.44	3.58
Bearing Resistance	1.87	1.33
Factored Bearing Resistance (ksf) ⁽¹⁾	3.50	2.97
Factored Bearing Stress (ksf)	1.89	2.24
Service Bearing Stress (ksf)	1.37	1.62
Notes: 1. Bearing Resistance calculated in accordance to Section 11.10.5.4 of 2018 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of 2018 LRFD BDS.		

5.3.3.2. Global Stability

For purposes of evaluating the stability of the proposed retaining wall site, NEAS reviewed the available 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, proposed cut into existing embankment slopes, weak or thick soil layer, etc.). Based on our review of the available information at the referenced locations and the associated soil properties, one (1) cross-section was estimated to be most "critical" and was analyzed for global stability. The cross-sections analyzed for global stability was the maximum total wall height section at approximate STA. 31+81.6 (RW-1 alignment).

For the referenced 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) preliminary cross-sections and design information provided by B&N via email at various dates throughout the preliminary design process; 2) a live load surcharge of 250 pounds per square foot (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 and Estimated Engineering Properties presented in Section 5.3.2. 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 Slide2 by Rocscience, Inc. Specifically, the Modified Bishop and Spencer analysis methods were used to calculate a factor of safety (FOS) for circular and block type slope failures, respectively. The FOS is the ratio of the resisting forces and the driving forces, with the desired safety factor being more than about 1.33 which equates to an AASHTO resistance factor less than 0.75 (per AASHTO's LRFD BDS the specified resistance factors are essentially the inverse of the FOS that should be targeted in slope stability programs). For this analysis, a resistance factor of 0.75 or lower is targeted as the slope does not contain or support a structural element.

Based on our slope stability analysis for the referenced retaining wall section, the minimum slope stability safety factor is about 2.03 (0.49 resistance factor). The graphical output of the slope stability program (cross-sectional model, calculated safety factor, and critical failure plane) is included within Appendix 5D.

5.3.3.3. Settlement

Settlement is not anticipated to be a concern at either wall location based on: 1) the over-consolidated nature of the glacial till soils encountered at the site; 2) the relatively uniform nature of the encountered soils underlying the proposed wall; and, 2) the fact that a majority of the proposed wall will be located within the existing slope and therefore the increase in loading is minimal. Potential settlement is not anticipated to be a concern as it is reasonable to assume that the settlement will be relatively minimal in magnitude, will be relatively uniform across the length of the wall and will take place over the life of the wall. As the design of the proposed wall is more refined, if it is determined that proposed fill heights may indicate the potential for relatively significant settlement, an evaluation of the settlement will be performed.

5.3.3.4. Temporary Excavations

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

5.3.3.5. Drainage Considerations

It is recommended that adequate drainage is maintained/controlled during and after construction of the retaining wall, and that trail/roadway drainage is carefully controlled around the wall location in order to prevent ponding, erosion of retained backfill soil, loss of shear strength of foundation soils due to saturation, and other drainage related issues.

It is recommended that internal drainage of the retaining wall be designed to provide positive drainage behind the wall and limit the buildup of hydrostatic pressure. Furthermore, it is recommended that the barrier or curb at the roadway extend at least 25 ft beyond wall limits, and outlet to a piped collection system

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(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.

APPENDIX 5A

GENERALIZED SUBSURFACE PROFILE

RETAINING WALL

STRATIGRAPHY & GW - B SIZE - OH DOT.GDT - 9/25/24 09:41 - P:\24-0005 HILLIARD CEMETERY ROAD (B&N)\120103\GEOTECHNICAL\RETAINING WALL X (LOCATION\DESCRIPTION)\ANALYSIS\GINT PROFILE\CEMETERY_RD_81224.GPJ



OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

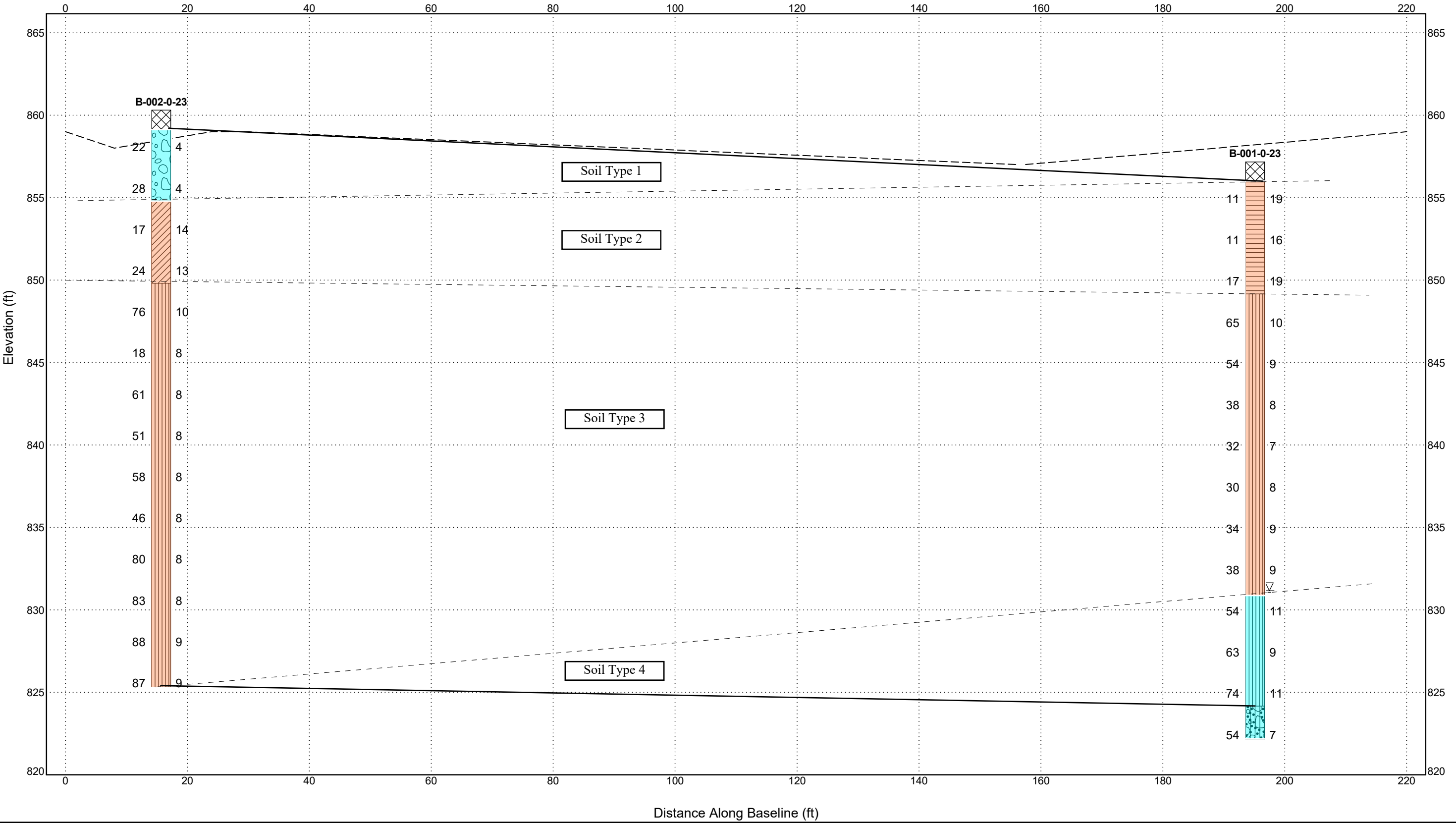
CLIENT _____
PROJECT NUMBER _____

 Cohesive
 Non-cohesive

PROJECT NAME _____
PROJECT LOCATION _____

SUBSURFACE DIAGRAM

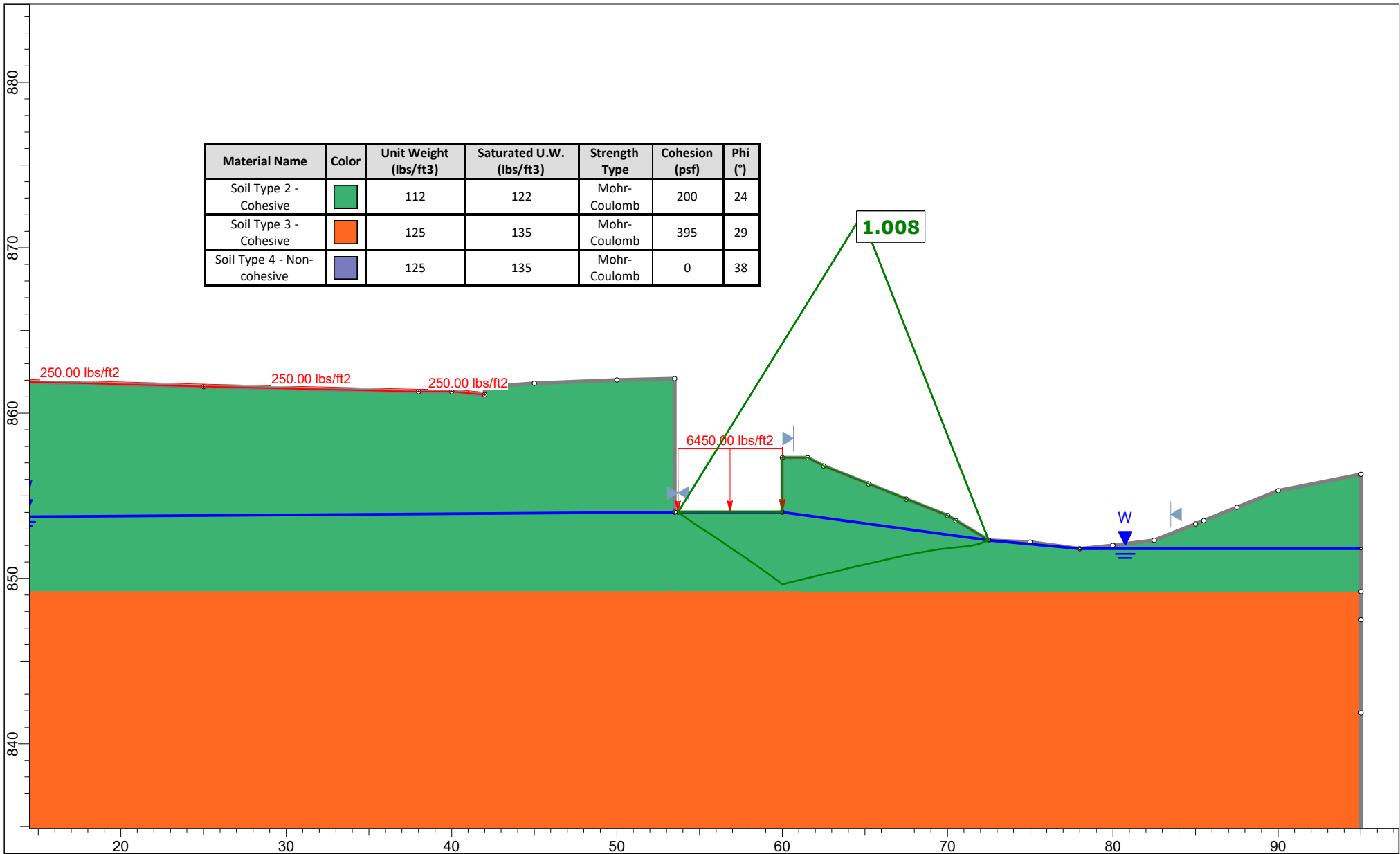
-  Ohio DOT: Pavement or Aggregate base
-  Ohio DOT: A-2-4, gravel and/or stone fragments with sand and silt
-  Ohio DOT: A-6b, silty clay
-  Ohio DOT: A-1-a, gravel and/or stone fragments
-  Ohio DOT: A-4a, sandy silt
-  Ohio DOT: A-6a, silt and clay



