DRAFT REPORT
GEOTECHNICAL EXPLORATION
BRO-68-33.36
BROWN COUNTY, OHIO
PID#: 120095

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NEAS PROJECT 25-0011

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EXECUTIVE SUMMARY

An intersection improvement project, BRO-68-33.36 (PID 120095), has been proposed in Green Township, Brown County, Ohio. The project is located at the intersection of U.S. 68 and Greenbush Road and includes subgrade explorations for a new roundabout with drainage improvements, including the replacement of two box culverts.

National Engineering & Architectural Services, Inc. (NEAS) has been contracted to perform geotechnical engineering services for the project. The purpose of the geotechnical engineering services was to perform geotechnical explorations within the project limits to obtain information concerning the subsurface soil and groundwater conditions relevant to the design and construction of the project. The scope of work performed by NEAS as part of the referenced project included: a review of published geotechnical information; performing 9 test borings for the proposed culverts and roundabout; laboratory testing of soil samples in accordance with the SGE; performing geotechnical engineering analysis to assess culvert/roundabout design and construction considerations; and development of this summary report.

NEAS understands that the proposed project improvements include construction of a new roundabout and new full-height headwalls for two culverts. The culvert under U.S. 68 is a precast four-sided box culvert with a 14 ft span and 6 ft rise, while the culvert under Greenbush West Road is a precast four-sided box culvert with a 16 ft span and 7 ft rise. This report presents a summary of the encountered surficial and subsurface conditions and provides recommendations for culvert foundation design and construction in accordance with Load and Resistance Factor Design (LRFD) method, as set forth in AASHTO's Publication LRFD Bridge Design Specifications, 9th Edition (AASHTO LRFD, 2020), ODOT's 2025 Bridge Design Manual (BDM) and ODOT’s 2025 Geotechnical Design Manual (ODOT GDM, 2025).

According to our subgrade analysis, unstable materials, including weak and high moisture content soils, were encountered in 20% of the proposed subgrade. The construction of the new roundabout will require both filling and cutting. NEAS recommends that spot stabilization be performed on unstable subgrade areas using Item 204 Proof Rolling for the entire project. Spot stabilization should be conducted using the Excavate and Replace method (Item 204 with Geotextile). Excavations are estimated to extend to a depth of 12 inches, with the excavated material being replaced with material in accordance with Section 608 "Excavate and Replace (Item 204)" of the ODOT GDM (ODOT, 2025). Stabilization limits should extend 18 inches beyond the edge of the paved roadway, shoulder or median. Additionally, it is recommended to remove any topsoil, existing pavement materials, or abandoned structure foundation materials. It is NEAS’s opinion that the subgrade soils will provide adequate pavement support, assuming the pavement is designed and constructed in accordance with the recommendations provided in this report and all applicable ODOT standards and specifications.

For the culvert under U.S. 68, geotechnical analyses were performed to evaluate external stability (i.e., overall stability, bearing capacity, and sliding) and global stability. Based on our analyses, the factored bearing resistances range from 11.0 ksf to 25.3 ksf. The minimum slope stability safety factors for both short-term and long-term conditions exceeded the desired value of 1.5.

For the culvert under Greenbush West Road, geotechnical analyses were conducted to assess external stability (i.e., overall stability, bearing capacity, and sliding) and global stability. The factored bearing resistances range from 10.1 ksf to 28.3 ksf. The minimum slope stability safety factors for both short-term and long-term conditions also exceeded the desired value of 1.5.

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1. INTRODUCTION

1.1. General

NEAS presents our Structure Foundation Exploration Report for the proposed intersection improvement project BRO-68-33.36 (PID 120095) in Green Township, Brown County, Ohio. The project is located at the intersection of U.S. 68 and Greenbush West Road and includes subgrade explorations for a new roundabout with drainage improvements, including the replacement of two box culverts.

This report presents a summary of the encountered surficial and subsurface conditions and our recommendations for culverts foundation design and construction in accordance with Load and Resistance Factor Design (LRFD) method as set forth in AASHTO's Publication LRFD Bridge Design Specifications, 9th Edition (AASHTO LRFD, 2020), ODOT's 2025 Bridge Design Manual (BDM) and ODOT's 2025 Geotechnical Design Manual (ODOT GDM, 2025).

The exploration was conducted in general accordance with National Engineering & Architectural Services, Inc.'s (NEAS's) proposal to WSP USA Inc., dated November 21, 2024, and with the provisions of ODOT's *Specifications for Geotechnical Explorations* (SGE) (ODOT, 2025).

The scope of work performed by NEAS as part of the referenced project included: a review of published geotechnical information; performing 9 test borings for culverts and subgrade; laboratory testing of soil samples in accordance with the SGE; performing geotechnical engineering analysis to assess culvert wall design and construction considerations; and development of this summary report.

1.2. Proposed Construction

NEAS understands that the proposed project improvements include the construction of a new roundabout and the replacement of two box culverts. The culvert under U.S. 68 will be a precast four-sided box culvert with a 14-ft span and a 6-ft rise. The culvert under Greenbush West Road will be a precast four-sided box culvert with a 16-ft span and a 7-ft rise.

2. GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1. Geology and Physiography

The project site is located within the Illinoian Till Plain. This area is characterized by broad, level to rolling uplands of moderately low relief (50 ft) dissected by steep-sided stream valleys (ODGS, 1998). The uplands consist of ground moraine of older till generally lacking ice-constructional features such as moraines, kames and eskers, but contains many buried valleys, as well as modern valleys alternating between broad floodplains and bedrock gorge.

The bedrock underlying the project site is mapped as Waynesville and Arnheim Formations, Undivided (USGS & ODGS, 2006). The Waynesville Fromation consists of 70% Shale and 30% limestone interbeded, is characterized as gray to blueish gray weathers light gray, planar to irregular, thin to thick bedded and about 90-120 feet thick. The Arnheim Formations consists of 60% Shale and 40% limestone interbeded, is characterized as gray to blueish gray weathers light gray, planar, wavy, irregular to nodular, thin to thick bedded, and about 50-100 feet thick. Based on the ODNR bedrock-topography data for Ohio

(ODNR DGS, 2004), bedrock elevations at the proposed project site can be expected to be approximately 900 ft amsl.

The soils at the proposed project site have primarily been mapped (Web Soil Survey) by the Natural Resources Conservation Service as, Westboro-Schaffeer silt loam with 2 to 4 percent slopes. Those soil units are described as being poorly drained to moderately well drained and classified as both cohesive and non-cohesive A-4 soils as well as cohesive A-6 and A-7 soils (AASHTO method of soil classification). (USDA, 2015).

2.2. Hydrology/Hydrogeology

Groundwater can be expected at an elevation consistent with that of the major local surface water body, Little Sterling Run. The flow line elevation of Little Sterling Run at the project site is approximately 935 ft.

Local variations in the groundwater table may exist in one of two reasons. First, if there has been extensive groundwater abstraction, water levels may be depressed by tens of feet over significantly large areas. Second, the presence of discontinuous bodies of glacial till provides the opportunity for localized pockets of perched groundwater to form.

The proposed project site is located within 1% annual chance flood hazard zone based on available mapping by the Federal Emergency Management Agency's National Flood Hazard mapping program (FEMA, 2019).

2.3. Mining and Oil/Gas Production

No abandoned mines are noted on ODNR’s Abandoned Underground Mine Locator within 1.0-mile distance of the proposed project site (ODNR [1], 2025).

No oil or gas wells are noted on ODNR’s Ohio Oil & Gas Locator within 1.0-mile distance of the proposed project site (ODNR [1], 2025).

2.4. Historical Records and Previous Phases of Project Exploration

A historical record search was performed through ODOT's Transportation Information Mapping System (TIMS). No historical projects were found within the project area.

3. GEOTECHNICAL EXPLORATION

3.1. Field Exploration Program

NEAS conducted the project’s subsurface exploration between March 11 and March 14, 2025, which included 9 borings drilled to depths between 7.5 ft and 35 ft below ground surface. The boring locations were selected by NEAS in general accordance with the guidelines contained in the SGE with the intent to evaluate subsurface soil and groundwater conditions. Borings were typically located within the planned roadway/structure construction areas that were not restricted by underground utilities or dictated by terrain (e.g. steep embankment slopes). Project boring locations were located and surveyed in the field by NEAS before drilling. Each individual project boring log (to be included within Appendix B) will include

the recorded boring latitude and longitude location (based on the surveyed County Coordinate System, NAD83, location) and the corresponding ground surface elevation, as shown in Table 1.

Table 1: Project Borings Summary

Boring Number	Alignment	Location (Sta/offset)	Latitude	Longitude	Elevation (NAVD 88) (ft)	Depth (ft)	Structure
B-001-0-24	Proposed U.S. 68	1866+67, 15' LT	39.074816	-83.925335	945.9	7.5	Subgrade
B-002-0-24	Proposed Greenbush West Rd	15+65, 9' RT	39.075312	-83.926059	944.5	35.0	Culvert
B-003-0-24	Proposed U.S. 68	1868+20, 40' RT	39.075245	-83.925177	942.4	10.0	Roadway
B-004-0-24	Proposed Greenbush East Rd	20+62, 4' LT	39.075307	-83.924312	948.3	7.5	Subgrade
B-005-0-24	Proposed Greenbush West Rd	14+95, 26' LT	39.075500	-83.926200	940.3	35.0	Culvert
B-006-0-24	Proposed Greenbush West Rd	12+98, 9' LT	39.075709	-83.926859	947.8	7.5	Subgrade
B-007-0-24	Proposed U.S. 68	1870+66, 38' LT	39.075785	-83.925722	940.6	31.5	Culvert
B-008-0-24	Proposed U.S. 68	1870+30, 36' RT	39.075840	-83.925440	940.1	31.5	Culvert
B-009-0-24	Proposed U.S. 68	1872+20, 1' RT	39.076151	-83.926012	944.4	10.0	Roadway

Project borings were drilled using either a CME 55X track-mounted or a or CME 45B truck-mounted drilling rig, utilizing 3.25-inch (inner diameter) hollow stem augers. Soil samples were recovered continuously or at 2.5-ft intervals drilled to end of boring (EOB) using an 18-inch split spoon sampler (AASHTO T-206 "Standard Method for Penetration Test and Split Barrel Sampling of Soils."). The soil samples obtained from the exploration program were visually observed in the field by the NEAS field representative and preserved for review by a Geologist and possible laboratory testing. Standard penetration tests (SPT) were conducted using CME auto hammers that have been calibrated to be 80% or 79% efficient (indicated on the boring logs) on March 8, 2024.

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 sealed or backfilled with auger cuttings, bentonite chips, or a combination of these materials.

3.2. Laboratory Testing Program

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

3.2.1. Classification Testing

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

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

3.2.2. Standard Penetration Test Results

Standard Penetration Tests (SPT) and split-barrel (commonly known as split-spoon) sampling of soils were performed at varying intervals (i.e., continuously or at 2.5-ft or 5 intervals) in the project borings performed. To account for the high efficiency (automatic) hammers used during SPT sampling, field SPT N-values were converted based on the calibrated efficiency (energy ratio) of the specific drill rig's hammer. Field N-values were converted to an equivalent rod energy of 60% (N_{60}) for use in analysis or for correlation purposes. The resulting N_{60} values are presented on the boring logs provided in Appendix B.

3.2.3. D_{50} Values for Scour Evaluation

Grain size distribution testing was performed on the obtained streambed samples to develop D_{50} values (i.e., the diameter in the particle-size distribution curve corresponding to 50% finer). The calculated D_{50} values are shown in Table 2 below and the developed particle-size distribution curves are included with the associated boring log within Appendix B.

Table 2: D_{50} Values for Scour Evaluation

Boring Number	Specimen ID	Specimen Elevation (ft)	ODOT (Modified AASHTO) ~ USCS Classification	D_{50} (mm)
B-002-0-24	SS-1	942.0' - 940.5'	A-7-6 ~ SANDY LEAN CLAY(CL)	0.029
	SS-2	939.5' - 938.0'	A-6a ~ SANDY LEAN CLAY(CL)	0.021
	SS-3	937.0' - 935.5'	A-4a ~ SILTY, CLAYEY SAND with GRAVEL(SC-SM)	0.080
	SS-4	934.5' - 933.0'	A-4a ~ SANDY LEAN CLAY(CL)	0.036
B-005-0-24	SS-1	592.9' - 591.4'	A-6b ~ SANDY LEAN CLAY(CL)	0.017
	SS-2	586.4' - 584.9'	A-4a ~ SILTY SAND(SM)	0.077
	SS-3	583.9' - 582.4'	A-4a ~ SANDY LEAN CLAY(CL)	0.030
	SS-4	578.9' - 577.4'	A-4a ~ SILTY, CLAYEY SAND(SC-SM)	0.093
B-007-0-23	SS-1	594.5' - 593.2'	A-6b ~ SANDY LEAN CLAY(CL)	0.028
	SS-2	589.2' - 587.7'	A-4a ~ SANDY LEAN CLAY(CL)	0.043
	SS-3	586.7' - 585.2'	A-4a ~ SANDY LEAN CLAY(CL)	0.023
	SS-4	584.2' - 582.7'	A-4a ~ SANDY LEAN CLAY(CL)	0.035
B-008-0-24	SS-1	592.9' - 591.4'	A-7-6 ~ SANDY LEAN CLAY(CL)	0.019
	SS-2	586.4' - 584.9'	A-4a ~ SANDY LEAN CLAY(CL)	0.046
	SS-3	583.9' - 582.4'	A-4a ~ CLAYEY SAND with GRAVEL(SC)	0.112
	SS-4	578.9' - 577.4'	A-4a ~ SANDY LEAN CLAY(CL)	0.058

3.2.4. Sulfate Testing

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

Table 3: Sulfate Test Summary by Boring

Boring ID	Sample	Depth (ft)	Dilution Ratio	Average Sulfate Content (ppm)
B-001-0-24	SS-1	1.5 - 3.0	20	93
B-003-0-24	SS-1	1.0 - 2.5	20	147
B-004-0-24	SS-1	1.5 - 3.0	20	20
B-006-0-24	SS-2	3.0 - 4.5	20	80
B-009-0-24	SS-1	1.0 - 2.5	20	293

4. GEOTECHNICAL FINDINGS

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

4.1. Subsurface Conditions

4.1.1. Overburden Soil

At the site of culvert under U.S. 68, the natural overburden soils primarily consist of cohesive materials, including Sandy Silt (A-4a), Silt (A-4b), Silty Clay (A-6b), and Clay (A-7-6). Granular materials were only encountered at the inlet site, between 19.5 and 24.5 ft below ground surface, corresponding to approximate elevations of 920.6 ft to 915.6 ft. The granular soils encountered were characterized as Coarse and Fine Sand (A-3a) and Silt (A-4b). The cohesive soils can be described as having a very stiff to hard consistency based on N_{60} values between 8 and 108 blows per foot (bpf), and unconfined compressive strengths (estimated by means of hand penetrometer) between approximately 2.25 and 4.50 ton per square foot (tsf). Natural moisture contents of the fine-grained till soils ranged from 7 to 21 percent in moisture. Based on Atterberg Limits test performed on representative samples of the natural till soils, the liquid and plastic limits ranged from 21 to 46 percent and 13 to 20 percent, respectively. The non-cohesive soils are described as very dense in compactness correlating to N_{60} values between 108 bpf and refusal. Natural moisture contents of the non-cohesive till soils ranged from 13 to 15 percent in moisture.

At the site of culvert under Greenbush West Road, the natural overburden soils primarily consist of cohesive materials, including Sandy Silt (A-4a), Silt and Clay (A-6a), Silty Clay (A-6b), and Clay (A-7-6). Granular materials were only encountered at the inlet site between 4.5 and 7.0 ft below ground surface, corresponding to approximate elevations of 935.8 ft to 933.3 ft. The granular soils encountered were characterized as Sandy Silt (A-4a). The cohesive soils can be described as having a very stiff to hard consistency based on N_{60} values between 7 bpf and refusal, and unconfined compressive strengths (estimated by means of hand penetrometer) between approximately 2.25 and 4.50 ton per square foot (tsf). Natural moisture contents of the fine-grained till soils ranged from 7 to 20 percent in moisture. Based on Atterberg Limits test performed on representative samples of the natural till soils, the liquid and plastic limits ranged from 20 to 41 percent and 13 to 20 percent, respectively. The non-cohesive soils are

described as dense in compactness correlating to N_{60} value of 45 bpf. Natural moisture content of the non-cohesive till soils was 14 percent in moisture.

Bedrock was not encountered in any of the structure borings performed.

Groundwater was not encountered during drilling in any of the structure borings performed.

5. ANALYSIS AND RECOMMENDATIONS

The analyses performed are based on the information: 1) the soil characteristics gathered during the subsurface exploration (i.e., SPT results, laboratory test results, etc.) presented in Section 5.1 of this report; 2) the developed generalized soil profile at the proposed culverts locations and other design assumptions presented in subsequent sections of this report; and, 3) the basemap and site plans including the culverts details provided by WSP USA, Inc., geotechnical analyses consisting of subgrade analysis, external stability (i.e., overall stability, bearing capacity, and sliding), global stability analyses, were performed for the proposed project. The geotechnical engineering analyses were performed in accordance with LRFD Bridge Design Specifications, (AASHTO LRFD, 2020), ODOT's 2025 Bridge Design Manual (BDM) and ODOT's 2025 Geotechnical Design Manual (GDM) (ODOT GDM, 2025).

5.1. Generalized Soil Profile for Analysis

For analysis purposes, each boring log was reviewed and the engineering properties for each soil strata were estimated based on their field (i.e., SPT N_{60} Values, hand penetrometer values, etc.) and laboratory (i.e., Atterberg Limits, grain size, etc.) test results using correlations provided in published engineering manuals, research reports and guidance documents. Engineering soil properties were estimated for each individual classified layer per boring location. Soil layers 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 4-7 below.

Table 4: Soil Profile and Estimated Engineering Properties – B-002-0-24

Culvert Under Greenbush West Road : Soil Profile B-002-0-24							
Soil Description	**Unit Weight[1] (pcf)**	**Moist Unit Weight[1] (pcf)**	**Saturated Unit Weight[1] (pcf)**	**Undrained Shear Strength[2] (psf)**	**Effective Cohesion[3] (psf)**	**Effective Friction Angle[3] (degrees)**	**Setup Factor (f_{su})**
Clay (A-7-6) Depth (944.5 ft - 940 ft)	108	108	118	850	100	21	2.00
Silt and Clay (A-6a) Depth (940 ft - 937.5 ft)	118	118	128	4350	300	27	1.50
Sandy Silt (A-4a) Depth (937.5 ft - 927.5 ft)	140	130	140	7600	450	30	1.50
Sandy Silt (A-4a) Depth (927.5 ft - 912.2 ft)	132	122	132	5300	375	28	1.50
Clay (A-7-6) Depth (912.2 ft - 909.5 ft)	128	118	128	4100	300	25	2.00

Notes:
1. *Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405.*
2. *Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.*

Table 5: Soil Profile and Estimated Engineering Properties – B-005-0-24

Culvert Under Greenbush West Road : Soil Profile B-005-0-24							
Soil Description	Unit Weight[1] (pcf)	Moist Unit Weight[1] (pcf)	Saturated Unit Weight[1] (pcf)	Undrained Shear Strength[2] (psf)	Effective Cohesion[3] (psf)	Effective Friction Angle[3] (degrees)	Setup Factor (f_{su})
Silty Clay (A-6b) Depth (940.3 ft - 935.8 ft)	110	110	120	1350	150	23	1.75
Sandy Silt (A-4a) Depth (935.8 ft - 933.3 ft)	122	122	132	-	-	39	1.20
Sandy Silt (A-4a) Depth (933.3 ft - 923.3 ft)	140	130	140	9700	450	32	1.50
Sandy Silt (A-4a) Depth (923.3 ft - 908 ft)	132	122	132	5200	375	28	1.50
Silt and Clay (A-6a) Depth (908 ft - 905.3 ft)	128	118	128	4100	300	26	1.50

Notes:
1. *Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405.*
2. *Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.*

Table 6: Soil Profile and Estimated Engineering Properties – B-007-0-24

Culvert Under U.S. 68: Soil Profile B-007-0-24							
Soil Description	Unit Weight[1] (pcf)	Moist Unit Weight[1] (pcf)	Saturated Unit Weight[1] (pcf)	Undrained Shear Strength[2] (psf)	Effective Cohesion[3] (psf)	Effective Friction Angle[3] (degrees)	Setup Factor (f_{su})
Silty Clay (A-6b) Depth (940.6 ft - 936.1 ft)	108	108	118	1100	100	22	1.75
Sandy Silt (A-4a) Depth (936.1 ft - 933.6 ft)	128	118	128	4000	300	27	1.50
Sandy Silt (A-4a) Depth (933.6 ft - 909.1 ft)	140	130	140	6600	450	29	1.50

Notes:
1. *Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405.*
2. *Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.*

Table 7: Soil Profile and Estimated Engineering Properties – B-008-0-24

Culvert Under U.S. 68: Soil Profile B-008-0-24							
Soil Description	Unit Weight[1] (pcf)	Moist Unit Weight[1] (pcf)	Saturated Unit Weight[1] (pcf)	Undrained Shear Strength[2] (psf)	Effective Cohesion[3] (psf)	Effective Friction Angle[3] (degrees)	Setup Factor (f_{su})
Clay (A-7-6) Depth (940.1 ft - 935.6 ft)	108	108	118	1000	100	21	2.00
Sandy Silt (A-4a) Depth (935.6 ft - 933.1 ft)	128	118	128	4350	300	28	1.50
Sandy Silt (A-4a) Depth (933.1 ft - 930.6 ft)	140	130	140	8650	450	31	1.50
Sandy Silt (A-4a) Depth (930.6 ft - 920.6 ft)	140	130	140	9200	450	32	1.50
Coarse and Fine Sand (A-3a) Depth (920.6 ft - 918.1 ft)	140	130	140	-	-	40	1.00
Silt (A-4b) Depth (918.1 ft - 915.6 ft)	140	130	140	-	-	40	1.50
Sandy Silt (A-4a) Depth (915.6 ft - 908.6 ft)	135	125	135	5850	400	29	1.50

Notes:
1. *Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405.*
2. *Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.*

5.2. Subgrade Analysis

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

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

5.2.1. Pavement Design Recommendations

It is our understanding that pavement analysis and design is to be performed to determine the proposed pavement section within the project limits to undergo the proposed improvements. A subgrade analysis was performed using the subgrade soil data obtained during our field exploration program to evaluate the soil characteristics to develop pavement parameters for use in pavement design. The subgrade analysis parameters recommended for use in pavement design are presented in Table 8 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 8: Pavement Design Parameters

BRO-68-33.36 Subgrade Analysis Summary					
Segment	**Maximum N_{60L}**	**Minimum N_{60L}**	**Average N_{60L}**	**Average PI Values**	**Design CBR**
U.S. 68	9	8	9	22	4
Greenbush Road	12	8	11	23	5
Entire Project	**12**	**8**	**10**	**24**	**4**

5.2.2. Unsuitable Soils

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

5.2.2.1. Rock

Rock was not encountered within subgrade elevation on the boring logs performed within the project limits. Per ODOT's GDM, if rock is encountered within 24 inches of the bottom of the proposed asphalt or concrete pavement it is to be removed in accordance with 204.05 of the ODOT CMS and replaced with Item 204 Embankment.

5.2.2.2. Prohibited Soils

Prohibited soil types per the GDM include A-4b, A-2-5, A-5, A-7-5, A-8a, A-8b, and soils with liquid limits greater than 65. Prohibited soils were not encountered within subgrade elevation on the boring logs performed within the project limits.

5.2.3. Unstable Soils

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

boring is analyzed. For example, if the GDM recommends an excavate and replace of 12 inches within a weak soil underlying 18 inches of stable material, it would be unreasonable to recommend the removal of both the stable and unstable material for a total of 30 inches of excavate and replace.

Information on the boring location where weak soils were encountered and determined to have a potential impact on subgrade performance is shown in Table 9 below.

Table 9: Unstable (Weak) Soils Location Summary

Boring ID	N_{60L}	Depth Below Subgrade (ft)
B-004-0-24	12	1.2 - 2.7

5.2.3.1. High Moisture Content Soils

High moisture content soils are defined by the GDM as soils that exceed the estimated optimum moisture content (per *Figure A - Optimum Moisture Content* within the GDM) for a given classification by 3 percent or more. Per the GDM, soils determined to be above the identified moisture content levels are a likely indication of the presence of an unstable subgrade and may require some form of subgrade stabilization. Similar to our analysis of weak soils, although a soil sample's moisture content may meet the criteria to be considered high, the depth in which the high moisture soil is encountered in relation to the proposed subgrade is considered when each individual subgrade boring is analyzed for stabilization recommendations. Based on the subsurface exploration performed, high moisture content soils within the proposed subgrade of the project were encountered in borings as shown in Table 10 below.

Table 10: High Moisture Content Soils Summary

Boring ID	Sample ID	Soil Type	Moisture Content (%)	Optimum Moisture Content	Moisture Above Optimum	Depth Below Subgrad
B-004-0-24	SS-2	A-7-6	22	18	4	1.2 - 2.7

5.2.4. Stabilization Recommendations

According to our subgrade analysis, unstable materials, including weak and high moisture content soils were encountered in 20 % of the proposed subgrade materials. These soils exhibited an N_{60} value below 12 bpf and/or a moisture content exceeding the optimum level by more than 3 percent within 3 feet of subgrade.

The construction of the new roundabout will require both filling and cutting. NEAS recommends performing spot stabilization on unstable subgrade areas identified through Item 204 Proof Rolling for the entire project. Spot stabilization should be conducted using the Excavate and Replace method (Item 204 with Geotextile). Excavations are estimated to extend to a depth of 12 inches, with the excavated material replaced in accordance with Section 608 "Excavate and Replace (Item 204)" of the ODOT GDM (ODOT, 2025). Stabilization limits should extend 18-inches beyond the edge of the proposed paved roadway, shoulder or median and it is recommended removing any topsoil, existing pavement materials or abandoned structure foundation materials.

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

5.3. Culvert Under U.S. 68 Analysis

A review was conducted for the proposed full-height headwalls at the culvert carrying U.S. 68 over Little Sterling Run.

5.3.1. Headwall Design Assumptions

For the design of the proposed culvert substructure, ODOT's BDM, ODOT's GDM AASHTO's LRFD BDS, and the project conditions dictate analysis parameters and design minimums/constraints are to be used in the analysis and design process. The referenced parameters and design minimums/constraints that where significant to our analyses consist of the following:

- 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).
- Retained fill soils will meet the minimum design soil parameters per ODOT's BDM Table 307-1 as shown in Table 11 below.

Table 11: Design Soil Parameters for Fill Materials

Type of Soil	Soil Unit Weight (pcf)	Friction Angle (°)	Cohesion (psf)
Granular Material Type B, per 703.16.C	120	30	0

5.3.2. External Stability

Using our estimated engineering soil properties, the headwall design assumptions outlined in Section 5.3.1 of this report, and the basemap and site plan provided by WSP USA Inc., NEAS conducted an external stability analysis of the proposed full-height inlet/outlet headwalls and wingwalls for the culvert under U.S. 68. The assumed footing dimensions are as follows: a footing width of 5.5 ft, a toe projection of 1 ft 11 in, a footing thickness of 1.5 ft, a shear key depth of 2.5 ft from the bottom of the footing, a shear key width of 1 ft, and a distance of 0 ft from the toe to the shear key.

LRFD Parameters

The headwall was evaluated resistance to bearing pressure, sliding force and overturning at the Strength Limit State in accordance with Section 11.5.3 of AASHTO's LRFD BDS. These design issues have been evaluated using a MathCAD-based software solution that follows the AASHTO BDS, as described in Appendix C. Load and resistance factors applicable to the design of a headwall are presented below in Table 12 based on AASHTO LRFD Sections 11.5.6, 11.5.7 and 10.5.5.2.2.

Table 12: Load and Resistance Factors for Culvert Headwall Analysis

Load Factors						
Group	Condition	Earth-Vertical	Earth-Horizontal	Dead load of structural components	Live Load Surcharge	Reference
		EV	EH	DC	LL	
Strength Ia	Sliding and Eccentricity	1.00	1.50	0.90	1.75	11.5.6
Strength Ib	Bearing Capacity	1.35	1.50	1.25	1.75	11.5.6
Resistance Factors						
Bearing resistance					0.55	11.5.7-1
Sliding resistance of permanent wall					1.00	11.5.7-1
Sliding resistance of passive earth pressure					0.50	10.5.5.2.2-1

Results are expressed in terms of Capacity to Demand Ratios (CDR) that compare the available factored resistances to the factored load. CDRs >=1 indicate a safe design. The CDRs calculated for the referenced cross section with respect to bearing, sliding and overturning, as well as the calculated factored bearing resistances are presented in Table 13 below (External Stability Results can be found in Appendix C).

Table 13: External Stability Analysis Summary at the Culvert under U.S. 68

Culvert Under U.S. 68		
Location	Inlet Headwall	Outlet Headwall
Top of Wall (feet)	942.8	942.5
Invert Elevation (feet)	935.3	935.0
Bottom of Footing (feet)	932.8	932.5
Exposed Wall Height (feet)	7.5	7.5
Design Wall Height (feet)	10.0	10.0
Boring Log Used in Calculation	B-008-0-24	B-007-0-24
Capactiy Demand Ratio (CDR)		
Sliding (Undrained/Drained)	9.5 / 3.4	7.6 / 3.2
Overturning / Eccentricity	4.5	4.5
Bearing Capacity (Undrained/Drained)	17.2 / 9.1	13.2 / 7.5
Factored Bearing Resistance (ksf)[(1)] (Undrained/Drained)	25.3 / 13.4	19.3 / 11.0

Notes:
1. *Bearing Resistance calculated in accordance to Section 11.10.5.4 of 2020 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of 2020 LRFD BDS.*

5.3.3. Global Stability

The slope geometry at the culvert under U.S. 68 site is assumed to be consistent with that shown in the site plans provided by WSP USA, Inc. ODOT's SGE and AASHTO's LRFD BDS dictate analysis parameters to be used in the analysis process. Based on planned roadway grades and alignment, AASHTO's LRFD BDS dictates that the slopes shall be evaluated for a live load surcharge of 250 pound per square foot (psf).

For purposes of evaluating the stability of the inlet and outlet headwall, NEAS developed one representative cross-sectional model of the tallest headwall section at each outlet location 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 site plan for the culvert provided by WSP USA, Inc.; and, 2) test borings and laboratory data developed as part of this report. With respect to the soil's engineering properties, the estimated engineering properties of the Soil Profile resented in Section 5.1. of this report were used in our analyses.

The above referenced slope stability models were analyzed for long-term (Effective Stress) and short-term (Total Stress) slope stability utilizing the software entitled *Slide 7.0* by Rocscience, Inc. Specifically, the Bishop, Spencer and GLE analysis methods were used to calculate a factor of safety (FOS) for circular type slope failure. The FOS is the ratio of the resisting forces to the driving forces, with the desired safety factor being more than about 1.5 which approximately equates to an AASHTO resistance factor of 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 the analysis, a resistance factor of 0.65 or lower is targeted as the slope does support a structural element.

Based on our slope stability analyses for the referenced headwall section, the minimum slope stability safety factor for both short-term and long-term conditions exceeded the desired value of 1.5. The results of the analyses are summarized in Table 14. The graphical output of the slope stability program (cross-sectional model, calculated safety factor, and critical failure plane) is presented in Appendix C.

Table 14: Global Stability Analysis Summary for Culvert under U.S. 68

Global Stability Analsysis at Culvert Under U.S. 68					
Location	**Boring No.**	**Description**	**Minimum Factor of Safety**	**Equivalent Resistance Factor**	**Status (OK/NG)**
Inlet	B-008-0-24	Effective Stress	3.99	0.25	OK
		Total Stress	21.42	0.05	OK
Outlet	B-007-0-24	Effective Stress	3.81	0.26	OK
		Total Stress	25.85	0.04	OK

5.1. Culvert Under Greenbush West Road Analysis

A review was conducted for the proposed full-height headwalls at the culvert carrying Greenbush West Road over Little Sterling Run.

5.1.1. External Stability

Based on the basemap and site plan provided by WSP USA, Inc., external stability was evaluated at both inlet and outlet cross-sections using the closest boring B-005-0-24 and B-002-0-24, respectively. Each cross-section was evaluated for resistance to bearing pressure, sliding force and overturning at the Strength Limit State in accordance with Section 11.5.3 of AASHTO's LRFD BDS. The assumed footing dimensions are as follows: a footing width of 6.25 ft, a toe projection of 2.25 ft, a footing thickness of 1.5 ft, a shear key depth of 2.5 ft from the bottom of the footing, a shear key width of 1 ft, and a distance of 0 ft from the toe to the shear key.

The CDRs calculated for the referenced cross sections with respect to bearing, sliding and overturning, as well as the calculated factored bearing resistances are presented in Table 15 below (External Stability Results can be found in Appendix C).

Table 15: External Stability Summary for Culvert under Greenbush West Road

Culvert Under Greenbush West Road		
Location	Inlet Headwall	Outlet Headwall
Top of Wall (feet)	942.9	942.6
Invert Elevation (feet)	934.4	934.1
Bottom of Footing (feet)	931.9	931.6
Exposed Wall Height (feet)	8.5	8.5
Design Wall Height (feet)	11.0	11.0
Boring Log Used in Calculation	B-005-0-24	B-002-0-24
Capactiy Demand Ratio (CDR)		
Sliding (Undrained/Drained)	7.0 / 2.6	4.9 / 2.1
Overturning / Eccentricity	2.9	2.8
Bearing Capacity (Undrained/Drained)	16.0 / 8.5	18.7 / 10.1
Factored Bearing Resistance (ksf)[1] (Undrained/Drained)	28.3 / 14.9	18.7 / 10.1
Notes: 1. *Bearing Resistance calculated in accordance to Section 11.10.5.4 of 2020 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of 2020 LRFD BDS.*		

5.1.2. Global Stability

The slope geometry at the culvert under Greenbush West Road site is assumed to be consistent with that shown in the site plans provided by WSP USA, Inc. ODOT's SGE and AASHTO's LRFD BDS dictate analysis parameters to be used in the analysis process. Based on planned roadway grades and alignment, AASHTO's LRFD BDS dictates that the slopes shall be evaluated for a live load surcharge of 250 pound per square foot (psf).

For purposes of evaluating the stability of the inlet and outlet headwall, NEAS developed one representative cross-sectional model of the tallest headwall section at each outlet location 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 site plan for the culvert provided by WSP USA, Inc.; and, 2) test borings and laboratory data developed as part of this report. With respect to the soil's engineering properties, the estimated engineering properties of the Soil Profile resented in Section 5.1. of this report were used in our analyses.

The above referenced slope stability models were analyzed for long-term (Effective Stress) and short-term (Total Stress) slope stability utilizing the software entitled *Slide 7.0* by Rocscience, Inc. Specifically, the Bishop, Spencer and GLE analysis methods were used to calculate a factor of safety (FOS) for circular type slope failure. The FOS is the ratio of the resisting forces to the driving forces, with the desired safety factor being more than about 1.5 which approximately equates to an AASHTO resistance factor of 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 the analysis, a resistance factor of 0.65 or lower is targeted as the slope does support a structural element.

Based on our slope stability analyses for the referenced headwall section, the minimum slope stability safety factor for both short-term and long-term conditions exceeded the desired value of 1.5. The results of the analyses are summarized in Table 16. The graphical output of the slope stability program (cross-sectional model, calculated safety factor, and critical failure plane) is presented in Appendix C.

Table 16: Global Stability Analysis Summary for Culvert under Greenbush West Road

Global Stability Analsysis at Culvert Under Greenbush West Road					
Location	Boring No.	Description	Minimum Factor of Safety	Equivalent Resistance Factor	Status (OK/NG)
Inlet	B-005-0-24	Effective Stress	3.58	0.28	OK
		Total Stress	22.62	0.04	OK
Outlet	B-002-0-24	Effective Stress	3.68	0.27	OK
		Total Stress	19.27	0.05	OK

5.2. Seismic Design Parameters

Based on the results of the subsurface exploration, laboratory test data, and the AASHTO Site Class Definitions indicated in Table 3.10.3.1-1 of the *LRFD Bridge Design Specifications, 9th Edition* (AASHTO LRFD, 2020), the average Standard Penetration Test blow count $\overline{N}$ at the project site is 36 blows/ft. Therefore, the project site is classified as Site Class of D - Stiff Soil, with $15<\overline{N}<50$ blows/ft.

6. QUALIFICATIONS

This investigation was performed in accordance with accepted geotechnical engineering practice for the purpose of characterizing the subsurface conditions for the BRO-68-33.36 project. This report has been prepared for WSP USA Inc., ODOT and their design consultants to be used solely in evaluating the soils underlying the project site and presenting geotechnical engineering recommendations specific to this project. The assessment of general site environmental conditions or the presence of pollutants in the soil, rock and groundwater of the site was beyond the scope of this geotechnical exploration. Our recommendations are based on the results of our field explorations, laboratory 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 in the nature, design or location of the proposed culverts are made, the conclusions and recommendations contained in this report should not be considered valid until they are reviewed and have been modified or verified in writing by a geotechnical engineer.

It has been a pleasure to be of service to WSP USA Inc. in performing this geotechnical exploration for the BRO-68-33.36 project. Please call if there are any questions, or if we can be of further service.