APPENDIX 5B

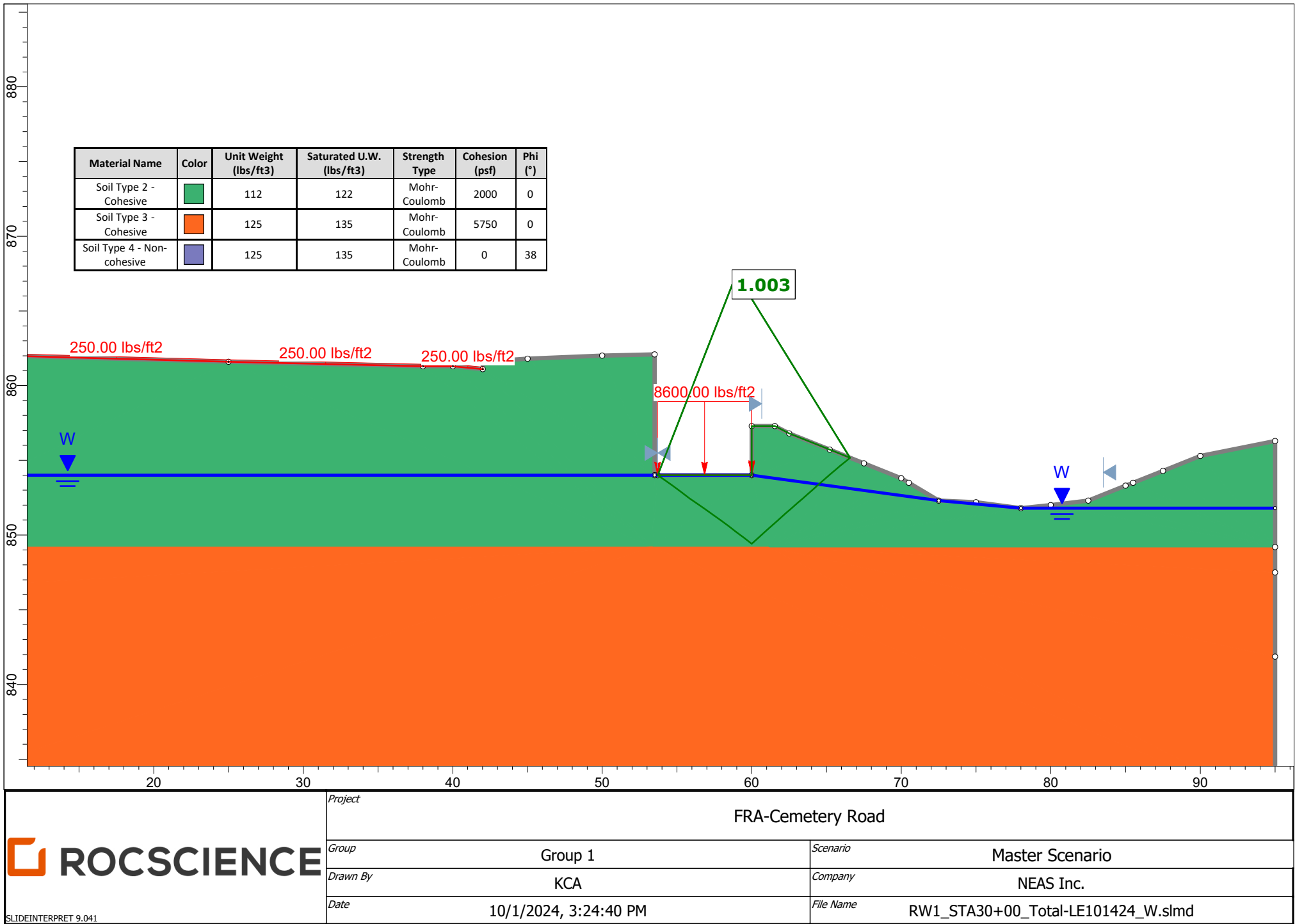
LIMIT EQUILIBRIUM ANALYSIS

RETAINING WALL AT STA. 30+00

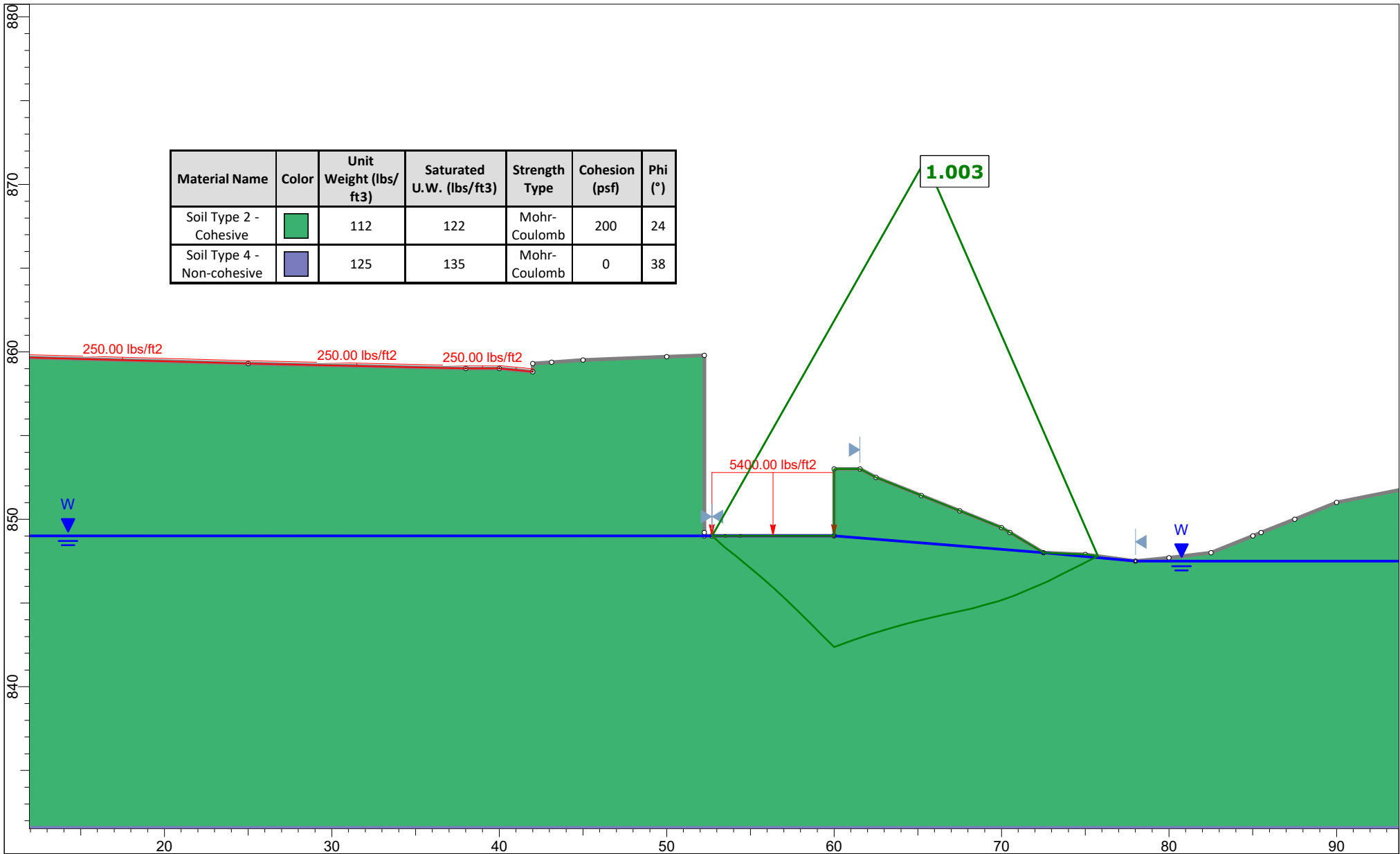


SLIDEINTERPRET 9.041

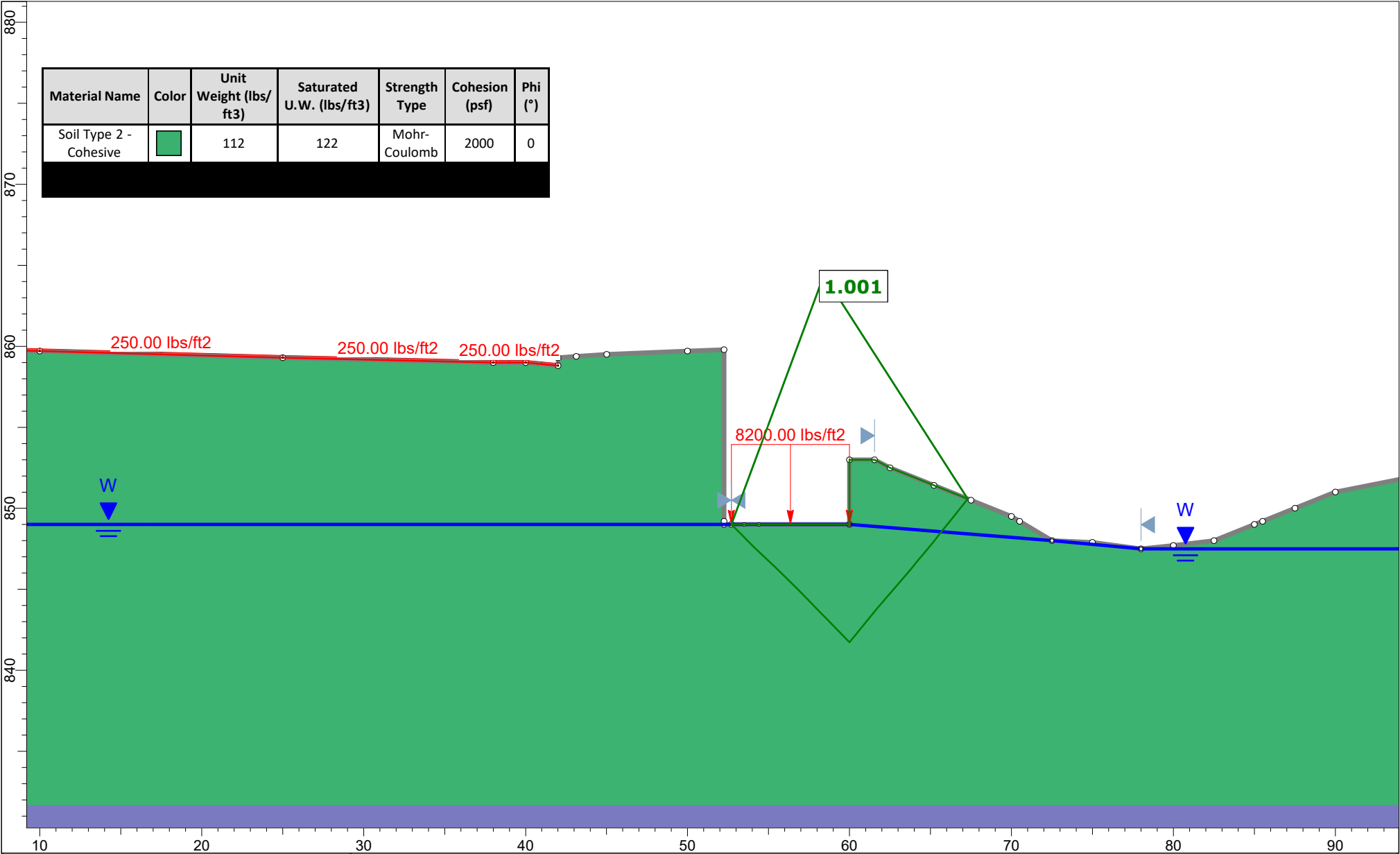
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Group		Group 1	Scenario Master Scenario
Drawn By		KCA	Company NEAS Inc.
Date		10/1/2024, 3:24:40 PM	File Name RW1_STA30+00_Eff-LE101424_W.slmd



RETAINING WALL AT STA. 31+81.6



 ROCSCIENCE	Project		FRA-Cemetery Road	
	Group		Group 1	Scenario
	Drawn By		KCA	Company
	Date		10/1/2024, 3:24:40 PM	NEAS Inc.
	File Name		RW1_STA31+81.6_Eff-LE020226_W_new.slmd	



Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Soil Type 2 - Cohesive		112	122	Mohr-Coulomb	2000	0



SLIDEINTERPRET 9.041

Project	FRA-Cemetery Road				
Group	Group 1		Scenario	Master Scenario	
Drawn By	KCA		Company	NEAS Inc.	
Date	10/1/2024, 3:24:40 PM		File Name	RW1_STA31+81.6_Total-LE020226_W_new.slmd	

APPENDIX 5C

EXTERNAL STABILITY ANALYSIS

RETAINING WALL AT STA. 30+00

Objective: To evaluate the external stability of CIP wall's with level backfill (no backslope).
Method: In accordance with ODOT Bridge Design Manual, 2019 [Sect. 204.6.2.2] LRFD Bridge Design Specifications, 8th Ed., Nov. 2017, [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:

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

$$\delta_{fd} := 0.67 \cdot \phi'_{fd}$$

$$\delta_{fd} = 16.1 \text{ deg}$$

Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Undrained Conditions (Total Stress):

$$\phi_{fdu} := 0 \text{ deg}$$

Angle of internal friction (Same as Drained Conditions if granular soils)

$$\gamma_{fd} = 112 \frac{\text{lbf}}{\text{ft}^3}$$

Unit weight

$$Su_{fdu} := 2000 \frac{\text{lbf}}{\text{ft}^2}$$

Undrained Shear Strength

$$\delta_{fdu} := 0.67 \cdot \phi_{fdu}$$

$$\delta_{fdu} = 0 \text{ deg}$$

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)

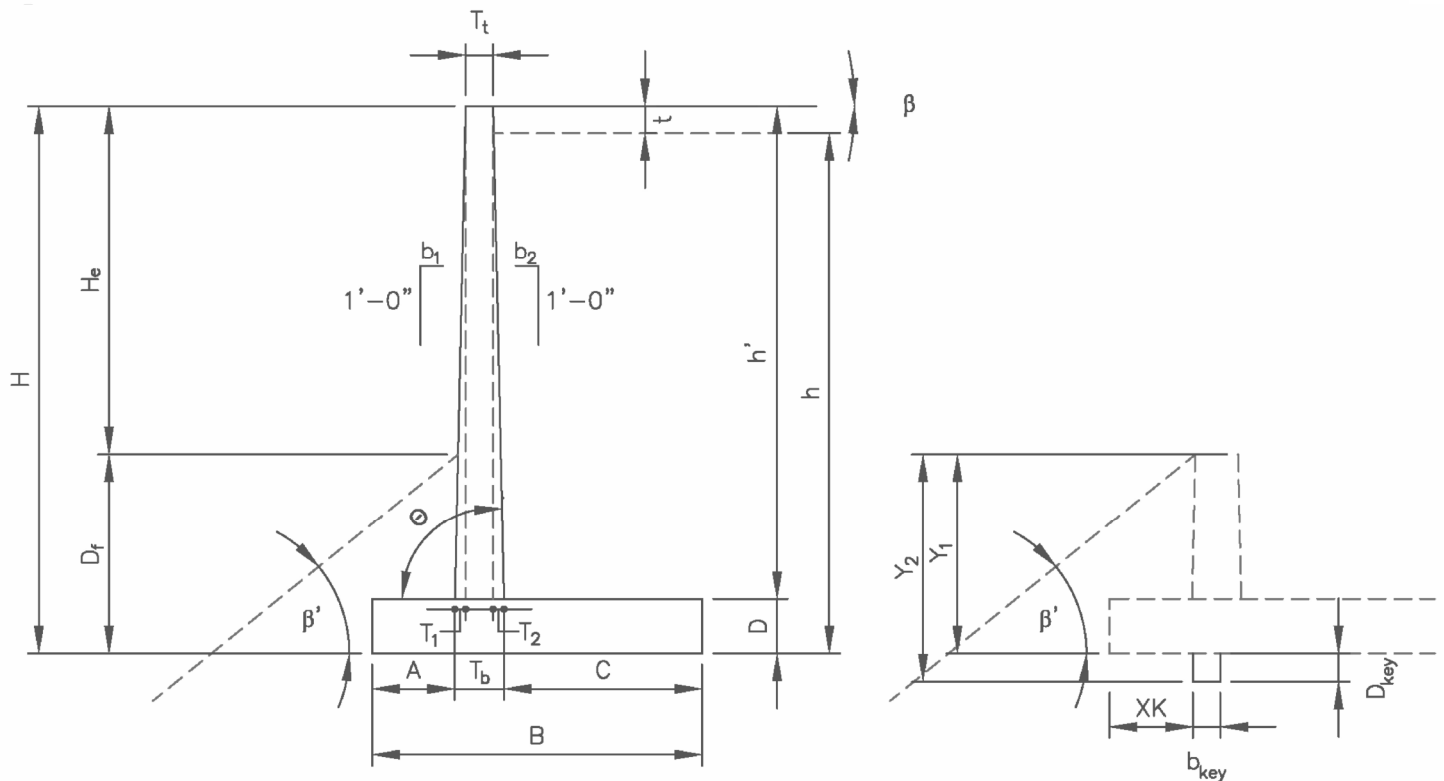
Other Parameters:

$$\gamma_c := 150 \frac{\text{lbf}}{\text{ft}^3}$$

Concrete Unit weight

$$\gamma_p := 150 \frac{\text{lbf}}{\text{ft}^3}$$

Pavement Unit weight



Wall Geometry:

$$H_e := 4.4 \text{ ft}$$

$$D_f := 4 \text{ ft}$$

$$H := H_e + D_f$$

$$H = 8.4 \text{ ft}$$

$$T_t := 16 \text{ in}$$

$$b_1 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

$$b_2 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

$$\beta := 0 \text{ deg}$$

$$\beta' := 26.565 \text{ deg}$$

$$t := 0 \text{ in}$$

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.

Pavement thickness

Preliminary Wall Dimensioning:

$B := 6.5 \text{ ft}$	$\frac{2}{5} \cdot H = 3.36 \text{ ft}$	to	$\frac{3}{5} \cdot H = 5.04 \text{ ft}$	Footing base width (2/5H to 3/5H)
$A := 8 \cdot \text{in}$	$\frac{H}{8} = 1.05 \text{ ft}$	to	$\frac{H}{5} = 1.68 \text{ ft}$	Toe projection (H/8 to H/5)
$D := 1.5 \text{ ft}$	$\frac{H}{8} = 1.05 \text{ ft}$	to	$\frac{H}{5} = 1.68 \text{ ft}$	Footing thickness (H/8 to H/5)

Shear Key Dimensioning:

$D_{key} := 0 \text{ ft}$	Depth of shear key from bottom of footing
$b_{key} := 0 \text{ ft}$	Note: Footings on rock typically require shear key Width of shear key
$XK := A$	Distance from toe to shear key

Other Wall Dimensions:

$h' := H - D$	$h' = 6.9 \text{ ft}$	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
$T_b := T_1 + T_2 + T_t$	$T_b = 1.333 \text{ ft}$	Stem thickness at bottom of wall
$C := B - A - T_b$	$C = 4.5 \text{ ft}$	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)
$y_1 := 4 \cdot \text{ft}$	$y_1 = 4 \text{ ft}$	Depth to where passive pressure may begin to be utilized in front of wall. (Typically Df)
$y_2 := D_f + D_{key}$	$y_2 = 4 \text{ ft}$	Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.
$h := H - t$	$h = 8.4 \text{ ft}$	Height of retained fill at back of heel

Live Load Surcharge Parameters:

$\lambda := 0 \text{ ft}$	Horizontal distance from the back of the wall to point of traffic surcharge load
$SUR := \text{if} \left(\lambda < \frac{H}{2}, 250 \frac{\text{lb}}{\text{ft}^2}, 100 \frac{\text{lb}}{\text{ft}^2} \right) = 250 \frac{\text{lb}}{\text{ft}^2}$	Live load surcharge (per LRFD BDS [3.11.6.4]) Note: 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

$$\frac{-\beta'}{\phi'_{fd}} = -1.107$$

$$\frac{-\delta_{fd}}{\phi'_{fd}} = -0.67$$

$$k'_p := 1.66$$

Passive Earth Pressure Coefficient
from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.848$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p$$

$$k_{pd} = 1.408$$

Passive Earth Pressure Coefficient for
Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

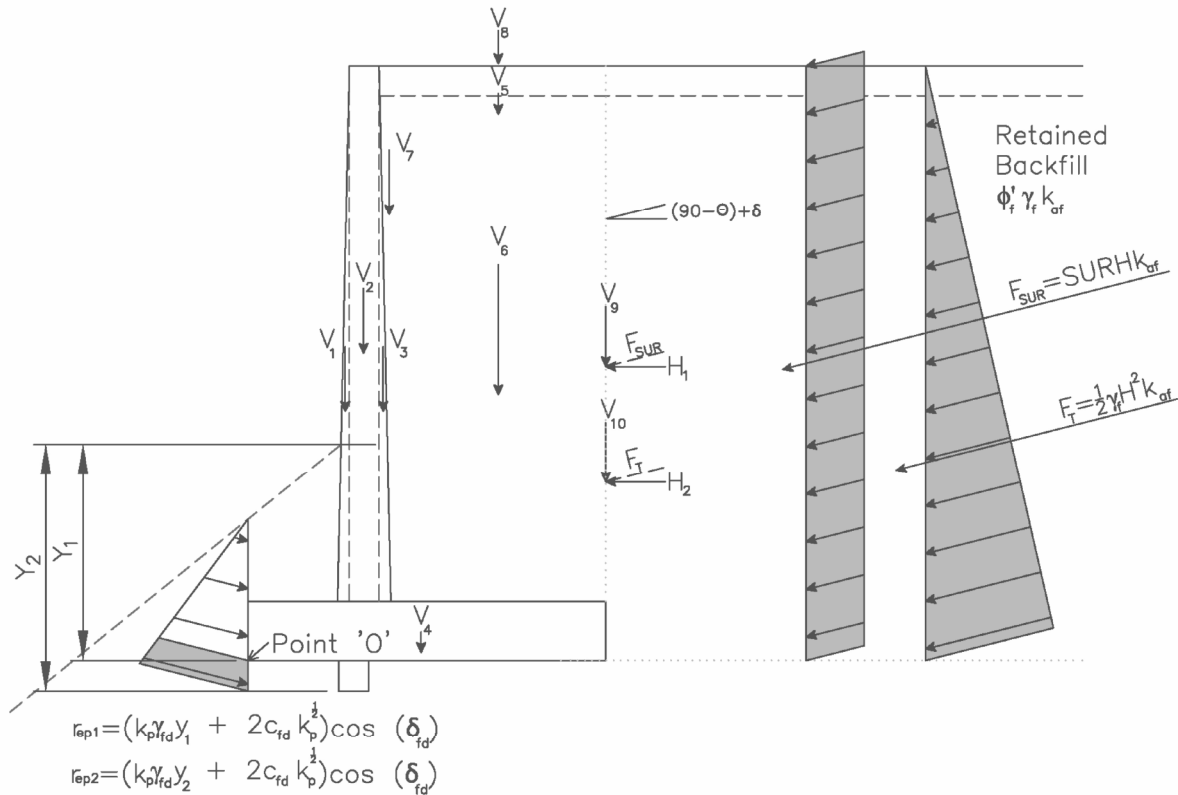
Undrained Conditions:

$$k_{pu} := \text{if}(\phi_{fdu} > 0, k_{pu}, 1)$$

$$k_{pu} = 1$$

Passive Earth Pressure Coefficient for
Resistance Undrained Conditions

Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:



$$F_T := \frac{1}{2} \cdot \gamma_f \cdot H^2 \cdot k_{af}$$

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

Vertical Loads:

$$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$$

$$V_2 := T_1 \cdot h' \cdot \gamma_c$$

$$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$$

$$V_4 := D \cdot B \cdot \gamma_c$$

$$V_5 := t \cdot (T_2 + C) \cdot \gamma_p$$

$$V_6 := C \cdot (h' - t) \cdot \gamma_f$$

$$V_7 := \frac{1}{2} \cdot b_2 \cdot (h' - t)^2 \cdot \gamma_f$$

$$V_8 := SUR \cdot (T_2 + C)$$

$$V_9 := F_{SUR} \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{10} := F_T \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$F_T = 1258.5 \frac{\text{lbf}}{\text{ft}}$$

$$F_{SUR} = 624.2 \frac{\text{lbf}}{\text{ft}}$$

$$V_1 = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_2 = 1380 \frac{\text{lbf}}{\text{ft}}$$

$$V_3 = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_4 = 1462.5 \frac{\text{lbf}}{\text{ft}}$$

$$V_5 = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_6 = 3726 \frac{\text{lbf}}{\text{ft}}$$

$$V_7 = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_8 = 1125 \frac{\text{lbf}}{\text{ft}}$$

$$V_9 = 214.5 \frac{\text{lbf}}{\text{ft}}$$

$$V_{10} = 432.5 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (EH)

Live Load Surcharge (LS)

Wall stem front batter (DC)

Wall stem (DC)

Wall stem back batter (DC)

Wall Footing (DC)

Pavement (DC)

Soil Backfill - Heel (EV)

Soil Backfill - Batter (EV)

Live Load Surcharge above Heel- (LS)
- Strength lb

Live Load Surcharge Resultant (vertical
comp. - LS) - Strength lb

Active earth force resultant (vertical
component - EH)

Moment Arm:

Moments produced from vertical loads about Point 'O'

$$d_{v1} := A + \frac{2}{3} \cdot T_1 = 0.7 \text{ ft}$$

$$d_{v2} := A + T_1 + \frac{T_1}{2} = 1.3 \text{ ft}$$

$$d_{v3} := A + T_1 + T_1 + \frac{T_2}{3} = 2 \text{ ft}$$

$$d_{v4} := \frac{B}{2} = 3.3 \text{ ft}$$

$$d_{v5} := B - \frac{T_2 + C}{2} = 4.3 \text{ ft}$$

$$d_{v6} := B - \frac{C}{2} = 4.3 \text{ ft}$$

$$d_{v7} := A + T_1 + T_1 + \left(\frac{2}{3} \cdot b_2 \cdot (h' - t) \right) = 2 \text{ ft}$$

$$d_{v8} := B - \frac{T_2 + C}{2} = 4.3 \text{ ft}$$

$$d_{v9} := B = 6.5 \text{ ft}$$

$$d_{v10} := B = 6.5 \text{ ft}$$

Horizontal Loads:

$$H_1 := F_{SUR} \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_1 = 586.2 \frac{\text{lbf}}{\text{ft}}$$

$$H_2 := F_T \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_2 = 1181.8 \frac{\text{lbf}}{\text{ft}}$$

Moment Arm:

$$d_{h1} := \frac{H}{2} \quad d_{h1} = 4.2 \text{ ft}$$

$$d_{h2} := \frac{H}{3} \quad d_{h2} = 2.8 \text{ ft}$$

Unfactored Loads by Load Type:

$$V_{DC} := V_1 + V_2 + V_3 + V_4 + V_5 \quad V_{DC} = 2842.5 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_1a} := V_9 \quad V_{LS_1a} = 214.5 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_{10} \quad V_{EH} = 432.5 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_2 \quad H_{EH} = 1181.8 \frac{\text{lbf}}{\text{ft}}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1} = 0 \text{ lbf}$$

$$MV_2 := V_2 \cdot d_{v2} = 1840 \text{ lbf}$$

$$MV_3 := V_3 \cdot d_{v3} = 0 \text{ lbf}$$

$$MV_4 := V_4 \cdot d_{v4} = 4753.1 \text{ lbf}$$

$$MV_5 := V_5 \cdot d_{v5} = 0 \text{ lbf}$$

$$MV_6 := V_6 \cdot d_{v6} = 15835.5 \text{ lbf}$$

$$MV_7 := V_7 \cdot d_{v7} = 0 \text{ lbf}$$

$$MV_8 := V_8 \cdot d_{v8} = 4781.3 \text{ lbf}$$

$$MV_9 := V_9 \cdot d_{v9} = 1394.4 \text{ lbf}$$

$$MV_{10} := V_{10} \cdot d_{v10} = 2811.2 \text{ lbf}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Active Earth Force Resultant (horizontal comp. - EH)

Moment:

$$MH_1 := H_1 \cdot d_{h1} \quad MH_1 = 2462.1 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MH_2 := H_2 \cdot d_{h2} \quad MH_2 = 3309.1 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$V_{EV} := V_6 + V_7 \quad V_{EV} = 3726 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_1b} := V_8 + V_9 \quad V_{LS_1b} = 1339.5 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_1 \quad H_{LS} = 586.2 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4 + MV_5$$

$$M_{DC} = 6593.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EV} := MV_6 + MV_7$$

$$M_{EV} = 15835.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ia} := MV_8$$

$$M_{LSV_Ia} = 1394.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ib} := MV_8 + MV_9$$

$$M_{LSV_Ib} = 6175.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_{10}$$

$$M_{EH1} = 2811.2 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSH} := MH_1$$

$$M_{LSH} = 2462.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_2$$

$$M_{EH2} = 3309.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

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)

$$Ia_{DC} := 0.9$$

$$Ia_{EV} := 1$$

$$Ia_{EH} := 1.5$$

$$Ia_{LS} := 1.75$$

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:

$$V_{Ia} := \eta \cdot ((Ia_{DC} \cdot V_{DC}) + (Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + (Ia_{LS} \cdot V_{LS_Ia})) \quad V_{Ia} = 7308.4 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot ((Ib_{DC} \cdot V_{DC}) + (Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + (Ib_{LS} \cdot V_{LS_Ib})) \quad V_{Ib} = 11576.1 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot ((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH})) \quad H_{Ia} = 2798.6 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot ((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH})) \quad H_{Ib} = 2798.6 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot ((Ia_{DC} \cdot M_{DC}) + (Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + (Ia_{LS} \cdot M_{LSV_Ia})) \quad MV_{Ia} = 28426.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot ((Ib_{DC} \cdot M_{DC}) + (Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + (Ib_{LS} \cdot M_{LSV_Ib})) \quad MV_{Ib} = 44643.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot ((Ia_{LS} \cdot M_{LSH}) + (Ia_{EH} \cdot M_{EH2})) \quad MH_{Ia} = 9272.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot ((Ib_{LS} \cdot M_{LSH}) + (Ib_{EH} \cdot M_{EH2})) \quad MH_{Ib} = 9272.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength lb:

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 44643.5 \frac{lb \cdot ft}{ft}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 9272.5 \frac{lb \cdot ft}{ft}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 11576.1 \frac{lb}{ft}$	Sum of Vertical Loads (Strength lb)
$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$	$x = 3.1 \text{ ft}$	Distance from Point "O" the resultant intersects the base
$e := \left \frac{B}{2} - x \right $	$e = 0.19 \text{ ft}$	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. When the foundation eccentricity is negative the absolute value is used.
$B' := B - 2e = 6.1 \text{ ft}$		

Drained nominal bearing resistance from limit equilibrium analysis (**Attached**):

$q_{nd} := 6450 \text{ psf}$	$q_{nd} = 6450 \frac{lb}{ft^2}$
------------------------------	---------------------------------

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

$\phi_b := 0.55$	Bearing resistance factor LRFD Table 11.5.7-1.
$q_{Rd} := \phi_b \cdot q_{nd}$	$q_{Rd} = 3.5 \text{ ksf}$
Factored bearing resistance Drained Conditions	

Undrained nominal bearing resistance from limit equilibrium analysis (**Attached**):

$q_{nu} := 8600 \text{ psf}$	$q_{nu} = 8600 \frac{lb}{ft^2}$
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Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]**:

$\phi_b := 0.55$	Bearing resistance factor LRFD Table 11.5.7-1.
$q_{Ru} := \phi_b \cdot q_{nu}$	$q_{Ru} = 4.7 \text{ ksf}$
Factored bearing resistance Undrained Conditions	

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions:	$q_{Rd} = 3.5 \text{ ksf}$
Undrained Conditions:	$q_{Ru} = 4.7 \text{ ksf}$

Evaluate External Stability of Wall:

Compute the bearing stress :

$$e = 0.19 \text{ ft}$$

$$\sigma := \frac{V_{DC} + V_{EV} + V_{EH} + V_{LS_lb}}{B - 2 \cdot e}$$

$$\sigma = 1.365 \text{ ksf}$$

Nominal Bearing Stress

$$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e}$$

$$\sigma_V = 1.894 \text{ ksf}$$

Factored Bearing Stress

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{Bearing_D} := \frac{q_{Rd}}{\sigma_V}$

Is the CDR > or = to 1.0?

$$CDR_{Bearing_D} = 1.87$$

Undrained Conditions: $CDR_{Bearing_U} := \frac{q_{Ru}}{\sigma_V}$

Is the CDR > or = to 1.0?

$$CDR_{Bearing_U} = 2.50$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{B}{3}$$

$$e_{max} = 2.2 \text{ ft}$$

Maximum Eccentricity **LRFD [11.6.3.3.]**
Equals B/3 for soil.

$$\Sigma M_R := MV_{Ia}$$

$$\Sigma M_R = 28426.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia}$$

$$\Sigma M_O = 9272.5 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 7308.4 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 2.6 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \text{abs} \left(\frac{B}{2} - x \right)$$

$$e = 0.63 \text{ ft}$$

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?