Respectfully Submitted,

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

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

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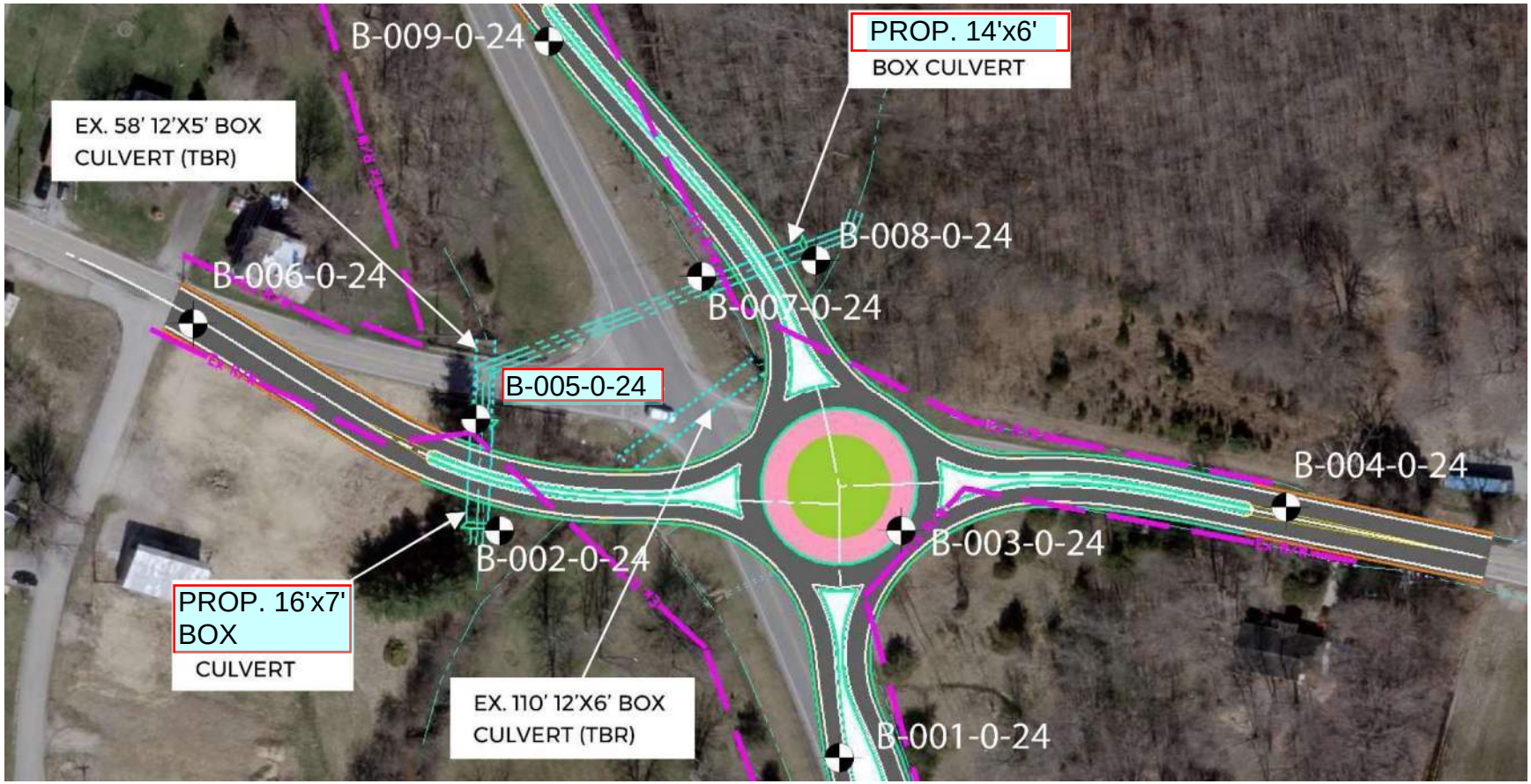
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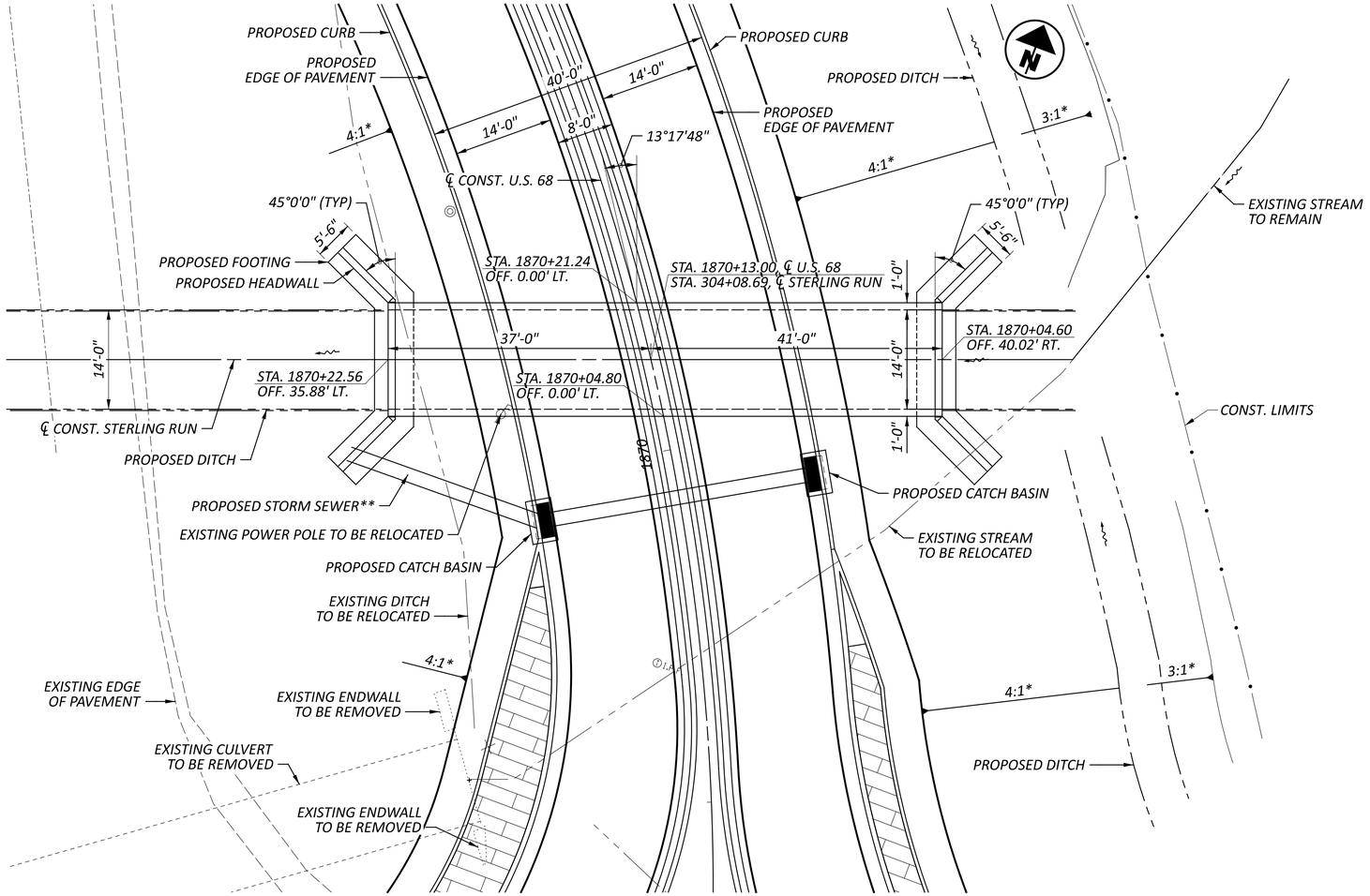
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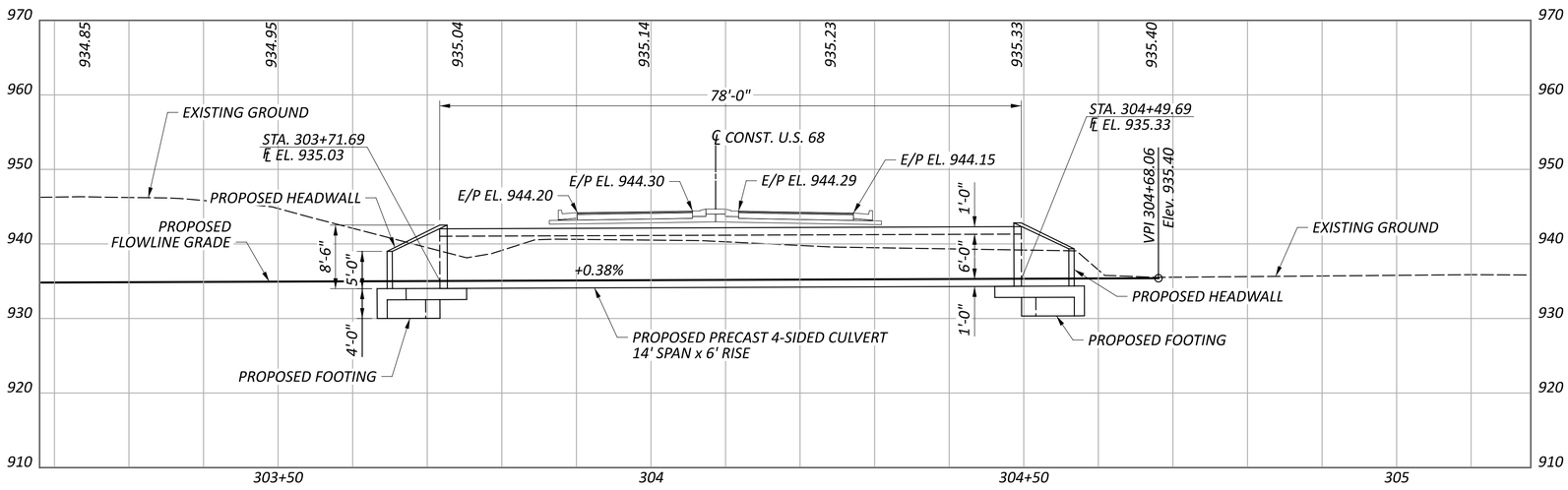
SITE PLAN



Proposed Boring Location Plan

BRO-68-33.36 / PID 120095





* GRADING TO BE FINALIZED DURING STAGE 2 DESIGN

** DRAINAGE DETAILS TO BE FINALIZED DURING STAGE 2 DESIGN

HYDRAULIC DATA

DRAINAGE AREA = 1555 ACRES

Q (1%) =	822.50 CFS	V (1%) =	14.19 FT/S	HW (1%) =	943.21 FT
Q (2%) =	732.47 CFS	V (2%) =	8.72 FT/S	HW (2%) =	942.85 FT
Q (4%) =	638.79 CFS	V (4%) =	8.18 FT/S	HW (4%) =	941.79 FT

ORDINARY HIGH WATER MARK: 937.38 FT

DESIGN SERVICE LIFE: 75 YEARS

ABRASION LEVEL: 2

pH: 8.1

EXISTING STRUCTURE

TYPE: 4-SIDED BOX (CONCRETE PRECAST)
12'-0" x 6'-0"

SPANS: 12'-0" F/F

ROADWAY: 39'-4" F/F SAFETY CURB

LOADING: HS20

SKEW: 20°

WEARING SURFACE: ASPHALT CONCRETE

APPROACH SLABS: N/A

ALIGNMENT: TANGENT

CROWN: 0.016

STRUCTURE FILE NUMBER: 0801722

DATE BUILT: 7/1/1998

DISPOSITION: TO BE REPLACED

PROPOSED STRUCTURE

TYPE: 4-SIDED BOX (CONCRETE PRECAST)
14'-0" x 6'-0"

SPANS: 14'-4½" F/F

ROADWAY: 40'-0" F/F SAFETY CURB

LOADING: HL93 AND 60 PSF FUTURE WEARING SURFACE

SKEW: 13°17'48"

WEARING SURFACE: ASPHALT CONCRETE

APPROACH SLABS: N/A

ALIGNMENT: TANGENT

CROWN: 0.016 FT/FT

DECK AREA: 640 SF

COORDINATES: LATITUDE 39.075726

LONGITUDE -83.925516

CULVERT DETAILS
US 68 OVER STERLING RUN
STA. 1870+13.00

SFN TBD

DESIGN AGENCY TRC

DESIGNER	CHECKER
JCP	CEJ

REVIEWER CJW 01/30/25

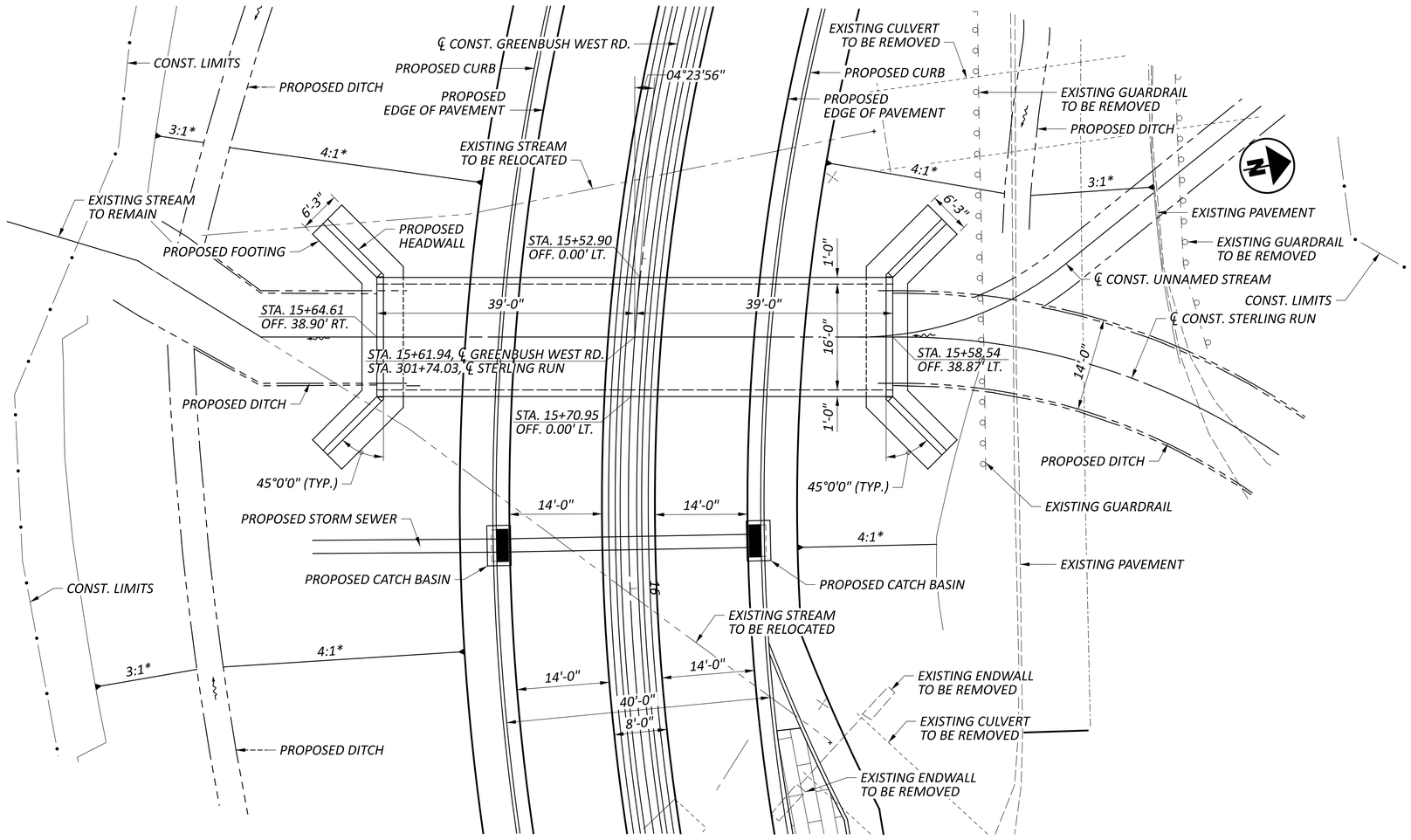
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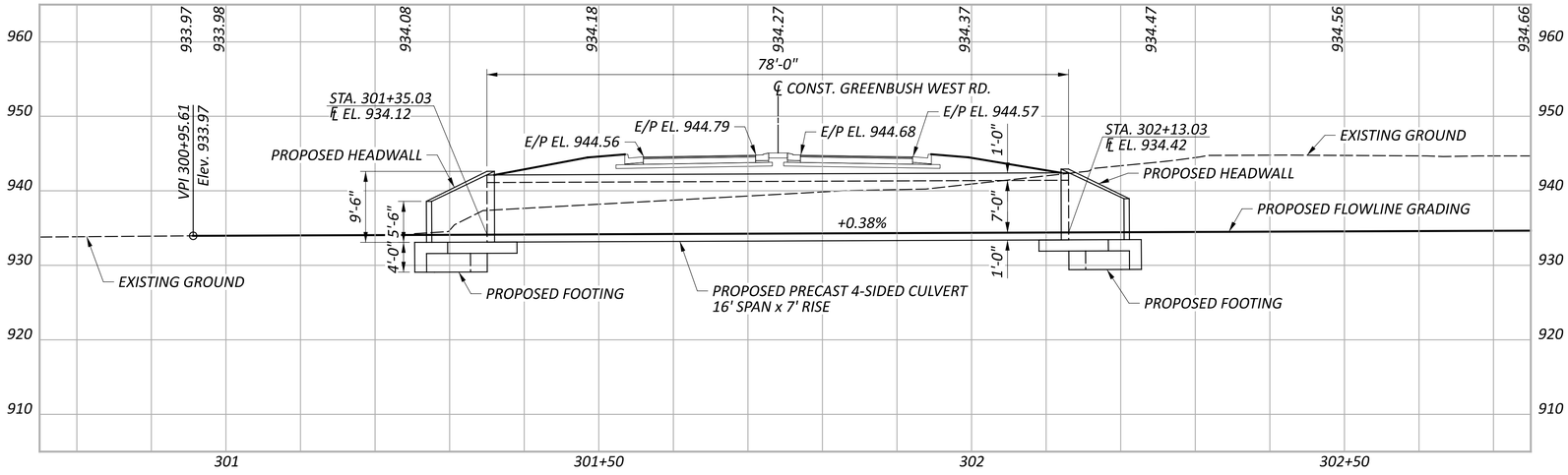
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1	2

SHEET	TOTAL
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BRO-68-33.36

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* GRADING TO BE FINALIZED DURING STAGE 2 DESIGN

HYDRAULIC DATA

DRAINAGE AREA = 1882 ACRES

Q (1%) =	950.79 CFS	V (1%) =	14.50 FT/S	HW (1%) =	942.01 FT
Q (2%) =	846.88 CFS	V (2%) =	13.99 FT/S	HW (2%) =	941.29 FT
Q (4%) =	738.99 CFS	V (4%) =	13.40 FT/S	HW (4%) =	940.60 FT

ORDINARY HIGH WATER MARK: 937.69 FT
DESIGN SERVICE LIFE: 75 YEARS
ABRASION LEVEL: 2
pH: 8.1

EXISTING STRUCTURE

TYPE: 4-SIDED BOX (CONCRETE PRECAST)
12'-0" x 5'-0"

SPANS: 12'-0" F/F
ROADWAY: 29'-10" F/F GUARDRAIL
LOADING: HS20
SKEW: 0°
WEARING SURFACE: ASPHALT CONCRETE
APPROACH SLABS: N/A
ALIGNMENT: TANGENT
CROWN: 0.016
STRUCTURE FILE NUMBER: 0835218
DATE BUILT: 7/1/2000
DISPOSITION: TO BE REPLACED

PROPOSED STRUCTURE

TYPE: 4-SIDED BOX (CONCRETE PRECAST)
16'-0" x 7'-0"

SPANS: 16'-½" F/F
ROADWAY: 40'-0" F/F SAFETY CURB
LOADING: HL93 AND 60 PSF FUTURE WEARING SURFACE
SKEW: 4°23'56"
WEARING SURFACE: ASPHALT CONCRETE
APPROACH SLABS: N/A
ALIGNMENT: TANGENT
CROWN: 0.016 FT/FT
DECK AREA: 720 SF

COORDINATES: LATITUDE 39.075358
LONGITUDE -83.926077

CULVERT DETAILS
GREENBUSH ROAD OVER STERLING RUN
STA. 15+61.94

SFN	TBD
DESIGN AGENCY	TRC
DESIGNER	JCP
CHECKER	CEJ
REVIEWER	CJW 01/30/25
PROJECT ID	120095
SUBSET	2
TOTAL	2
SHEET	P.76
TOTAL	76

BRO-68-33.36

MODEL: CLP_Sterling - West Culvert Plan [Sheet] PAPERSIZE: 34x22 (in.) DATE: 1/31/2025 TIME: 6:53:56 PM USER: USEK712839
pw:\\wsp-us-pw.bentley.com:wsp-us-pw-02\Documents\OH - OHIO Projects\120095\401-Engineering_TRC\Drainage\Sheets\120095_DC102.dgn

APPENDIX B

BORING LOGS

PROJECT: BRO-68-33.36	DRILLING FIRM / OPERATOR: NEAS INC. / RW	DRILL RIG: CME 45B	STATION / OFFSET: 1866+67, 15' LT.
TYPE: SUBGRADE	SAMPLING FIRM / LOGGER: NEAS INC. / RW	HAMMER: CME AUTOMATIC	ALIGNMENT: PR. U.S. 68
PID: 120095 SFN:	DRILLING METHOD: 2.25" HSA	CALIBRATION DATE: 3/8/24	ELEVATION: 945.9 (MSL) EOB: 7.5 ft.
START: 3/11/25 END: 3/11/25	SAMPLING METHOD: SPT	ENERGY RATIO (%): 79	LAT / LONG: 39.074816, -83.925335

EXPLORATION ID B-001-0-24

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
16.0" ASPHALT AND 4.0" BASE (DRILLERS DESCRIPTION)	945.9																		
	944.2																		
STIFF TO VERY STIFF, BROWN AND GRAY, **CLAY**, "AND" SILT, SOME SAND, TRACE GRAVEL, SLIGHTLY ORGANIC, DAMP		1	6 5 7	16	28	SS-1	2.25	4	5	19	39	33	42	18	24	17	A-7-6 (13)	93	
	941.4	2	1 6 6	16	39	SS-2	2.00	-	-	-	-	-	-	-	-	18	A-7-6 (V)	-	
STIFF, GRAY AND BROWN, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, MOIST		3 4 5	1 3 4	9	33	SS-3	1.50	1	3	12	51	33	37	19	18	21	A-6b (11)	-	
	938.4	6 7	5 6 7	17	72	SS-4	1.25	-	-	-	-	-	-	-	-	22	A-6b (V)	-	
		EOB																	

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE CAVE-IN AT 1.5'. DRILLED AS STAKED.

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

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 3/25/25 09:48 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

PROJECT: BRO-68-33.36	DRILLING FIRM / OPERATOR: NEAS INC. / J. HODGES	DRILL RIG: CME 55X	STATION / OFFSET: 15+65, 9' RT.	EXPLORATION ID B-002-0-24
TYPE: CULVERT	SAMPLING FIRM / LOGGER: NEAS INC. / J. HODGES	HAMMER: CME AUTOMATIC	ALIGNMENT: PR. GREENBUSH WEST RD	
PID: 120095 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/8/24	ELEVATION: 944.5 (MSL) EOB: 35.0 ft.	PAGE 1 OF 2
START: 3/13/25 END: 3/13/25	SAMPLING METHOD: SPT	ENERGY RATIO (%): 80	LAT / LONG: 39.075312, -83.926059	

MATERIAL DESCRIPTION AND NOTES	ELEV. 944.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
VERY STIFF, BROWN MOTTLED WITH GRAY AND ORANGISH BROWN, **CLAY**, SOME SILT, SOME SAND, TRACE GRAVEL, IRON STAINING, MOIST		1 2 3 4	1 2 3	7	67	SS-1	2.25	9	6	24	33	28	41	15	26	20	A-7-6 (12)	-	
	940.0																		
HARD, BROWN MOTTLED WITH GRAY AND ORANGISH BROWN, **SILT AND CLAY**, SOME SAND, TRACE GRAVEL, IRON STAINING, DAMP		5 6 7	6 10 16	35	100	SS-2	4.50	7	11	17	36	29	27	16	11	11	A-6a (6)	-	
	937.5																		
HARD, GRAY, **SANDY SILT**, LITTLE TO SOME CLAY, LITTLE TO SOME GRAVEL, DAMP		8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29	7 18 27	60	100	SS-3	4.50	23	10	18	31	18	20	13	7	7	A-4a (3)	-	
			14 27 11	51	56	SS-4	4.50	17	10	17	32	24	26	16	10	11	A-4a (4)	-	
			10 23 40	84	100	SS-5	4.50	-	-	-	-	-	-	-	-	9	A-4a (V)	-	
			25 25 22	63	100	SS-6	4.50	-	-	-	-	-	-	-	-	10	A-4a (V)	-	
			10 16 21	49	100	SS-7	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	
			11 13 22	47	100	SS-8	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	
			5 15 22	49	100	SS-9	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	
			11 12 18	40	100	SS-10	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	
			9 12 17	39	100	SS-11	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 3/25/25 09:48 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

PID: 120095	SFN:	PROJECT: BRO-68-33.36	STATION / OFFSET: 15+65, 9' RT.	START: 3/13/25	END: 3/13/25	PG 2 OF 2	B-002-0-24

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
HARD, GRAY, **SANDY SILT**, LITTLE TO SOME CLAY, LITTLE TO SOME GRAVEL, DAMP *(continued)*	914.5	31	6 9 15	32	72	SS-12	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	
	912.2	32																	
VERY STIFF, GRAYISH BROWN, **CLAY**, "AND" SILT, LITTLE SAND, TRACE GRAVEL, DAMP		33																	
		34	5 9 16	33	100	SS-13	3.00	0	2	14	39	45	41	18	23	18	A-7-6 (13)	-	
	909.5	EOB 35																	

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

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 3/25/25 09:48 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

PROJECT: BRO-68-33.36	DRILLING FIRM / OPERATOR: NEAS INC. / RW	DRILL RIG: CME 45B	STATION / OFFSET: 1868+20, 40' RT.	EXPLORATION ID B-003-0-24
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: NEAS INC. / RW	HAMMER: CME AUTOMATIC	ALIGNMENT: PR. U.S. 68	
PID: 120095 SFN:	DRILLING METHOD: 2.25" HSA	CALIBRATION DATE: 3/8/24	ELEVATION: 942.4 (MSL) EOB: 10.0 ft.	PAGE 1 OF 1
START: 3/11/25 END: 3/11/25	SAMPLING METHOD: SPT	ENERGY RATIO (%): 79	LAT / LONG: 39.075245, -83.925177	

MATERIAL DESCRIPTION AND NOTES	ELEV. 942.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
6.0" TOPSOIL (DRILLERS DESCRIPTION)	941.9																		
STIFF TO VERY STIFF, BROWN MOTTLED WITH GRAY AND ORANGISH BROWN, **CLAY**, "AND" SILT, LITTLE SAND, TRACE GRAVEL, MOIST TO DAMP		1	3 3 3	8	61	SS-1	1.75	1	3	13	38	45	58	24	34	26	A-7-6 (20)	147	
		2																	
		3																	
		4	6 8 7	20	44	SS-2	2.00	-	-	-	-	-	-	-	-	13	A-7-6 (V)	-	
	936.9	5																	
VERY STIFF TO HARD, GRAY AND BROWN, **SANDY SILT**, LITTLE TO SOME CLAY, LITTLE TO SOME GRAVEL, DAMP		6	10 25 17	55	61	SS-3	3.75	17	12	16	32	23	24	15	9	12	A-4a (4)	-	
		7																	
		8																	
		9	20 25 27	68	50	SS-4	4.50	22	13	17	28	20	23	13	10	8	A-4a (3)	-	
	932.4	EOB 10																	

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE CAVE-IN AT 8.5'. DRILLED AS STAKED.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 3/25/25 09:48 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

PROJECT: BRO-68-33.36	DRILLING FIRM / OPERATOR: NEAS INC. / RW	DRILL RIG: CME 45B	STATION / OFFSET: 20+62, 4' LT.
TYPE: SUBGRADE	SAMPLING FIRM / LOGGER: NEAS INC. / RW	HAMMER: CME AUTOMATIC	ALIGNMENT: PR. GREENBUSH EAST RD
PID: 120095 SFN:	DRILLING METHOD: 2.25" HSA	CALIBRATION DATE: 3/8/24	ELEVATION: 948.3 (MSL) EOB: 7.5 ft.
START: 3/11/25 END: 3/11/25	SAMPLING METHOD: SPT	ENERGY RATIO (%): 79	LAT / LONG: 39.075307, -83.924312

EXPLORATION ID B-004-0-24

MATERIAL DESCRIPTION AND NOTES	ELEV. 948.3	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
14.0" ASPHALT AND 4.0" BASE (DRILLERS DESCRIPTION)	946.8	1																	
MEDIUM DENSE, BROWN, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP	945.3	2	9 10 6	21	17	SS-1	-	17	21	28	24	10	NP	NP	NP	8	A-2-4 (0)	20	
VERY STIFF, BROWN MOTTLED WITH GRAY AND ORANGISH BROWN, **CLAY**, "AND" SILT, LITTLE SAND, TRACE GRAVEL, IRON STAINING, MOIST TO DAMP		3	1 3 6	12	39	SS-2	2.25	1	3	11	47	38	44	21	23	22	A-7-6 (14)	-	
		4	2 3 7	13	44	SS-3	4.00	-	-	-	-	-	-	-	-	21	A-7-6 (V)	-	
	940.8	5	2 5 8	17	61	SS-4	4.00	-	-	-	-	-	-	-	-	19	A-7-6 (V)	-	
		6 EOB																	
		7																	

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE CAVE-IN AT 2.3'. DRILLED AS STAKED.

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

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 3/25/25 09:48 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

PROJECT: BRO-68-33.36	DRILLING FIRM / OPERATOR: NEAS INC. / J. HODGES	DRILL RIG: CME 55X	STATION / OFFSET: 14+94, 39' LT.	EXPLORATION ID B-005-0-24	
TYPE: CULVERT	SAMPLING FIRM / LOGGER: NEAS INC. / J. HODGES	HAMMER: CME AUTOMATIC	ALIGNMENT: PR. GREENBUSH WEST RD		
PID: 120095 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/8/24	ELEVATION: 940.3 (MSL) EOB: 35.0 ft.		PAGE 1 OF 2
START: 3/13/25 END: 3/13/25	SAMPLING METHOD: SPT	ENERGY RATIO (%): 80	LAT / LONG: 39.075500, -83.926200		

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
VERY STIFF, ORANGISH BROWN AND GRAY, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, IRON STAINING, DAMP	940.3																		
	935.8	1	1 3 5	11	44	SS-1	2.75	7	15	13	31	34	38	20	18	19	A-6b (9)	-	
DENSE, BROWN AND GRAY, **SANDY SILT**, TRACE CLAY, TRACE GRAVEL, DAMP	933.3	5	9 16 18	45	100	SS-2	-	4	12	35	39	10	NP	NP	NP	14	A-4a (3)	-	
HARD, GRAY, **SANDY SILT**, LITTLE TO SOME CLAY, LITTLE GRAVEL, DAMP		8	7 17 32	65	89	SS-3	4.50	13	12	16	33	26	25	15	10	8	A-4a (5)	-	
		11	11 25 48	97	100	SS-4	4.50	18	15	19	31	17	21	14	7	7	A-4a (3)	-	
		13	12 29 34	84	100	SS-5	4.50	-	-	-	-	-	-	-	-	8	A-4a (V)	-	
		15	35 50	-	92	SS-6	4.50	-	-	-	-	-	-	-	-	8	A-4a (V)	-	
		18	7 14 20	45	100	SS-7	4.50	-	-	-	-	-	-	-	-	10	A-4a (V)	-	
		20	9 14 25	52	100	SS-8	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	
		23	4 12 18	40	100	SS-9	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	
		25	5 10 16	35	100	SS-10	4.50	-	-	-	-	-	-	-	-	12	A-4a (V)	-	
		28	5 9 15	32	100	SS-11	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 4/17/25 08:21 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

PID: 120095	SFN:	PROJECT: BRO-68-33.36	STATION / OFFSET: 14+95, 26' LT.	START: 3/13/25	END: 3/13/25	PG 2 OF 2	B-005-0-24

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
HARD, GRAY, **SANDY SILT**, LITTLE TO SOME CLAY, LITTLE GRAVEL, DAMP *(continued)*	910.3	31	7 14 21	47	100	SS-12	4.50	-	-	-	-	-	-	-	-	12	A-4a (V)	-	
	908.0	32																	
HARD, GRAYISH BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP		33																	
	905.3	34	5 9 16	33	100	SS-13	4.25	9	2	13	44	32	30	16	14	14	A-6a (10)	-	
		EOB 35																	

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. BORING OFFSET 6.0' WEST.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 3/25/25 09:48 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

PROJECT: BRO-68-33.36	DRILLING FIRM / OPERATOR: NEAS INC. / RW	DRILL RIG: CME 45B	STATION / OFFSET: 12+98, 9' LT.
TYPE: SUBGRADE	SAMPLING FIRM / LOGGER: NEAS INC. / RW	HAMMER: CME AUTOMATIC	ALIGNMENT: PR. GREENBUSH WEST RD
PID: 120095 SFN:	DRILLING METHOD: 2.25" HSA	CALIBRATION DATE: 3/8/24	ELEVATION: 947.8 (MSL) EOB: 7.5 ft.
START: 3/11/25 END: 3/11/25	SAMPLING METHOD: SPT	ENERGY RATIO (%): 79	LAT / LONG: 39.075709, -83.926859

EXPLORATION ID B-006-0-24

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 947.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
16.0" ASPHALT AND 4.0" BASE (DRILLERS DESCRIPTION)	946.1	1																	
HARD, BROWN, **SANDY SILT**, LITTLE CLAY, TRACE GRAVEL, DAMP	944.8	2	6 4 5	12	11	SS-1	4.50	-	-	-	-	-	-	-	-	12	A-4a (V)	-	
VERY STIFF, ORANGISH BROWN AND GRAY, **CLAY**, "AND" SILT, LITTLE TO SOME SAND, TRACE GRAVEL, IRON STAINING, MOIST TO DAMP		3	1 5 7	16	50	SS-2	2.75	1	3	15	46	35	48	19	29	20	A-7-6 (17)	80	
		4	3 6 8	18	56	SS-3	2.50	0	3	18	40	39	52	20	32	20	A-7-6 (18)	-	
		5	6 7 9	21	61	SS-4	2.75	-	-	-	-	-	-	-	-	19	A-7-6 (V)	-	
	940.3	6 EOB																	
		7																	

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE CAVE-IN AT 1.5'. DRILLED AS STAKED.

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

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 3/25/25 09:48 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

PROJECT: BRO-68-33.36	DRILLING FIRM / OPERATOR: NEAS INC. / J. HODGES	DRILL RIG: CME 55X	STATION / OFFSET: 1870+66, 38' LT.	EXPLORATION ID B-007-0-24
TYPE: CULVERT	SAMPLING FIRM / LOGGER: NEAS INC. / J. HODGES	HAMMER: CME AUTOMATIC	ALIGNMENT: PR. U.S. 68	
PID: 120095 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/8/24	ELEVATION: 940.6 (MSL) EOB: 31.5 ft.	PAGE 1 OF 2
START: 3/14/25 END: 3/14/25	SAMPLING METHOD: SPT	ENERGY RATIO (%): 80	LAT / LONG: 39.075785, -83.925722	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
VERY STIFF, BROWN MOTTLED WITH ORANGISH BROWN AND GRAY, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, IRON STAINING, MOIST	940.6																		
	936.1	3	3 3 4	9	44	SS-1	3.00	0	2	33	40	25	35	16	19	18	A-6b (10)	-	
HARD, BROWN, **SANDY SILT**, SOME CLAY, LITTLE GRAVEL, DAMP	933.6	6	3 7 17	32	100	SS-2	4.50	12	14	18	33	23	24	15	9	10	A-4a (4)	-	
HARD, GRAY, **SANDY SILT**, SOME CLAY, TRACE TO LITTLE GRAVEL, DAMP		8	5 14 21	47	100	SS-3	4.50	11	11	16	33	29	26	16	10	9	A-4a (5)	-	
		11	12 27 40	89	100	SS-4	4.50	8	12	20	37	23	22	14	8	7	A-4a (5)	-	
		13	25 24 27	68	100	SS-5	4.50	-	-	-	-	-	-	-	-	9	A-4a (V)	-	
		16	13 16 30	61	100	SS-6	4.50	-	-	-	-	-	-	-	-	9	A-4a (V)	-	
		18	15 21 25	61	50	SS-7	4.50	-	-	-	-	-	-	-	-	7	A-4a (V)	-	
		21	18 16 23	52	89	SS-8	4.50	-	-	-	-	-	-	-	-	9	A-4a (V)	-	
		23	17 18 26	59	100	SS-9	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	
		26	9 13 24	49	89	SS-10	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	
		28	16 19 24	57	100	SS-11	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 3/25/25 09:48 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

PID: 120095	SFN:	PROJECT: BRO-68-33.36	STATION / OFFSET: 1870+66, 38' LT.	START: 3/14/25	END: 3/14/25	PG 2 OF 2	B-007-0-24

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
HARD, GRAY, **SANDY SILT**, SOME CLAY, TRACE TO LITTLE GRAVEL, DAMP *(continued)*	910.6																		
	909.1	EOB 31	14 14 22	48	72	SS-12	4.50	-	-	-	-	-	-	-	-	12	A-4a (V)	-	

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

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 3/25/25 09:48 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

PROJECT: BRO-68-33.36	DRILLING FIRM / OPERATOR: NEAS INC. / J. HODGES	DRILL RIG: CME 55X	STATION / OFFSET: 1870+30, 36' RT.	EXPLORATION ID B-008-0-24
TYPE: CULVERT	SAMPLING FIRM / LOGGER: NEAS INC. / J. HODGES	HAMMER: CME AUTOMATIC	ALIGNMENT: PR. U.S. 68	
PID: 120095 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/8/24	ELEVATION: 940.1 (MSL) EOB: 31.5 ft.	PAGE 1 OF 2
START: 3/14/25 END: 3/14/25	SAMPLING METHOD: SPT	ENERGY RATIO (%): 80	LAT / LONG: 39.075840, -83.925440	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
VERY STIFF, DARK GRAY, **CLAY**, SOME SILT, SOME SAND, TRACE GRAVEL, SLIGHTLY ORGANIC, MOIST	940.1																		
		1																	
		2																	
		3	2 3 3	8	89	SS-1	2.25	1	5	26	33	35	46	20	26	21	A-7-6 (14)	-	
	935.6	4																	
HARD, ORANGISH BROWN, **SANDY SILT**, SOME CLAY, LITTLE GRAVEL, DAMP		5																	
		6	3 10 16	35	33	SS-2	4.50	16	12	17	32	23	26	16	10	10	A-4a (4)	-	
	933.1	7																	
HARD, GRAY, **SANDY SILT**, SOME STONE FRAGMENTS, LITTLE CLAY, CONTAINS NO INTACT SOIL FOR HP READINGS, DAMP		8	12 22 33	73	100	SS-3	-	28	11	14	27	20	24	14	10	7	A-4a (2)	-	
	930.6	9																	
HARD, GRAY AND BROWN, **SANDY SILT**, LITTLE CLAY, LITTLE GRAVEL, DAMP		10																	
		11	16 31 50	108	100	SS-4	4.50	14	12	20	35	19	21	13	8	7	A-4a (4)	-	
		12																	
		13	17 24 30	72	100	SS-5	4.50	-	-	-	-	-	-	-	-	8	A-4a (V)	-	
		14																	
		15																	
		16	14 23 27	67	100	SS-6	4.50	-	-	-	-	-	-	-	-	9	A-4a (V)	-	
		17																	
		18	7 42 29	95	100	SS-7	4.50	-	-	-	-	-	-	-	-	8	A-4a (V)	-	
	920.6	19																	
VERY DENSE, GRAY, **COARSE AND FINE SAND**, TRACE SILT, TRACE CLAY, TRACE GRAVEL, MOIST		20																	
		21	14 31 50	108	89	SS-8	-	-	-	-	-	-	-	-	-	13	A-3a (V)	-	
	918.1	22																	
VERY DENSE, GRAY, **SILT**, SOME TO "AND" SAND, TRACE CLAY, TRACE GRAVEL, DAMP		23	16 50	-	83	SS-9	-	-	-	-	-	-	-	-	-	15	A-4b (V)	-	
	915.6	24																	
HARD, GRAY, **SANDY SILT**, LITTLE CLAY, TRACE GRAVEL, DAMP		25																	
		26	5 15 24	52	100	SS-10	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	
		27																	
		28	6 12 25	49	100	SS-11	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	
		29																	

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 3/25/25 09:49 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

PID: 120095	SFN:	PROJECT: BRO-68-33.36	STATION / OFFSET: 1870+30, 36' RT.	START: 3/14/25	END: 3/14/25	PG 2 OF 2	B-008-0-24

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
HARD, GRAY, **SANDY SILT**, LITTLE CLAY, TRACE GRAVEL, DAMP *(continued)*	910.1																		
	908.6	EOB 31	7 11 19	40	100	SS-12	4.50	-	-	-	-	-	-	-	-	11	A-4a (V)	-	

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

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 3/25/25 09:49 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

PROJECT: BRO-68-33.36	DRILLING FIRM / OPERATOR: NEAS INC. / RW	DRILL RIG: CME 45B	STATION / OFFSET: 1872+20, 1' RT.	EXPLORATION ID B-009-0-24
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: NEAS INC. / RW	HAMMER: CME AUTOMATIC	ALIGNMENT: PR. US-68	
PID: 120095 SFN:	DRILLING METHOD: 2.25" HSA	CALIBRATION DATE: 3/8/24	ELEVATION: 944.4 (MSL) EOB: 10.0 ft.	PAGE 1 OF 1
START: 3/11/25 END: 3/11/25	SAMPLING METHOD: SPT	ENERGY RATIO (%): 79	LAT / LONG: 39.076151, -83.926012	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
6.0" TOPSOIL (DRILLERS DESCRIPTION)	944.4																		
STIFF TO VERY STIFF, BROWN MOTTLED WITH ORANGISH BROWN AND GRAY, **CLAY**, "AND" SILT, LITTLE SAND, TRACE GRAVEL, IRON STAINING, MOIST	943.9	1	2 3 4	9	56	SS-1	2.25	1	3	13	44	39	53	19	34	23	A-7-6 (19)	293	
		2																	
		3																	
		4	2 4 7	14	67	SS-2	2.00	1	2	11	50	36	42	19	23	21	A-7-6 (14)	-	
		5																	
		6	2 3 6	12	61	SS-3	2.50	-	-	-	-	-	-	-	-	22	A-7-6 (V)	-	
		7																	
		8																	
		9	3 3 5	11	67	SS-4	2.25	-	-	-	-	-	-	-	-	22	A-7-6 (V)	-	
	934.4	EOB 10																	

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE CAVE-IN AT 8.0'. DRILLED AS STAKED.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT LOG W/ SULFATES (8.5 X 11) - OH DOT.GDT - 3/25/25 09:49 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

OHIO DEPARTMENT OF TRANSPORTATION

DETERMINING SULFATE CONTENT IN SOILS

SUPPLEMENT 1122

Project C-R-S:	BRO-68-33.36
PID No:	120095
Report Date:	3/19/2025
Consultant:	NEAS Inc.
Technician:	E. Gurhan

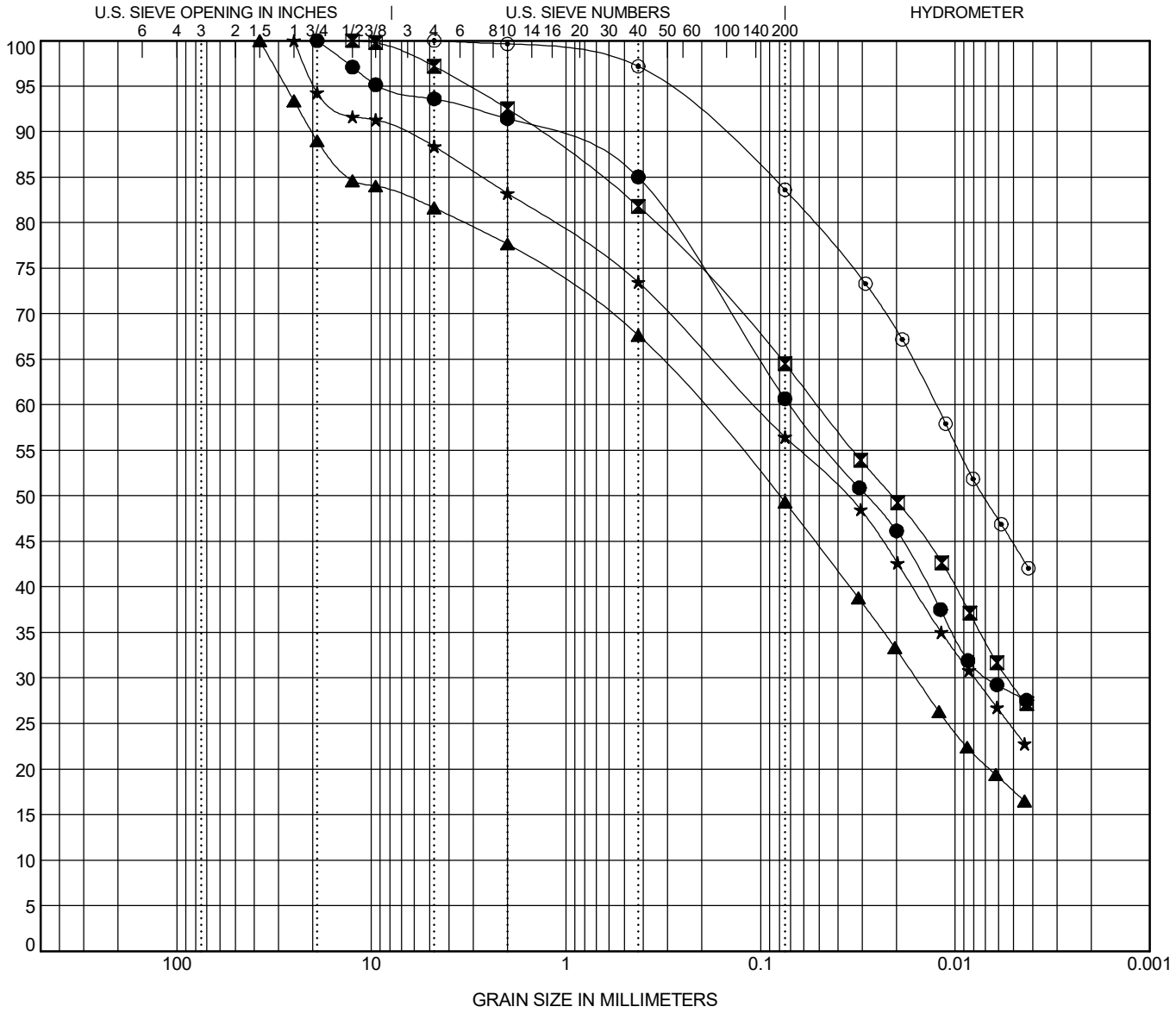
Boring ID & Sample #	Station	Offset	Latitude & Longitude or State Plane Coordinates		Elevation	Soaking Time (hr)	Replicate Sample Readings						Sulfate Content (ppm)
							1		2		3		
							Dilution	Reading	Dilution	Reading	Dilution	Reading	
B-001-0-24 SS-1	1866+67	15L	39.07481562	-83.92533479	945.89	19.4	20	5	20	6	20	3	**93**
B-003-0-24 SS-1	1868+20	40R	39.07524512	-83.92517664	942.37	19.4	20	7	20	7	20	8	**147**
B-004-0-24 SS-1	20+62	4L	39.07530689	-83.92431174	948.348	19.4	20	1	20	1	20	1	**20**
B-006-0-24 SS-2	12+98	9L	39.07570861	-83.9268593	947.791	19.4	20	4	20	5	20	3	**80**
B-009-0-24 SS-1	1872+20	1R	39.07615093	-83.9260115	944.376	19.4	20	13	20	15	20	16	**293**

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT BRO-68-33.36 **PID** 120095

OGE NUMBER 0 **PROJECT TYPE**



COBBLES	GRAVEL	SAND		SILT	CLAY
		coarse	fine		

Specimen Identification			ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	B-002-0-24	2.5	**A-7-6 ~ SANDY LEAN CLAY(CL)**	**41**	**15**	**26**
⊠	B-002-0-24	5.0	**A-6a ~ SANDY LEAN CLAY(CL)**	**27**	**16**	**11**
▲	B-002-0-24	7.5	**A-4a ~ SILTY, CLAYEY SAND with GRAVEL(SC-SM)**	**20**	**13**	**7**
★	B-002-0-24	10.0	**A-4a ~ SANDY LEAN CLAY(CL)**	**26**	**16**	**10**
⊙	B-002-0-24	33.5	**A-7-6 ~ LEAN CLAY with SAND(CL)**	**41**	**18**	**23**

Specimen Identification			D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	B-002-0-24	2.5	**1.416**	**0.029**	**0.007**		**9**	**6**	**24**	**33**	**28**		
⊠	B-002-0-24	5.0	**1.388**	**0.021**	**0.005**		**7**	**11**	**17**	**36**	**29**		
▲	B-002-0-24	7.5	**20.22**	**0.08**	**0.016**		**23**	**10**	**18**	**31**	**18**		
★	B-002-0-24	10.0	**6.973**	**0.036**	**0.008**		**17**	**10**	**17**	**32**	**24**		
⊙	B-002-0-24	33.5	**0.17**	**0.007**			**0**	**2**	**14**	**39**	**45**		

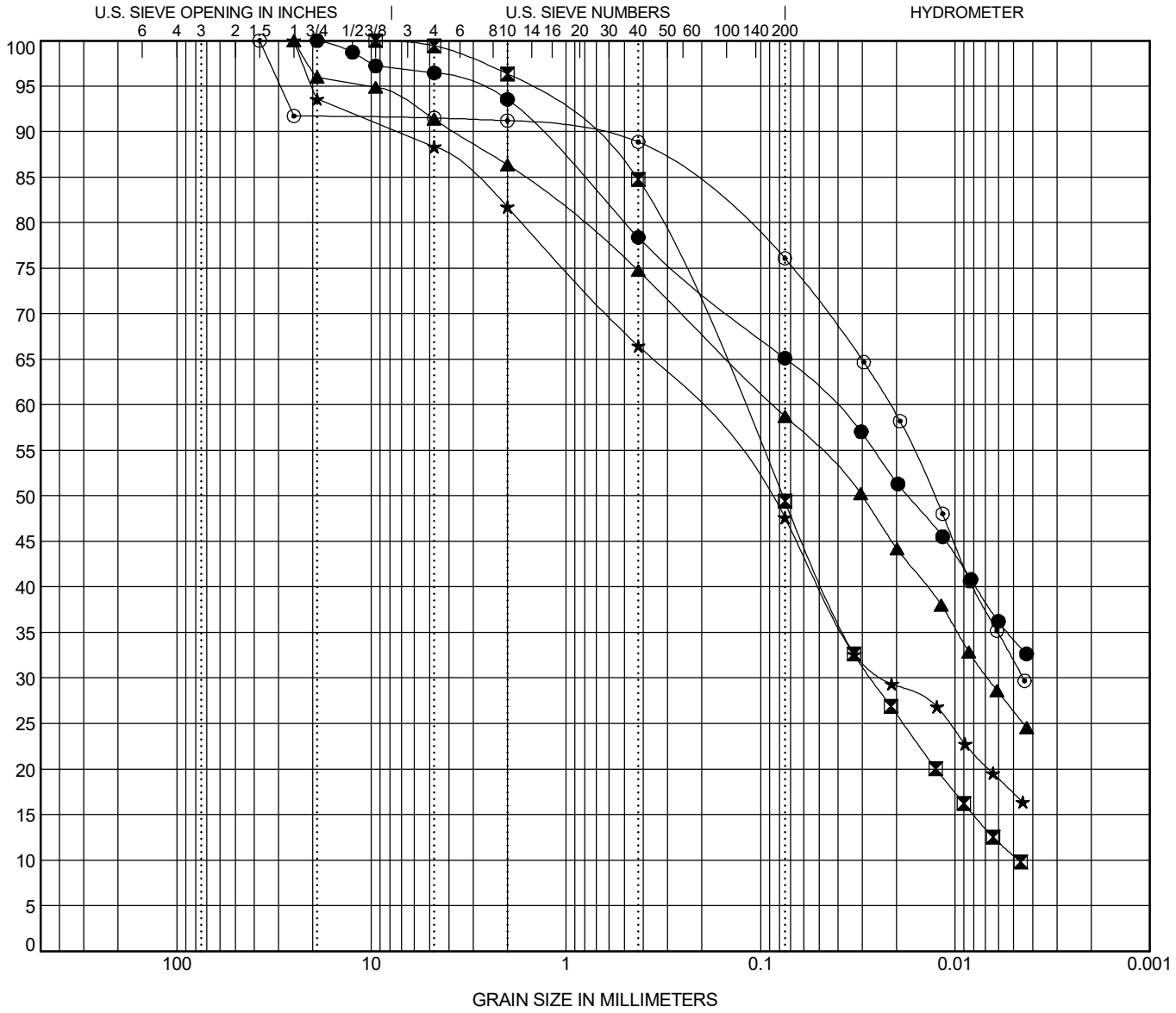
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GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT BRO-68-33.36 **PID** 120095

OGE NUMBER 0 **PROJECT TYPE**



COBBLES	GRAVEL	SAND		SILT	CLAY
		coarse	fine		

Specimen Identification			ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	B-005-0-24	2.5	A-6b ~ SANDY LEAN CLAY(CL)	38	20	18
⊠	B-005-0-24	5.0	A-4a ~ SILTY SAND(SM)	NP	NP	NP
▲	B-005-0-24	7.5	A-4a ~ SANDY LEAN CLAY(CL)	25	15	10
★	B-005-0-24	10.0	A-4a ~ SILTY, CLAYEY SAND(SC-SM)	21	14	7
⊙	B-005-0-24	33.5	A-6a ~ LEAN CLAY with SAND(CL)	30	16	14

Specimen Identification			D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	B-005-0-24	2.5	1.391	0.017			7	15	13	31	34		
⊠	B-005-0-24	5.0	0.854	0.077	0.027	0.005	4	12	35	39	10	1.24	26.78
▲	B-005-0-24	7.5	3.745	0.03	0.007		13	12	16	33	26		
★	B-005-0-24	10.0	7.361	0.093	0.023		18	15	19	31	17		
⊙	B-005-0-24	33.5	0.894	0.013	0.004		9	2	13	44	32		

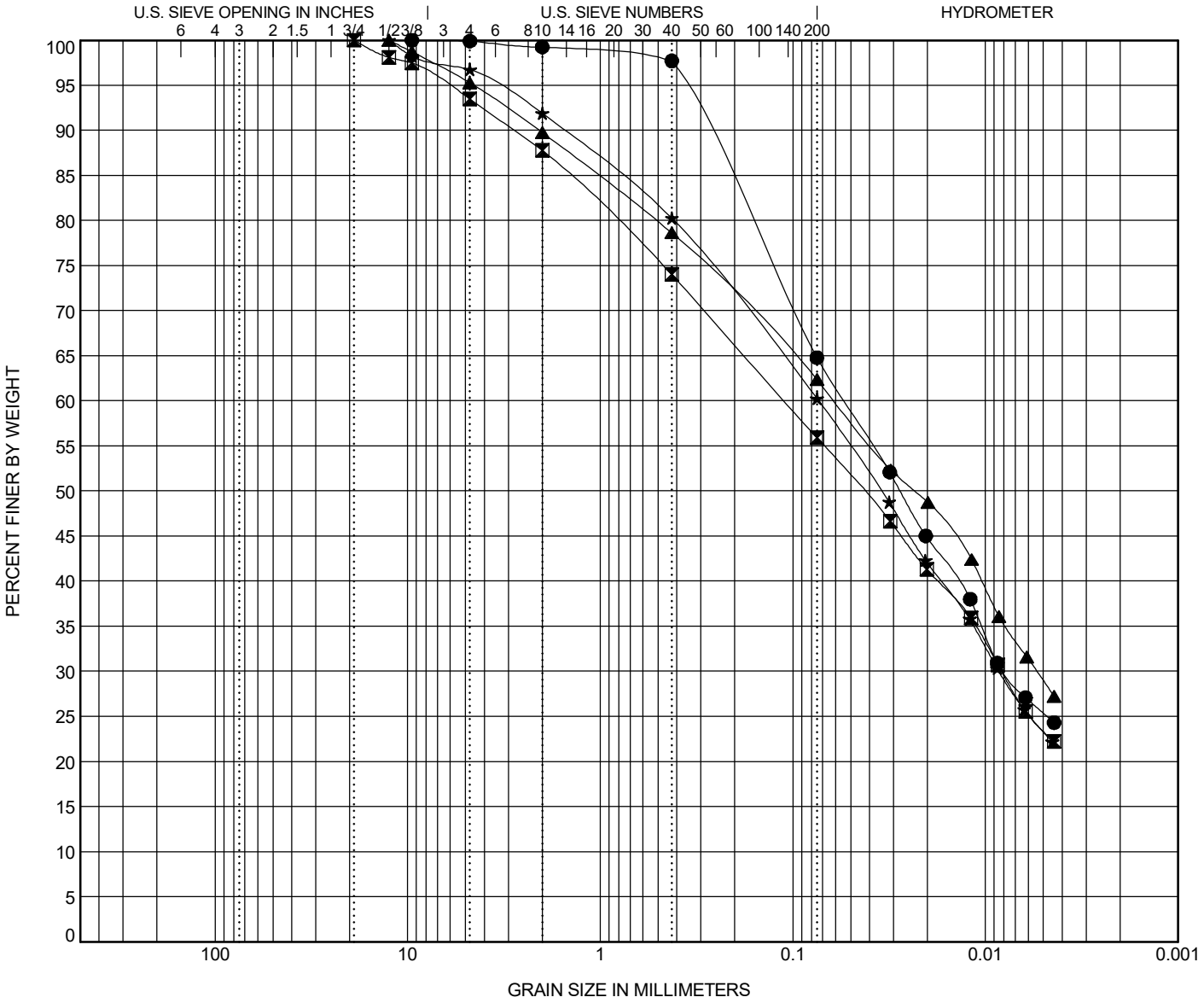
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GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT BRO-68-33.36 **PID** 120095

OGE NUMBER 0 **PROJECT TYPE**



COBBLES	GRAVEL	SAND		SILT	CLAY
		coarse	fine		

	Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	B-007-0-24	2.5	A-6b ~ SANDY LEAN CLAY(CL)	35	16	19
⊠	B-007-0-24	5.0	A-4a ~ SANDY LEAN CLAY(CL)	24	15	9
▲	B-007-0-24	7.5	A-4a ~ SANDY LEAN CLAY(CL)	26	16	10
★	B-007-0-24	10.0	A-4a ~ SANDY LEAN CLAY(CL)	22	14	8

	Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	B-007-0-24	2.5	0.283	0.028	0.008		0	2	33	40	25		
⊠	B-007-0-24	5.0	2.806	0.043	0.008		12	14	18	33	23		
▲	B-007-0-24	7.5	2.075	0.023	0.005		11	11	16	33	29		
★	B-007-0-24	10.0	1.553	0.035	0.009		8	12	20	37	23		

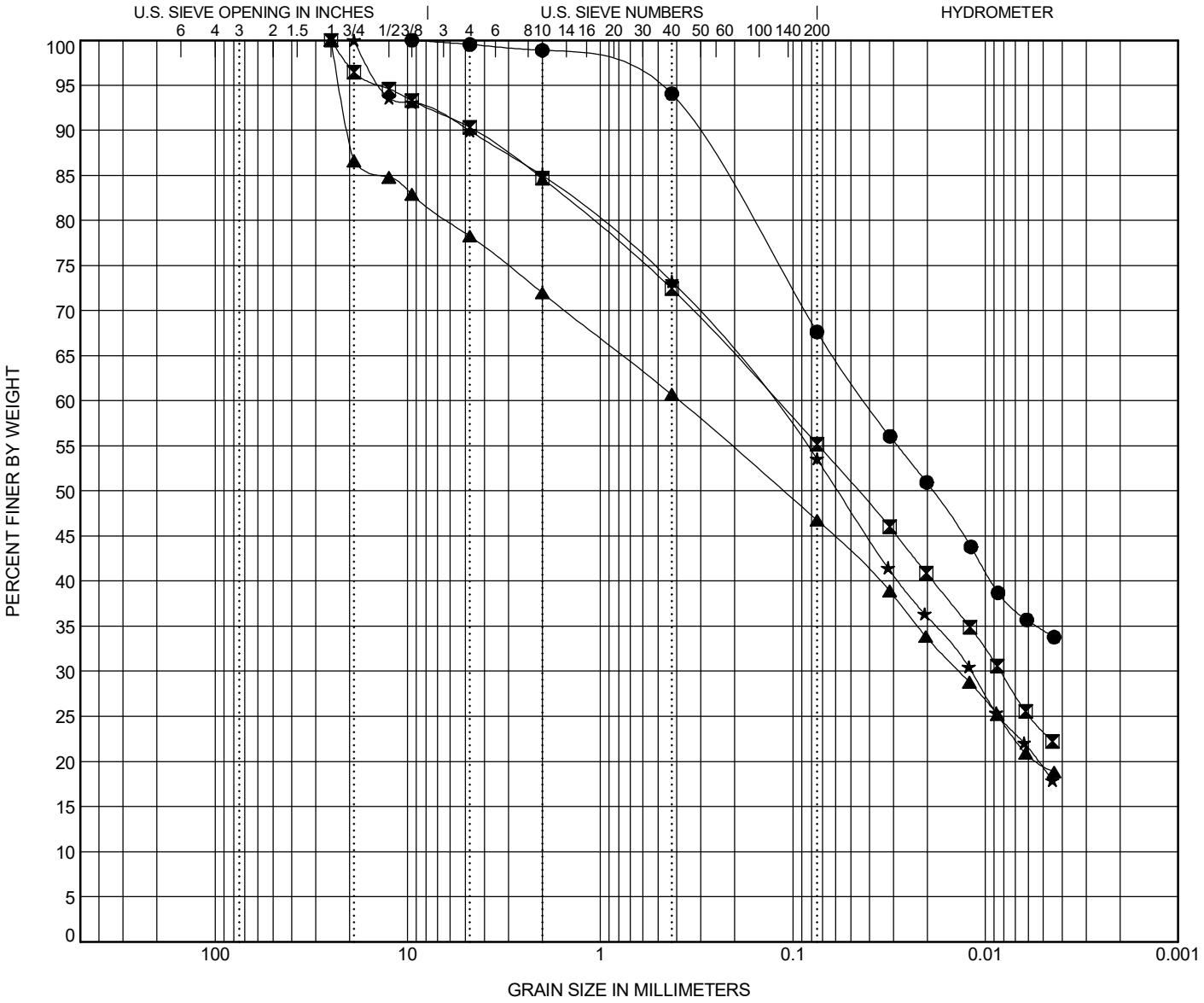
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GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT BRO-68-33.36 **PID** 120095

OGE NUMBER 0 **PROJECT TYPE**



COBBLES	GRAVEL	SAND		SILT	CLAY
		coarse	fine		

	Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	B-008-0-24	2.5	A-7-6 ~ SANDY LEAN CLAY(CL)	46	20	26
⊠	B-008-0-24	5.0	A-4a ~ SANDY LEAN CLAY(CL)	26	16	10
▲	B-008-0-24	7.5	A-4a ~ CLAYEY SAND with GRAVEL(SC)	24	14	10
★	B-008-0-24	10.0	A-4a ~ SANDY LEAN CLAY(CL)	21	13	8

	Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	B-008-0-24	2.5	0.326	0.019			1	5	26	33	35		
⊠	B-008-0-24	5.0	4.507	0.046	0.008		16	12	17	32	23		
▲	B-008-0-24	7.5	20.367	0.112	0.014		28	11	14	27	20		
★	B-008-0-24	10.0	4.802	0.058	0.012		14	12	20	35	19		

GRAIN SIZE - OH DOT.GDT - 3/19/25 15:39 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\BRO-68-33.36\GINT FILES\BRO-68-33.36.GPJ

APPENDIX D

CALCULATION

SUBGRADE ANALYSIS

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.)

BRO-68-33.36
PID #120095

Subgrade explorations for a new roundabout with drainage improvements to include new full height headwalls.

NEAS, Inc.

Prepared By: Zhao Mankoci
Date prepared: Friday, March 21, 2025

Chunmei (Melinda) He, Ph.D., P.E.
2800 Corporate Exchange Drive
Suite 240
Columbus, OH 43231
614.714.0299 Ext 111
che@neasinc.com

NO. OF BORINGS: 5

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.7 11/6/2024

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	B-001-0-24	US-68	1866+67	15	LT	CME 45B	79	945.9	944.1	1.9 C
2	B-003-0-24	E Greenbush Road	18+13	40	RT	CME 45B	79	942.4	944.5	2.1 F
3	B-004-0-24	E Greenbush Road	20+62	4	LT	CME 45B	79	948.3	946.5	1.8 C
4	B-006-0-24	W Greenbush Road	12+98	9	LT	CME 45B	79	947.8	946.1	1.7 C
5	B-009-0-24	US-68	1872+20	1	RT	CME 45B	79	944.4	946.3	1.9 F

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.7 11/6/2024

#	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	SS-1	1.5	3.0	-0.4	1.2	16		2.25	42	18	24	39	33	72	17	18	A-7-6	13	93					
	001-0	SS-2	3.0	4.5	1.2	2.7	16		2							18	18	A-7-6	16						
	24	SS-3	4.5	6.0	2.7	4.2	9		1.5	37	19	18	51	33	84	21	16	A-6b	11						
		SS-4	6.0	7.5	4.2	5.7	17	9	1.25							22	16	A-6b	16						
2	B	SS-1	1.0	2.5	3.1	4.6	8		1.75	58	24	34	38	45	83	26	21	A-7-6	20	147					
	003-0	SS-2	3.5	5.0	5.6	7.1	20		2							13	18	A-7-6							
	24	SS-3	6.0	7.5	8.1	9.6	55		3.75	24	15	9	32	23	55	12	10	A-4a							
		SS-4	8.5	10.0	10.6	12.1	68	8	4.5	23	13	10	28	20	48	8	10	A-4a							
3	B	SS-1	1.5	3.0	-0.3	1.2	21			NP	NP	NP	24	10	34	8	10	A-2-4	0	20					
	004-0	SS-2	3.0	4.5	1.2	2.7	12		2.25	44	21	23	47	38	85	22	18	A-7-6	14			N_{60} & Mc			
	24	SS-3	4.5	6.0	2.7	4.2	13		4							21	18	A-7-6	16						
		SS-4	6.0	7.5	4.2	5.7	17	12	4							19	18	A-7-6	16						
4	B	SS-1	1.5	3.0	-0.2	1.3	12		4.5							12	10	A-4a	8						
	006-0	SS-2	3.0	4.5	1.3	2.8	16		2.75	48	19	29	46	35	81	20	18	A-7-6	17	80					
	24	SS-3	4.5	6.0	2.8	4.3	18		2.5	52	20	32	40	39	79	20	18	A-7-6	18						
		SS-4	6.0	7.5	4.3	5.8	21	12	2.75							19	18	A-7-6	16						
5	B	SS-1	1.0	2.5	2.9	4.4	9		2.25	53	19	34	44	39	83	23	18	A-7-6	19	293					
	009-0	SS-2	3.5	5.0	5.4	6.9	14		2	42	19	23	50	36	86	21	18	A-7-6							
	24	SS-3	6.0	7.5	7.9	9.4	12		2.5							22	18	A-7-6							
		SS-4	8.5	10.0	10.4	11.9	11	9	2.25							22	18	A-7-6							

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.7 11/6/2024

PID: PID #12009

County-Route-Section: BRO-68-33.36

No. of Borings: 5

Geotechnical Consultant: NEAS, Inc.

Prepared By: Zhao Mankoci

Date prepared: 3/21/2025

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

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

Design CBR	4

% Samples within 3 feet of subgrade			
$N_{60} \leq 5$	0%	HP ≤ 0.5	0%
$N_{60} < 12$	13%	0.5 < HP ≤ 1	0%
$12 \leq N_{60} < 15$	19%	1 < HP ≤ 2	13%
$N_{60} \geq 20$	6%	HP > 2	44%
M+	6%		
Rock	0%		
Unsuitable Soil	0%		

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

% Proposed Subgrade Surface	
Unstable & Unsuitable	13%
Unstable	13%
Unsuitable (Soil & Rock)	0%

	N_{60}	N_{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M_C	M_{OPT}	GI
Average	19	10	2.67	42	19	24	40	32	72	18	16	14
Maximum	68	12	4.50	58	24	34	51	45	86	26	21	20
Minimum	8	8	1.25	23	13	9	24	10	34	8	10	0

Classification Counts by Sample

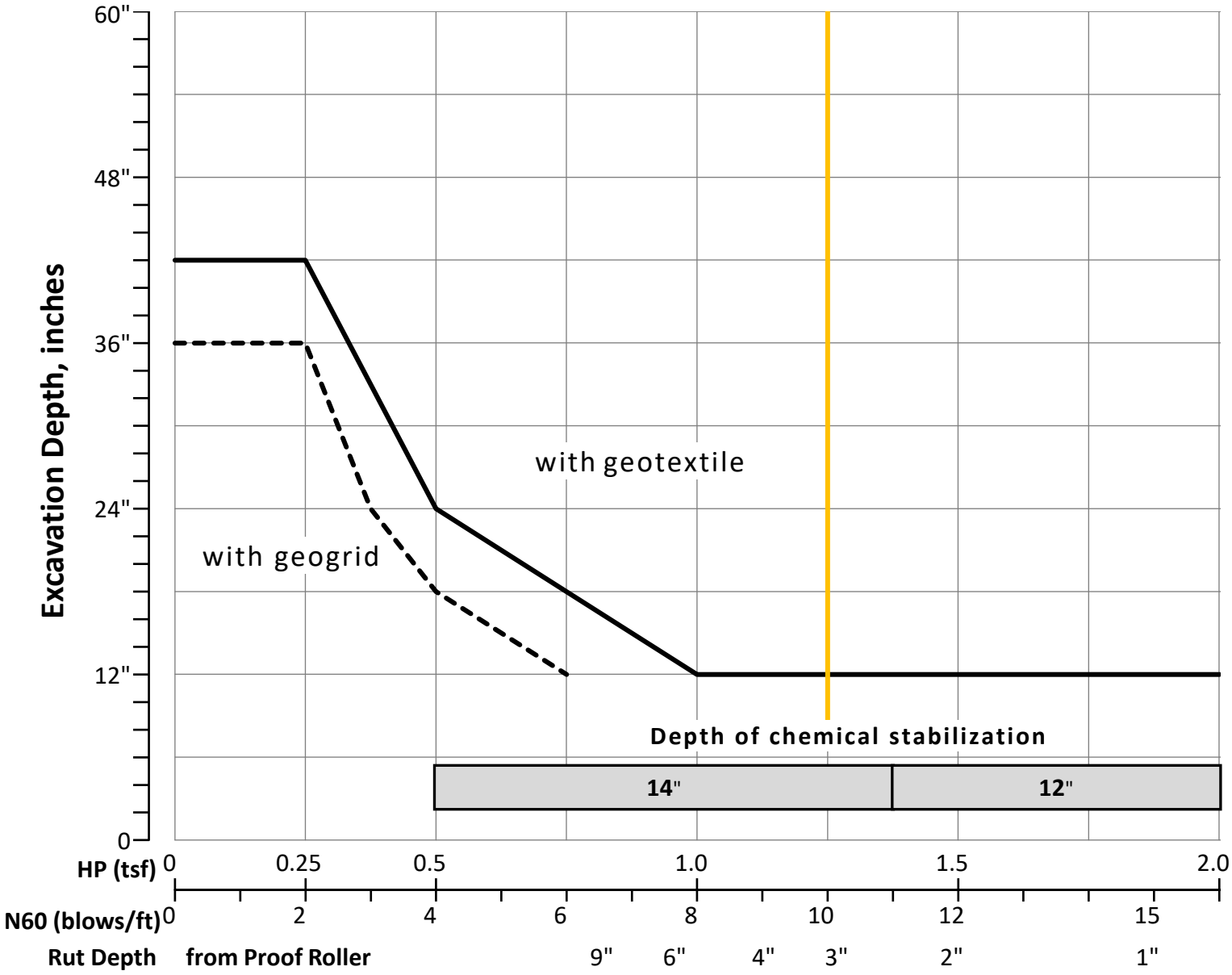
ODOT Class	UCF	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals
Count	0	0	0	0	1	0	0	0	0	0	3	0	0	0	2	0	14	0	0	20
Percent	0%	0%	0%	0%	5%	0%	0%	0%	0%	0%	15%	0%	0%	0%	10%	0%	70%	0%	0%	100%
% Rock|Granular|Cohesive	0%	0%	20%									80%								100%
Surface Class Count	0	0	0	0	1	0	0	0	0	0	1	0	0	0	1	0	5	0	0	8
Surface Class Percent	0%	0%	0%	0%	13%	0%	0%	0%	0%	0%	13%	0%	0%	0%	13%	0%	63%	0%	0%	100%

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.7 11/6/2024

Fig. 600-1 – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
2.67	0.50	☐ HP
10.00	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.)

BRO-68-33.36
PID #120095

Subgrade explorations for a new roundabout with drainage improvements to include new full height headwalls - Greenbush Road

NEAS, Inc.