$$CDR_{Eccentricity} = 3.44$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$R_u := H_{Ia} \quad R_u = 2798.6 \frac{\text{lb}}{\text{ft}}$$

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

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 7308.4 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_\tau := c \cdot \Sigma V \cdot \tan(\phi'_{fd})$$

$$R_\tau = 3253.9 \frac{\text{lb}}{\text{ft}}$$

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:

$$R_R = 3253.913 \frac{\text{lb}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding} = 1.16$$

Undrained Conditions (Total Stress):

Compute passive resistance throughout the design life of the wall **LRFD [Eq 3.11.5.4-1]**:

$$r_{ep1} := (k_{pu} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot Su_{fdu} \cdot \sqrt{k_{pu}}) \cdot \cos(\delta_{fd})$$

Nominal passive pressure at y1

$$r_{ep2} := (k_{pu} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot Su_{fdu} \cdot \sqrt{k_{pu}}) \cdot \cos(\delta_{fd})$$

Nominal passive pressure at y2

$$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot (y_2 - y_1) \quad R_{ep} = 0 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

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

$$\Sigma V := V_{Ia} \quad \Sigma V = 7308.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$e = 0.63 \text{ ft}$$

Wall eccentricity, Calculated in above Limiting Eccentricity at Base of Wall (Strength Ia) Section.

$$B = 6.5 \text{ ft}$$

Footing base width

$$\frac{B}{6} = 1.1 \text{ ft}$$

If e < B/6 the resultant is in the middle one-third

$$\sigma_{vmax} := \frac{\Sigma V}{B} \cdot \left(1 + 6 \cdot \frac{e}{B}\right) \quad \sigma_{vmax} = 1777.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}^2}$$

Max vertical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$$\sigma_{vmin} := \frac{\Sigma V}{B} \cdot \left(1 - 6 \cdot \frac{e}{B}\right) \quad \sigma_{vmin} = 471.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}^2}$$

Max vertical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$$q_{max} := \frac{1}{2} \cdot \sigma_{vmax} \quad q_{max} = 888.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}^2}$$

Max unit shear resistance as 1/2 max vertical stress **LRFD [10.6.3.4]**.

$$q_{min} := \frac{1}{2} \cdot \sigma_{vmin} \quad q_{min} = 235.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}^2}$$

Minimum unit shear resistance as 1/2 minimum vertical stress **LRFD [10.6.3.4]**.

Determine which Cohesive Soil Resistance Case is Present:

$$\text{Case}_1 := \text{if}(q_{max} > Su_{fdu} > q_{min} \geq 0, 1, 0) \quad \text{Case}_1 = 0$$

$$\text{Case}_2 := \text{if}(Su_{fdu} > q_{max} > q_{min} \geq 0, 1, 0) \quad \text{Case}_2 = 1$$

$$\text{Case}_3 := \text{if}(q_{max} > q_{min} > Su_{fdu}, 1, 0) \quad \text{Case}_3 = 0$$

$$\text{Case}_4 := \text{if}(q_{min} < 0, \text{if}(Su_{fdu} < q_{max}, 1, 0), 0) \quad \text{Case}_4 = 0$$

$$\text{Case}_5 := \text{if}(q_{min} < 0, \text{if}(Su_{fdu} > q_{max}, 1, 0), 0) \quad \text{Case}_5 = 0$$

Unit Shear Resistance for Case 1:

$$S_1 := Su_{fdu} - q_{min} = 1764.3 \frac{\text{lbf}}{\text{ft}^2}$$

$$S_2 := q_{min} = 235.7 \frac{\text{lbf}}{\text{ft}^2}$$

$$B_1 := \frac{B \cdot (Su_{fdu} - q_{min})}{q_{max} - q_{min}} = 17.6 \text{ ft}$$

$$B_2 := \frac{B \cdot (q_{max} - Su_{fdu})}{q_{max} - q_{min}} = -11.1 \text{ ft}$$

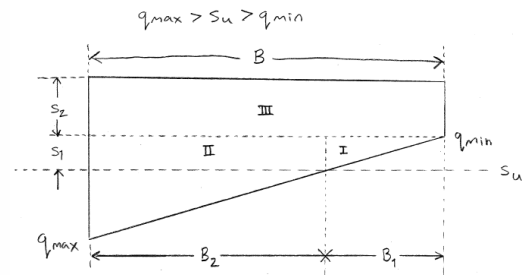
$$B_3 := B = 6.5 \text{ ft}$$

$$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 15492 \frac{\text{lbf}}{\text{ft}}$$

$$II := S_1 \cdot B_2 = -19515.9 \frac{\text{lbf}}{\text{ft}}$$

$$III := S_2 \cdot B_3 = 1531.8 \frac{\text{lbf}}{\text{ft}}$$

$$R_{\tau_case1} := I + II + III = -2492 \frac{\text{lbf}}{\text{ft}}$$



Unit Shear Resistance for Case 2:

$$S_1 := q_{max} - q_{min} = 653 \frac{\text{lbf}}{\text{ft}^2}$$

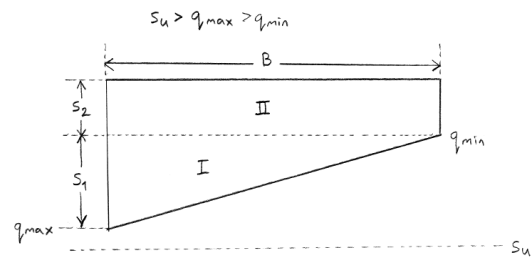
$$S_2 := q_{min} = 235.7 \frac{\text{lbf}}{\text{ft}^2}$$

$$B = 6.5 \text{ ft}$$

$$I := \frac{1}{2} \cdot S_1 \cdot B = 2122.4 \frac{\text{lbf}}{\text{ft}}$$

$$II := S_2 \cdot B = 1531.8 \frac{\text{lbf}}{\text{ft}}$$

$$R_{\tau_case2} := I + II = 3654.2 \frac{\text{lbf}}{\text{ft}}$$



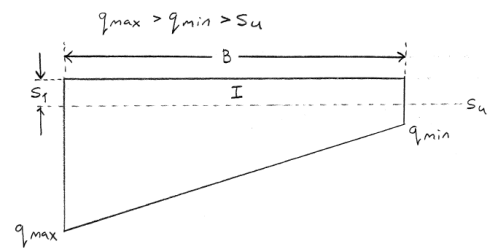
Unit Shear Resistance for Case 3:

$$S_1 := Su_{fdu} = 2000 \frac{\text{lbf}}{\text{ft}^2}$$

$$B = 6.5 \text{ ft}$$

$$I := \frac{1}{2} \cdot S_1 \cdot B = 6500 \frac{\text{lbf}}{\text{ft}}$$

$$R_{\tau_case3} := I = 6500 \frac{\text{lbf}}{\text{ft}}$$



Unit Shear Resistance for Case 4:

$$S_1 := Su_{fdu} = 2000 \frac{\text{lbf}}{\text{ft}^2}$$

$$B_3 := \frac{B \cdot (-q_{min})}{q_{max} - q_{min}} = -2.3 \text{ ft}$$

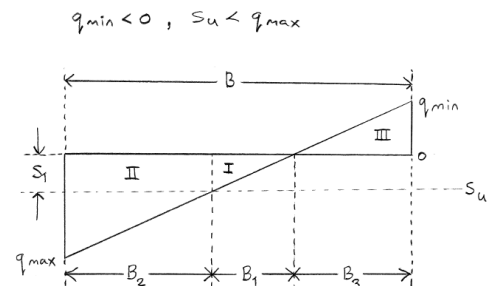
$$B_1 := \left(\frac{Su_{fdu}}{q_{max}} \right) \cdot (B - B_3) = 19.9 \text{ ft}$$

$$B_2 := B - (B_1 + B_3) = -11.1 \text{ ft}$$

$$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 19907.1 \frac{\text{lbf}}{\text{ft}}$$

$$II := S_1 \cdot B_2 = -22122.7 \frac{\text{lbf}}{\text{ft}}$$

$$R_{\tau_case4} := I + II = -2215.6 \frac{\text{lbf}}{\text{ft}}$$



Unit Shear Resistance for Case 5:

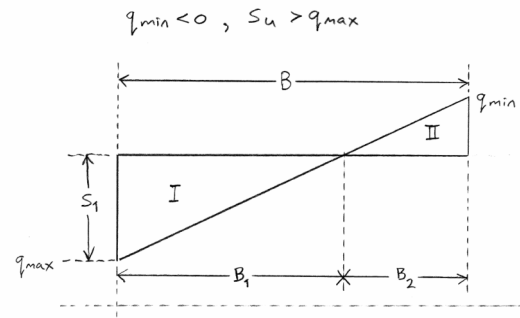
$$S_1 := q_{max} = 888.7 \frac{\text{lb}}{\text{ft}^2}$$

$$B_1 := \frac{B \cdot q_{max}}{q_{max} - q_{min}} = 8.8 \text{ ft}$$

$$B_2 := B - B_1 = -2.3 \text{ ft}$$

$$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 3930.6 \frac{\text{lb}}{\text{ft}}$$

$$R_{\tau_case5} := I = 3930.6 \frac{\text{lb}}{\text{ft}}$$



Define the Applicable Case:

$$R_{\tau} := R_{\tau_case2} \quad R_{\tau} = 3654.2 \frac{\text{lb}}{\text{ft}}$$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau} + \phi_{ep} \cdot R_{ep}$$

$$R_R := \phi R_n$$

Factored Sliding Resistance to be used in CDR Calculations:

$$R_R = 3654.204 \frac{\text{lb}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding} = 1.31$$

RETAINING WALL AT STA. 31+81.6

Objective: To evaluate the external stability of CIP wall's with level backfill (no backslope).
Method: In accordance with ODOT Bridge Design Manual, 2019 [Sect. 204.6.2.2] LRFD Bridge Design Specifications, 8th Ed., Nov. 2017, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].