Prepared By: Zhao Mankoci
Date prepared: Monday, March 24, 2025

Chunmei (Melinda) He, Ph.D., P.E.
2800 Corporate Exchange Drive
Suite 240
Columbus, OH 43231
614.714.0299 Ext 111
che@neasinc.com

NO. OF BORINGS: 3

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.7 11/6/2024

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	B-003-0-24	E Greenbush Road	18+13	40	RT	CME 45B	79	942.4	944.5	2.1 F
2	B-004-0-24	E Greenbush Road	20+62	4	LT	CME 45B	79	948.3	946.5	1.8 C
3	B-006-0-24	W Greenbush Road	12+98	9	LT	CME 45B	79	947.8	946.1	1.7 C

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.7 11/6/2024

#	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	SS-1	1.0	2.5	3.1	4.6	8		1.75	58	24	34	38	45	83	26	21	A-7-6	20	147					
	003-0	SS-2	3.5	5.0	5.6	7.1	20		2							13	18	A-7-6							
	24	SS-3	6.0	7.5	8.1	9.6	55		3.75	24	15	9	32	23	55	12	10	A-4a							
		SS-4	8.5	10.0	10.6	12.1	68	8	4.5	23	13	10	28	20	48	8	10	A-4a							
2	B	SS-1	1.5	3.0	-0.3	1.2	21			NP	NP	NP	24	10	34	8	10	A-2-4	0	20					
	004-0	SS-2	3.0	4.5	1.2	2.7	12		2.25	44	21	23	47	38	85	22	18	A-7-6	14			N_{60} & Mc			
	24	SS-3	4.5	6.0	2.7	4.2	13		4							21	18	A-7-6	16						
		SS-4	6.0	7.5	4.2	5.7	17	12	4							19	18	A-7-6	16						
3	B	SS-1	1.5	3.0	-0.2	1.3	12		4.5							12	10	A-4a	8						
	006-0	SS-2	3.0	4.5	1.3	2.8	16		2.75	48	19	29	46	35	81	20	18	A-7-6	17	80					
	24	SS-3	4.5	6.0	2.8	4.3	18		2.5	52	20	32	40	39	79	20	18	A-7-6	18						
		SS-4	6.0	7.5	4.3	5.8	21	12	2.75							19	18	A-7-6	16						

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.7 11/6/2024

PID: PID #12009

County-Route-Section: BRO-68-33.36
No. of Borings: 3

Geotechnical Consultant: NEAS, Inc.
Prepared By: Zhao Mankoci
Date prepared: 3/24/2025

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

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

Design CBR	5

% Samples within 3 feet of subgrade			
$N_{60} \leq 5$	0%	HP ≤ 0.5	0%
$N_{60} < 12$	0%	0.5 < HP ≤ 1	0%
$12 \leq N_{60} < 15$	30%	1 < HP ≤ 2	0%
$N_{60} \geq 20$	10%	HP > 2	50%
M+	10%		
Rock	0%		
Unsuitable Soil	0%		

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

% Proposed Subgrade Surface	
Unstable & Unsuitable	20%
Unstable	20%
Unsuitable (Soil & Rock)	0%

	N_{60}	N_{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M_C	M_{OPT}	GI
Average	23	11	3.16	42	19	23	36	30	66	17	16	14
Maximum	68	12	4.50	58	24	34	47	45	85	26	21	20
Minimum	8	8	1.75	23	13	9	24	10	34	8	10	0

Classification Counts by Sample

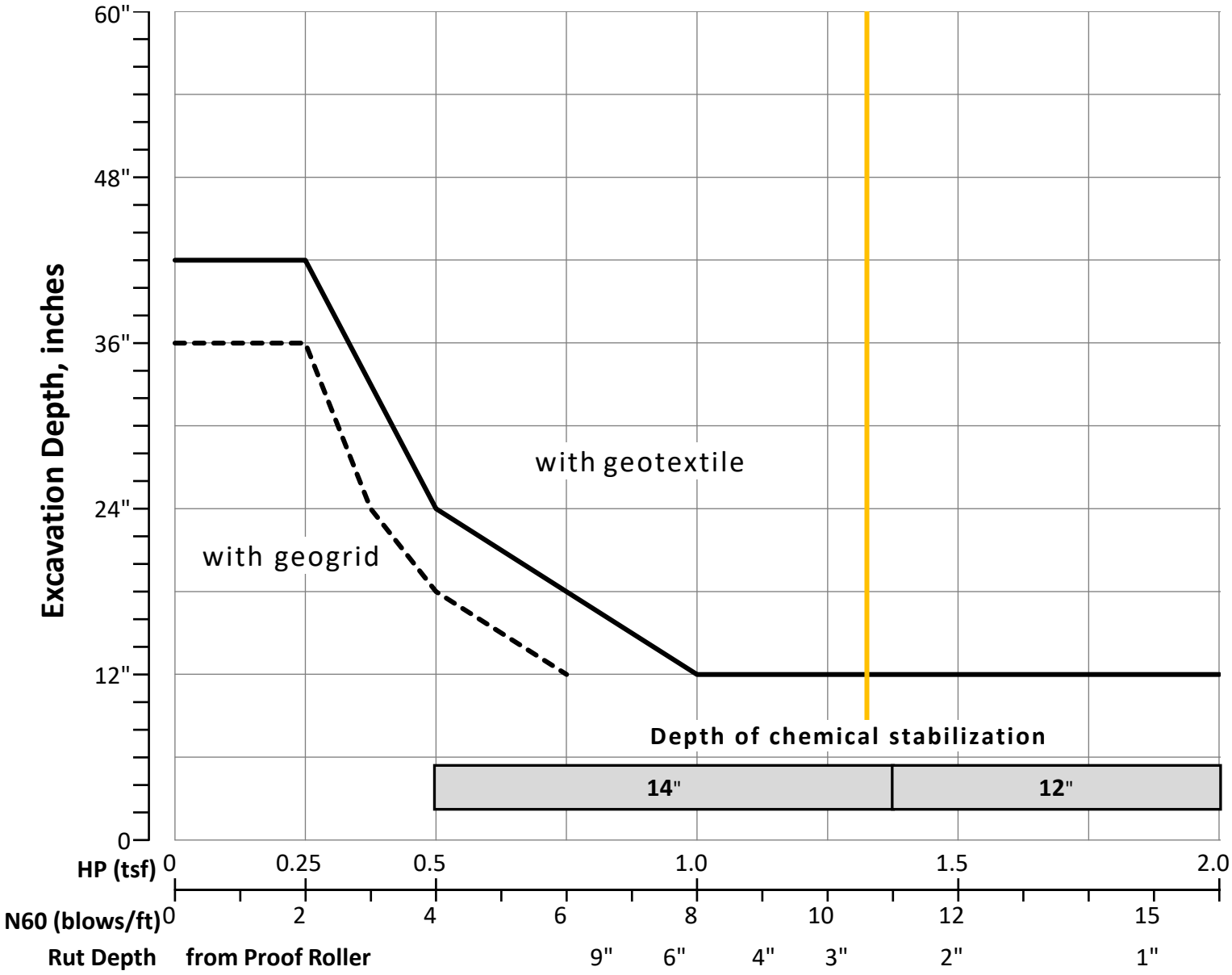
ODOT Class	UCF	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals
Count	0	0	0	0	1	0	0	0	0	0	3	0	0	0	0	0	8	0	0	12
Percent	0%	0%	0%	0%	8%	0%	0%	0%	0%	0%	25%	0%	0%	0%	0%	0%	67%	0%	0%	100%
% Rock|Granular|Cohesive	0%	0%	33%									67%								100%
Surface Class Count	0	0	0	0	1	0	0	0	0	0	1	0	0	0	0	0	3	0	0	5
Surface Class Percent	0%	0%	0%	0%	20%	0%	0%	0%	0%	0%	20%	0%	0%	0%	0%	0%	60%	0%	0%	100%

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.7 11/6/2024

Fig. 600-1 – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
3.16	0.50	☐ HP
10.67	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.)

BRO-68-33.36
PID #120095

Subgrade explorations for a new roundabout with drainage improvements to include new full height headwalls - US-68

NEAS, Inc.

Prepared By: Zhao Mankoci
Date prepared: Monday, March 24, 2025

Chunmei (Melinda) He, Ph.D., P.E.
2800 Corporate Exchange Drive
Suite 240
Columbus, OH 43231
614.714.0299 Ext 111
che@neasinc.com

NO. OF BORINGS: 3

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.7 11/6/2024

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	B-001-0-24	US-68	1866+67	15	LT	CME 45B	79	945.9	944.1	1.9 C
2	B-003-0-24	US-68	1868+20	40	RT	CME 45B	79	942.4	946.0	3.6 F
3	B-009-0-24	US-68	1872+20	1	RT	CME 45B	79	944.4	946.3	1.9 F

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.7 11/6/2024

#	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	SS-1	1.5	3.0	-0.4	1.2	16		2.25	42	18	24	39	33	72	17	18	A-7-6	13	93					
	001-0	SS-2	3.0	4.5	1.2	2.7	16		2							18	18	A-7-6	16						
	24	SS-3	4.5	6.0	2.7	4.2	9		1.5	37	19	18	51	33	84	21	16	A-6b	11						
		SS-4	6.0	7.5	4.2	5.7	17	9	1.25							22	16	A-6b	16						
2	B	SS-1	1.0	2.5	4.6	6.1	8		1.75	58	24	34	38	45	83	26	21	A-7-6	20	147					
	003-0	SS-2	3.5	5.0	7.1	8.6	20		2							13	18	A-7-6							
	24	SS-3	6.0	7.5	9.6	11.1	55		3.75	24	15	9	32	23	55	12	10	A-4a							
		SS-4	8.5	10.0	12.1	13.6	68	8	4.5	23	13	10	28	20	48	8	10	A-4a							
3	B	SS-1	1.0	2.5	2.9	4.4	9		2.25	53	19	34	44	39	83	23	18	A-7-6	19	293					
	009-0	SS-2	3.5	5.0	5.4	6.9	14		2	42	19	23	50	36	86	21	18	A-7-6							
	24	SS-3	6.0	7.5	7.9	9.4	12		2.5							22	18	A-7-6							
		SS-4	8.5	10.0	10.4	11.9	11	9	2.25							22	18	A-7-6							

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.7 11/6/2024

PID: PID #12009

County-Route-Section: BRO-68-33.36

No. of Borings: 3

Geotechnical Consultant: NEAS, Inc.

Prepared By: Zhao Mankoci

Date prepared: 3/24/2025

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

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

Design CBR	4

% Samples within 3 feet of subgrade			
$N_{60} \leq 5$	0%	HP ≤ 0.5	0%
$N_{60} < 12$	29%	0.5 < HP ≤ 1	0%
$12 \leq N_{60} < 15$	0%	1 < HP ≤ 2	29%
$N_{60} \geq 20$	0%	HP > 2	29%
M+	0%		
Rock	0%		
Unsuitable Soil	0%		

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

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

	N_{60}	N_{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M_C	M_{OPT}	GI
Average	21	9	2.33	40	18	22	40	33	73	19	17	16
Maximum	68	9	4.50	58	24	34	51	45	86	26	21	20
Minimum	8	8	1.25	23	13	9	28	20	48	8	10	11

Classification Counts by Sample

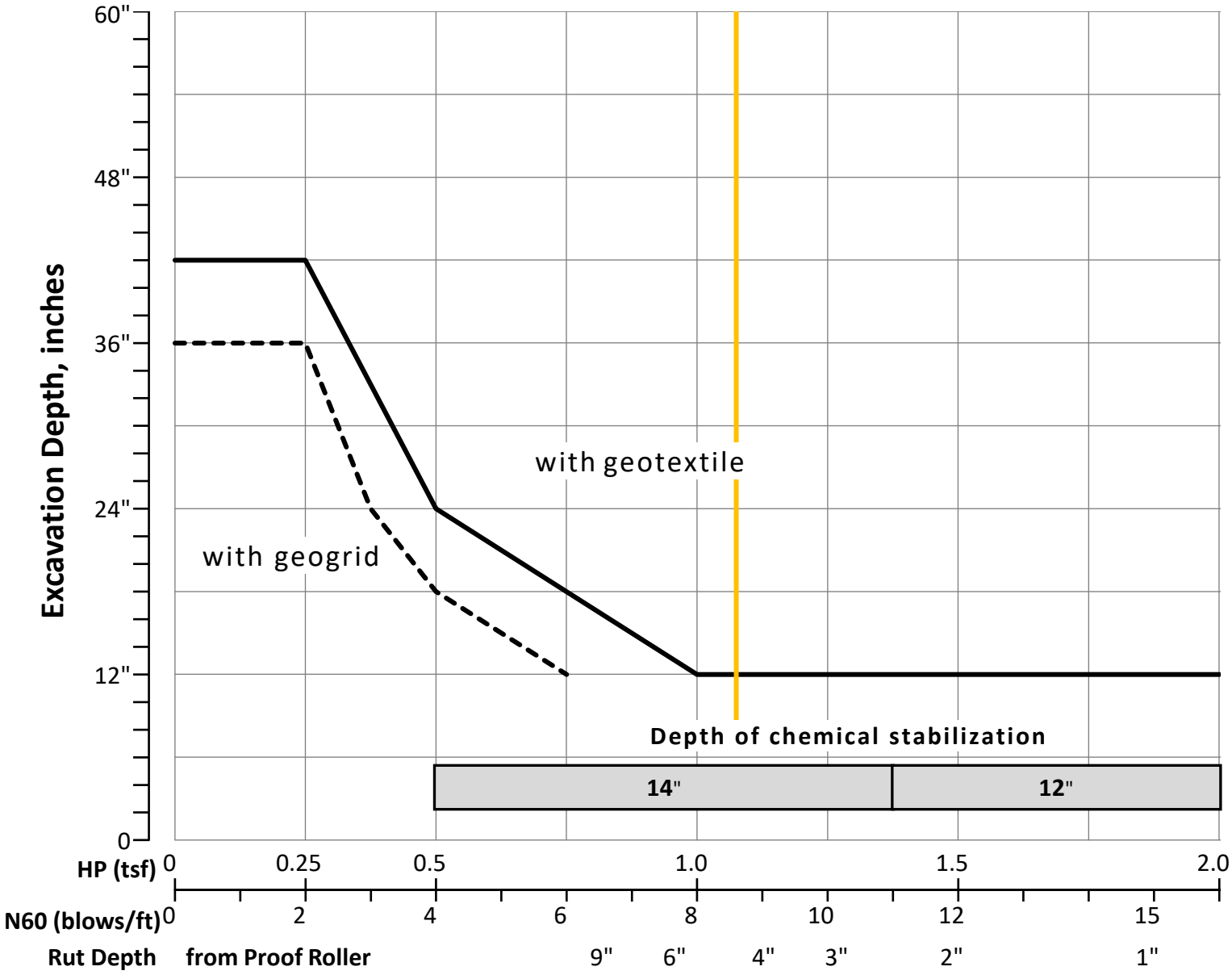
ODOT Class	UCF	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals
Count	0	0	0	0	0	0	0	0	0	0	2	0	0	0	2	0	8	0	0	12
Percent	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	17%	0%	0%	0%	17%	0%	67%	0%	0%	100%
% Rock|Granular|Cohesive	0%	0%	17%									83%								100%
Surface Class Count	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	2	0	0	3
Surface Class Percent	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	33%	0%	67%	0%	0%	100%

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.7 11/6/2024

Fig. 600-1 – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
2.33	0.50	☐ HP
8.67	6.00	☐ N60L

Average HP

Average N_{60L}

EXTERNAL STABILITY

Objective: **To evaluate the external stability of CIP wall design with broken backsloping backfill.**
Method: **In accordance with ODOT Bridge Design Manual [Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].**

Givens:

Backfill Soil Design Parameters:

$\phi'_f := 30\ deg$ — Effective angle of internal friction

$\gamma_f := 120\ \frac{lbf}{ft^3}$ — Unit weight

$c'_f := 0\ \frac{lbf}{ft^2}$ — Effective Cohesion

$\delta := 0.67 \cdot \phi'_f$ $\quad \delta = 20.1\ deg$ — 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):

$\phi'_{fd} := 31\ deg$ — Effective angle of internal friction

$\gamma_{fd} := 140\ \frac{lbf}{ft^3}$ — Unit weight

$c'_{fd} := 450\ \frac{lbf}{ft^2}$ — Effective Cohesion

$\delta_{fd} := 0.67 \cdot \phi'_{fd}$ $\quad \delta_{fd} = 20.8\ deg$ — Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Undrained Conditions (Total Stress):

$\phi_{fdu} := 0\ deg$ — Angle of internal friction (Same as Drained Conditions if granular soils)

$\gamma_{fd} = 140\ \frac{lbf}{ft^3}$ — Unit weight

$Su_{fdu} := 8650\ \frac{lbf}{ft^2}$ — Undrained Shear Strength

$\delta_{fdu} := 0.67 \cdot \phi_{fdu}$ $\quad \delta_{fdu} = 0\ deg$ — Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

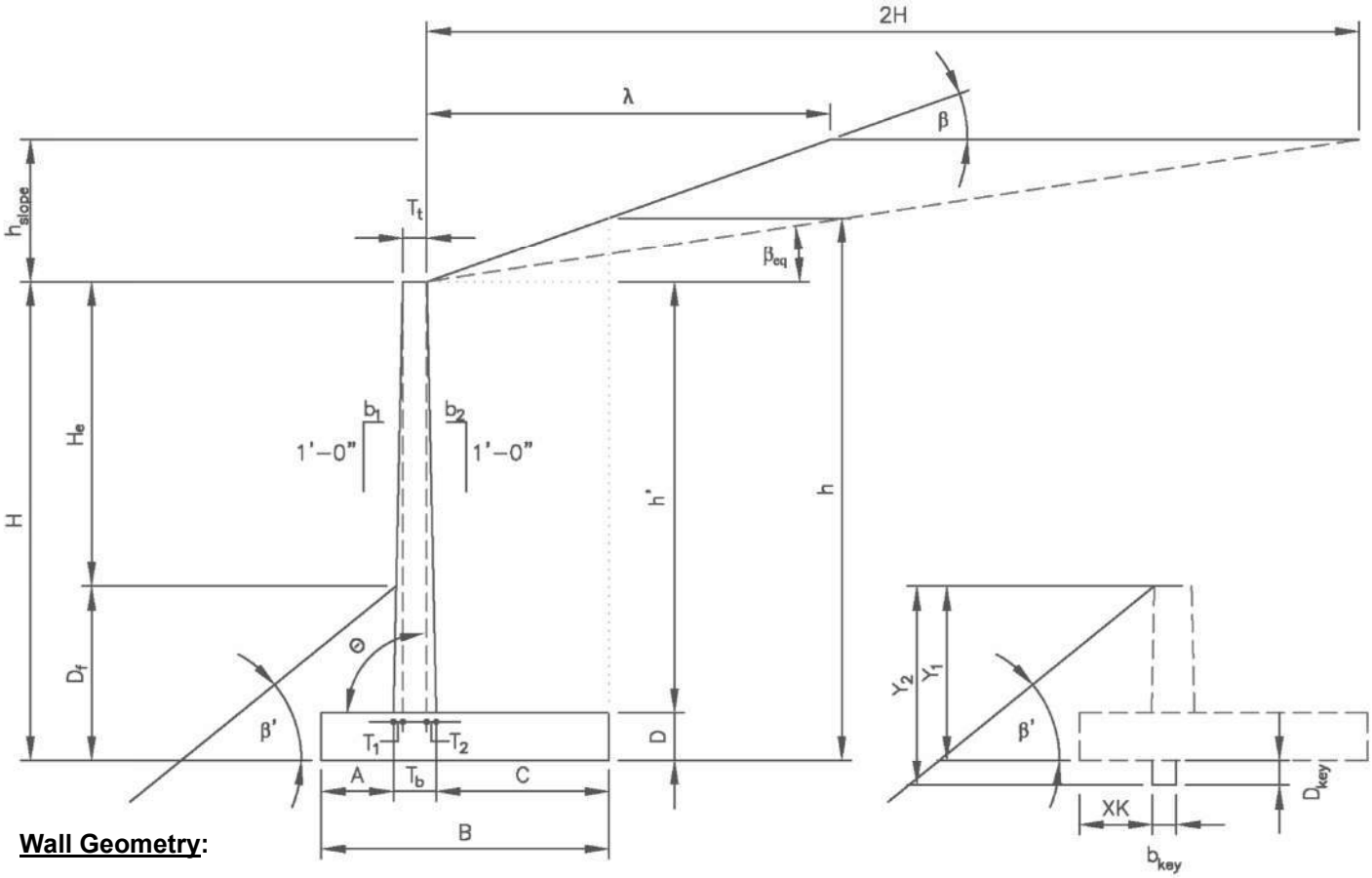
Foundation Surcharge Soil Parameters:

$\gamma_q := 120\ \frac{lbf}{ft^3}$ — Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Other Parameters:

$\gamma_c := 150\ \frac{lbf}{ft^3}$ — Concrete Unit weight

Created with PTC Mathcad Express. See www.mathcad.com for more information.



Wall Geometry:

$H_e := 7.5\ ft$ — Exposed wall height

$D_f := 2.5\ ft$ — 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**

$H := H_e + D_f$ $H = 10\ ft$ — Design Wall Height

$T_t := 12\ in$ — Stem thickness at top of wall

$b_1 := 0 \cdot \left(\frac{in}{ft}\right)$ — Frontwall batter, (b1H:12V)

$b_2 := 0 \cdot \left(\frac{in}{ft}\right)$ — Backwall batter, (b2H:12V)

Preliminary Wall Dimensioning:

$B := 5.5\ ft$ $\frac{2}{5} \cdot H = 4\ ft$ to $\frac{3}{5} \cdot H = 6\ ft$ — Footing base width (2/5H to 3/5H)

$A := 1.92\ ft$ $\frac{H}{8} = 1.25\ ft$ to $\frac{H}{5} = 2\ ft$ — Toe projection (H/8 to H/5)

$D := 1.5\ ft$ $\frac{H}{8} = 1.25\ ft$ to $\frac{H}{5} = 2\ ft$ — Footing thickness (H/8 to H/5)

www.mathcad.com for more information.

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Shear Key Dimensioning:

$D_{key} := 2.5\ ft$ — Depth of shear key from bottom of footing
Note: Footings on rock typically require shear key

$b_{key} := 1\ ft$ — Width of shear key

$XK := 0\ ft$ — Distance from toe to shear key

Other Wall Dimensions:

$h' := H - D$ $\quad h' = 8.5\ ft$ — Stem height

$T_1 := b_1 \cdot h'$ $\quad T_1 = 0\ ft$ — Stem front batter width

$T_2 := b_2 \cdot h'$ $\quad T_2 = 0\ ft$ — Stem back batter width

$T_b := T_1 + T_2 + T_t$ $\quad T_b = 1\ ft$ — Stem thickness at bottom of wall

$C := B - A - T_b$ $\quad C = 2.58\ ft$ — Heel projection

$\theta := 90\ deg$ — Angle of back face of wall to horizontal = *atan(12/b2)*

$b := 12\ in$ $\quad b = 1\ ft$ — Concrete strip width (for design)

$y_1 := D_f$ $\quad y_1 = 2.5\ ft$ — Depth to where passive pressure may begin to be utilized in front of wall.

$y_2 := D_f + D_{key}$ $\quad y_2 = 5\ ft$ — Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

Site Grading and Slope Dimensions:

$\beta := 0\ deg$ — Inclination of ground slope behind face of wall. Horizontal backfill behind CIP wall, β = 0 deg

Inclination of ground slope:

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

$\beta' := 0\ deg$ — Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, β' = 0 deg. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

$\lambda := 0\ ft$ — Horizontal distance from the back of the wall to point of slope crest .

$L_{Traffic} := 8\ ft$ — Horizontal distance from assumed traffic surcharge load to Backface of Wall.

$2 \cdot H = 20\ ft$ **IF λ IS GREATER THAN 2*H - USE INFINITE SLOPE CALCULATION SHEET**

$h_{slope} := \lambda \cdot \tan(\beta)$ $\quad h_{slope} = 0\ ft$ — Height of broken slope behind wall

$\beta_{eq} := \operatorname{atan}\left(\frac{h_{slope}}{2 \cdot H}\right) = 0\ deg$ — Equivalent backslope angle

$h := \text{if}\left(\lambda \le T_2 + C,\ H + h_{slope},\ H + (T_2 + C) \cdot \tan(\beta)\right) = 10\ ft$ — Height of retained fill at back of heel

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Live Load Surcharge Parameters:

$$SUR := \text{if}\left(L_{Traffic} < \frac{H}{2}, 250\ \frac{lbf}{ft^2}, 0\ \frac{lbf}{ft^2}\right) = 0\ \frac{lbf}{ft^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])
Note: ,A live load ssurcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2 .

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{\left(\sin\left(\phi'_f + \delta\right) \cdot \sin\left(\phi'_f - \beta_{eq}\right)\right)}{\left(\sin\left(\theta - \delta\right) \cdot \sin\left(\theta + \beta_{eq}\right)\right)}}\right)^2 \quad \Gamma = 2.687$$

$$k_{af} := \left(\frac{\left(\sin\left(\theta + \phi'_f\right)\right)^2}{\left(\Gamma \cdot \left(\sin\left(\theta\right)\right)^2 \cdot \sin\left(\theta - \delta\right)\right)}\right) \quad k_{af} = 0.297$$

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 θ = 90 degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0 \qquad \frac{-\delta_{fd}}{\phi'_{fd}} = -0.67$$

$k'_p := 7$

Passive Earth Pressure Coefficient from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$R_d := 0.752$

Reduction Factor

$k_{pd} := R_d \cdot k'_p \qquad k_{pd} = 5.264$

Passive Earth Pressure Coefficient for Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

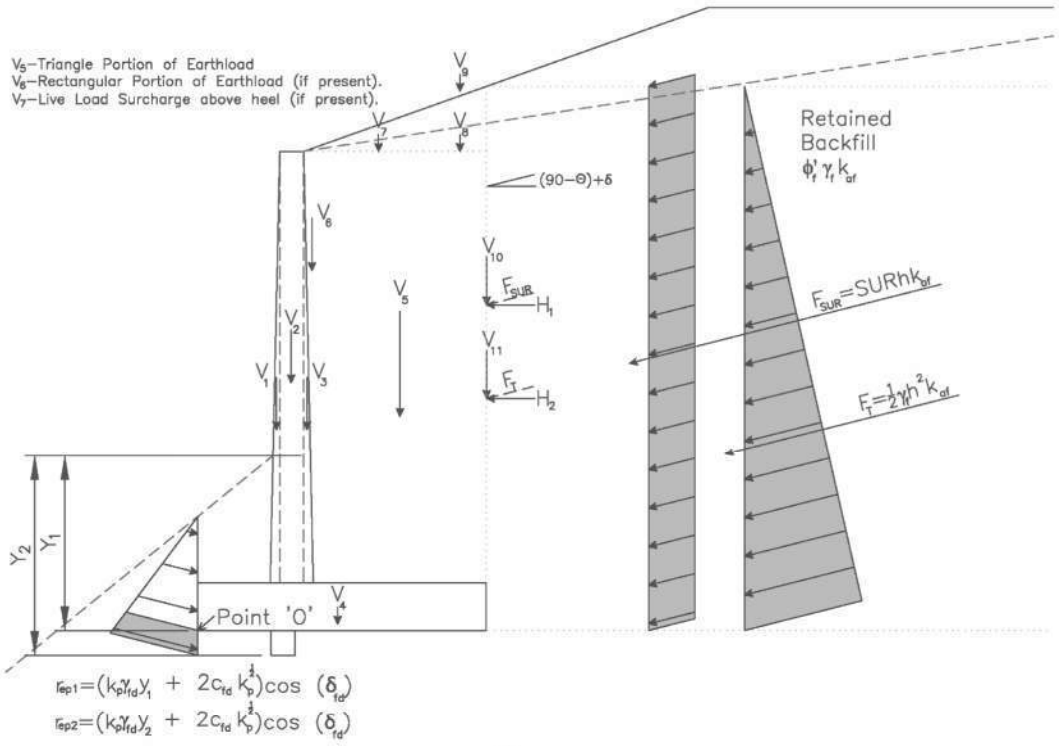
Undrained Conditions:

$k_{pu} := \text{if}\left(\phi_{fdu} > 0, k_{pu}, 1\right) \qquad k_{pu} = 1$

Passive Earth Pressure Coefficient for Resistance Undrained Conditions

Created

Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:



$F_T := \frac{1}{2} \cdot \gamma_f \cdot h^2 \cdot k_{af}$ $F_T = 1783.56 \frac{lbf}{ft}$ Active Earth Force Resultant (EH)

$F_{SUR} := SUR \cdot h \cdot k_{af}$ $F_{SUR} = 0 \frac{lbf}{ft}$ Live Load Surcharge (LS)

Vertical Loads:

$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$ $V_1 = 0 \frac{lbf}{ft}$ Wall stem front batter (DC)

$V_2 := T_t \cdot h' \cdot \gamma_c$ $V_2 = 1275 \frac{lbf}{ft}$ Wall stem (DC)

$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$ $V_3 = 0 \frac{lbf}{ft}$ Wall stem back batter (DC)

$V_4 := D \cdot B \cdot \gamma_c$ $V_4 = 1237.5 \frac{lbf}{ft}$ Wall Footing (DC)

$V_5 := C \cdot h' \cdot \gamma_f$ $V_5 = 2631.6 \frac{lbf}{ft}$ Soil Backfill - Heel (EV)

$V_6 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_f$ $V_6 = 0 \frac{lbf}{ft}$ Soil Backfill - Batter (EV)

$V_7 := \text{if}\left(\lambda \le T_2 + C, \frac{1}{2} \cdot \lambda^2 \cdot \tan(\beta) \cdot \gamma_f, \frac{1}{2} \cdot (T_2 + C)^2 \cdot \tan(\beta) \cdot \gamma_f\right)$

$V_7 = 0 \frac{lbf}{ft}$ Soil Backfill - Backslope Triangle Portion (EV)

$V_8 := \text{if}\left(\lambda \le T_2 + C, (T_2 + C - \lambda) \cdot h_{slope} \cdot \gamma_f, 0 \cdot \frac{lbf}{ft}\right)$

$V_8 = 0 \frac{lbf}{ft}$ Soil Backfill - Backslope Rectangular Portion (EV)

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$V_9 := \text{if}\left(\lambda \le T_2 + C,\ (T_2 + C - \lambda) \cdot SUR,\ 0 \cdot \frac{lbf}{ft}\right)$

$V_9 = 0\ \frac{lbf}{ft}$ Live Load Surcharge Above Heel - (LS)

$V_{10} := F_{SUR} \cdot \sin(90 \cdot deg - \theta + \delta)$ $V_{10} = 0\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (vertical comp. - LS)

$V_{11} := F_T \cdot \sin(90 \cdot deg - \theta + \delta)$ $V_{11} = 612.938\ \frac{lbf}{ft}$ Active earth force resultant (vertical component - EH)

Moment Arm: Moment:

Moments produced from vertical loads about Point 'O'

$d_{v1} := A + \frac{2}{3} \cdot T_1 = 1.92\ ft$ $MV_1 := V_1 \cdot d_{v1} = 0\ lbf$

$d_{v2} := A + T_1 + \frac{T_t}{2} = 2.42\ ft$ $MV_2 := V_2 \cdot d_{v2} = 3085.5\ lbf$

$d_{v3} := A + T_1 + T_t + \frac{T_2}{3} = 2.92\ ft$ $MV_3 := V_3 \cdot d_{v3} = 0\ lbf$

$d_{v4} := \frac{B}{2} = 2.75\ ft$ $MV_4 := V_4 \cdot d_{v4} = 3403.125\ lbf$

$d_{v5} := B - \frac{C}{2} = 4.21\ ft$ $MV_5 := V_5 \cdot d_{v5} = 11079.036\ lbf$

$d_{v6} := A + T_1 + T_t + \frac{2\ T_2}{3} = 2.92\ ft$ $MV_6 := V_6 \cdot d_{v6} = 0\ lbf$

$d_{v7} := \text{if}\left(\lambda \le T_2 + C,\ A + T_1 + T_t + \left(\frac{2}{3}\ (\lambda)\right),\ A + T_1 + T_t + \left(\frac{2}{3}\ (T_2 + C)\right)\right) = 2.92\ ft$ $MV_7 := V_7 \cdot d_{v7} = 0\ lbf$

$d_{v8} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2}\right) = 4.21\ ft$ $MV_8 := V_8 \cdot d_{v8} = 0\ lbf$

$d_{v9} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2}\right) = 4.21\ ft$ $MV_9 := V_9 \cdot d_{v9} = 0\ lbf$

$d_{v10} := B = 5.5\ ft$ $MV_{10} := V_{10} \cdot d_{v10} = 0\ lbf$

$d_{v11} := B = 5.5\ ft$ $MV_{11} := V_{11} \cdot d_{v11} = 3371.157\ lbf$

Horizontal Loads:

$H_1 := F_{SUR} \cdot \cos(90 \cdot deg - \theta + \delta)$ $H_1 = 0\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (horizontal comp. - LS)

$H_2 := F_T \cdot \cos(90 \cdot deg - \theta + \delta)$ $H_2 = 1674.931\ \frac{lbf}{ft}$ Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm: Moment:

$d_{h1} := \frac{h}{2}$ $d_{h1} = 5\ ft$ $MH_1 := H_1 \cdot d_{h1}$ $MH_1 = 0\ \frac{lbf \cdot ft}{ft}$

$d_{h2} := \frac{h}{3}$ $d_{h2} = 3.333\ ft$ $MH_2 := H_2 \cdot d_{h2}$ $MH_2 = 5583.103\ \frac{lbf \cdot ft}{ft}$

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Unfactored Loads by Load Type:

$V_{DC} := V_1 + V_2 + V_3 + V_4$ $V_{DC} = 2512.5 \frac{lbf}{ft}$ $V_{EV} := V_5 + V_6 + V_7 + V_8$ $V_{EV} = 2631.6 \frac{lbf}{ft}$

$V_{LS_Ia} := V_{10}$ $V_{LS_Ia} = 0 \frac{lbf}{ft}$ $V_{LS_Ib} := V_9 + V_{10}$ $V_{LS_Ia} = 0 \frac{lbf}{ft}$

$V_{EH} := V_{11}$ $V_{EH} = 612.938 \frac{lbf}{ft}$

$H_{EH} := H_2$ $H_{EH} = 1674.931 \frac{lbf}{ft}$ $H_{LS} := H_1$ $H_{LS} = 0 \frac{lbf}{ft}$

Unfactored Moments by Load Type

$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4$ $M_{DC} = 6488.625 \frac{lbf \cdot ft}{ft}$

$M_{EV} := MV_5 + MV_6 + MV_7 + MV_8$ $M_{EV} = 11079.036 \frac{lbf \cdot ft}{ft}$

$M_{LSV_Ia} := MV_{10}$ $M_{LSV_Ia} = 0 \frac{lbf \cdot ft}{ft}$

$M_{LSV_Ib} := MV_9 + MV_{10}$ $M_{LSV_Ib} = 0 \frac{lbf \cdot ft}{ft}$

$M_{EH1} := MV_{11}$ $M_{EH1} = 3371.157 \frac{lbf \cdot ft}{ft}$

$M_{LSH} := MH_1$ $M_{LSH} = 0 \frac{lbf \cdot ft}{ft}$

$M_{EH2} := MH_2$ $M_{EH2} = 5583.103 \frac{lbf \cdot ft}{ft}$

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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:

$V_{Ia} := \eta \cdot \left(\left(Ia_{DC} \cdot V_{DC} \right) + \left(Ia_{EV} \cdot V_{EV} \right) + \left(Ia_{EH} \cdot V_{EH} \right) + \left(Ia_{LS} \cdot V_{LS_Ia} \right) \right)$ $V_{Ia} = 5812.257 \ \frac{lbf}{ft}$

$V_{Ib} := \eta \cdot \left(\left(Ib_{DC} \cdot V_{DC} \right) + \left(Ib_{EV} \cdot V_{EV} \right) + \left(Ib_{EH} \cdot V_{EH} \right) + \left(Ib_{LS} \cdot V_{LS_Ib} \right) \right)$ $V_{Ib} = 7612.692 \ \frac{lbf}{ft}$

Factored Horizontal Loads by Limit State:

$H_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot H_{LS} \right) + \left(Ia_{EH} \cdot H_{EH} \right) \right)$ $H_{Ia} = 2512.396 \ \frac{lbf}{ft}$

$H_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot H_{LS} \right) + \left(Ib_{EH} \cdot H_{EH} \right) \right)$ $H_{Ib} = 2512.396 \ \frac{lbf}{ft}$

Factored Moments Produced by Vertical Loads by Limit State:

$MV_{Ia} := \eta \cdot \left(\left(Ia_{DC} \cdot M_{DC} \right) + \left(Ia_{EV} \cdot M_{EV} \right) + \left(Ia_{EH} \cdot M_{EH1} \right) + \left(Ia_{LS} \cdot M_{LSV_Ia} \right) \right)$ $MV_{Ia} = 21975.534 \ \frac{lbf \cdot ft}{ft}$

$MV_{Ib} := \eta \cdot \left(\left(Ib_{DC} \cdot M_{DC} \right) + \left(Ib_{EV} \cdot M_{EV} \right) + \left(Ib_{EH} \cdot M_{EH1} \right) + \left(Ib_{LS} \cdot M_{LSV_Ib} \right) \right)$ $MV_{Ib} = 28124.216 \ \frac{lbf \cdot ft}{ft}$

Factored Moments Produced by Horizontal Loads by Limit State:

$MH_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot M_{LSH} \right) + \left(Ia_{EH} \cdot M_{EH2} \right) \right)$ $MH_{Ia} = 8374.655 \ \frac{lbf \cdot ft}{ft}$

$MH_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot M_{LSH} \right) + \left(Ib_{EH} \cdot M_{EH2} \right) \right)$ $MH_{Ib} = 8374.655 \ \frac{lbf \cdot ft}{ft}$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength Ib:

$\Sigma M_R := MV_{Ib}$ $\Sigma M_R = 28124.216 \frac{lbf \cdot ft}{ft}$ Sum of Resisting Moments (Strength Ib)

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 8374.655 \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 7612.692 \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$ $x = 2.594\ ft$ Distance from Point "O" the resultant intersects the base

$e := \left| \frac{B}{2} - x \right|$ $e = 0.16\ ft$ Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

Foundation Layout:

$B' := B - 2 \cdot e$ $B' = 5.189\ ft$ Effective Footing Width

$L' := 35\ ft$ Effective Footing Length (Assumed)

$H' := H_{Ib}$ $H' = 2512.396 \frac{lbf}{ft}$ Summation of Horizontal Loads (Strength Ib)

$V' := V_{Ib}$ $V' = 7612.692 \frac{lbf}{ft}$ Summation of Vertical Loads (Strength Ib)

$D_f = 2.5\ ft$ Footing embedment

$d_w := 0\ ft$ Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan\left(45\ deg + \frac{\phi'_{fd}}{2}\right)^2, 1.0\right)$ $N_q = 20.63$

$N_c := \text{if}\left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14\right)$ $N_c = 32.67$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$ $N_\gamma = 25.994$

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

$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L'}\right)\right)$ $s_c = 1.094$

$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd})\right), 1\right)$ $s_q = 1.089$

$s_\gamma := \text{if}\left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'}\right), 1\right)$ $s_\gamma = 0.941$

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Load inclination factors:

$i_q := 1$

$i_\gamma := 1$

$i_c := 1$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**. "Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$C_{wq} := \text{if}\left(d_w \geq D_f, 1.0, 0.5\right)$ $C_{wq} = 0.5$

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

Depth Correction Factor per Hanson (1970):

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

$d_q = 1.13$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $N_{cm} = 35.73$

$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q$ $N_{qm} = 25.32$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $N_{\gamma m} = 24.453$

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

$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $q_{nd} = 24316.9 \ \frac{lbf}{ft^2}$

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

$\phi_b := .55$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Rd} := \phi_b \cdot q_{nd}$ $q_{Rd} = 13.374 \ ksf$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

$N_\gamma := 2 \cdot \left(N_q + 1\right) \cdot \tan\left(\phi_{fdu}\right)$ $N_\gamma = 0$

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

$s_c := \text{if}\left(\phi_{fdu} > 0\,,\, 1 + \left(\frac{B'}{L'}\right)\cdot\left(\frac{N_q}{N_c}\right),\, 1 + \left(\frac{B'}{5\cdot L'}\right)\right)$ $\quad s_c = 1.03$

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

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

Load inclination factors:

$i_q := 1$

$i_\gamma := 1$

$i_c := 1$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**. "Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

Depth Correction Factor per Hanson (1970):

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

$d_q = 1.13$

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

$N_{cm} := N_c\cdot s_c\cdot i_c$ $\quad N_{cm} = 5.292$

$N_{qm} := N_q\cdot s_q\cdot d_q\cdot i_q$ $\quad N_{qm} = 1.127$

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

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

$q_{nu} := Su_{fdu}\cdot N_{cm} + \gamma_q\cdot D_f\cdot N_{qm}\cdot C_{wq} + 0.5\cdot\gamma_{fd}\cdot B'\cdot N_{\gamma m}\cdot C_{w\gamma}$ $\quad q_{nu} = 45948.263\ \frac{lbf}{ft^2}$

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

$\phi_b := .55$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Ru} := \phi_b\cdot q_{nu}$ $\quad q_{Ru} = 25.272\ ksf$ Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 13.374\ ksf$

Undrained Conditions: $q_{Ru} = 25.272\ ksf$

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Evaluate External Stability of Wall:

Compute the factored bearing stress :

$e = 0.16\ ft$

$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e}$ $\sigma_V = 1.467\ ksf$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{Bearing_D} := \frac{q_{Rd}}{\sigma_V}$ Is the CDR > or = to 1.0? $CDR_{Bearing_D} = 9.12$

Undrained Conditions: $CDR_{Bearing_U} := \frac{q_{Ru}}{\sigma_V}$ Is the CDR > or = to 1.0? $CDR_{Bearing_U} = 17.22$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$e_{max} := \frac{B}{3}$ $e_{max} = 1.833\ ft$ Maximum Eccentricity **LRFD [C11.6.3.3.] Equals B/3 for soil.**

$\Sigma M_R := MV_{Ia}$ $\Sigma M_R = 21975.534\ \frac{lbf \cdot ft}{ft}$ Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$ $\Sigma M_O = 8374.655\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$ $\Sigma V = 5812.257\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$ $x = 2.34\ ft$ Distance from Point "O" the resultant intersects the base

$e := \left| \frac{B}{2} - x \right|$ $e = 0.41\ ft$ Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2**]. The effective bearing width is equal to B-2e. .

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e}$ Is the CDR > or = to 1.0? $CDR_{Eccentricity} = 4.47$

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Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$ $\quad R_u = 2512.396 \ \frac{lbf}{ft}$

Drained Conditions (Effective Stress):

Compute passive resistance throughout the design life of the wall **LRFD [Eq 3.11.5.4-1]:**:

$r_{ep1} := \left(k_{pd} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y1

$r_{ep2} := \left(k_{pd} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y2

$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot \left(y_2 - y_1\right)$ $\quad R_{ep} = 11286.773 \ \frac{lbf}{ft}$ 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:

$c := 0.8$

c = 1.0 for Cast-in-Place
c = 0.8 for Precast

$\Sigma V := V_{Ia}$ $\quad \Sigma V = 5812.257 \ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$R_\tau := c \cdot \Sigma V \cdot \tan\left(\phi'_{fd}\right)$ $\quad R_\tau = 2793.885 \ \frac{lbf}{ft}$ Nominal sliding resistance Under Drained Condition

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]**:

$\phi_{ep} := 0.5$ Resistance factor for passive resistance specified in **LRFD Table 10.5.5.2.2-1**

$\phi_\tau := 1.0$ Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

$\phi R_n := \phi_\tau \cdot R_\tau + \phi_{ep} \cdot R_{ep}$ $\quad R_R := \phi R_n$

Factored Sliding Resistance to be used in CDR Calculations: $R_R = 8437.271 \ \frac{lbf}{ft}$

Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{R_u}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 3.36$

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Undrained Conditions (Total Stress):

Compute passive resistance throughout the design life of the wall **LRFD [Eq 3.11.5.4-1]**::

$r_{ep1} := \left(k_{pu} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot Su_{fdu} \cdot \sqrt{k_{pu}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y1

$r_{ep2} := \left(k_{pu} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot Su_{fdu} \cdot \sqrt{k_{pu}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y2

$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot \left(y_2 - y_1\right)$ $R_{ep} = 41666.449 \ \frac{lbf}{ft}$ 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:

$c := 0.8$ c = 1.0 for Cast-in-Place
c = 0.8 for Precast

$\Sigma V := V_{Ia}$ $\Sigma V = 5812.257 \ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$e = 0.41 \ ft$ Wall eccentricity, Calculated in above Limiting Eccentricity at Base of Wall (Strength Ia) Section.

$B = 5.5 \ ft$ Footing base width

$\frac{B}{6} = 0.917 \ ft$ If e < B/6 the resultant is in the middle one-third

$\sigma_{vmax} := \frac{\Sigma V}{B} \cdot \left(1 + 6 \cdot \frac{e}{B}\right)$ $\sigma_{vmax} = 1529.401 \ \frac{lbf}{ft^2}$ Max vertical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$\sigma_{vmin} := \frac{\Sigma V}{B} \cdot \left(1 - 6 \cdot \frac{e}{B}\right)$ $\sigma_{vmin} = 584.147 \ \frac{lbf}{ft^2}$ Max verical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$q_{max} := \frac{1}{2} \cdot \sigma_{vmax}$ $q_{max} = 764.7 \ \frac{lbf}{ft^2}$ Max unit shear resistance as 1/2 max vertical stress **LRFD [10.6.3.4]**.

$q_{min} := \frac{1}{2} \cdot \sigma_{vmin}$ $q_{min} = 292.074 \ \frac{lbf}{ft^2}$ Minimum unit shear resistance as 1/2 minimum vertical stress **LRFD [10.6.3.4]**.

Determine which Cohesive Soil Resistance Case is Present:

$Case_1 := \text{if}\left(q_{max} > Su_{fdu} > q_{min} \geq 0 \ , 1 \ , 0\right)$ $Case_1 = 0$

$Case_2 := \text{if}\left(Su_{fdu} > q_{max} > q_{min} \geq 0 \ , 1 \ , 0\right)$ $Case_2 = 1$

$Case_3 := \text{if}\left(q_{max} > q_{min} > Su_{fdu} \ , 1 \ , 0\right)$ $Case_3 = 0$

$Case_4 := \text{if}\left(q_{min} < 0 \ , \text{if}\left(Su_{fdu} < q_{max} \ , 1 \ , 0\right) \ , 0\right)$ $Case_4 = 0$

$Case_5 := \text{if}\left(q_{min} < 0 \ , \text{if}\left(Su_{fdu} > q_{max} \ , 1 \ , 0\right) \ , 0\right)$ $Case_5 = 0$

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Unit Shear Resistance for Case 1:

$S_1 := Su_{fdu} - q_{min} = 8357.926 \ \frac{lbf}{ft^2}$

$S_2 := q_{min} = 292.074 \ \frac{lbf}{ft^2}$

$B_1 := \frac{B \cdot (Su_{fdu} - q_{min})}{q_{max} - q_{min}} = 97.262 \ ft$

$B_2 := \frac{B \cdot (q_{max} - Su_{fdu})}{q_{max} - q_{min}} = -91.762 \ ft$

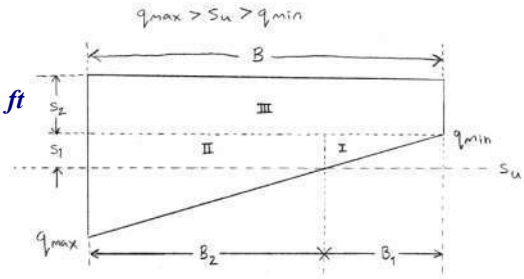
$B_3 := B = 5.5 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 406454.182 \ \frac{lbf}{ft}$

$II := S_1 \cdot B_2 = -766939.769 \ \frac{lbf}{ft}$

$III := S_2 \cdot B_3 = 1606.405 \ \frac{lbf}{ft}$

$R_{\tau_case1} := I + II + III = -358879.182 \ \frac{lbf}{ft}$



Unit Shear Resistance for Case 2:

$S_1 := q_{max} - q_{min} = 472.627 \ \frac{lbf}{ft^2}$

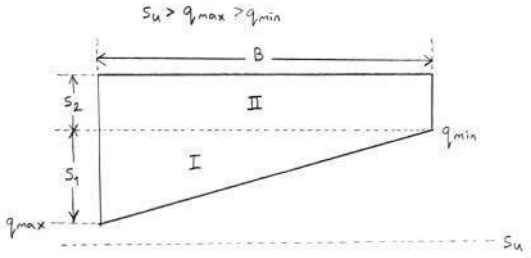
$S_2 := q_{min} = 292.074 \ \frac{lbf}{ft^2}$

$B = 5.5 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B = 1299.723 \ \frac{lbf}{ft}$

$II := S_2 \cdot B = 1606.405 \ \frac{lbf}{ft}$

$R_{\tau_case2} := I + II = 2906.128 \ \frac{lbf}{ft}$



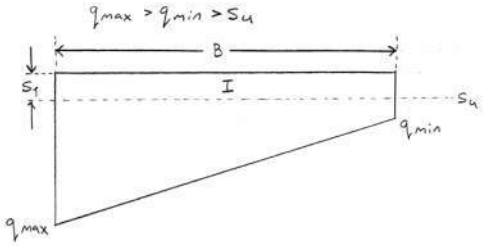
Unit Shear Resistance for Case 3:

$S_1 := Su_{fdu} = 8650 \ \frac{lbf}{ft^2}$

$B = 5.5 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B = 23787.5 \ \frac{lbf}{ft}$

$R_{\tau_case3} := I = 23787.5 \ \frac{lbf}{ft}$



Unit Shear Resistance for Case 4:

$S_1 := Su_{fdu} = 8650 \ \frac{lbf}{ft^2}$

$B_3 := \frac{B \cdot (-q_{min})}{q_{max} - q_{min}} = -3.399 \ ft$

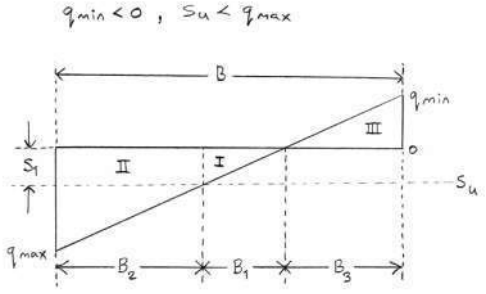
$B_1 := \left(\frac{Su_{fdu}}{q_{max}}\right) \cdot (B - B_3) = 100.661 \ ft$

$B_2 := B - (B_1 + B_3) = -91.762 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 435358.203 \ \frac{lbf}{ft}$

$II := S_1 \cdot B_2 = -793741.022 \ \frac{lbf}{ft}$

$R_{\tau_case4} := I + II = -358382.819 \ \frac{lbf}{ft}$



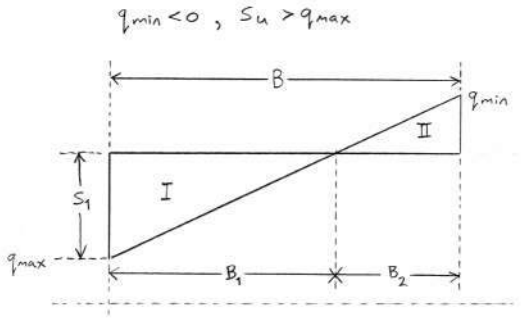
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Unit Shear Resistance for Case 5:

$S_1 := q_{max} = 764.7 \ \frac{lbf}{ft^2}$

$B_1 := \frac{B \cdot q_{max}}{q_{max} - q_{min}} = 8.899 \ ft$

$B_2 := B - B_1 = -3.399 \ ft$



$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 3402.491 \ \frac{lbf}{ft}$

$R_{\tau_case5} := I = 3402.491 \ \frac{lbf}{ft}$

Define the Applicable Case:

$R_\tau := R_{\tau_case2}$ $\quad R_\tau = 2906.128 \ \frac{lbf}{ft}$ Nominal sliding resistance Under Undrained Condition

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]**:

$\phi_{ep} := 0.5$ Resistance factor for passive resistance specified in **LRFD Table 10.5.5.2.2-1**

$\phi_\tau := 1.0$ Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

$\phi R_n := \phi_\tau \cdot R_\tau + \phi_{ep} \cdot R_{ep}$ $\quad R_R := \phi R_n$

Factored Sliding Resistance to be used in CDR Calculations: $R_R = 23739.353 \ \frac{lbf}{ft}$

Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{R_u}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 9.45$

Objective: **To evaluate the external stability of CIP wall design with broken backsloping backfill.**
Method: **In accordance with ODOT Bridge Design Manual [Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].**

Givens:

Backfill Soil Design Parameters:

$\phi'_f := 30\ deg$ — Effective angle of internal friction

$\gamma_f := 120\ \frac{lbf}{ft^3}$ — Unit weight

$c'_f := 0\ \frac{lbf}{ft^2}$ — Effective Cohesion

$\delta := 0.67 \cdot \phi'_f$ $\quad \delta = 20.1\ deg$ — 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):

$\phi'_{fd} := 29\ deg$ — Effective angle of internal friction

$\gamma_{fd} := 140\ \frac{lbf}{ft^3}$ — Unit weight

$c'_{fd} := 450\ \frac{lbf}{ft^2}$ — Effective Cohesion

$\delta_{fd} := 0.67 \cdot \phi'_{fd}$ $\quad \delta_{fd} = 19.4\ deg$ — Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Undrained Conditions (Total Stress):

$\phi_{fdu} := 0\ deg$ — Angle of internal friction (Same as Drained Conditions if granular soils)

$\gamma_{fd} = 140\ \frac{lbf}{ft^3}$ — Unit weight

$Su_{fdu} := 6600\ \frac{lbf}{ft^2}$ — Undrained Shear Strength

$\delta_{fdu} := 0.67 \cdot \phi_{fdu}$ $\quad \delta_{fdu} = 0\ deg$ — Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

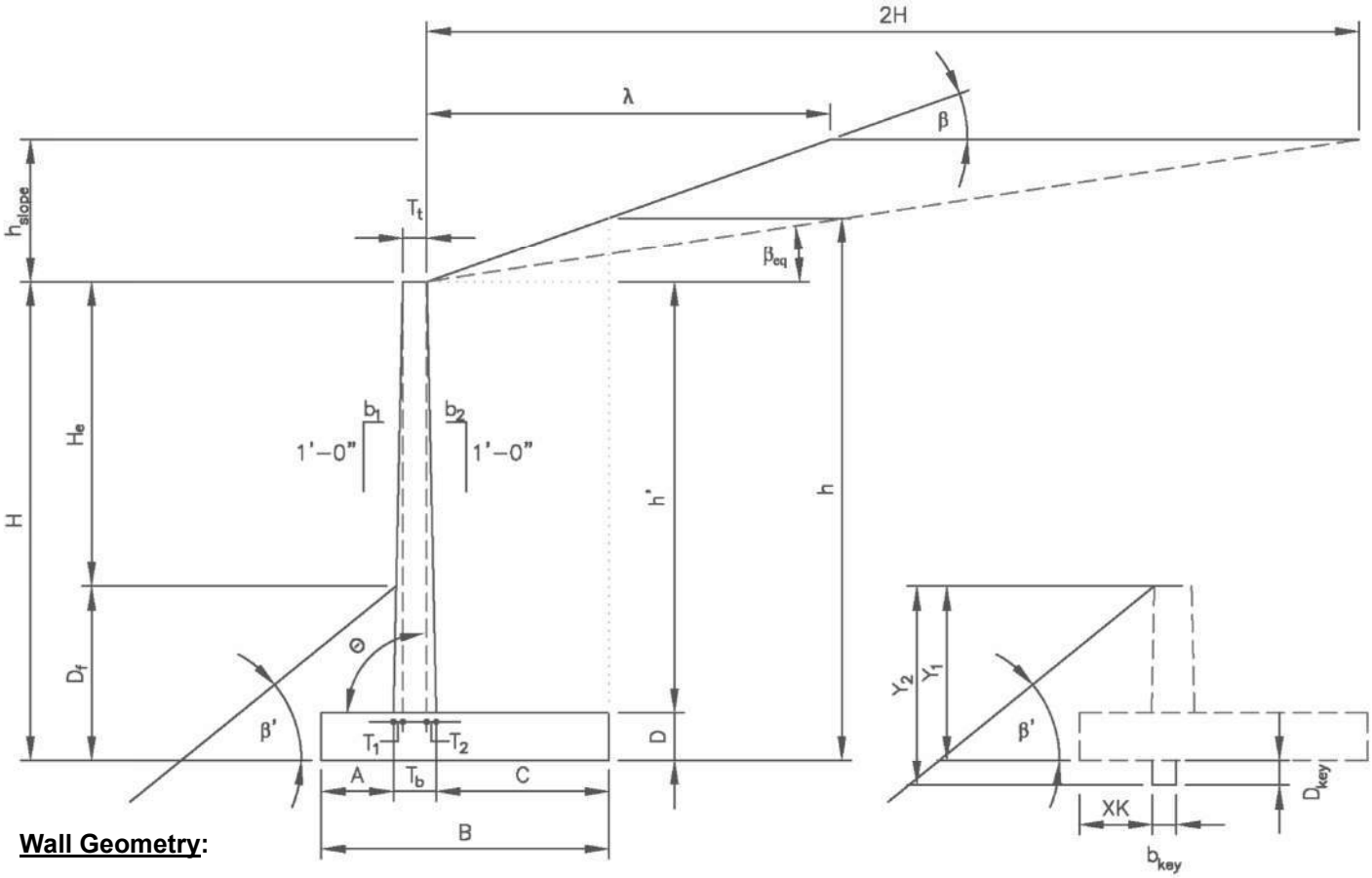
Foundation Surcharge Soil Parameters:

$\gamma_q := 120\ \frac{lbf}{ft^3}$ — Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Other Parameters:

$\gamma_c := 150\ \frac{lbf}{ft^3}$ — Concrete Unit weight

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Wall Geometry:

$H_e := 7.5\ ft$ — Exposed wall height

$D_f := 2.5\ ft$ — 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**

$H := H_e + D_f$ $\quad H = 10\ ft$ — Design Wall Height

$T_t := 12\ in$ — Stem thickness at top of wall

$b_1 := 0 \cdot \left(\frac{in}{ft}\right)$ — Frontwall batter, (b1H:12V)

$b_2 := 0 \cdot \left(\frac{in}{ft}\right)$ — Backwall batter, (b2H:12V)

Preliminary Wall Dimensioning:

$B := 5.5\ ft$ $\quad \frac{2}{5} \cdot H = 4\ ft$ to $\frac{3}{5} \cdot H = 6\ ft$ — Footing base width (2/5H to 3/5H)

$A := 1.92\ ft$ $\quad \frac{H}{8} = 1.25\ ft$ to $\frac{H}{5} = 2\ ft$ — Toe projection (H/8 to H/5)

$D := 1.5\ ft$ $\quad \frac{H}{8} = 1.25\ ft$ to $\frac{H}{5} = 2\ ft$ — Footing thickness (H/8 to H/5)

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Shear Key Dimensioning:

$D_{key} := 2.5\ ft$ — Depth of shear key from bottom of footing
Note: Footings on rock typically require shear key

$b_{key} := 1\ ft$ — Width of shear key

$XK := 0\ ft$ — Distance from toe to shear key

Other Wall Dimensions:

$h' := H - D$ $\quad h' = 8.5\ ft$ — Stem height

$T_1 := b_1 \cdot h'$ $\quad T_1 = 0\ ft$ — Stem front batter width

$T_2 := b_2 \cdot h'$ $\quad T_2 = 0\ ft$ — Stem back batter width

$T_b := T_1 + T_2 + T_t$ $\quad T_b = 1\ ft$ — Stem thickness at bottom of wall

$C := B - A - T_b$ $\quad C = 2.58\ ft$ — Heel projection

$\theta := 90\ deg$ — Angle of back face of wall to horizontal = *atan(12/b2)*

$b := 12\ in$ $\quad b = 1\ ft$ — Concrete strip width (for design)

$y_1 := D_f$ $\quad y_1 = 2.5\ ft$ — Depth to where passive pressure may begin to be utilized in front of wall.

$y_2 := D_f + D_{key}$ $\quad y_2 = 5\ ft$ — Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

Site Grading and Slope Dimensions:

$\beta := 0\ deg$ — Inclination of ground slope behind face of wall. Horizontal backfill behind CIP wall, β = 0 deg

Inclination of ground slope:

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

$\beta' := 0\ deg$ — Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, β' = 0 deg. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

$\lambda := 0\ ft$ — Horizontal distance from the back of the wall to point of slope crest .

$L_{Traffic} := 8\ ft$ — Horizontal distance from assumed traffic surcharge load to Backface of Wall.

$2 \cdot H = 20\ ft$ **IF λ IS GREATER THAN 2*H - USE INFINITE SLOPE CALCULATION SHEET**

$h_{slope} := \lambda \cdot \tan(\beta)$ $\quad h_{slope} = 0\ ft$ — Height of broken slope behind wall

$\beta_{eq} := \operatorname{atan}\left(\frac{h_{slope}}{2 \cdot H}\right) = 0\ deg$ — Equivalent backslope angle

$h := \text{if}\left(\lambda \le T_2 + C,\ H + h_{slope},\ H + \left(T_2 + C\right) \cdot \tan(\beta)\right) = 10\ ft$ — Height of retained fill at back of heel

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Live Load Surcharge Parameters:

$$SUR := \text{if}\left(L_{Traffic} < \frac{H}{2}, 250\ \frac{lbf}{ft^2}, 0\ \frac{lbf}{ft^2}\right) = 0\ \frac{lbf}{ft^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])
Note: ,A live load ssurcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2 .

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{\left(\sin\left(\phi'_f + \delta\right) \cdot \sin\left(\phi'_f - \beta_{eq}\right)\right)}{\left(\sin\left(\theta - \delta\right) \cdot \sin\left(\theta + \beta_{eq}\right)\right)}}\right)^2 \quad \Gamma = 2.687$$

$$k_{af} := \left(\frac{\left(\sin\left(\theta + \phi'_f\right)\right)^2}{\left(\Gamma \cdot \left(\sin\left(\theta\right)\right)^2 \cdot \sin\left(\theta - \delta\right)\right)}\right) \quad k_{af} = 0.297$$

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 θ = 90 degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0 \qquad \frac{-\delta_{fd}}{\phi'_{fd}} = -0.67$$

$$k'_p := 6$$

Passive Earth Pressure Coefficient from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.811$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p \qquad k_{pd} = 4.866$$

Passive Earth Pressure Coefficient for Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

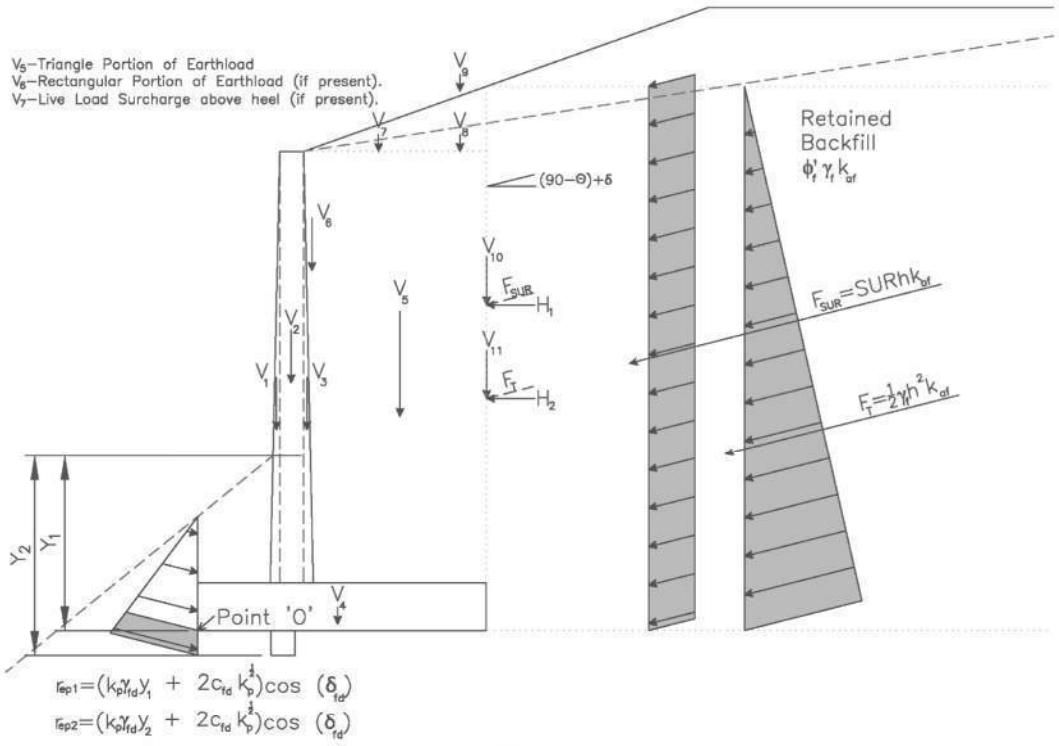
Undrained Conditions:

$$k_{pu} := \text{if}\left(\phi_{fdu} > 0, k_{pu}, 1\right) \qquad k_{pu} = 1$$

Passive Earth Pressure Coefficient for Resistance Undrained Conditions

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Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:



$F_T := \frac{1}{2} \cdot \gamma_f \cdot h^2 \cdot k_{af}$ $F_T = 1783.56 \ \frac{lbf}{ft}$ Active Earth Force Resultant (EH)

$F_{SUR} := SUR \cdot h \cdot k_{af}$ $F_{SUR} = 0 \ \frac{lbf}{ft}$ Live Load Surcharge (LS)

Vertical Loads:

$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$ $V_1 = 0 \ \frac{lbf}{ft}$ Wall stem front batter (DC)

$V_2 := T_t \cdot h' \cdot \gamma_c$ $V_2 = 1275 \ \frac{lbf}{ft}$ Wall stem (DC)

$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$ $V_3 = 0 \ \frac{lbf}{ft}$ Wall stem back batter (DC)

$V_4 := D \cdot B \cdot \gamma_c$ $V_4 = 1237.5 \ \frac{lbf}{ft}$ Wall Footing (DC)

$V_5 := C \cdot h' \cdot \gamma_f$ $V_5 = 2631.6 \ \frac{lbf}{ft}$ Soil Backfill - Heel (EV)

$V_6 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_f$ $V_6 = 0 \ \frac{lbf}{ft}$ Soil Backfill - Batter (EV)

$V_7 := \text{if}\left(\lambda \le T_2 + C, \frac{1}{2} \cdot \lambda^2 \cdot \tan(\beta) \cdot \gamma_f, \frac{1}{2} \cdot (T_2 + C)^2 \cdot \tan(\beta) \cdot \gamma_f\right)$

$V_7 = 0 \ \frac{lbf}{ft}$ Soil Backfill - Backslope Triangle Portion (EV)

$V_8 := \text{if}\left(\lambda \le T_2 + C, (T_2 + C - \lambda) \cdot h_{slope} \cdot \gamma_f, 0 \cdot \frac{lbf}{ft}\right)$

$V_8 = 0 \ \frac{lbf}{ft}$ Soil Backfill - Backslope Rectangular Portion (EV)

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$V_9 := \text{if}\left(\lambda \le T_2 + C,\ (T_2 + C - \lambda) \cdot SUR,\ 0 \cdot \frac{lbf}{ft}\right)$

$V_9 = 0\ \frac{lbf}{ft}$ Live Load Surcharge Above Heel - (LS)

$V_{10} := F_{SUR} \cdot \sin(90 \cdot deg - \theta + \delta)$ $V_{10} = 0\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (vertical comp. - LS)

$V_{11} := F_T \cdot \sin(90 \cdot deg - \theta + \delta)$ $V_{11} = 612.938\ \frac{lbf}{ft}$ Active earth force resultant (vertical component - EH)

Moment Arm: Moment:

Moments produced from vertical loads about Point 'O'

$d_{v1} := A + \frac{2}{3} \cdot T_1 = 1.92\ ft$ $MV_1 := V_1 \cdot d_{v1} = 0\ lbf$

$d_{v2} := A + T_1 + \frac{T_t}{2} = 2.42\ ft$ $MV_2 := V_2 \cdot d_{v2} = 3085.5\ lbf$

$d_{v3} := A + T_1 + T_t + \frac{T_2}{3} = 2.92\ ft$ $MV_3 := V_3 \cdot d_{v3} = 0\ lbf$

$d_{v4} := \frac{B}{2} = 2.75\ ft$ $MV_4 := V_4 \cdot d_{v4} = 3403.125\ lbf$

$d_{v5} := B - \frac{C}{2} = 4.21\ ft$ $MV_5 := V_5 \cdot d_{v5} = 11079.036\ lbf$

$d_{v6} := A + T_1 + T_t + \frac{2\ T_2}{3} = 2.92\ ft$ $MV_6 := V_6 \cdot d_{v6} = 0\ lbf$

$d_{v7} := \text{if}\left(\lambda \le T_2 + C,\ A + T_1 + T_t + \left(\frac{2}{3}\ (\lambda)\right),\ A + T_1 + T_t + \left(\frac{2}{3}\ (T_2 + C)\right)\right) = 2.92\ ft$ $MV_7 := V_7 \cdot d_{v7} = 0\ lbf$

$d_{v8} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2}\right) = 4.21\ ft$ $MV_8 := V_8 \cdot d_{v8} = 0\ lbf$

$d_{v9} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2}\right) = 4.21\ ft$ $MV_9 := V_9 \cdot d_{v9} = 0\ lbf$

$d_{v10} := B = 5.5\ ft$ $MV_{10} := V_{10} \cdot d_{v10} = 0\ lbf$

$d_{v11} := B = 5.5\ ft$ $MV_{11} := V_{11} \cdot d_{v11} = 3371.157\ lbf$

Horizontal Loads:

$H_1 := F_{SUR} \cdot \cos(90 \cdot deg - \theta + \delta)$ $H_1 = 0\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (horizontal comp. - LS)

$H_2 := F_T \cdot \cos(90 \cdot deg - \theta + \delta)$ $H_2 = 1674.931\ \frac{lbf}{ft}$ Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm: Moment:

$d_{h1} := \frac{h}{2}$ $d_{h1} = 5\ ft$ $MH_1 := H_1 \cdot d_{h1}$ $MH_1 = 0\ \frac{lbf \cdot ft}{ft}$

$d_{h2} := \frac{h}{3}$ $d_{h2} = 3.333\ ft$ $MH_2 := H_2 \cdot d_{h2}$ $MH_2 = 5583.103\ \frac{lbf \cdot ft}{ft}$

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Unfactored Loads by Load Type:

$V_{DC} := V_1 + V_2 + V_3 + V_4$ $V_{DC} = 2512.5 \frac{lbf}{ft}$ $V_{EV} := V_5 + V_6 + V_7 + V_8$ $V_{EV} = 2631.6 \frac{lbf}{ft}$

$V_{LS_Ia} := V_{10}$ $V_{LS_Ia} = 0 \frac{lbf}{ft}$ $V_{LS_Ib} := V_9 + V_{10}$ $V_{LS_Ia} = 0 \frac{lbf}{ft}$

$V_{EH} := V_{11}$ $V_{EH} = 612.938 \frac{lbf}{ft}$

$H_{EH} := H_2$ $H_{EH} = 1674.931 \frac{lbf}{ft}$ $H_{LS} := H_1$ $H_{LS} = 0 \frac{lbf}{ft}$

Unfactored Moments by Load Type

$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4$ $M_{DC} = 6488.625 \frac{lbf \cdot ft}{ft}$

$M_{EV} := MV_5 + MV_6 + MV_7 + MV_8$ $M_{EV} = 11079.036 \frac{lbf \cdot ft}{ft}$

$M_{LSV_Ia} := MV_{10}$ $M_{LSV_Ia} = 0 \frac{lbf \cdot ft}{ft}$

$M_{LSV_Ib} := MV_9 + MV_{10}$ $M_{LSV_Ib} = 0 \frac{lbf \cdot ft}{ft}$

$M_{EH1} := MV_{11}$ $M_{EH1} = 3371.157 \frac{lbf \cdot ft}{ft}$

$M_{LSH} := MH_1$ $M_{LSH} = 0 \frac{lbf \cdot ft}{ft}$

$M_{EH2} := MH_2$ $M_{EH2} = 5583.103 \frac{lbf \cdot ft}{ft}$

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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:

$V_{Ia} := \eta \cdot \left(\left(Ia_{DC} \cdot V_{DC} \right) + \left(Ia_{EV} \cdot V_{EV} \right) + \left(Ia_{EH} \cdot V_{EH} \right) + \left(Ia_{LS} \cdot V_{LS_Ia} \right) \right)$ $V_{Ia} = 5812.257 \ \frac{lbf}{ft}$

$V_{Ib} := \eta \cdot \left(\left(Ib_{DC} \cdot V_{DC} \right) + \left(Ib_{EV} \cdot V_{EV} \right) + \left(Ib_{EH} \cdot V_{EH} \right) + \left(Ib_{LS} \cdot V_{LS_Ib} \right) \right)$ $V_{Ib} = 7612.692 \ \frac{lbf}{ft}$

Factored Horizontal Loads by Limit State:

$H_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot H_{LS} \right) + \left(Ia_{EH} \cdot H_{EH} \right) \right)$ $H_{Ia} = 2512.396 \ \frac{lbf}{ft}$

$H_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot H_{LS} \right) + \left(Ib_{EH} \cdot H_{EH} \right) \right)$ $H_{Ib} = 2512.396 \ \frac{lbf}{ft}$

Factored Moments Produced by Vertical Loads by Limit State:

$MV_{Ia} := \eta \cdot \left(\left(Ia_{DC} \cdot M_{DC} \right) + \left(Ia_{EV} \cdot M_{EV} \right) + \left(Ia_{EH} \cdot M_{EH1} \right) + \left(Ia_{LS} \cdot M_{LSV_Ia} \right) \right)$ $MV_{Ia} = 21975.534 \ \frac{lbf \cdot ft}{ft}$

$MV_{Ib} := \eta \cdot \left(\left(Ib_{DC} \cdot M_{DC} \right) + \left(Ib_{EV} \cdot M_{EV} \right) + \left(Ib_{EH} \cdot M_{EH1} \right) + \left(Ib_{LS} \cdot M_{LSV_Ib} \right) \right)$ $MV_{Ib} = 28124.216 \ \frac{lbf \cdot ft}{ft}$

Factored Moments Produced by Horizontal Loads by Limit State:

$MH_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot M_{LSH} \right) + \left(Ia_{EH} \cdot M_{EH2} \right) \right)$ $MH_{Ia} = 8374.655 \ \frac{lbf \cdot ft}{ft}$

$MH_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot M_{LSH} \right) + \left(Ib_{EH} \cdot M_{EH2} \right) \right)$ $MH_{Ib} = 8374.655 \ \frac{lbf \cdot ft}{ft}$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength Ib:

$\Sigma M_R := MV_{Ib}$ $\Sigma M_R = 28124.216 \frac{lbf \cdot ft}{ft}$ Sum of Resisting Moments (Strength Ib)

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 8374.655 \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 7612.692 \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$ $x = 2.594\ ft$ Distance from Point "O" the resultant intersects the base

$e := \left| \frac{B}{2} - x \right|$ $e = 0.16\ ft$ Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

Foundation Layout:

$B' := B - 2 \cdot e$ $B' = 5.189\ ft$ Effective Footing Width

$L' := 35\ ft$ Effective Footing Length (Assumed)

$H' := H_{Ib}$ $H' = 2512.396 \frac{lbf}{ft}$ Summation of Horizontal Loads (Strength Ib)

$V' := V_{Ib}$ $V' = 7612.692 \frac{lbf}{ft}$ Summation of Vertical Loads (Strength Ib)

$D_f = 2.5\ ft$ Footing embedment

$d_w := 0\ ft$ Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan\left(45\ deg + \frac{\phi'_{fd}}{2}\right)^2, 1.0\right)$ $N_q = 16.44$

$N_c := \text{if}\left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14\right)$ $N_c = 27.86$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$ $N_\gamma = 19.338$

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

$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L'}\right)\right)$ $s_c = 1.087$

$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd})\right), 1\right)$ $s_q = 1.082$

$s_\gamma := \text{if}\left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'}\right), 1\right)$ $s_\gamma = 0.941$

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Load inclination factors:

$i_q := 1$

$i_\gamma := 1$

$i_c := 1$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**. "Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$C_{wq} := \text{if}\left(d_w \geq D_f, 1.0, 0.5\right)$ $\quad C_{wq} = 0.5$

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

Depth Correction Factor per Hanson (1970):

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

$d_q = 1.13$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\quad N_{cm} = 30.298$

$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q$ $\quad N_{qm} = 20.146$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\quad N_{\gamma m} = 18.191$

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

$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\quad q_{nd} = 19959.521 \ \frac{lbf}{ft^2}$

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

$\phi_b := .55$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Rd} := \phi_b \cdot q_{nd}$ $\quad q_{Rd} = 10.978 \ ksf$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

$N_\gamma := 2 \cdot \left(N_q + 1\right) \cdot \tan\left(\phi_{fdu}\right)$ $\quad N_\gamma = 0$

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

$s_c := \text{if}\left(\phi_{fdu} > 0\,,\, 1 + \left(\frac{B'}{L'}\right)\cdot\left(\frac{N_q}{N_c}\right),\, 1 + \left(\frac{B'}{5\cdot L'}\right)\right)$ $\quad s_c = 1.03$

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

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

Load inclination factors:

$i_q := 1$

$i_\gamma := 1$

$i_c := 1$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**. "Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

Depth Correction Factor per Hanson (1970):

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

$d_q = 1.13$

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

$N_{cm} := N_c\cdot s_c\cdot i_c$ $\quad N_{cm} = 5.292$

$N_{qm} := N_q\cdot s_q\cdot d_q\cdot i_q$ $\quad N_{qm} = 1.132$

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

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

$q_{nu} := Su_{fdu}\cdot N_{cm} + \gamma_q\cdot D_f\cdot N_{qm}\cdot C_{wq} + 0.5\cdot\gamma_{fd}\cdot B'\cdot N_{\gamma m}\cdot C_{w\gamma}$ $\quad q_{nu} = 35099.633\ \frac{lbf}{ft^2}$

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

$\phi_b := .55$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Ru} := \phi_b\cdot q_{nu}$ $\quad q_{Ru} = 19.305\ ksf$ Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 10.978\ ksf$

Undrained Conditions: $q_{Ru} = 19.305\ ksf$

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Evaluate External Stability of Wall:

Compute the factored bearing stress :

$e = 0.16\ ft$

$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e}$ $\sigma_V = 1.467\ ksf$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{Bearing_D} := \frac{q_{Rd}}{\sigma_V}$ Is the CDR > or = to 1.0? $CDR_{Bearing_D} = 7.48$

Undrained Conditions: $CDR_{Bearing_U} := \frac{q_{Ru}}{\sigma_V}$ Is the CDR > or = to 1.0? $CDR_{Bearing_U} = 13.16$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$e_{max} := \frac{B}{3}$ $e_{max} = 1.833\ ft$ Maximum Eccentricity **LRFD [C11.6.3.3.] Equals B/3 for soil.**

$\Sigma M_R := MV_{Ia}$ $\Sigma M_R = 21975.534\ \frac{lbf \cdot ft}{ft}$ Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$ $\Sigma M_O = 8374.655\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$ $\Sigma V = 5812.257\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$ $x = 2.34\ ft$ Distance from Point "O" the resultant intersects the base

$e := \left| \frac{B}{2} - x \right|$ $e = 0.41\ ft$ Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e. .