Givens:

Backfill Soil Design Parameters:

$$\phi'_f := 30 \text{ deg} \quad \text{Effective angle of internal friction}$$

$$\gamma_f := 120 \frac{\text{lbf}}{\text{ft}^3} \quad \text{Unit weight}$$

$$c'_f := 0 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Effective Cohesion}$$

$$\delta := 0.67 \cdot \phi'_f \quad \delta = 20.1 \text{ deg} \quad \text{Friction angle between backfill and wall taken as specified in LRFD BDS C3.11.5.3 (degrees)}$$

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$$\phi'_{fd} := 24 \text{ deg} \quad \text{Effective angle of internal friction}$$

$$\gamma_{fd} := 112 \frac{\text{lbf}}{\text{ft}^3} \quad \text{Unit weight}$$

$$c'_{fd} := 200 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Effective Cohesion}$$

$$\delta_{fd} := 0.67 \cdot \phi'_{fd} \quad \delta_{fd} = 16.1 \text{ deg} \quad \text{Friction angle between foundation soils and footing taken as specified in LRFD BDS C3.11.5.3 (degrees)}$$

Undrained Conditions (Total Stress):

$$\phi_{fdu} := 0 \text{ deg} \quad \text{Angle of internal friction (Same as Drained Conditions if granular soils)}$$

$$\gamma_{fd} = 112 \frac{\text{lbf}}{\text{ft}^3} \quad \text{Unit weight}$$

$$Su_{fdu} := 2000 \frac{\text{lbf}}{\text{ft}^2} \quad \text{Undrained Shear Strength}$$

$$\delta_{fdu} := 0.67 \cdot \phi_{fdu} \quad \delta_{fdu} = 0 \text{ deg} \quad \text{Friction angle between foundation soils and footing taken as specified in LRFD BDS C3.11.5.3 (degrees)}$$

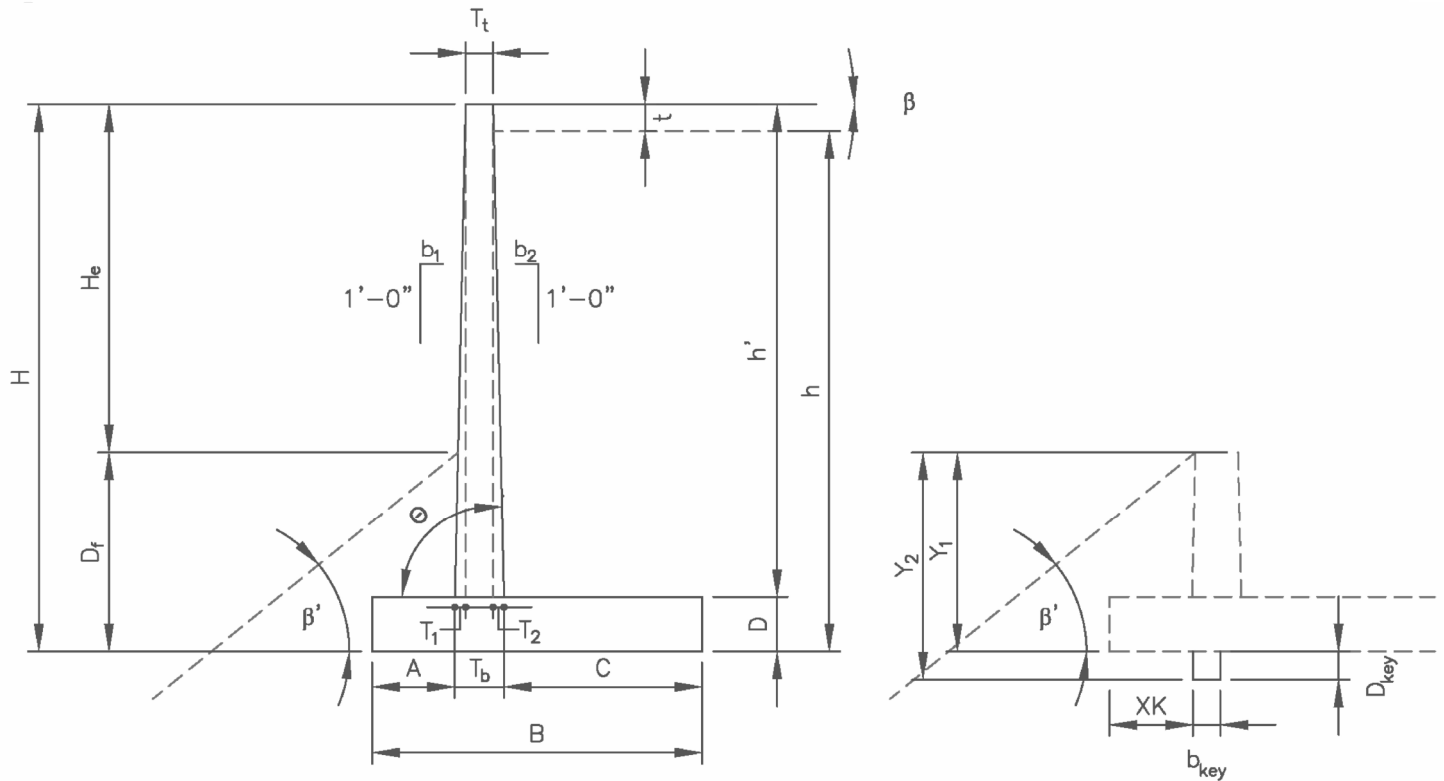
Foundation Surcharge Soil Parameters:

$$\gamma_q := 120 \frac{\text{lbf}}{\text{ft}^3} \quad \text{Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)}$$

Other Parameters:

$$\gamma_c := 150 \frac{\text{lbf}}{\text{ft}^3} \quad \text{Concrete Unit weight}$$

$$\gamma_p := 150 \frac{\text{lbf}}{\text{ft}^3} \quad \text{Pavement Unit weight}$$



Wall Geometry:

$$H_e := 6.4 \text{ ft} + .18 \cdot \text{ft} = 6.6 \text{ ft}$$

$$D_f := 4 \text{ ft}$$

$$H := H_e + D_f$$

$$H = 10.58 \text{ ft}$$

$$T_t := 16 \text{ in}$$

$$b_1 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

$$b_2 := 0 \cdot \left(\frac{\text{in}}{\text{ft}} \right)$$

$$\beta := 0 \text{ deg}$$

$$\beta' := 26.565 \text{ deg}$$

$$t := 0 \text{ in}$$

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.

Pavement thickness

Preliminary Wall Dimensioning:

$B := 7 \text{ ft} + 9 \cdot \text{in}$	$\frac{2}{5} \cdot H = 4.23 \text{ ft}$	to	$\frac{3}{5} \cdot H = 6.35 \text{ ft}$	Footing base width (2/5H to 3/5H)
$A := 12 \cdot \text{in}$	$\frac{H}{8} = 1.32 \text{ ft}$	to	$\frac{H}{5} = 2.12 \text{ ft}$	Toe projection (H/8 to H/5)
$D := 1.5 \text{ ft}$	$\frac{H}{8} = 1.32 \text{ ft}$	to	$\frac{H}{5} = 2.12 \text{ ft}$	Footing thickness (H/8 to H/5)

Shear Key Dimensioning:

$D_{key} := 0 \text{ ft}$	Depth of shear key from bottom of footing
$b_{key} := 0 \text{ ft}$	Note: Footings on rock typically require shear key Width of shear key
$XK := A$	Distance from toe to shear key

Other Wall Dimensions:

$h' := H - D$	$h' = 9.1 \text{ ft}$	Stem height
$T_1 := b_1 \cdot h'$	$T_1 = 0 \text{ ft}$	Stem front batter width
$T_2 := b_2 \cdot h'$	$T_2 = 0 \text{ ft}$	Stem back batter width
$T_b := T_1 + T_2 + T_t$	$T_b = 1.333 \text{ ft}$	Stem thickness at bottom of wall
$C := B - A - T_b$	$C = 5.417 \text{ ft}$	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)
$y_1 := 4 \cdot \text{ft}$	$y_1 = 4 \text{ ft}$	Depth to where passive pressure may begin to be utilized in front of wall. (Typically Df)
$y_2 := D_f + D_{key}$	$y_2 = 4 \text{ ft}$	Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.
$h := H - t$	$h = 10.58 \text{ ft}$	Height of retained fill at back of heel

Live Load Surcharge Parameters:

$\lambda := 0 \text{ ft}$	Horizontal distance from the back of the wall to point of traffic surcharge load
$SUR := \text{if} \left(\lambda < \frac{H}{2}, 250 \frac{\text{lb}}{\text{ft}^2}, 100 \frac{\text{lb}}{\text{ft}^2} \right) = 250 \frac{\text{lb}}{\text{ft}^2}$	Live load surcharge (per LRFD BDS [3.11.6.4]) Note: 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

$$\frac{-\beta'}{\phi'_{fd}} = -1.107$$

$$\frac{-\delta_{fd}}{\phi'_{fd}} = -0.67$$

$$k'_p := 1.66$$

Passive Earth Pressure Coefficient
from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.848$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p$$

$$k_{pd} = 1.408$$

Passive Earth Pressure Coefficient for
Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

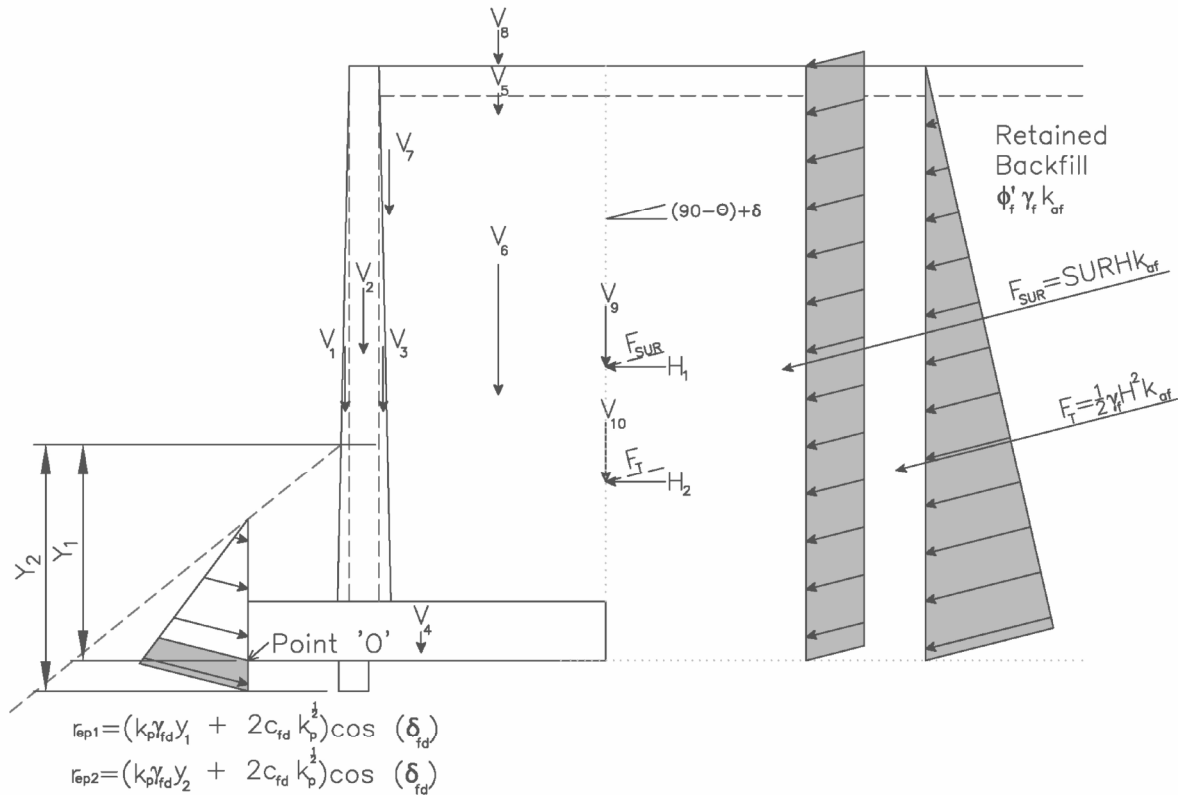
Undrained Conditions:

$$k_{pu} := \text{if}(\phi_{fdu} > 0, k_{pu}, 1)$$

$$k_{pu} = 1$$

Passive Earth Pressure Coefficient for
Resistance Undrained Conditions

Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:



$$F_T := \frac{1}{2} \cdot \gamma_f \cdot H^2 \cdot k_{af}$$

$$F_{SUR} := SUR \cdot H \cdot k_{af}$$

Vertical Loads:

$$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$$

$$V_2 := T_1 \cdot h' \cdot \gamma_c$$

$$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$$

$$V_4 := D \cdot B \cdot \gamma_c$$

$$V_5 := t \cdot (T_2 + C) \cdot \gamma_p$$

$$V_6 := C \cdot (h' - t) \cdot \gamma_f$$

$$V_7 := \frac{1}{2} \cdot b_2 \cdot (h' - t)^2 \cdot \gamma_f$$

$$V_8 := SUR \cdot (T_2 + C)$$

$$V_9 := F_{SUR} \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$V_{10} := F_T \cdot \sin(90 \cdot \text{deg} - \theta + \delta)$$

$$F_T = 1996.5 \frac{\text{lbf}}{\text{ft}}$$

$$F_{SUR} = 786.3 \frac{\text{lbf}}{\text{ft}}$$

$$V_1 = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_2 = 1816 \frac{\text{lbf}}{\text{ft}}$$

$$V_3 = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_4 = 1743.8 \frac{\text{lbf}}{\text{ft}}$$

$$V_5 = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_6 = 5902 \frac{\text{lbf}}{\text{ft}}$$

$$V_7 = 0 \frac{\text{lbf}}{\text{ft}}$$

$$V_8 = 1354.2 \frac{\text{lbf}}{\text{ft}}$$

$$V_9 = 270.2 \frac{\text{lbf}}{\text{ft}}$$

$$V_{10} = 686.1 \frac{\text{lbf}}{\text{ft}}$$

Active Earth Force Resultant (EH)