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e}$ Is the CDR > or = to 1.0? $CDR_{Eccentricity} = 4.47$

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Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$ $\quad R_u = 2512.396\ \frac{lbf}{ft}$

Drained Conditions (Effective Stress):

Compute passive resistance throughout the design life of the wall **LRFD [Eq 3.11.5.4-1]:**:

$r_{ep1} := \left(k_{pd} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y1

$r_{ep2} := \left(k_{pd} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y2

$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot \left(y_2 - y_1\right)$ $\quad R_{ep} = 10703.51\ \frac{lbf}{ft}$ 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:

$c := 0.8$

c = 1.0 for Cast-in-Place
c = 0.8 for Precast

$\Sigma V := V_{Ia}$ $\quad \Sigma V = 5812.257\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$R_\tau := c \cdot \Sigma V \cdot \tan\left(\phi'_{fd}\right)$ $\quad R_\tau = 2577.429\ \frac{lbf}{ft}$ Nominal sliding resistance Under Drained Condition

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]**:

$\phi_{ep} := 0.5$ Resistance factor for passive resistance specified in **LRFD Table 10.5.5.2.2-1**

$\phi_\tau := 1.0$ Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

$\phi R_n := \phi_\tau \cdot R_\tau + \phi_{ep} \cdot R_{ep}$ $\quad R_R := \phi R_n$

Factored Sliding Resistance to be used in CDR Calculations: $R_R = 7929.184\ \frac{lbf}{ft}$

Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{R_u}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 3.16$

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Undrained Conditions (Total Stress):

Compute passive resistance throughout the design life of the wall **LRFD [Eq 3.11.5.4-1]**::

$r_{ep1} := \left(k_{pu} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot Su_{fdu} \cdot \sqrt{k_{pu}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y1

$r_{ep2} := \left(k_{pu} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot Su_{fdu} \cdot \sqrt{k_{pu}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y2

$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot \left(y_2 - y_1\right)$ $R_{ep} = 32358.355 \ \frac{lbf}{ft}$ 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:

$c := 0.8$ c = 1.0 for Cast-in-Place
c = 0.8 for Precast

$\Sigma V := V_{Ia}$ $\Sigma V = 5812.257 \ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$e = 0.41 \ ft$ Wall eccentricity, Calculated in above Limiting Eccentricity at Base of Wall (Strength Ia) Section.

$B = 5.5 \ ft$ Footing base width

$\frac{B}{6} = 0.917 \ ft$ If e < B/6 the resultant is in the middle one-third

$\sigma_{vmax} := \frac{\Sigma V}{B} \cdot \left(1 + 6 \cdot \frac{e}{B}\right)$ $\sigma_{vmax} = 1529.401 \ \frac{lbf}{ft^2}$ Max vertical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$\sigma_{vmin} := \frac{\Sigma V}{B} \cdot \left(1 - 6 \cdot \frac{e}{B}\right)$ $\sigma_{vmin} = 584.147 \ \frac{lbf}{ft^2}$ Max verical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$q_{max} := \frac{1}{2} \cdot \sigma_{vmax}$ $q_{max} = 764.7 \ \frac{lbf}{ft^2}$ Max unit shear resistance as 1/2 max vertical stress **LRFD [10.6.3.4]**.

$q_{min} := \frac{1}{2} \cdot \sigma_{vmin}$ $q_{min} = 292.074 \ \frac{lbf}{ft^2}$ Minimum unit shear resistance as 1/2 minimum vertical stress **LRFD [10.6.3.4]**.

Determine which Cohesive Soil Resistance Case is Present:

$Case_1 := \text{if}\left(q_{max} > Su_{fdu} > q_{min} \geq 0 \ , 1 \ , 0\right)$ $Case_1 = 0$

$Case_2 := \text{if}\left(Su_{fdu} > q_{max} > q_{min} \geq 0 \ , 1 \ , 0\right)$ $Case_2 = 1$

$Case_3 := \text{if}\left(q_{max} > q_{min} > Su_{fdu} \ , 1 \ , 0\right)$ $Case_3 = 0$

$Case_4 := \text{if}\left(q_{min} < 0 \ , \text{if}\left(Su_{fdu} < q_{max} \ , 1 \ , 0\right) \ , 0\right)$ $Case_4 = 0$

$Case_5 := \text{if}\left(q_{min} < 0 \ , \text{if}\left(Su_{fdu} > q_{max} \ , 1 \ , 0\right) \ , 0\right)$ $Case_5 = 0$

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Unit Shear Resistance for Case 1:

$S_1 := Su_{fdu} - q_{min} = 6307.926 \ \frac{lbf}{ft^2}$

$S_2 := q_{min} = 292.074 \ \frac{lbf}{ft^2}$

$B_1 := \frac{B \cdot (Su_{fdu} - q_{min})}{q_{max} - q_{min}} = 73.406 \ ft$

$B_2 := \frac{B \cdot (q_{max} - Su_{fdu})}{q_{max} - q_{min}} = -67.906 \ ft$

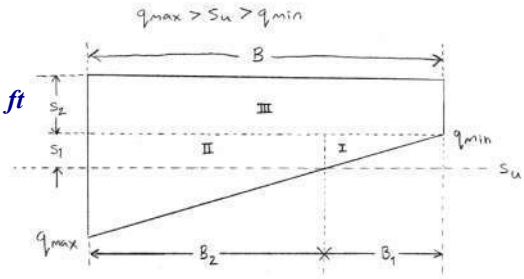
$B_3 := B = 5.5 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 231519.59 \ \frac{lbf}{ft}$

$II := S_1 \cdot B_2 = -428345.585 \ \frac{lbf}{ft}$

$III := S_2 \cdot B_3 = 1606.405 \ \frac{lbf}{ft}$

$R_{\tau_case1} := I + II + III = -195219.59 \ \frac{lbf}{ft}$



Unit Shear Resistance for Case 2:

$S_1 := q_{max} - q_{min} = 472.627 \ \frac{lbf}{ft^2}$

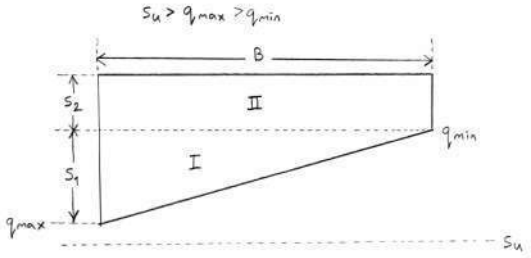
$S_2 := q_{min} = 292.074 \ \frac{lbf}{ft^2}$

$B = 5.5 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B = 1299.723 \ \frac{lbf}{ft}$

$II := S_2 \cdot B = 1606.405 \ \frac{lbf}{ft}$

$R_{\tau_case2} := I + II = 2906.128 \ \frac{lbf}{ft}$



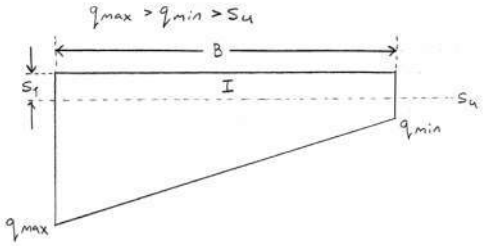
Unit Shear Resistance for Case 3:

$S_1 := Su_{fdu} = 6600 \ \frac{lbf}{ft^2}$

$B = 5.5 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B = 18150 \ \frac{lbf}{ft}$

$R_{\tau_case3} := I = 18150 \ \frac{lbf}{ft}$



Unit Shear Resistance for Case 4:

$S_1 := Su_{fdu} = 6600 \ \frac{lbf}{ft^2}$

$B_3 := \frac{B \cdot (-q_{min})}{q_{max} - q_{min}} = -3.399 \ ft$

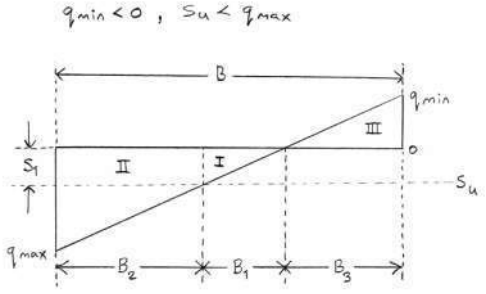
$B_1 := \left(\frac{Su_{fdu}}{q_{max}}\right) \cdot (B - B_3) = 76.805 \ ft$

$B_2 := B - (B_1 + B_3) = -67.906 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 253455.89 \ \frac{lbf}{ft}$

$II := S_1 \cdot B_2 = -448179.117 \ \frac{lbf}{ft}$

$R_{\tau_case4} := I + II = -194723.227 \ \frac{lbf}{ft}$



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Unit Shear Resistance for Case 5:

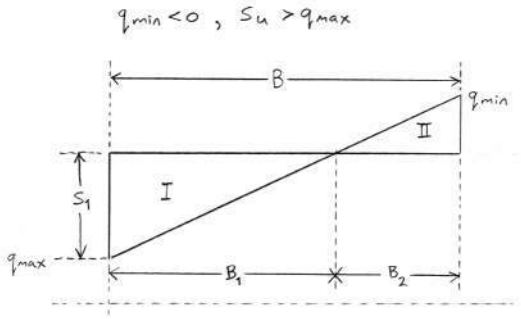
$S_1 := q_{max} = 764.7 \ \frac{lbf}{ft^2}$

$B_1 := \frac{B \cdot q_{max}}{q_{max} - q_{min}} = 8.899 \ ft$

$B_2 := B - B_1 = -3.399 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 3402.491 \ \frac{lbf}{ft}$

$R_{\tau_case5} := I = 3402.491 \ \frac{lbf}{ft}$



Define the Applicable Case:

$R_\tau := R_{\tau_case2}$ $R_\tau = 2906.128 \ \frac{lbf}{ft}$ Nominal sliding resistance Under Undrained Condition

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]**:

$\phi_{ep} := 0.5$ Resistance factor for passive resistance specified in **LRFD Table 10.5.5.2.2-1**

$\phi_\tau := 1.0$ Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

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

Factored Sliding Resistance to be used in CDR Calculations: $R_R = 19085.306 \ \frac{lbf}{ft}$

Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{R_u}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 7.60$

Objective: **To evaluate the external stability of CIP wall design with broken backsloping backfill.**
Method: **In accordance with ODOT Bridge Design Manual [Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].**

Givens:

Backfill Soil Design Parameters:

$\phi'_f := 30\ deg$ — Effective angle of internal friction

$\gamma_f := 120\ \frac{lbf}{ft^3}$ — Unit weight

$c'_f := 0\ \frac{lbf}{ft^2}$ — Effective Cohesion

$\delta := 0.67 \cdot \phi'_f$ $\quad \delta = 20.1\ deg$ — 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):

$\phi'_{fd} := 32\ deg$ — Effective angle of internal friction

$\gamma_{fd} := 140\ \frac{lbf}{ft^3}$ — Unit weight

$c'_{fd} := 450\ \frac{lbf}{ft^2}$ — Effective Cohesion

$\delta_{fd} := 0.67 \cdot \phi'_{fd}$ $\quad \delta_{fd} = 21.4\ deg$ — Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Undrained Conditions (Total Stress):

$\phi_{fdu} := 0\ deg$ — Angle of internal friction (Same as Drained Conditions if granular soils)

$\gamma_{fd} = 140\ \frac{lbf}{ft^3}$ — Unit weight

$Su_{fdu} := 9700\ \frac{lbf}{ft^2}$ — Undrained Shear Strength

$\delta_{fdu} := 0.67 \cdot \phi_{fdu}$ $\quad \delta_{fdu} = 0\ deg$ — Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

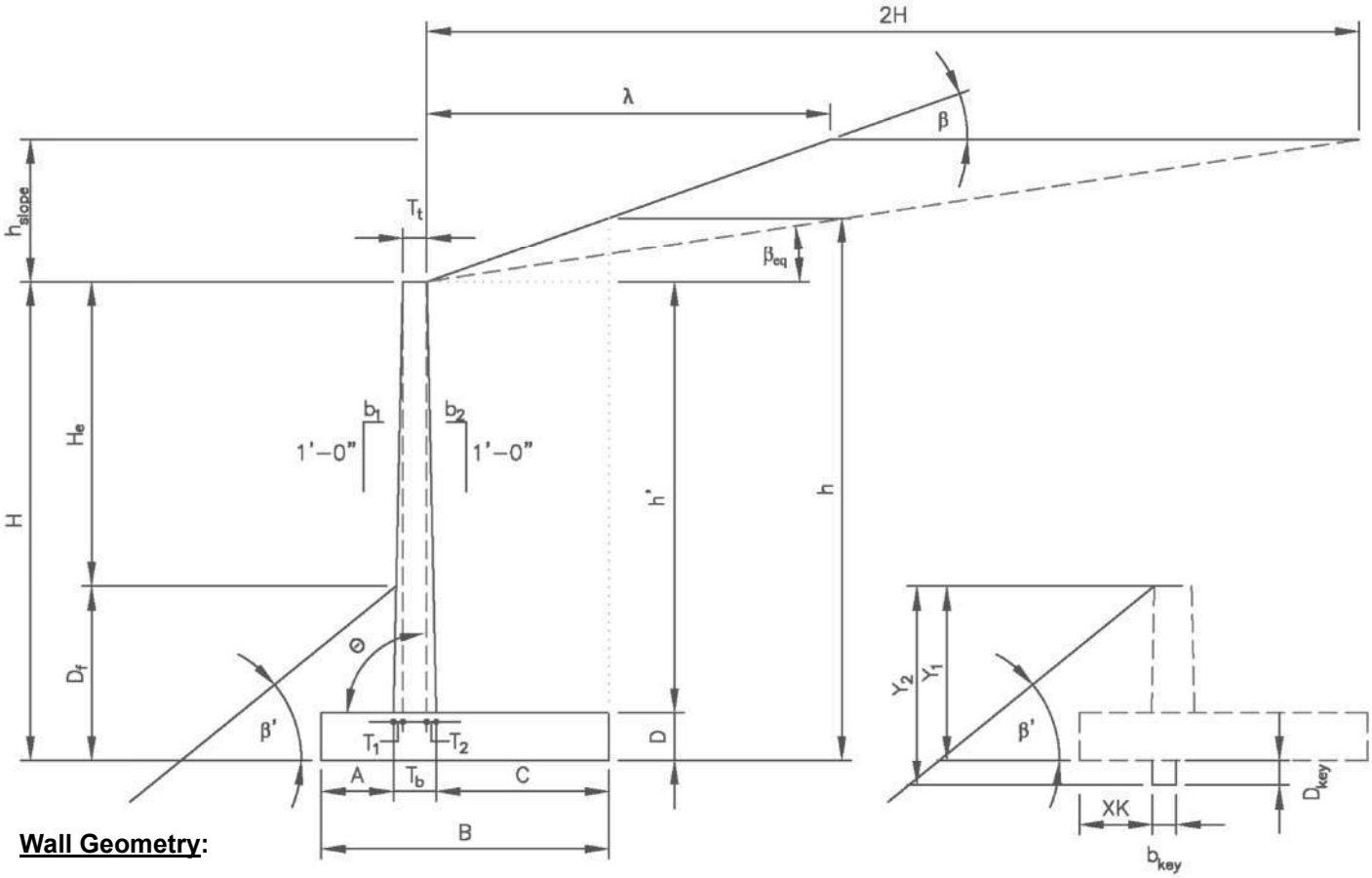
Foundation Surcharge Soil Parameters:

$\gamma_q := 120\ \frac{lbf}{ft^3}$ — Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Other Parameters:

$\gamma_c := 150\ \frac{lbf}{ft^3}$ — Concrete Unit weight

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Wall Geometry:

$H_e := 8.5\ ft$ — Exposed wall height

$D_f := 2.5\ ft$ — 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**

$H := H_e + D_f$ $\quad H = 11\ ft$ — Design Wall Height

$T_t := 12\ in$ — Stem thickness at top of wall

$b_1 := 0 \cdot \left(\frac{in}{ft}\right)$ — Frontwall batter, (b1H:12V)

$b_2 := 0 \cdot \left(\frac{in}{ft}\right)$ — Backwall batter, (b2H:12V)

Preliminary Wall Dimensioning:

$B := 6.25\ ft$ $\quad \frac{2}{5} \cdot H = 4.4\ ft$ to $\frac{3}{5} \cdot H = 6.6\ ft$ — Footing base width (2/5H to 3/5H)

$A := 2.25\ ft$ $\quad \frac{H}{8} = 1.38\ ft$ to $\frac{H}{5} = 2.2\ ft$ — Toe projection (H/8 to H/5)

$D := 1.5\ ft$ $\quad \frac{H}{8} = 1.38\ ft$ to $\frac{H}{5} = 2.2\ ft$ — Footing thickness (H/8 to H/5)

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Shear Key Dimensioning:

$D_{key} := 2.5\ ft$ — Depth of shear key from bottom of footing
Note: Footings on rock typically require shear key

$b_{key} := 1\ ft$ — Width of shear key

$XK := 0\ ft$ — Distance from toe to shear key

Other Wall Dimensions:

$h' := H - D$ $\quad h' = 9.5\ ft$ — Stem height

$T_1 := b_1 \cdot h'$ $\quad T_1 = 0\ ft$ — Stem front batter width

$T_2 := b_2 \cdot h'$ $\quad T_2 = 0\ ft$ — Stem back batter width

$T_b := T_1 + T_2 + T_t$ $\quad T_b = 1\ ft$ — Stem thickness at bottom of wall

$C := B - A - T_b$ $\quad C = 3\ ft$ — Heel projection

$\theta := 90\ deg$ — Angle of back face of wall to horizontal = *atan(12/b2)*

$b := 12\ in$ $\quad b = 1\ ft$ — Concrete strip width (for design)

$y_1 := D_f$ $\quad y_1 = 2.5\ ft$ — Depth to where passive pressure may begin to be utilized in front of wall.

$y_2 := D_f + D_{key}$ $\quad y_2 = 5\ ft$ — Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

Site Grading and Slope Dimensions:

$\beta := 14.036\ deg$ — Inclination of ground slope behind face of wall. Horizontal backfill behind CIP wall, β = 0 deg

Inclination of ground slope:

- Horizontal: **0**
- 4H:1V: **14.036**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$\beta' := 0\ deg$ — Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, β' = 0 deg. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

$\lambda := 13\ ft$ — Horizontal distance from the back of the wall to point of slope crest .

$L_{Traffic} := 10\ ft$ — Horizontal distance from assumed traffic surcharge load to Backface of Wall.

$2 \cdot H = 22\ ft$ **IF λ IS GREATER THAN 2*H - USE INFINITE SLOPE CALCULATION SHEET**

$h_{slope} := \lambda \cdot \tan(\beta)$ $\quad h_{slope} = 3.25\ ft$ — Height of broken slope behind wall

$\beta_{eq} := \operatorname{atan}\left(\frac{h_{slope}}{2 \cdot H}\right) = 8.403\ deg$ — Equivalent backslope angle

$h := \text{if}\left(\lambda \le T_2 + C,\ H + h_{slope},\ H + \left(T_2 + C\right) \cdot \tan(\beta)\right) = 11.75\ ft$ — Height of retained fill at back of heel

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Live Load Surcharge Parameters:

$$SUR := \text{if}\left(L_{Traffic} < \frac{H}{2}, 250\ \frac{lbf}{ft^2}, 0\ \frac{lbf}{ft^2}\right) = 0\ \frac{lbf}{ft^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])
Note: ,A live load ssurcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2 .

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{\left(\sin\left(\phi'_f + \delta\right) \cdot \sin\left(\phi'_f - \beta_{eq}\right)\right)}{\left(\sin\left(\theta - \delta\right) \cdot \sin\left(\theta + \beta_{eq}\right)\right)}}\right)^2 \quad \Gamma = 2.407$$

$$k_{af} := \left(\frac{\left(\sin\left(\theta + \phi'_f\right)\right)^2}{\left(\Gamma \cdot \left(\sin\left(\theta\right)\right)^2 \cdot \sin\left(\theta - \delta\right)\right)}\right) \quad k_{af} = 0.332$$

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 θ = 90 degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0 \qquad \frac{-\delta_{fd}}{\phi'_{fd}} = -0.67$$

$$k'_p := 8$$

Passive Earth Pressure Coefficient from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.752$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p \qquad k_{pd} = 6.016$$

Passive Earth Pressure Coefficient for Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

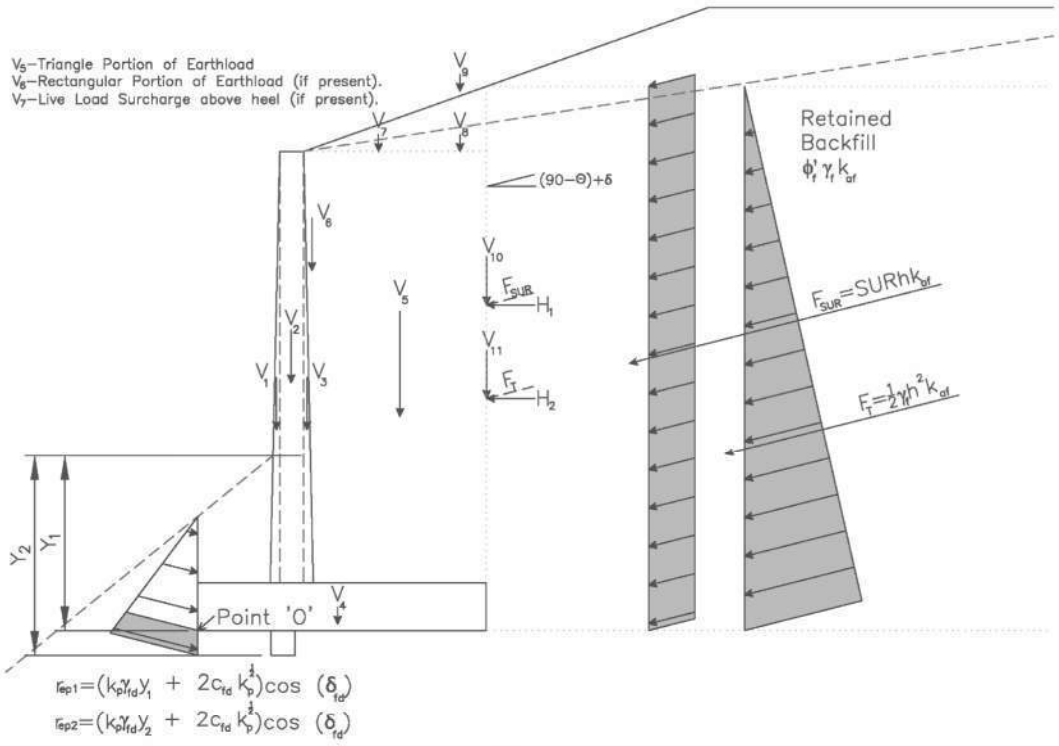
Undrained Conditions:

$$k_{pu} := \text{if}\left(\phi_{fdu} > 0, k_{pu}, 1\right) \qquad k_{pu} = 1$$

Passive Earth Pressure Coefficient for Resistance Undrained Conditions

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Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:



$F_T := \frac{1}{2} \cdot \gamma_f \cdot h^2 \cdot k_{af}$ $F_T = 2749.019 \frac{lbf}{ft}$ Active Earth Force Resultant (EH)

$F_{SUR} := SUR \cdot h \cdot k_{af}$ $F_{SUR} = 0 \frac{lbf}{ft}$ Live Load Surcharge (LS)

Vertical Loads:

$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$ $V_1 = 0 \frac{lbf}{ft}$ Wall stem front batter (DC)

$V_2 := T_t \cdot h' \cdot \gamma_c$ $V_2 = 1425 \frac{lbf}{ft}$ Wall stem (DC)

$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$ $V_3 = 0 \frac{lbf}{ft}$ Wall stem back batter (DC)

$V_4 := D \cdot B \cdot \gamma_c$ $V_4 = 1406.25 \frac{lbf}{ft}$ Wall Footing (DC)

$V_5 := C \cdot h' \cdot \gamma_f$ $V_5 = 3420 \frac{lbf}{ft}$ Soil Backfill - Heel (EV)

$V_6 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_f$ $V_6 = 0 \frac{lbf}{ft}$ Soil Backfill - Batter (EV)

$V_7 := \text{if}\left(\lambda \le T_2 + C, \frac{1}{2} \cdot \lambda^2 \cdot \tan(\beta) \cdot \gamma_f, \frac{1}{2} \cdot (T_2 + C)^2 \cdot \tan(\beta) \cdot \gamma_f\right)$

$V_7 = 134.998 \frac{lbf}{ft}$ Soil Backfill - Backslope Triangle Portion (EV)

$V_8 := \text{if}\left(\lambda \le T_2 + C, (T_2 + C - \lambda) \cdot h_{slope} \cdot \gamma_f, 0 \cdot \frac{lbf}{ft}\right)$

$V_8 = 0 \frac{lbf}{ft}$ Soil Backfill - Backslope Rectangular Portion (EV)

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$V_9 := \text{if}\left(\lambda \le T_2 + C,\ (T_2 + C - \lambda)\cdot SUR,\ 0\cdot \frac{lbf}{ft}\right)$

$V_9 = 0\ \frac{lbf}{ft}$ Live Load Surcharge Above Heel - (LS)

$V_{10} := F_{SUR}\cdot \sin(90\cdot deg - \theta + \delta)$ $V_{10} = 0\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (vertical comp. - LS)

$V_{11} := F_T\cdot \sin(90\cdot deg - \theta + \delta)$ $V_{11} = 944.727\ \frac{lbf}{ft}$ Active earth force resultant (vertical component - EH)

Moment Arm: Moment:

Moments produced from vertical loads about Point 'O'

$d_{v1} := A + \frac{2}{3}\cdot T_1 = 2.25\ ft$ $MV_1 := V_1\cdot d_{v1} = 0\ lbf$

$d_{v2} := A + T_1 + \frac{T_t}{2} = 2.75\ ft$ $MV_2 := V_2\cdot d_{v2} = 3918.75\ lbf$

$d_{v3} := A + T_1 + T_t + \frac{T_2}{3} = 3.25\ ft$ $MV_3 := V_3\cdot d_{v3} = 0\ lbf$

$d_{v4} := \frac{B}{2} = 3.125\ ft$ $MV_4 := V_4\cdot d_{v4} = 4394.531\ lbf$

$d_{v5} := B - \frac{C}{2} = 4.75\ ft$ $MV_5 := V_5\cdot d_{v5} = 16245\ lbf$

$d_{v6} := A + T_1 + T_t + \frac{2\ T_2}{3} = 3.25\ ft$ $MV_6 := V_6\cdot d_{v6} = 0\ lbf$

$d_{v7} := \text{if}\left(\lambda \le T_2 + C,\ A + T_1 + T_t + \left(\frac{2}{3}\ (\lambda)\right),\ A + T_1 + T_t + \left(\frac{2}{3}\ (T_2 + C)\right)\right) = 5.25\ ft$ $MV_7 := V_7\cdot d_{v7} = 708.737\ lbf$

$d_{v8} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2}\right) = 11.25\ ft$ $MV_8 := V_8\cdot d_{v8} = 0\ lbf$

$d_{v9} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2}\right) = 11.25\ ft$ $MV_9 := V_9\cdot d_{v9} = 0\ lbf$

$d_{v10} := B = 6.25\ ft$ $MV_{10} := V_{10}\cdot d_{v10} = 0\ lbf$

$d_{v11} := B = 6.25\ ft$ $MV_{11} := V_{11}\cdot d_{v11} = 5904.544\ lbf$

Horizontal Loads:

$H_1 := F_{SUR}\cdot \cos(90\cdot deg - \theta + \delta)$ $H_1 = 0\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (horizontal comp. - LS)

$H_2 := F_T\cdot \cos(90\cdot deg - \theta + \delta)$ $H_2 = 2581.588\ \frac{lbf}{ft}$ Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm: Moment:

$d_{h1} := \frac{h}{2}$ $d_{h1} = 5.875\ ft$ $MH_1 := H_1\cdot d_{h1}$ $MH_1 = 0\ \frac{lbf\cdot ft}{ft}$

$d_{h2} := \frac{h}{3}$ $d_{h2} = 3.917\ ft$ $MH_2 := H_2\cdot d_{h2}$ $MH_2 = 10111.208\ \frac{lbf\cdot ft}{ft}$

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Unfactored Loads by Load Type:

$V_{DC} := V_1 + V_2 + V_3 + V_4$ $V_{DC} = 2831.25 \frac{lbf}{ft}$ $V_{EV} := V_5 + V_6 + V_7 + V_8$ $V_{EV} = 3554.998 \frac{lbf}{ft}$

$V_{LS_Ia} := V_{10}$ $V_{LS_Ia} = 0 \frac{lbf}{ft}$ $V_{LS_Ib} := V_9 + V_{10}$ $V_{LS_Ia} = 0 \frac{lbf}{ft}$

$V_{EH} := V_{11}$ $V_{EH} = 944.727 \frac{lbf}{ft}$

$H_{EH} := H_2$ $H_{EH} = 2581.588 \frac{lbf}{ft}$ $H_{LS} := H_1$ $H_{LS} = 0 \frac{lbf}{ft}$

Unfactored Moments by Load Type

$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4$ $M_{DC} = 8313.281 \frac{lbf \cdot ft}{ft}$

$M_{EV} := MV_5 + MV_6 + MV_7 + MV_8$ $M_{EV} = 16953.737 \frac{lbf \cdot ft}{ft}$

$M_{LSV_Ia} := MV_{10}$ $M_{LSV_Ia} = 0 \frac{lbf \cdot ft}{ft}$

$M_{LSV_Ib} := MV_9 + MV_{10}$ $M_{LSV_Ib} = 0 \frac{lbf \cdot ft}{ft}$

$M_{EH1} := MV_{11}$ $M_{EH1} = 5904.544 \frac{lbf \cdot ft}{ft}$

$M_{LSH} := MH_1$ $M_{LSH} = 0 \frac{lbf \cdot ft}{ft}$

$M_{EH2} := MH_2$ $M_{EH2} = 10111.208 \frac{lbf \cdot ft}{ft}$

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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:

$V_{Ia} := \eta \cdot \left(\left(Ia_{DC} \cdot V_{DC} \right) + \left(Ia_{EV} \cdot V_{EV} \right) + \left(Ia_{EH} \cdot V_{EH} \right) + \left(Ia_{LS} \cdot V_{LS_Ia} \right) \right)$ $V_{Ia} = 7520.213 \ \frac{lbf}{ft}$

$V_{Ib} := \eta \cdot \left(\left(Ib_{DC} \cdot V_{DC} \right) + \left(Ib_{EV} \cdot V_{EV} \right) + \left(Ib_{EH} \cdot V_{EH} \right) + \left(Ib_{LS} \cdot V_{LS_Ib} \right) \right)$ $V_{Ib} = 9755.4 \ \frac{lbf}{ft}$

Factored Horizontal Loads by Limit State:

$H_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot H_{LS} \right) + \left(Ia_{EH} \cdot H_{EH} \right) \right)$ $H_{Ia} = 3872.382 \ \frac{lbf}{ft}$

$H_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot H_{LS} \right) + \left(Ib_{EH} \cdot H_{EH} \right) \right)$ $H_{Ib} = 3872.382 \ \frac{lbf}{ft}$

Factored Moments Produced by Vertical Loads by Limit State:

$MV_{Ia} := \eta \cdot \left(\left(Ia_{DC} \cdot M_{DC} \right) + \left(Ia_{EV} \cdot M_{EV} \right) + \left(Ia_{EH} \cdot M_{EH1} \right) + \left(Ia_{LS} \cdot M_{LSV_Ia} \right) \right)$ $MV_{Ia} = 33292.506 \ \frac{lbf \cdot ft}{ft}$

$MV_{Ib} := \eta \cdot \left(\left(Ib_{DC} \cdot M_{DC} \right) + \left(Ib_{EV} \cdot M_{EV} \right) + \left(Ib_{EH} \cdot M_{EH1} \right) + \left(Ib_{LS} \cdot M_{LSV_Ib} \right) \right)$ $MV_{Ib} = 42135.963 \ \frac{lbf \cdot ft}{ft}$

Factored Moments Produced by Horizontal Loads by Limit State:

$MH_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot M_{LSH} \right) + \left(Ia_{EH} \cdot M_{EH2} \right) \right)$ $MH_{Ia} = 15166.812 \ \frac{lbf \cdot ft}{ft}$

$MH_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot M_{LSH} \right) + \left(Ib_{EH} \cdot M_{EH2} \right) \right)$ $MH_{Ib} = 15166.812 \ \frac{lbf \cdot ft}{ft}$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength Ib:

$\Sigma M_R := MV_{Ib}$ $\Sigma M_R = 42135.963 \frac{lbf \cdot ft}{ft}$ Sum of Resisting Moments (Strength Ib)

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 15166.812 \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 9755.4 \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$ $x = 2.765\ ft$ Distance from Point "O" the resultant intersects the base

$e := \left| \frac{B}{2} - x \right|$ $e = 0.36\ ft$ Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

Foundation Layout:

$B' := B - 2 \cdot e$ $B' = 5.529\ ft$ Effective Footing Width

$L' := 41\ ft$ Effective Footing Length (Assumed)

$H' := H_{Ib}$ $H' = 3872.382 \frac{lbf}{ft}$ Summation of Horizontal Loads (Strength Ib)

$V' := V_{Ib}$ $V' = 9755.4 \frac{lbf}{ft}$ Summation of Vertical Loads (Strength Ib)

$D_f = 2.5\ ft$ Footing embedment

$d_w := 0\ ft$ Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan\left(45\ deg + \frac{\phi'_{fd}}{2}\right)^2, 1.0\right)$ $N_q = 23.18$

$N_c := \text{if}\left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14\right)$ $N_c = 35.49$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$ $N_\gamma = 30.215$