Live Load Surcharge (LS)

Wall stem front batter (DC)

Wall stem (DC)

Wall stem back batter (DC)

Wall Footing (DC)

Pavement (DC)

Soil Backfill - Heel (EV)

Soil Backfill - Batter (EV)

Live Load Surcharge above Heel- (LS)
- Strength lb

Live Load Surcharge Resultant (vertical
comp. - LS) - Strength lb

Active earth force resultant (vertical
component - EH)

Moment Arm:

Moments produced from vertical loads about Point 'O'

$$d_{v1} := A + \frac{2}{3} \cdot T_1 = 1 \text{ ft}$$

$$d_{v2} := A + T_1 + \frac{T_1}{2} = 1.7 \text{ ft}$$

$$d_{v3} := A + T_1 + T_1 + \frac{T_2}{3} = 2.3 \text{ ft}$$

$$d_{v4} := \frac{B}{2} = 3.9 \text{ ft}$$

$$d_{v5} := B - \frac{T_2 + C}{2} = 5 \text{ ft}$$

$$d_{v6} := B - \frac{C}{2} = 5 \text{ ft}$$

$$d_{v7} := A + T_1 + T_1 + \left(\frac{2}{3} \cdot b_2 \cdot (h' - t) \right) = 2.3 \text{ ft}$$

$$d_{v8} := B - \frac{T_2 + C}{2} = 5 \text{ ft}$$

$$d_{v9} := B = 7.8 \text{ ft}$$

$$d_{v10} := B = 7.8 \text{ ft}$$

Moment:

$$MV_1 := V_1 \cdot d_{v1} = 0 \text{ lbf}$$

$$MV_2 := V_2 \cdot d_{v2} = 3026.7 \text{ lbf}$$

$$MV_3 := V_3 \cdot d_{v3} = 0 \text{ lbf}$$

$$MV_4 := V_4 \cdot d_{v4} = 6757 \text{ lbf}$$

$$MV_5 := V_5 \cdot d_{v5} = 0 \text{ lbf}$$

$$MV_6 := V_6 \cdot d_{v6} = 29755.9 \text{ lbf}$$

$$MV_7 := V_7 \cdot d_{v7} = 0 \text{ lbf}$$

$$MV_8 := V_8 \cdot d_{v8} = 6827.3 \text{ lbf}$$

$$MV_9 := V_9 \cdot d_{v9} = 2094.1 \text{ lbf}$$

$$MV_{10} := V_{10} \cdot d_{v10} = 5317.3 \text{ lbf}$$

Horizontal Loads:

$$H_1 := F_{SUR} \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_1 = 738.4 \frac{\text{lbf}}{\text{ft}}$$

$$H_2 := F_T \cdot \cos(90 \cdot \text{deg} - \theta + \delta) \quad H_2 = 1874.9 \frac{\text{lbf}}{\text{ft}}$$

Live Load Surcharge Resultant (horizontal comp. - LS)

Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm:

$$d_{h1} := \frac{H}{2} \quad d_{h1} = 5.3 \text{ ft}$$

$$d_{h2} := \frac{H}{3} \quad d_{h2} = 3.5 \text{ ft}$$

Moment:

$$MH_1 := H_1 \cdot d_{h1} \quad MH_1 = 3906 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

$$MH_2 := H_2 \cdot d_{h2} \quad MH_2 = 6612 \frac{\text{lbf} \cdot \text{ft}}{\text{ft}}$$

Unfactored Loads by Load Type:

$$V_{DC} := V_1 + V_2 + V_3 + V_4 + V_5 \quad V_{DC} = 3559.8 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EV} := V_6 + V_7 \quad V_{EV} = 5902 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_1a} := V_9 \quad V_{LS_1a} = 270.2 \frac{\text{lbf}}{\text{ft}}$$

$$V_{LS_1b} := V_8 + V_9 \quad V_{LS_1b} = 1624.4 \frac{\text{lbf}}{\text{ft}}$$

$$V_{EH} := V_{10} \quad V_{EH} = 686.1 \frac{\text{lbf}}{\text{ft}}$$

$$H_{LS} := H_1 \quad H_{LS} = 738.4 \frac{\text{lbf}}{\text{ft}}$$

$$H_{EH} := H_2 \quad H_{EH} = 1874.9 \frac{\text{lbf}}{\text{ft}}$$

Unfactored Moments by Load Type

$$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4 + MV_5$$

$$M_{DC} = 9783.7 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EV} := MV_6 + MV_7$$

$$M_{EV} = 29755.9 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ia} := MV_9$$

$$M_{LSV_Ia} = 2094.1 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSV_Ib} := MV_8 + MV_9$$

$$M_{LSV_Ib} = 8921.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH1} := MV_{10}$$

$$M_{EH1} = 5317.3 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{LSH} := MH_1$$

$$M_{LSH} = 3906 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$M_{EH2} := MH_2$$

$$M_{EH2} = 6612 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

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:

$$V_{Ia} := \eta \cdot \left((Ia_{DC} \cdot V_{DC}) + (Ia_{EV} \cdot V_{EV}) + (Ia_{EH} \cdot V_{EH}) + (Ia_{LS} \cdot V_{LS_Ia}) \right) \quad V_{Ia} = 10607.8 \frac{\text{lb}}{\text{ft}}$$

$$V_{Ib} := \eta \cdot \left((Ib_{DC} \cdot V_{DC}) + (Ib_{EV} \cdot V_{EV}) + (Ib_{EH} \cdot V_{EH}) + (Ib_{LS} \cdot V_{LS_Ib}) \right) \quad V_{Ib} = 16289.2 \frac{\text{lb}}{\text{ft}}$$

Factored Horizontal Loads by Limit State:

$$H_{Ia} := \eta \cdot \left((Ia_{LS} \cdot H_{LS}) + (Ia_{EH} \cdot H_{EH}) \right) \quad H_{Ia} = 4104.4 \frac{\text{lb}}{\text{ft}}$$

$$H_{Ib} := \eta \cdot \left((Ib_{LS} \cdot H_{LS}) + (Ib_{EH} \cdot H_{EH}) \right) \quad H_{Ib} = 4104.4 \frac{\text{lb}}{\text{ft}}$$

Factored Moments Produced by Vertical Loads by Limit State:

$$MV_{Ia} := \eta \cdot \left((Ia_{DC} \cdot M_{DC}) + (Ia_{EV} \cdot M_{EV}) + (Ia_{EH} \cdot M_{EH1}) + (Ia_{LS} \cdot M_{LSV_Ia}) \right) \quad MV_{Ia} = 50201.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MV_{Ib} := \eta \cdot \left((Ib_{DC} \cdot M_{DC}) + (Ib_{EV} \cdot M_{EV}) + (Ib_{EH} \cdot M_{EH1}) + (Ib_{LS} \cdot M_{LSV_Ib}) \right) \quad MV_{Ib} = 75988.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Factored Moments Produced by Horizontal Loads by Limit State:

$$MH_{Ia} := \eta \cdot \left((Ia_{LS} \cdot M_{LSH}) + (Ia_{EH} \cdot M_{EH2}) \right) \quad MH_{Ia} = 16753.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

$$MH_{Ib} := \eta \cdot \left((Ib_{LS} \cdot M_{LSH}) + (Ib_{EH} \cdot M_{EH2}) \right) \quad MH_{Ib} = 16753.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength lb:

$\Sigma M_R := MV_{lb}$	$\Sigma M_R = 75988.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Resisting Moments (Strength lb)
$\Sigma M_O := MH_{lb}$	$\Sigma M_O = 16753.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$	Sum of Overturning Moments (Strength lb)
$\Sigma V := V_{lb}$	$\Sigma V = 16289.2 \frac{\text{lb}}{\text{ft}}$	Sum of Vertical Loads (Strength lb)
$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$	$x = 3.6 \text{ ft}$	Distance from Point "O" the resultant intersects the base
$e := \left \frac{B}{2} - x \right $	$e = 0.24 \text{ ft}$	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.
$B' := B - 2e = 7.3 \text{ ft}$		

Drained nominal bearing resistance from limit equilibrium analysis (**Attached**):

$q_{nd} := 5400 \text{ psf}$	$q_{nd} = 5400 \frac{\text{lb}}{\text{ft}^2}$
------------------------------	---

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

$\phi_b := 0.55$	Bearing resistance factor LRFD Table 11.5.7-1.
$q_{Rd} := \phi_b \cdot q_{nd}$	$q_{Rd} = 2.97 \text{ ksf}$
Factored bearing resistance Drained Conditions	

Undrained nominal bearing resistance from limit equilibrium analysis (**Attached**):

$q_{nu} := 8200 \text{ psf}$	$q_{nu} = 8200 \frac{\text{lb}}{\text{ft}^2}$
------------------------------	---

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

$\phi_b := 0.55$	Bearing resistance factor LRFD Table 11.5.7-1.
$q_{Ru} := \phi_b \cdot q_{nu}$	$q_{Ru} = 4.5 \text{ ksf}$
Factored bearing resistance Undrained Conditions	

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions:	$q_{Rd} = 3 \text{ ksf}$
Undrained Conditions:	$q_{Ru} = 4.5 \text{ ksf}$

Evaluate External Stability of Wall:

Compute the ultimate bearing stress :

$$e = 0.24 \text{ ft}$$

$$\sigma := \frac{V_{DC} + V_{EV} + V_{EH} + V_{LS_lb}}{B - 2 \cdot e}$$

$$\sigma = 1.619 \text{ ksf}$$

Nominal Bearing Stress

$$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e}$$

$$\sigma_V = 2.24 \text{ ksf}$$

Factored Bearing Stress

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions:

$$CDR_{Bearing_D} := \frac{q_{Rd}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{Bearing_D} = 1.33$$

Undrained Conditions:

$$CDR_{Bearing_U} := \frac{q_{Ru}}{\sigma_V}$$

Is the CDR > or = to 1.0?