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

$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L'}\right)\right)$ $s_c = 1.088$

$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd})\right), 1\right)$ $s_q = 1.084$

$s_\gamma := \text{if}\left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'}\right), 1\right)$ $s_\gamma = 0.946$

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Load inclination factors:

$i_q := 1$

$i_\gamma := 1$

$i_c := 1$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**. "Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$C_{wq} := \text{if}\left(d_w \geq D_f, 1.0, 0.5\right)$ $C_{wq} = 0.5$

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

Depth Correction Factor per Hanson (1970):

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

$d_q = 1.12$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $N_{cm} = 38.616$

$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q$ $N_{qm} = 28.077$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $N_{\gamma m} = 28.585$

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

$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $q_{nd} = 27120.28 \ \frac{lbf}{ft^2}$

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

$\phi_b := .55$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Rd} := \phi_b \cdot q_{nd}$ $q_{Rd} = 14.916 \ ksf$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

$N_\gamma := 2 \cdot \left(N_q + 1\right) \cdot \tan\left(\phi_{fdu}\right)$ $N_\gamma = 0$

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

$s_c := \text{if}\left(\phi_{fdu} > 0\,,\, 1 + \left(\frac{B'}{L'}\right)\cdot\left(\frac{N_q}{N_c}\right)\,,\, 1 + \left(\frac{B'}{5\cdot L'}\right)\right)$ $\quad s_c = 1.027$

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

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

Load inclination factors:

$i_q := 1$

$i_\gamma := 1$

$i_c := 1$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**. "Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

Depth Correction Factor per Hanson (1970):

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

$d_q = 1.12$

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

$N_{cm} := N_c\cdot s_c\cdot i_c$ $\quad N_{cm} = 5.279$

$N_{qm} := N_q\cdot s_q\cdot d_q\cdot i_q$ $\quad N_{qm} = 1.117$

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

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

$q_{nu} := Su_{fdu}\cdot N_{cm} + \gamma_q\cdot D_f\cdot N_{qm}\cdot C_{wq} + 0.5\cdot\gamma_{fd}\cdot B'\cdot N_{\gamma m}\cdot C_{w\gamma}$ $\quad q_{nu} = 51370.315\ \frac{lbf}{ft^2}$

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

$\phi_b := .55$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Ru} := \phi_b\cdot q_{nu}$ $\quad q_{Ru} = 28.254\ ksf$ Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 14.916\ ksf$

Undrained Conditions: $q_{Ru} = 28.254\ ksf$

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Evaluate External Stability of Wall:

Compute the factored bearing stress :

$e = 0.36\ ft$

$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e}$ $\sigma_V = 1.764\ ksf$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{Bearing_D} := \frac{q_{Rd}}{\sigma_V}$ Is the CDR > or = to 1.0? $CDR_{Bearing_D} = 8.45$

Undrained Conditions: $CDR_{Bearing_U} := \frac{q_{Ru}}{\sigma_V}$ Is the CDR > or = to 1.0? $CDR_{Bearing_U} = 16.01$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$e_{max} := \frac{B}{3}$	$e_{max} = 2.083\ ft$	Maximum Eccentricity **LRFD [C11.6.3.3.] Equals B/3 for soil.**
$\Sigma M_R := MV_{Ia}$	$\Sigma M_R = 33292.506\ \frac{lbf \cdot ft}{ft}$	Sum of Resisting Moments (Strength Ia)
$\Sigma M_O := MH_{Ia}$	$\Sigma M_O = 15166.812\ \frac{lbf \cdot ft}{ft}$	Sum of Overturning Moments (Strength Ia)
$\Sigma V := V_{Ia}$	$\Sigma V = 7520.213\ \frac{lbf}{ft}$	Sum of Vertical Loads (Strength Ia)
$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$	$x = 2.41\ ft$	Distance from Point "O" the resultant intersects the base
$e := \left\lvert \frac{B}{2} - x \right\rvert$	$e = 0.71\ ft$	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? $CDR_{Eccentricity} = 2.91$

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Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$ $\quad R_u = 3872.382\ \frac{lbf}{ft}$

Drained Conditions (Effective Stress):

Compute passive resistance throughout the design life of the wall **LRFD [Eq 3.11.5.4-1]:**:

$r_{ep1} := \left(k_{pd} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y1

$r_{ep2} := \left(k_{pd} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y2

$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot \left(y_2 - y_1\right)$ $\quad R_{ep} = 12486.41\ \frac{lbf}{ft}$ 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:

$c := 0.8$

c = 1.0 for Cast-in-Place
c = 0.8 for Precast

$\Sigma V := V_{Ia}$ $\quad \Sigma V = 7520.213\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$R_\tau := c \cdot \Sigma V \cdot \tan\left(\phi'_{fd}\right)$ $\quad R_\tau = 3759.321\ \frac{lbf}{ft}$ Nominal sliding resistance Under Drained Condition

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]**:

$\phi_{ep} := 0.5$ Resistance factor for passive resistance specified in **LRFD Table 10.5.5.2.2-1**

$\phi_\tau := 1.0$ Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

$\phi R_n := \phi_\tau \cdot R_\tau + \phi_{ep} \cdot R_{ep}$ $\quad R_R := \phi R_n$

Factored Sliding Resistance to be used in CDR Calculations: $R_R = 10002.526\ \frac{lbf}{ft}$

Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{R_u}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 2.58$

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Undrained Conditions (Total Stress):

Compute passive resistance throughout the design life of the wall **LRFD [Eq 3.11.5.4-1]**::

$r_{ep1} := \left(k_{pu} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot Su_{fdu} \cdot \sqrt{k_{pu}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y1

$r_{ep2} := \left(k_{pu} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot Su_{fdu} \cdot \sqrt{k_{pu}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y2

$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot \left(y_2 - y_1\right)$ $R_{ep} = 46365.518 \ \frac{lbf}{ft}$ 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:

$c := 0.8$ c = 1.0 for Cast-in-Place
c = 0.8 for Precast

$\Sigma V := V_{Ia}$ $\Sigma V = 7520.213 \ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$e = 0.71 \ ft$ Wall eccentricity, Calculated in above Limiting Eccentricity at Base of Wall (Strength Ia) Section.

$B = 6.25 \ ft$ Footing base width

$\frac{B}{6} = 1.042 \ ft$ If e < B/6 the resultant is in the middle one-third

$\sigma_{vmax} := \frac{\Sigma V}{B} \cdot \left(1 + 6 \cdot \frac{e}{B}\right)$ $\sigma_{vmax} = 2028.83 \ \frac{lbf}{ft^2}$ Max vertical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$\sigma_{vmin} := \frac{\Sigma V}{B} \cdot \left(1 - 6 \cdot \frac{e}{B}\right)$ $\sigma_{vmin} = 377.638 \ \frac{lbf}{ft^2}$ Max verical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$q_{max} := \frac{1}{2} \cdot \sigma_{vmax}$ $q_{max} = 1014.415 \ \frac{lbf}{ft^2}$ Max unit shear resistance as 1/2 max vertical stress **LRFD [10.6.3.4]**.

$q_{min} := \frac{1}{2} \cdot \sigma_{vmin}$ $q_{min} = 188.819 \ \frac{lbf}{ft^2}$ Minimum unit shear resistance as 1/2 minimum vertical stress **LRFD [10.6.3.4]**.

Determine which Cohesive Soil Resistance Case is Present:

$Case_1 := \text{if}\left(q_{max} > Su_{fdu} > q_{min} \geq 0 \ , 1 \ , 0\right)$ $Case_1 = 0$

$Case_2 := \text{if}\left(Su_{fdu} > q_{max} > q_{min} \geq 0 \ , 1 \ , 0\right)$ $Case_2 = 1$

$Case_3 := \text{if}\left(q_{max} > q_{min} > Su_{fdu} \ , 1 \ , 0\right)$ $Case_3 = 0$

$Case_4 := \text{if}\left(q_{min} < 0 \ , \text{if}\left(Su_{fdu} < q_{max} \ , 1 \ , 0\right) \ , 0\right)$ $Case_4 = 0$

$Case_5 := \text{if}\left(q_{min} < 0 \ , \text{if}\left(Su_{fdu} > q_{max} \ , 1 \ , 0\right) \ , 0\right)$ $Case_5 = 0$

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Unit Shear Resistance for Case 1:

$S_1 := Su_{fdu} - q_{min} = 9511.181 \ \frac{lbf}{ft^2}$ $S_2 := q_{min} = 188.819 \ \frac{lbf}{ft^2}$

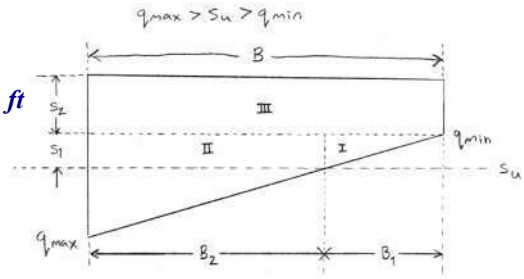
$B_1 := \frac{B \cdot (Su_{fdu} - q_{min})}{q_{max} - q_{min}} = 72.002 \ ft$ $B_2 := \frac{B \cdot (q_{max} - Su_{fdu})}{q_{max} - q_{min}} = -65.752 \ ft$

$B_3 := B = 6.25 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 342413.986 \ \frac{lbf}{ft}$ $II := S_1 \cdot B_2 = -625383.091 \ \frac{lbf}{ft}$

$III := S_2 \cdot B_3 = 1180.12 \ \frac{lbf}{ft}$

$R_{\tau_case1} := I + II + III = -281788.986 \ \frac{lbf}{ft}$



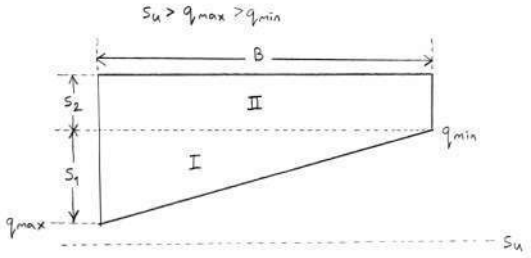
Unit Shear Resistance for Case 2:

$S_1 := q_{max} - q_{min} = 825.596 \ \frac{lbf}{ft^2}$ $S_2 := q_{min} = 188.819 \ \frac{lbf}{ft^2}$

$B = 6.25 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B = 2579.986 \ \frac{lbf}{ft}$ $II := S_2 \cdot B = 1180.12 \ \frac{lbf}{ft}$

$R_{\tau_case2} := I + II = 3760.107 \ \frac{lbf}{ft}$



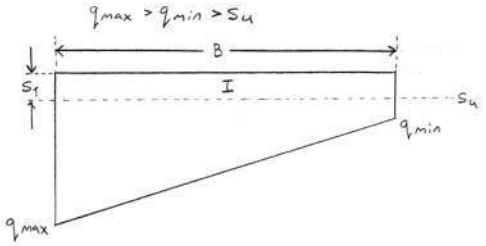
Unit Shear Resistance for Case 3:

$S_1 := Su_{fdu} = 9700 \ \frac{lbf}{ft^2}$

$B = 6.25 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B = 30312.5 \ \frac{lbf}{ft}$

$R_{\tau_case3} := I = 30312.5 \ \frac{lbf}{ft}$



Unit Shear Resistance for Case 4:

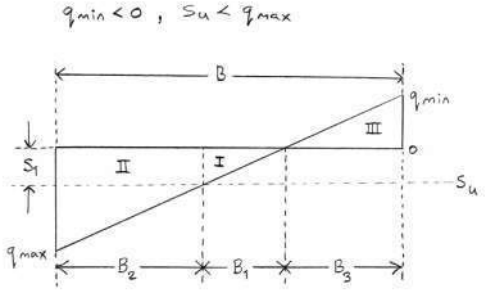
$S_1 := Su_{fdu} = 9700 \ \frac{lbf}{ft^2}$

$B_3 := \frac{B \cdot (-q_{min})}{q_{max} - q_{min}} = -1.429 \ ft$ $B_1 := \left(\frac{Su_{fdu}}{q_{max}}\right) \cdot (B - B_3) = 73.432 \ ft$

$B_2 := B - (B_1 + B_3) = -65.752 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 356144.376 \ \frac{lbf}{ft}$ $II := S_1 \cdot B_2 = -637798.411 \ \frac{lbf}{ft}$

$R_{\tau_case4} := I + II = -281654.035 \ \frac{lbf}{ft}$



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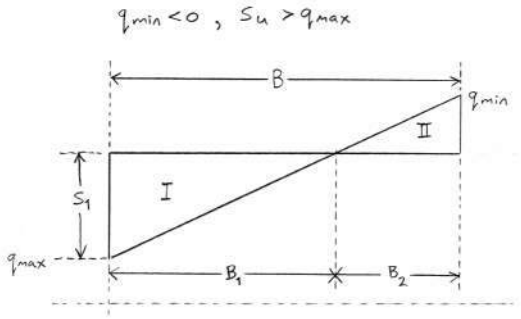
Unit Shear Resistance for Case 5:

$S_1 := q_{max} = 1014.415 \ \frac{lbf}{ft^2}$

$B_1 := \frac{B \cdot q_{max}}{q_{max} - q_{min}} = 7.679 \ ft$ $\quad B_2 := B - B_1 = -1.429 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 3895.057 \ \frac{lbf}{ft}$

$R_{\tau_case5} := I = 3895.057 \ \frac{lbf}{ft}$



Define the Applicable Case:

$R_\tau := R_{\tau_case2}$ $\quad R_\tau = 3760.107 \ \frac{lbf}{ft}$ Nominal sliding resistance Under Undrained Condition

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]**:

$\phi_{ep} := 0.5$ Resistance factor for passive resistance specified in **LRFD Table 10.5.5.2.2-1**

$\phi_\tau := 1.0$ Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

$\phi R_n := \phi_\tau \cdot R_\tau + \phi_{ep} \cdot R_{ep}$ $\quad R_R := \phi R_n$

Factored Sliding Resistance to be used in CDR Calculations: $R_R = 26942.865 \ \frac{lbf}{ft}$

Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{R_u}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 6.96$

Objective: **To evaluate the external stability of CIP wall design with broken backsloping backfill.**
Method: **In accordance with ODOT Bridge Design Manual [Sect. 305.2], ODOT Geotechnical Design Manual [Section 1500] and LRFD Bridge Design Specifications, 9th Ed., 2020, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].**

Givens:

Backfill Soil Design Parameters:

$\phi'_f := 30\ deg$ — Effective angle of internal friction

$\gamma_f := 120\ \frac{lbf}{ft^3}$ — Unit weight

$c'_f := 0\ \frac{lbf}{ft^2}$ — Effective Cohesion

$\delta := 0.67 \cdot \phi'_f$ $\quad \delta = 20.1\ deg$ — 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):

$\phi'_{fd} := 29\ deg$ — Effective angle of internal friction

$\gamma_{fd} := 135\ \frac{lbf}{ft^3}$ — Unit weight

$c'_{fd} := 400\ \frac{lbf}{ft^2}$ — Effective Cohesion

$\delta_{fd} := 0.67 \cdot \phi'_{fd}$ $\quad \delta_{fd} = 19.4\ deg$ — Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

Undrained Conditions (Total Stress):

$\phi_{fdu} := 0\ deg$ — Angle of internal friction (Same as Drained Conditions if granular soils)

$\gamma_{fd} = 135\ \frac{lbf}{ft^3}$ — Unit weight

$Su_{fdu} := 6400\ \frac{lbf}{ft^2}$ — Undrained Shear Strength

$\delta_{fdu} := 0.67 \cdot \phi_{fdu}$ $\quad \delta_{fdu} = 0\ deg$ — Friction angle between foundation soils and footing taken as specified in **LRFD BDS C3.11.5.3 (degrees)**

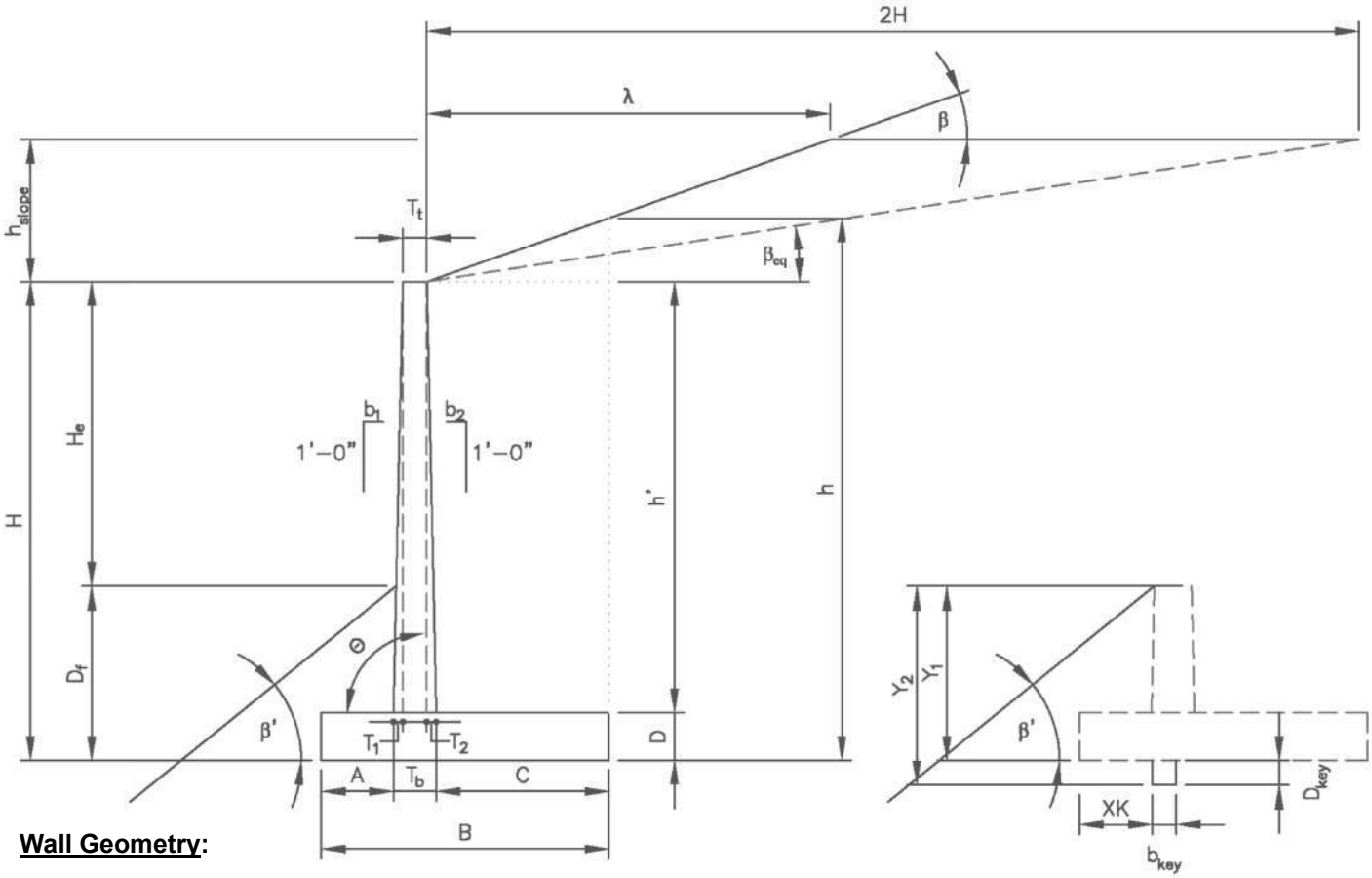
Foundation Surcharge Soil Parameters:

$\gamma_q := 120\ \frac{lbf}{ft^3}$ — Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Other Parameters:

$\gamma_c := 150\ \frac{lbf}{ft^3}$ — Concrete Unit weight

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Wall Geometry:

$H_e := 8.5\ ft$ — Exposed wall height

$D_f := 2.5\ ft$ — 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**

$H := H_e + D_f$ $H = 11\ ft$ — Design Wall Height

$T_t := 12\ in$ — Stem thickness at top of wall

$b_1 := 0 \cdot \left(\frac{in}{ft}\right)$ — Frontwall batter, (b1H:12V)

$b_2 := 0 \cdot \left(\frac{in}{ft}\right)$ — Backwall batter, (b2H:12V)

Preliminary Wall Dimensioning:

$B := 6.25\ ft$ $\frac{2}{5} \cdot H = 4.4\ ft$ to $\frac{3}{5} \cdot H = 6.6\ ft$ — Footing base width (2/5H to 3/5H)

$A := 2.25\ ft$ $\frac{H}{8} = 1.38\ ft$ to $\frac{H}{5} = 2.2\ ft$ — Toe projection (H/8 to H/5)

$D := 1.5\ ft$ $\frac{H}{8} = 1.38\ ft$ to $\frac{H}{5} = 2.2\ ft$ — Footing thickness (H/8 to H/5)

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Shear Key Dimensioning:

$D_{key} := 2.5\ ft$ — Depth of shear key from bottom of footing
Note: Footings on rock typically require shear key

$b_{key} := 1\ ft$ — Width of shear key

$XK := 0\ ft$ — Distance from toe to shear key

Other Wall Dimensions:

$h' := H - D$ $\quad h' = 9.5\ ft$ — Stem height

$T_1 := b_1 \cdot h'$ $\quad T_1 = 0\ ft$ — Stem front batter width

$T_2 := b_2 \cdot h'$ $\quad T_2 = 0\ ft$ — Stem back batter width

$T_b := T_1 + T_2 + T_t$ $\quad T_b = 1\ ft$ — Stem thickness at bottom of wall

$C := B - A - T_b$ $\quad C = 3\ ft$ — Heel projection

$\theta := 90\ deg$ — Angle of back face of wall to horizontal = *atan(12/b2)*

$b := 12\ in$ $\quad b = 1\ ft$ — Concrete strip width (for design)

$y_1 := D_f$ $\quad y_1 = 2.5\ ft$ — Depth to where passive pressure may begin to be utilized in front of wall.

$y_2 := D_f + D_{key}$ $\quad y_2 = 5\ ft$ — Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

Site Grading and Slope Dimensions:

$\beta := 14.036\ deg$ — Inclination of ground slope behind face of wall. Horizontal backfill behind CIP wall, β = 0 deg

Inclination of ground slope:

- Horizontal: **0**
- 4H:1V: **14.036**
- 3H:1V: **18.435**
- 2H:1V: **26.565**
- 1.5H:1V: **33.690**

$\beta' := 0\ deg$ — Inclination of ground slope in front of wall. If it is horizontal backfill in front of CIP wall, β' = 0 deg. A negative angle (-) indicates grades slope up from front of wall. Positive angle (+) indicates grade slope down from wall as shown in above figure.

$\lambda := 15\ ft$ — Horizontal distance from the back of the wall to point of slope crest .

$L_{Traffic} := 10\ ft$ — Horizontal distance from assumed traffic surcharge load to Backface of Wall.

$2 \cdot H = 22\ ft$ **IF λ IS GREATER THAN 2*H - USE INFINITE SLOPE CALCULATION SHEET**

$h_{slope} := \lambda \cdot \tan(\beta)$ $\quad h_{slope} = 3.75\ ft$ — Height of broken slope behind wall

$\beta_{eq} := \operatorname{atan}\left(\frac{h_{slope}}{2 \cdot H}\right) = 9.673\ deg$ — Equivalent backslope angle

$h := \text{if}\left(\lambda \le T_2 + C,\ H + h_{slope},\ H + \left(T_2 + C\right) \cdot \tan(\beta)\right) = 11.75\ ft$ — Height of retained fill at back of heel

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Live Load Surcharge Parameters:

$$SUR := \text{if}\left(L_{Traffic} < \frac{H}{2}, 250\ \frac{lbf}{ft^2}, 0\ \frac{lbf}{ft^2}\right) = 0\ \frac{lbf}{ft^2}$$

Live load surcharge (per LRFD BDS [3.11.6.4])
Note: ,A live load ssurcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2 .

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{\left(\sin\left(\phi'_f + \delta\right) \cdot \sin\left(\phi'_f - \beta_{eq}\right)\right)}{\left(\sin\left(\theta - \delta\right) \cdot \sin\left(\theta + \beta_{eq}\right)\right)}}\right)^2 \quad \Gamma = 2.361$$

$$k_{af} := \left(\frac{\left(\sin\left(\theta + \phi'_f\right)\right)^2}{\left(\Gamma \cdot \left(\sin\left(\theta\right)\right)^2 \cdot \sin\left(\theta - \delta\right)\right)}\right) \quad k_{af} = 0.338$$

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 θ = 90 degrees

$$\frac{-\beta'}{\phi'_{fd}} = 0 \qquad \frac{-\delta_{fd}}{\phi'_{fd}} = -0.67$$

$$k'_p := 6$$

Passive Earth Pressure Coefficient from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$$R_d := 0.811$$

Reduction Factor

$$k_{pd} := R_d \cdot k'_p \qquad k_{pd} = 4.866$$

Passive Earth Pressure Coefficient for Drained Conditions

Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

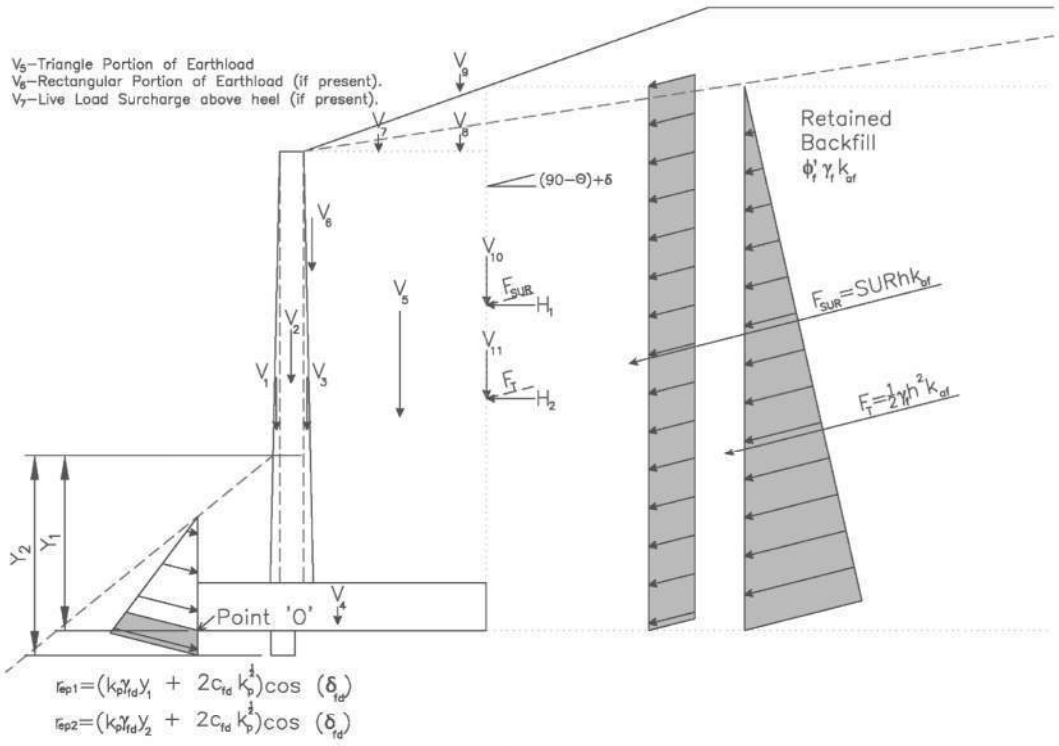
Undrained Conditions:

$$k_{pu} := \text{if}\left(\phi_{fdu} > 0, k_{pu}, 1\right) \qquad k_{pu} = 1$$

Passive Earth Pressure Coefficient for Resistance Undrained Conditions

Created

Compute Unfactored Loads LRFD [Tables 3.4.1-1 and 3.4.1-2]:



$F_T := \frac{1}{2} \cdot \gamma_f \cdot h^2 \cdot k_{af}$ $F_T = 2802.16 \ \frac{lbf}{ft}$ Active Earth Force Resultant (EH)

$F_{SUR} := SUR \cdot h \cdot k_{af}$ $F_{SUR} = 0 \ \frac{lbf}{ft}$ Live Load Surcharge (LS)

Vertical Loads:

$V_1 := \frac{1}{2} \cdot T_1 \cdot h' \cdot \gamma_c$ $V_1 = 0 \ \frac{lbf}{ft}$ Wall stem front batter (DC)

$V_2 := T_t \cdot h' \cdot \gamma_c$ $V_2 = 1425 \ \frac{lbf}{ft}$ Wall stem (DC)

$V_3 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_c$ $V_3 = 0 \ \frac{lbf}{ft}$ Wall stem back batter (DC)

$V_4 := D \cdot B \cdot \gamma_c$ $V_4 = 1406.25 \ \frac{lbf}{ft}$ Wall Footing (DC)

$V_5 := C \cdot h' \cdot \gamma_f$ $V_5 = 3420 \ \frac{lbf}{ft}$ Soil Backfill - Heel (EV)

$V_6 := \frac{1}{2} \cdot T_2 \cdot h' \cdot \gamma_f$ $V_6 = 0 \ \frac{lbf}{ft}$ Soil Backfill - Batter (EV)

$V_7 := \text{if}\left(\lambda \le T_2 + C, \frac{1}{2} \cdot \lambda^2 \cdot \tan(\beta) \cdot \gamma_f, \frac{1}{2} \cdot (T_2 + C)^2 \cdot \tan(\beta) \cdot \gamma_f\right)$

$V_7 = 134.998 \ \frac{lbf}{ft}$ Soil Backfill - Backslope Triangle Portion (EV)

$V_8 := \text{if}\left(\lambda \le T_2 + C, (T_2 + C - \lambda) \cdot h_{slope} \cdot \gamma_f, 0 \cdot \frac{lbf}{ft}\right)$

$V_8 = 0 \ \frac{lbf}{ft}$ Soil Backfill - Backslope Rectangular Portion (EV)

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$V_9 := \text{if}\left(\lambda \le T_2 + C,\ (T_2 + C - \lambda) \cdot SUR,\ 0 \cdot \frac{lbf}{ft}\right)$

$V_9 = 0\ \frac{lbf}{ft}$ Live Load Surcharge Above Heel - (LS)

$V_{10} := F_{SUR} \cdot \sin(90 \cdot deg - \theta + \delta)$ $V_{10} = 0\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (vertical comp. - LS)

$V_{11} := F_T \cdot \sin(90 \cdot deg - \theta + \delta)$ $V_{11} = 962.989\ \frac{lbf}{ft}$ Active earth force resultant (vertical component - EH)

Moment Arm: Moment:

Moments produced from vertical loads about Point 'O'

$d_{v1} := A + \frac{2}{3} \cdot T_1 = 2.25\ ft$ $MV_1 := V_1 \cdot d_{v1} = 0\ lbf$

$d_{v2} := A + T_1 + \frac{T_t}{2} = 2.75\ ft$ $MV_2 := V_2 \cdot d_{v2} = 3918.75\ lbf$

$d_{v3} := A + T_1 + T_t + \frac{T_2}{3} = 3.25\ ft$ $MV_3 := V_3 \cdot d_{v3} = 0\ lbf$

$d_{v4} := \frac{B}{2} = 3.125\ ft$ $MV_4 := V_4 \cdot d_{v4} = 4394.531\ lbf$

$d_{v5} := B - \frac{C}{2} = 4.75\ ft$ $MV_5 := V_5 \cdot d_{v5} = 16245\ lbf$

$d_{v6} := A + T_1 + T_t + \frac{2\ T_2}{3} = 3.25\ ft$ $MV_6 := V_6 \cdot d_{v6} = 0\ lbf$

$d_{v7} := \text{if}\left(\lambda \le T_2 + C,\ A + T_1 + T_t + \left(\frac{2}{3}\ (\lambda)\right),\ A + T_1 + T_t + \left(\frac{2}{3}\ (T_2 + C)\right)\right) = 5.25\ ft$ $MV_7 := V_7 \cdot d_{v7} = 708.737\ lbf$

$d_{v8} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2}\right) = 12.25\ ft$ $MV_8 := V_8 \cdot d_{v8} = 0\ lbf$

$d_{v9} := A + T_1 + T_t + \lambda + \left(\frac{T_2 + C - \lambda}{2}\right) = 12.25\ ft$ $MV_9 := V_9 \cdot d_{v9} = 0\ lbf$

$d_{v10} := B = 6.25\ ft$ $MV_{10} := V_{10} \cdot d_{v10} = 0\ lbf$

$d_{v11} := B = 6.25\ ft$ $MV_{11} := V_{11} \cdot d_{v11} = 6018.684\ lbf$

Horizontal Loads:

$H_1 := F_{SUR} \cdot \cos(90 \cdot deg - \theta + \delta)$ $H_1 = 0\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (horizontal comp. - LS)

$H_2 := F_T \cdot \cos(90 \cdot deg - \theta + \delta)$ $H_2 = 2631.492\ \frac{lbf}{ft}$ Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm: Moment:

$d_{h1} := \frac{h}{2}$ $d_{h1} = 5.875\ ft$ $MH_1 := H_1 \cdot d_{h1}$ $MH_1 = 0\ \frac{lbf \cdot ft}{ft}$

$d_{h2} := \frac{h}{3}$ $d_{h2} = 3.917\ ft$ $MH_2 := H_2 \cdot d_{h2}$ $MH_2 = 10306.667\ \frac{lbf \cdot ft}{ft}$

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Unfactored Loads by Load Type:

$V_{DC} := V_1 + V_2 + V_3 + V_4$ $V_{DC} = 2831.25 \frac{lbf}{ft}$ $V_{EV} := V_5 + V_6 + V_7 + V_8$ $V_{EV} = 3554.998 \frac{lbf}{ft}$

$V_{LS_Ia} := V_{10}$ $V_{LS_Ia} = 0 \frac{lbf}{ft}$ $V_{LS_Ib} := V_9 + V_{10}$ $V_{LS_Ia} = 0 \frac{lbf}{ft}$

$V_{EH} := V_{11}$ $V_{EH} = 962.989 \frac{lbf}{ft}$

$H_{EH} := H_2$ $H_{EH} = 2631.492 \frac{lbf}{ft}$ $H_{LS} := H_1$ $H_{LS} = 0 \frac{lbf}{ft}$

Unfactored Moments by Load Type

$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4$ $M_{DC} = 8313.281 \frac{lbf \cdot ft}{ft}$

$M_{EV} := MV_5 + MV_6 + MV_7 + MV_8$ $M_{EV} = 16953.737 \frac{lbf \cdot ft}{ft}$

$M_{LSV_Ia} := MV_{10}$ $M_{LSV_Ia} = 0 \frac{lbf \cdot ft}{ft}$

$M_{LSV_Ib} := MV_9 + MV_{10}$ $M_{LSV_Ib} = 0 \frac{lbf \cdot ft}{ft}$

$M_{EH1} := MV_{11}$ $M_{EH1} = 6018.684 \frac{lbf \cdot ft}{ft}$

$M_{LSH} := MH_1$ $M_{LSH} = 0 \frac{lbf \cdot ft}{ft}$

$M_{EH2} := MH_2$ $M_{EH2} = 10306.667 \frac{lbf \cdot ft}{ft}$

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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:

$V_{Ia} := \eta \cdot \left(\left(Ia_{DC} \cdot V_{DC} \right) + \left(Ia_{EV} \cdot V_{EV} \right) + \left(Ia_{EH} \cdot V_{EH} \right) + \left(Ia_{LS} \cdot V_{LS_Ia} \right) \right)$ $V_{Ia} = 7547.607 \ \frac{lbf}{ft}$

$V_{Ib} := \eta \cdot \left(\left(Ib_{DC} \cdot V_{DC} \right) + \left(Ib_{EV} \cdot V_{EV} \right) + \left(Ib_{EH} \cdot V_{EH} \right) + \left(Ib_{LS} \cdot V_{LS_Ib} \right) \right)$ $V_{Ib} = 9782.793 \ \frac{lbf}{ft}$

Factored Horizontal Loads by Limit State:

$H_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot H_{LS} \right) + \left(Ia_{EH} \cdot H_{EH} \right) \right)$ $H_{Ia} = 3947.239 \ \frac{lbf}{ft}$

$H_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot H_{LS} \right) + \left(Ib_{EH} \cdot H_{EH} \right) \right)$ $H_{Ib} = 3947.239 \ \frac{lbf}{ft}$

Factored Moments Produced by Vertical Loads by Limit State:

$MV_{Ia} := \eta \cdot \left(\left(Ia_{DC} \cdot M_{DC} \right) + \left(Ia_{EV} \cdot M_{EV} \right) + \left(Ia_{EH} \cdot M_{EH1} \right) + \left(Ia_{LS} \cdot M_{LSV_Ia} \right) \right)$ $MV_{Ia} = 33463.717 \ \frac{lbf \cdot ft}{ft}$

$MV_{Ib} := \eta \cdot \left(\left(Ib_{DC} \cdot M_{DC} \right) + \left(Ib_{EV} \cdot M_{EV} \right) + \left(Ib_{EH} \cdot M_{EH1} \right) + \left(Ib_{LS} \cdot M_{LSV_Ib} \right) \right)$ $MV_{Ib} = 42307.173 \ \frac{lbf \cdot ft}{ft}$

Factored Moments Produced by Horizontal Loads by Limit State:

$MH_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot M_{LSH} \right) + \left(Ia_{EH} \cdot M_{EH2} \right) \right)$ $MH_{Ia} = 15460 \ \frac{lbf \cdot ft}{ft}$

$MH_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot M_{LSH} \right) + \left(Ib_{EH} \cdot M_{EH2} \right) \right)$ $MH_{Ib} = 15460 \ \frac{lbf \cdot ft}{ft}$

Compute Bearing Resistance:

Compute the resultant location about the toe of the base length (distance from "O") Strength Ib:

$\Sigma M_R := MV_{Ib}$ $\Sigma M_R = 42307.173 \frac{lbf \cdot ft}{ft}$ Sum of Resisting Moments (Strength Ib)

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 15460 \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 9782.793 \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$ $x = 2.744\ ft$ Distance from Point "O" the resultant intersects the base

$e := \left| \frac{B}{2} - x \right|$ $e = 0.38\ ft$ Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e.