$$CDR_{Bearing_U} = 2.01$$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$$e_{max} := \frac{B}{3}$$

$$e_{max} = 2.6 \text{ ft}$$

Maximum Eccentricity **LRFD [11.6.3.3.]**
Equals B/3 for soil.

$$\Sigma M_R := MV_{Ia}$$

$$\Sigma M_R = 50201.8 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Resisting Moments (Strength Ia)

$$\Sigma M_O := MH_{Ia}$$

$$\Sigma M_O = 16753.4 \frac{\text{lb} \cdot \text{ft}}{\text{ft}}$$

Sum of Overturning Moments (Strength Ia)

$$\Sigma V := V_{Ia}$$

$$\Sigma V = 10607.8 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$$

$$x = 3.2 \text{ ft}$$

Distance from Point "O" the resultant intersects the base

$$e := \text{abs} \left(\frac{B}{2} - x \right)$$

$$e = 0.72 \text{ ft}$$

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?

$$CDR_{Eccentricity} = 3.58$$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$$R_u := H_{Ia} \quad R_u = 4104.4 \frac{\text{lb}}{\text{ft}}$$

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

$$\Sigma V := V_{Ia} \quad \Sigma V = 10607.8 \frac{\text{lb}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$R_{\tau} := c \cdot \Sigma V \cdot \tan(\phi'_{fd})$$

$$R_{\tau} = 4722.9 \frac{\text{lb}}{\text{ft}}$$

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:

$$R_R = 4722.889 \frac{\text{lb}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding} = 1.15$$

Undrained Conditions (Total Stress):

Compute passive resistance throughout the design life of the wall **LRFD [Eq 3.11.5.4-1]**:

$$r_{ep1} := (k_{pu} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot Su_{fdu} \cdot \sqrt{k_{pu}}) \cdot \cos(\delta_{fd})$$

Nominal passive pressure at y1

$$r_{ep2} := (k_{pu} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot Su_{fdu} \cdot \sqrt{k_{pu}}) \cdot \cos(\delta_{fd})$$

Nominal passive pressure at y2

$$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot (y_2 - y_1) \quad R_{ep} = 0 \frac{\text{lbft}}{\text{ft}}$$

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

$$\Sigma V := V_{la} \quad \Sigma V = 10607.8 \frac{\text{lbft}}{\text{ft}}$$

Sum of Vertical Loads (Strength Ia)

$$e = 0.72 \text{ ft}$$

Wall eccentricity, Calculated in above Limiting Eccentricity at Base of Wall (Strength Ia) Section.

$$B = 7.8 \text{ ft}$$

Footing base width

$$\frac{B}{6} = 1.3 \text{ ft}$$

If e < B/6 the resultant is in the middle one-third

$$\sigma_{vmax} := \frac{\Sigma V}{B} \cdot \left(1 + 6 \cdot \frac{e}{B}\right) \quad \sigma_{vmax} = 2133.6 \frac{\text{lbft}}{\text{ft}^2}$$

Max vertical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$$\sigma_{vmin} := \frac{\Sigma V}{B} \cdot \left(1 - 6 \cdot \frac{e}{B}\right) \quad \sigma_{vmin} = 603.9 \frac{\text{lbft}}{\text{ft}^2}$$

Max vertical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$$q_{max} := \frac{1}{2} \cdot \sigma_{vmax} \quad q_{max} = 1066.8 \frac{\text{lbft}}{\text{ft}^2}$$

Max unit shear resistance as 1/2 max vertical stress **LRFD [10.6.3.4]**.

$$q_{min} := \frac{1}{2} \cdot \sigma_{vmin} \quad q_{min} = 301.9 \frac{\text{lbft}}{\text{ft}^2}$$

Minimum unit shear resistance as 1/2 minimum vertical stress **LRFD [10.6.3.4]**.

Determine which Cohesive Soil Resistance Case is Present:

$$\text{Case}_1 := \text{if}(q_{max} > Su_{fdu} > q_{min} \geq 0, 1, 0) \quad \text{Case}_1 = 0$$

$$\text{Case}_2 := \text{if}(Su_{fdu} > q_{max} > q_{min} \geq 0, 1, 0) \quad \text{Case}_2 = 1$$

$$\text{Case}_3 := \text{if}(q_{max} > q_{min} > Su_{fdu}, 1, 0) \quad \text{Case}_3 = 0$$

$$\text{Case}_4 := \text{if}(q_{min} < 0, \text{if}(Su_{fdu} < q_{max}, 1, 0), 0) \quad \text{Case}_4 = 0$$

$$\text{Case}_5 := \text{if}(q_{min} < 0, \text{if}(Su_{fdu} > q_{max}, 1, 0), 0) \quad \text{Case}_5 = 0$$

Unit Shear Resistance for Case 1:

$$S_1 := Su_{fdu} - q_{min} = 1698.1 \frac{\text{lbf}}{\text{ft}^2}$$

$$S_2 := q_{min} = 301.9 \frac{\text{lbf}}{\text{ft}^2}$$

$$B_1 := \frac{B \cdot (Su_{fdu} - q_{min})}{q_{max} - q_{min}} = 17.2 \text{ ft}$$

$$B_2 := \frac{B \cdot (q_{max} - Su_{fdu})}{q_{max} - q_{min}} = -9.5 \text{ ft}$$

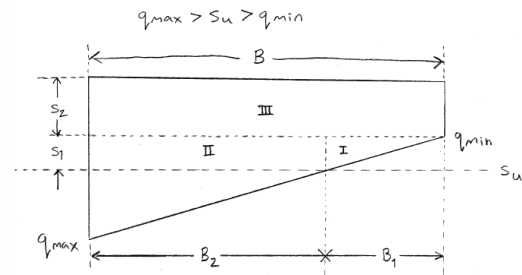
$$B_3 := B = 7.8 \text{ ft}$$

$$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 14607.9 \frac{\text{lbf}}{\text{ft}}$$

$$II := S_1 \cdot B_2 = -16055.8 \frac{\text{lbf}}{\text{ft}}$$

$$III := S_2 \cdot B_3 = 2340 \frac{\text{lbf}}{\text{ft}}$$

$$R_{\tau_case1} := I + II + III = 892.1 \frac{\text{lbf}}{\text{ft}}$$



Unit Shear Resistance for Case 2:

$$S_1 := q_{max} - q_{min} = 764.9 \frac{\text{lbf}}{\text{ft}^2}$$

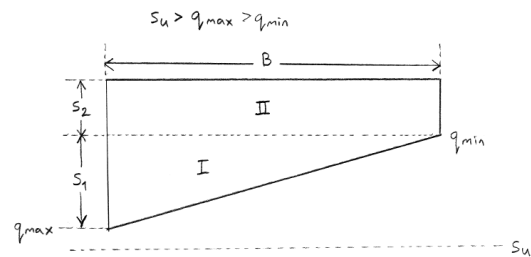
$$S_2 := q_{min} = 301.9 \frac{\text{lbf}}{\text{ft}^2}$$

$$B = 7.8 \text{ ft}$$

$$I := \frac{1}{2} \cdot S_1 \cdot B = 2963.9 \frac{\text{lbf}}{\text{ft}}$$

$$II := S_2 \cdot B = 2340 \frac{\text{lbf}}{\text{ft}}$$

$$R_{\tau_case2} := I + II = 5303.9 \frac{\text{lbf}}{\text{ft}}$$



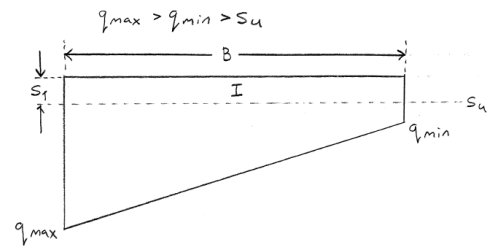
Unit Shear Resistance for Case 3:

$$S_1 := Su_{fdu} = 2000 \frac{\text{lbf}}{\text{ft}^2}$$

$$B = 7.8 \text{ ft}$$

$$I := \frac{1}{2} \cdot S_1 \cdot B = 7750 \frac{\text{lbf}}{\text{ft}}$$

$$R_{\tau_case3} := I = 7750 \frac{\text{lbf}}{\text{ft}}$$



Unit Shear Resistance for Case 4:

$$S_1 := Su_{fdu} = 2000 \frac{\text{lbf}}{\text{ft}^2}$$

$$B_3 := \frac{B \cdot (-q_{min})}{q_{max} - q_{min}} = -3.1 \text{ ft}$$

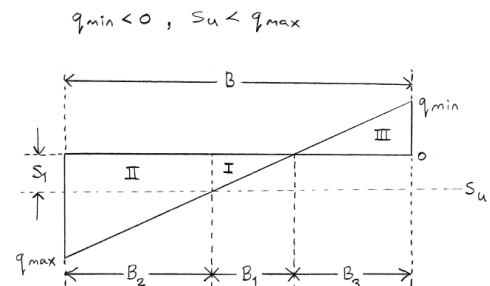
$$B_1 := \left(\frac{Su_{fdu}}{q_{max}} \right) \cdot (B - B_3) = 20.3 \text{ ft}$$

$$B_2 := B - (B_1 + B_3) = -9.5 \text{ ft}$$

$$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 20264.6 \frac{\text{lbf}}{\text{ft}}$$

$$II := S_1 \cdot B_2 = -18910.7 \frac{\text{lbf}}{\text{ft}}$$

$$R_{\tau_case4} := I + II = 1353.9 \frac{\text{lbf}}{\text{ft}}$$



Unit Shear Resistance for Case 5:

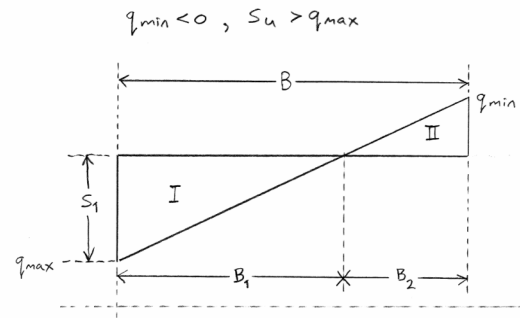
$$S_1 := q_{max} = 1066.8 \frac{\text{lb}}{\text{ft}^2}$$

$$B_1 := \frac{B \cdot q_{max}}{q_{max} - q_{min}} = 10.8 \text{ ft}$$

$$B_2 := B - B_1 = -3.1 \text{ ft}$$

$$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 5765.7 \frac{\text{lb}}{\text{ft}}$$

$$R_{\tau_case5} := I = 5765.7 \frac{\text{lb}}{\text{ft}}$$



Define the Applicable Case:

$$R_{\tau} := R_{\tau_case2}$$

$$R_{\tau} = 5303.9 \frac{\text{lb}}{\text{ft}}$$

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.

$$\phi R_n := \phi_{\tau} \cdot R_{\tau} + \phi_{ep} \cdot R_{ep}$$

$$R_R := \phi R_n$$

Factored Sliding Resistance to be used in CDR Calculations:

$$R_R = 5303.891 \frac{\text{lb}}{\text{ft}}$$

Sliding Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} := \frac{R_R}{R_u}$$

Is the CDR > or = to 1.0?

$$CDR_{Sliding} = 1.29$$

APPENDIX 5D

GLOBAL STABILITY ANALYSIS

RETAINING WALL AT STA. 31+81.6