Foundation Layout:

$B' := B - 2 \cdot e$ $B' = 5.489\ ft$ Effective Footing Width

$L' := 41\ ft$ Effective Footing Length (Assumed)

$H' := H_{Ib}$ $H' = 3947.239 \frac{lbf}{ft}$ Summation of Horizontal Loads (Strength Ib)

$V' := V_{Ib}$ $V' = 9782.793 \frac{lbf}{ft}$ Summation of Vertical Loads (Strength Ib)

$D_f = 2.5\ ft$ Footing embedment

$d_w := 0\ ft$ Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan\left(45\ deg + \frac{\phi'_{fd}}{2}\right)^2, 1.0\right)$ $N_q = 16.44$

$N_c := \text{if}\left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14\right)$ $N_c = 27.86$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$ $N_\gamma = 19.338$

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

$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L'}\right)\right)$ $s_c = 1.079$

$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd})\right), 1\right)$ $s_q = 1.074$

$s_\gamma := \text{if}\left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'}\right), 1\right)$ $s_\gamma = 0.946$

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Load inclination factors:

$i_q := 1$

$i_\gamma := 1$

$i_c := 1$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**. "Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

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

$C_{wq} := \text{if}\left(d_w \geq D_f, 1.0, 0.5\right)$ $C_{wq} = 0.5$

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

Depth Correction Factor per Hanson (1970):

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

$d_q = 1.13$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $N_{cm} = 30.062$

$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q$ $N_{qm} = 19.885$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $N_{\gamma m} = 18.302$

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

$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $q_{nd} = 18397.813 \ \frac{lbf}{ft^2}$

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

$\phi_b := .55$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Rd} := \phi_b \cdot q_{nd}$ $q_{Rd} = 10.119 \ ksf$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

$N_\gamma := 2 \cdot \left(N_q + 1\right) \cdot \tan\left(\phi_{fdu}\right)$ $N_\gamma = 0$

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

$s_c := \text{if}\left(\phi_{fdu} > 0\,,\, 1 + \left(\frac{B'}{L'}\right)\cdot\left(\frac{N_q}{N_c}\right)\,,\, 1 + \left(\frac{B'}{5\cdot L'}\right)\right)$ $\quad s_c = 1.027$

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

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

Load inclination factors:

$i_q := 1$

$i_\gamma := 1$

$i_c := 1$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**. "Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

Depth Correction Factor per Hanson (1970):

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

$d_q = 1.13$

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

$N_{cm} := N_c\cdot s_c\cdot i_c$ $\quad N_{cm} = 5.278$

$N_{qm} := N_q\cdot s_q\cdot d_q\cdot i_q$ $\quad N_{qm} = 1.126$

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

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

$q_{nu} := Su_{fdu}\cdot N_{cm} + \gamma_q\cdot D_f\cdot N_{qm}\cdot C_{wq} + 0.5\cdot\gamma_{fd}\cdot B'\cdot N_{\gamma m}\cdot C_{w\gamma}$ $\quad q_{nu} = 33945.619\ \frac{lbf}{ft^2}$

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

$\phi_b := .55$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Ru} := \phi_b\cdot q_{nu}$ $\quad q_{Ru} = 18.67\ ksf$ Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 10.119\ ksf$

Undrained Conditions: $q_{Ru} = 18.67\ ksf$

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Evaluate External Stability of Wall:

Compute the factored bearing stress :

$e = 0.38\ ft$

$\sigma_V := \frac{\Sigma V}{B - 2 \cdot e}$ $\sigma_V = 1.782\ ksf$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{Bearing_D} := \frac{q_{Rd}}{\sigma_V}$ Is the CDR > or = to 1.0? $CDR_{Bearing_D} = 5.68$

Undrained Conditions: $CDR_{Bearing_U} := \frac{q_{Ru}}{\sigma_V}$ Is the CDR > or = to 1.0? $CDR_{Bearing_U} = 10.47$

Limiting Eccentricity at Base of Wall (Strength Ia):

Compute the resultant location about the toe "O" of the base length (distance from Pivot):

$e_{max} := \frac{B}{3}$ $e_{max} = 2.083\ ft$ Maximum Eccentricity **LRFD [C11.6.3.3.] Equals B/3 for soil.**

$\Sigma M_R := MV_{Ia}$ $\Sigma M_R = 33463.717\ \frac{lbf \cdot ft}{ft}$ Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$ $\Sigma M_O = 15460\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$ $\Sigma V = 7547.607\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$ $x = 2.385\ ft$ Distance from Point "O" the resultant intersects the base

$e := \left| \frac{B}{2} - x \right|$ $e = 0.74\ ft$ 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? $CDR_{Eccentricity} = 2.82$

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Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$ $\quad R_u = 3947.239\ \frac{lbf}{ft}$

Drained Conditions (Effective Stress):

Compute passive resistance throughout the design life of the wall **LRFD [Eq 3.11.5.4-1]:**:

$r_{ep1} := \left(k_{pd} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y1

$r_{ep2} := \left(k_{pd} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot c'_{fd} \cdot \sqrt{k_{pd}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y2

$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot \left(y_2 - y_1\right)$ $\quad R_{ep} = 9968.338\ \frac{lbf}{ft}$ 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:

$c := 0.8$

c = 1.0 for Cast-in-Place
c = 0.8 for Precast

$\Sigma V := V_{Ia}$ $\quad \Sigma V = 7547.607\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$R_\tau := c \cdot \Sigma V \cdot \tan\left(\phi'_{fd}\right)$ $\quad R_\tau = 3346.965\ \frac{lbf}{ft}$ Nominal sliding resistance Under Drained Condition

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]**:

$\phi_{ep} := 0.5$ Resistance factor for passive resistance specified in **LRFD Table 10.5.5.2.2-1**

$\phi_\tau := 1.0$ Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

$\phi R_n := \phi_\tau \cdot R_\tau + \phi_{ep} \cdot R_{ep}$ $\quad R_R := \phi R_n$

Factored Sliding Resistance to be used in CDR Calculations: $R_R = 8331.135\ \frac{lbf}{ft}$

Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{R_u}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 2.11$

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Undrained Conditions (Total Stress):

Compute passive resistance throughout the design life of the wall **LRFD [Eq 3.11.5.4-1]**::

$r_{ep1} := \left(k_{pu} \cdot \gamma_{fd} \cdot y_1 + 2 \cdot Su_{fdu} \cdot \sqrt{k_{pu}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y1

$r_{ep2} := \left(k_{pu} \cdot \gamma_{fd} \cdot y_2 + 2 \cdot Su_{fdu} \cdot \sqrt{k_{pu}}\right) \cdot \cos\left(\delta_{fd}\right)$ Nominal passive pressure at y2

$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot \left(y_2 - y_1\right)$ $R_{ep} = 31371.101 \ \frac{lbf}{ft}$ 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:

$c := 0.8$ c = 1.0 for Cast-in-Place
c = 0.8 for Precast

$\Sigma V := V_{Ia}$ $\Sigma V = 7547.607 \ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$e = 0.74 \ ft$ Wall eccentricity, Calculated in above Limiting Eccentricity at Base of Wall (Strength Ia) Section.

$B = 6.25 \ ft$ Footing base width

$\frac{B}{6} = 1.042 \ ft$ If e < B/6 the resultant is in the middle one-third

$\sigma_{vmax} := \frac{\Sigma V}{B} \cdot \left(1 + 6 \cdot \frac{e}{B}\right)$ $\sigma_{vmax} = 2065.097 \ \frac{lbf}{ft^2}$ Max vertical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$\sigma_{vmin} := \frac{\Sigma V}{B} \cdot \left(1 - 6 \cdot \frac{e}{B}\right)$ $\sigma_{vmin} = 350.137 \ \frac{lbf}{ft^2}$ Max verical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$q_{max} := \frac{1}{2} \cdot \sigma_{vmax}$ $q_{max} = 1032.549 \ \frac{lbf}{ft^2}$ Max unit shear resistance as 1/2 max vertical stress **LRFD [10.6.3.4]**.

$q_{min} := \frac{1}{2} \cdot \sigma_{vmin}$ $q_{min} = 175.068 \ \frac{lbf}{ft^2}$ Minimum unit shear resistance as 1/2 minimum vertical stress **LRFD [10.6.3.4]**.

Determine which Cohesive Soil Resistance Case is Present:

$Case_1 := \text{if}\left(q_{max} > Su_{fdu} > q_{min} \geq 0 \ , 1 \ , 0\right)$ $Case_1 = 0$

$Case_2 := \text{if}\left(Su_{fdu} > q_{max} > q_{min} \geq 0 \ , 1 \ , 0\right)$ $Case_2 = 1$

$Case_3 := \text{if}\left(q_{max} > q_{min} > Su_{fdu} \ , 1 \ , 0\right)$ $Case_3 = 0$

$Case_4 := \text{if}\left(q_{min} < 0 \ , \text{if}\left(Su_{fdu} < q_{max} \ , 1 \ , 0\right) \ , 0\right)$ $Case_4 = 0$

$Case_5 := \text{if}\left(q_{min} < 0 \ , \text{if}\left(Su_{fdu} > q_{max} \ , 1 \ , 0\right) \ , 0\right)$ $Case_5 = 0$

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Unit Shear Resistance for Case 1:

$S_1 := Su_{fdu} - q_{min} = 6224.932 \ \frac{lbf}{ft^2}$ $S_2 := q_{min} = 175.068 \ \frac{lbf}{ft^2}$

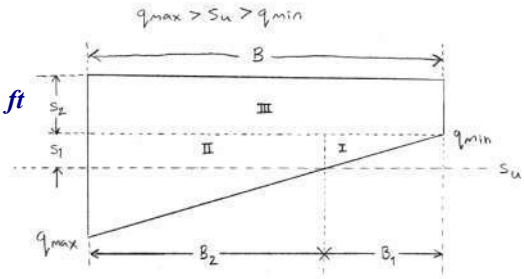
$B_1 := \frac{B \cdot (Su_{fdu} - q_{min})}{q_{max} - q_{min}} = 45.372 \ ft$ $B_2 := \frac{B \cdot (q_{max} - Su_{fdu})}{q_{max} - q_{min}} = -39.122 \ ft$

$B_3 := B = 6.25 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 141219.61 \ \frac{lbf}{ft}$ $II := S_1 \cdot B_2 = -243533.397 \ \frac{lbf}{ft}$

$III := S_2 \cdot B_3 = 1094.177 \ \frac{lbf}{ft}$

$R_{\tau_case1} := I + II + III = -101219.61 \ \frac{lbf}{ft}$



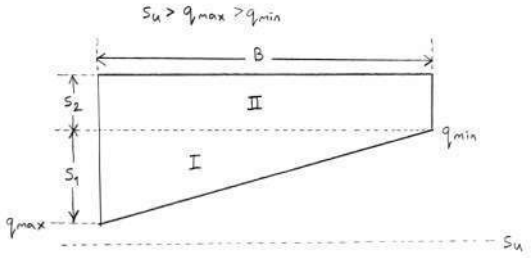
Unit Shear Resistance for Case 2:

$S_1 := q_{max} - q_{min} = 857.48 \ \frac{lbf}{ft^2}$ $S_2 := q_{min} = 175.068 \ \frac{lbf}{ft^2}$

$B = 6.25 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B = 2679.626 \ \frac{lbf}{ft}$ $II := S_2 \cdot B = 1094.177 \ \frac{lbf}{ft}$

$R_{\tau_case2} := I + II = 3773.803 \ \frac{lbf}{ft}$



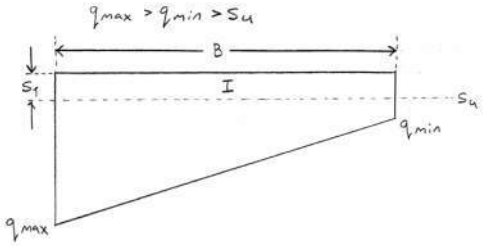
Unit Shear Resistance for Case 3:

$S_1 := Su_{fdu} = 6400 \ \frac{lbf}{ft^2}$

$B = 6.25 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B = 20000 \ \frac{lbf}{ft}$

$R_{\tau_case3} := I = 20000 \ \frac{lbf}{ft}$



Unit Shear Resistance for Case 4:

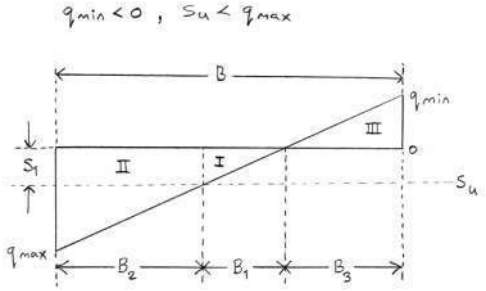
$S_1 := Su_{fdu} = 6400 \ \frac{lbf}{ft^2}$

$B_3 := \frac{B \cdot (-q_{min})}{q_{max} - q_{min}} = -1.276 \ ft$ $B_1 := \left(\frac{Su_{fdu}}{q_{max}}\right) \cdot (B - B_3) = 46.648 \ ft$

$B_2 := B - (B_1 + B_3) = -39.122 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 149274.554 \ \frac{lbf}{ft}$ $II := S_1 \cdot B_2 = -250382.467 \ \frac{lbf}{ft}$

$R_{\tau_case4} := I + II = -101107.913 \ \frac{lbf}{ft}$



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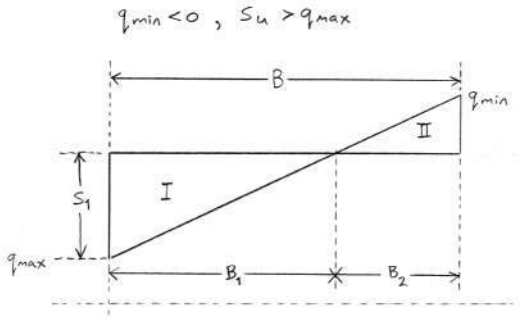
Unit Shear Resistance for Case 5:

$S_1 := q_{max} = 1032.549 \ \frac{lbf}{ft^2}$

$B_1 := \frac{B \cdot q_{max}}{q_{max} - q_{min}} = 7.526 \ ft$ $\quad B_2 := B - B_1 = -1.276 \ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 3885.5 \ \frac{lbf}{ft}$

$R_{\tau_case5} := I = 3885.5 \ \frac{lbf}{ft}$



Define the Applicable Case:

$R_\tau := R_{\tau_case2}$ $\quad R_\tau = 3773.803 \ \frac{lbf}{ft}$ Nominal sliding resistance Under Undrained Condition

Compute factored resistance against failure by sliding **LRFD [10.6.3.4]**:

$\phi_{ep} := 0.5$ Resistance factor for passive resistance specified in **LRFD Table 10.5.5.2.2-1**

$\phi_\tau := 1.0$ Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

$\phi R_n := \phi_\tau \cdot R_\tau + \phi_{ep} \cdot R_{ep}$ $\quad R_R := \phi R_n$

Factored Sliding Resistance to be used in CDR Calculations: $R_R = 19459.354 \ \frac{lbf}{ft}$

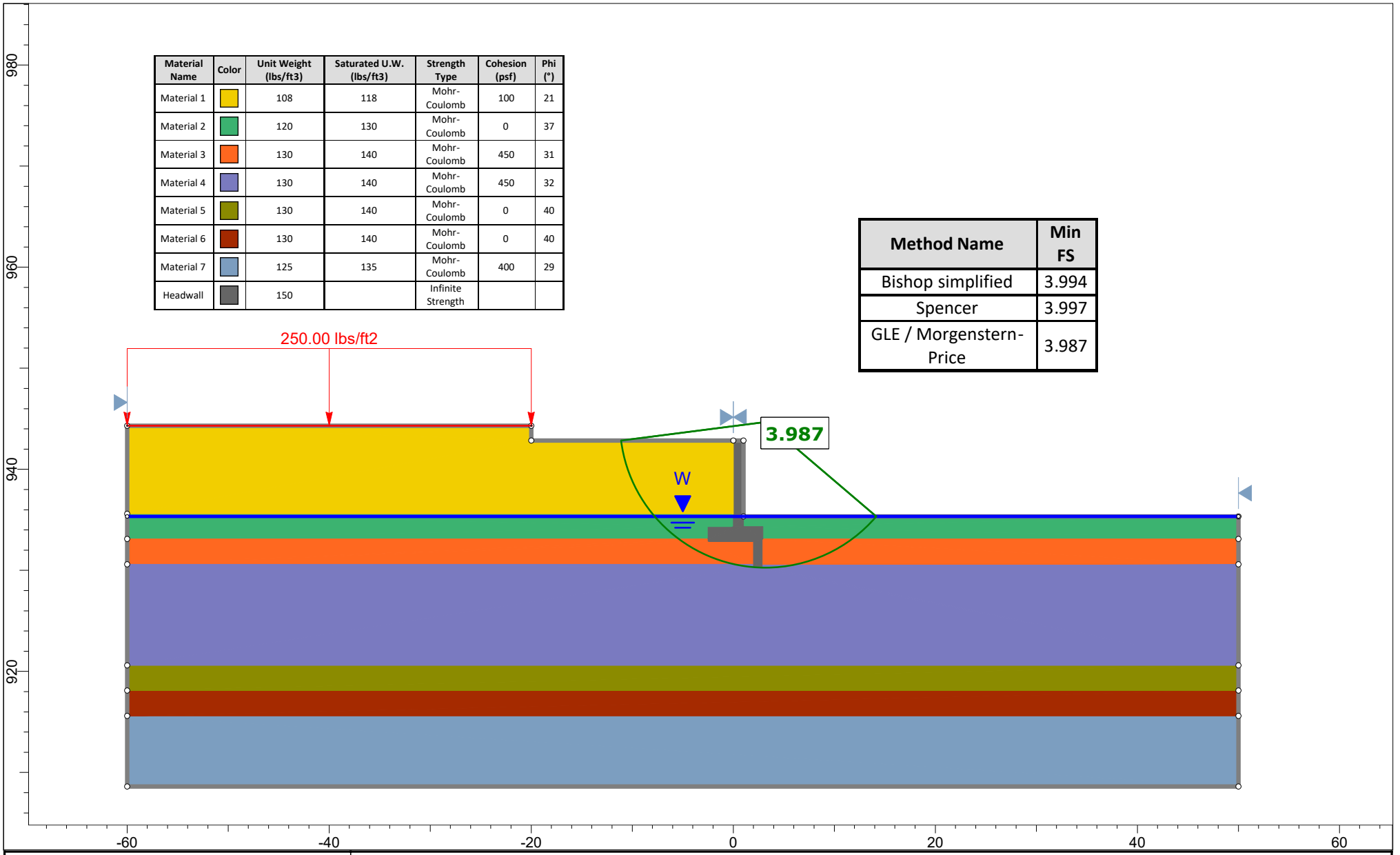
Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{R_u}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 4.93$

GLOBAL STABILITY

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		108	118	Mohr-Coulomb	100	21
Material 2		120	130	Mohr-Coulomb	0	37
Material 3		130	140	Mohr-Coulomb	450	31
Material 4		130	140	Mohr-Coulomb	450	32
Material 5		130	140	Mohr-Coulomb	0	40
Material 6		130	140	Mohr-Coulomb	0	40
Material 7		125	135	Mohr-Coulomb	400	29
Headwall		150		Infinite Strength		

Method Name	Min FS
Bishop simplified	3.994
Spencer	3.997
GLE / Morgenstern-Price	3.987



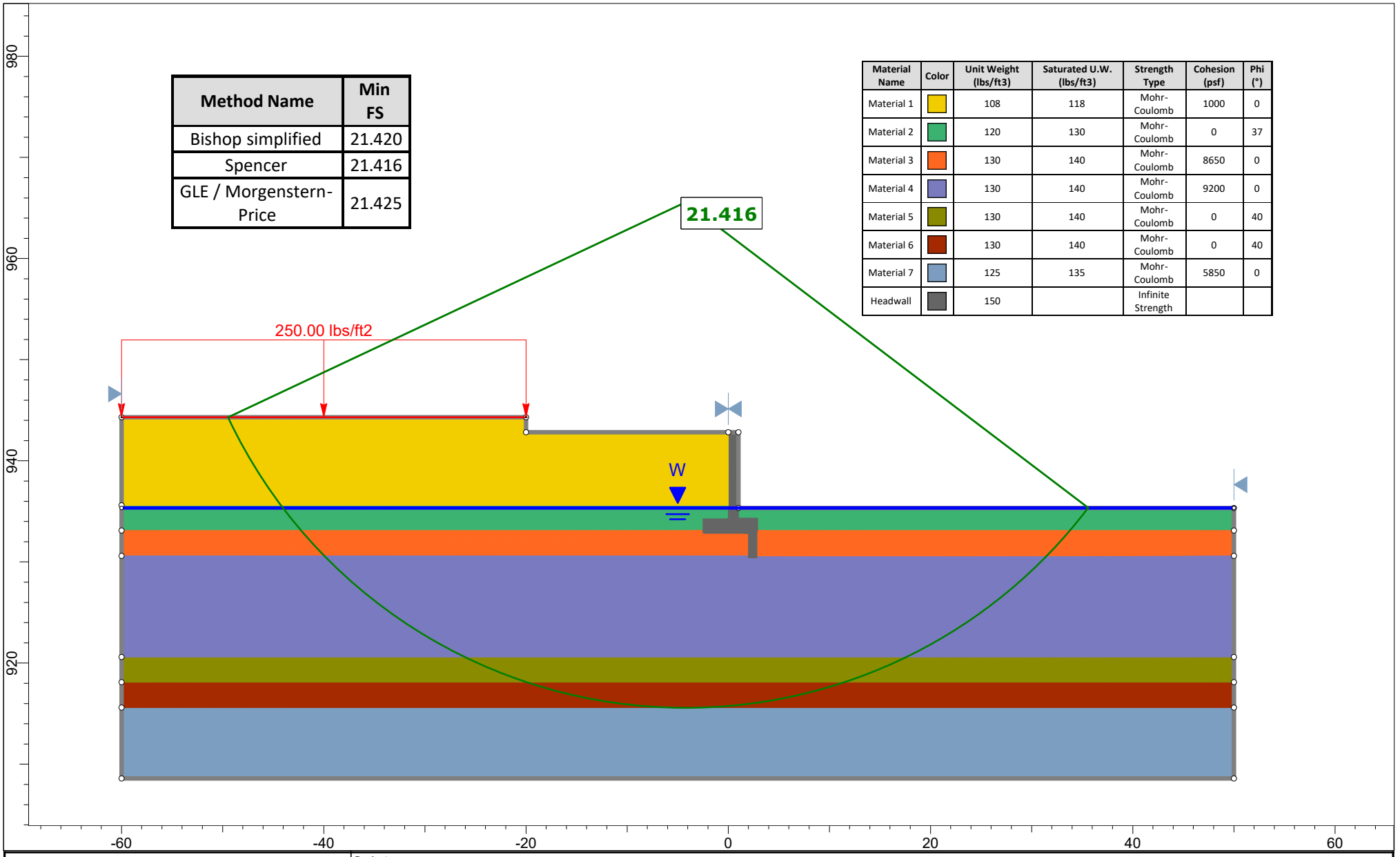
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SLIDEINTERPRET 9.036

Project	BRO-68-33.36		
Group	Group 1	Scenario	Master Scenario
Drawn By	ZM	Company	NEAS, inc.
Date	3/20/2025	File Name	BRO-68_Culvert Under US-68_Inlet_Effective_B-08.slmd

Method Name	Min FS
Bishop simplified	21.420
Spencer	21.416
GLE / Morgenstern-Price	21.425

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		108	118	Mohr-Coulomb	1000	0
Material 2		120	130	Mohr-Coulomb	0	37
Material 3		130	140	Mohr-Coulomb	8650	0
Material 4		130	140	Mohr-Coulomb	9200	0
Material 5		130	140	Mohr-Coulomb	0	40
Material 6		130	140	Mohr-Coulomb	0	40
Material 7		125	135	Mohr-Coulomb	5850	0
Headwall		150		Infinite Strength		



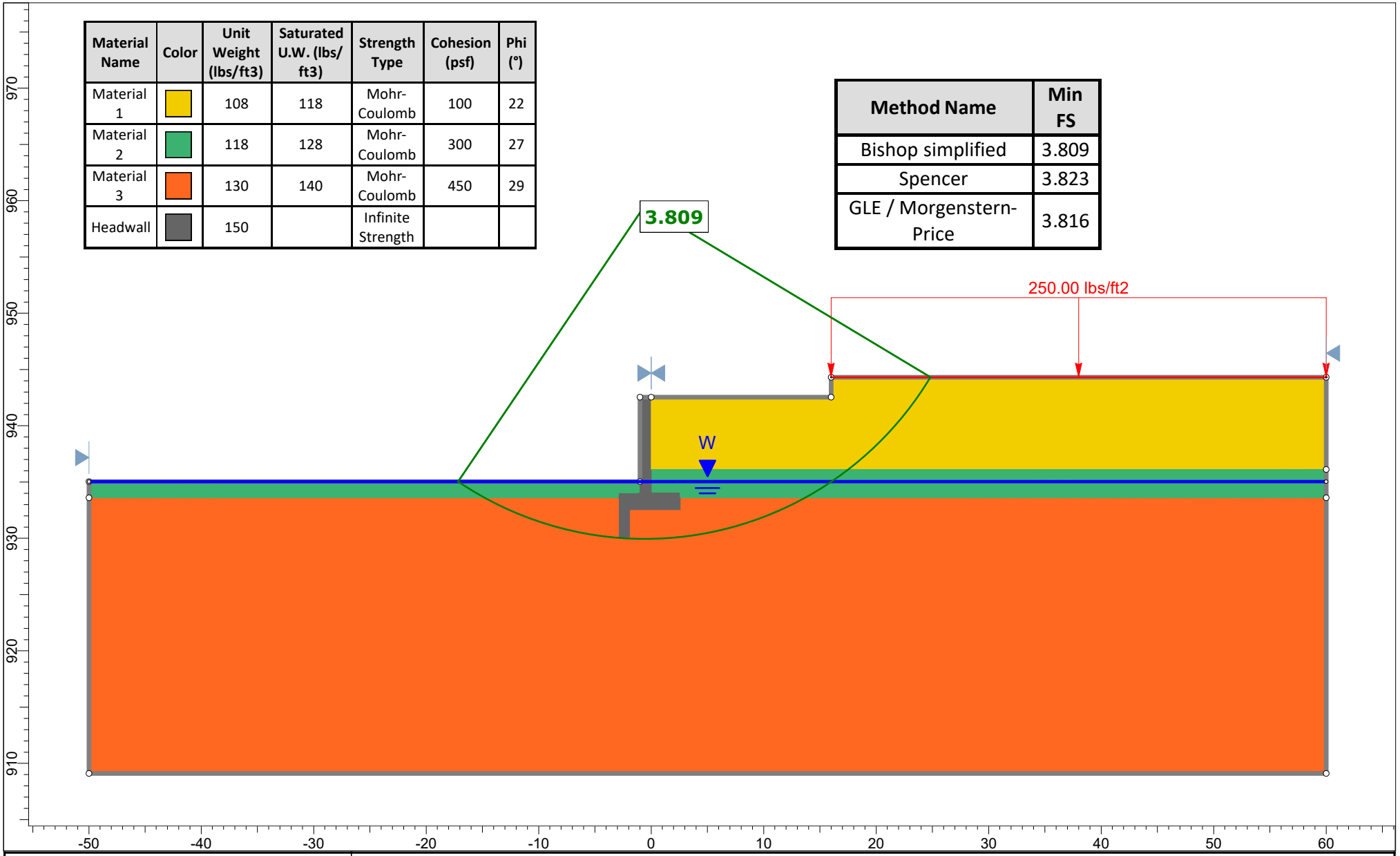
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SLIDEINTERPRET 9.036

Project	BRO-68-33.36		
Group	Group 1	Scenario	Master Scenario
Drawn By	ZM	Company	NEAS, inc.
Date	3/20/2025	File Name	BRO-68_Culvert Under US-68_Inlet_Total_B-08.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		108	118	Mohr-Coulomb	100	22
Material 2		118	128	Mohr-Coulomb	300	27
Material 3		130	140	Mohr-Coulomb	450	29
Headwall		150		Infinite Strength		

Method Name	Min FS
Bishop simplified	3.809
Spencer	3.823
GLE / Morgenstern-Price	3.816



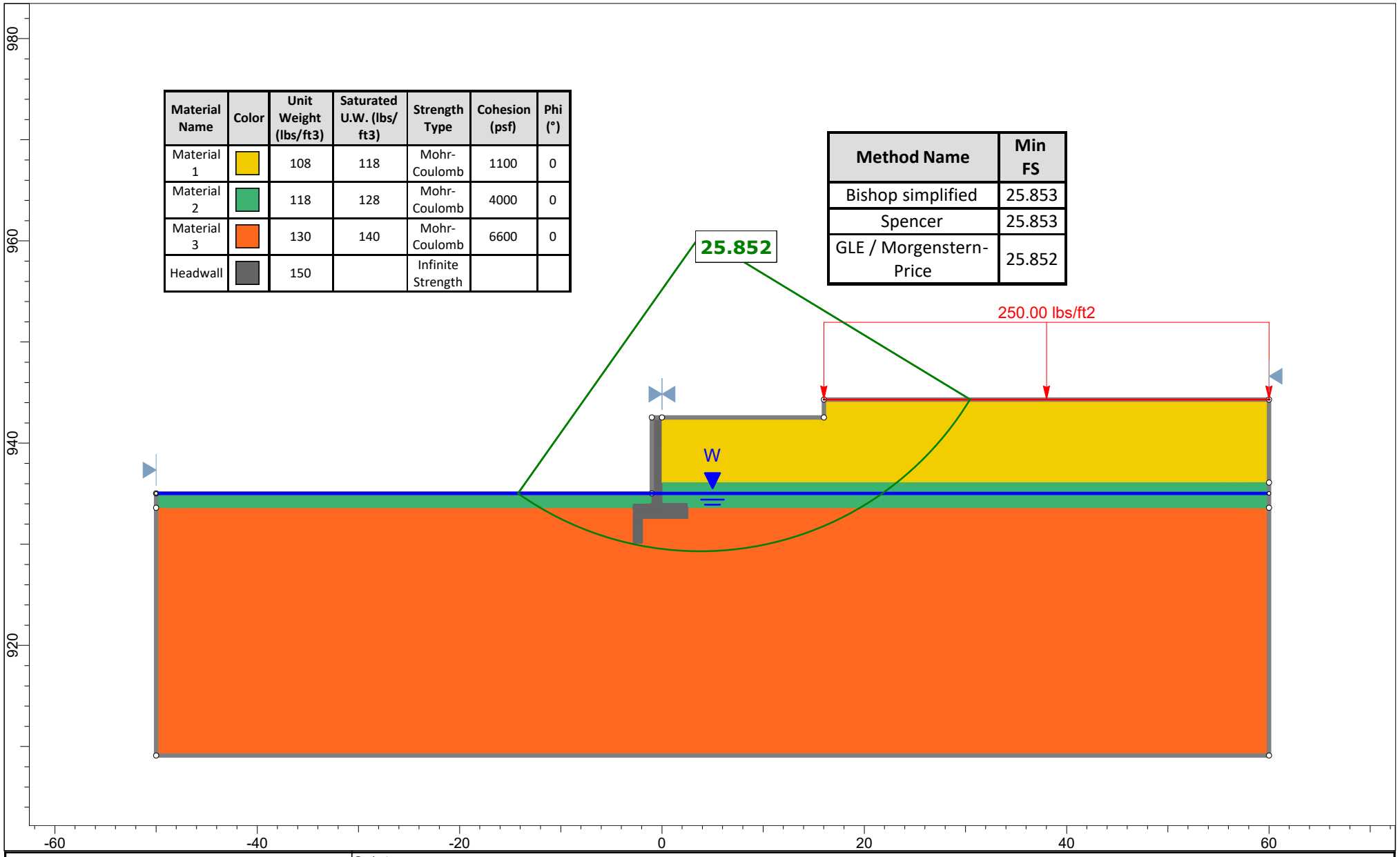
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SLIDEINTERPRET 9.036

Project	BRO-68-33.36		
Group	Group 1	Scenario	Master Scenario
Drawn By	ZM	Company	NEAS, inc.
Date	3/20/2025	File Name	BRO-68_Culvert Under US-68_Outlet_Effective_B-07.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		108	118	Mohr-Coulomb	1100	0
Material 2		118	128	Mohr-Coulomb	4000	0
Material 3		130	140	Mohr-Coulomb	6600	0
Headwall		150		Infinite Strength		

Method Name	Min FS
Bishop simplified	25.853
Spencer	25.853
GLE / Morgenstern-Price	25.852



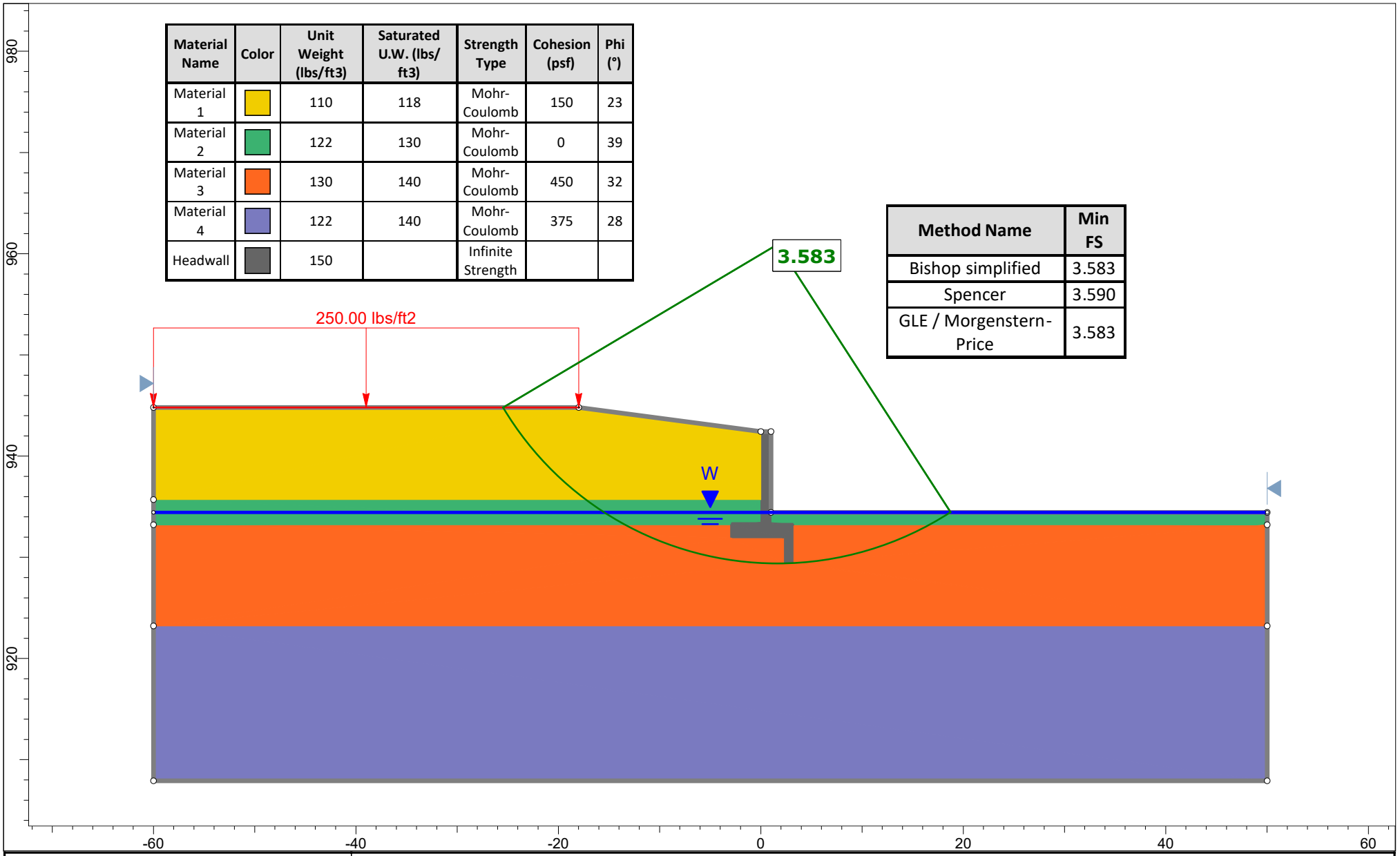
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SLIDEINTERPRET 9.036

Project	BRO-68-33.36		
Group	Group 1	Scenario	Master Scenario
Drawn By	ZM	Company	NEAS, inc.
Date	3/20/2025	File Name	BRO-68_Culvert Under US-68_Outlet_Total_B-07.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		110	118	Mohr-Coulomb	150	23
Material 2		122	130	Mohr-Coulomb	0	39
Material 3		130	140	Mohr-Coulomb	450	32
Material 4		122	140	Mohr-Coulomb	375	28
Headwall		150		Infinite Strength		

Method Name	Min FS
Bishop simplified	3.583
Spencer	3.590
GLE / Morgenstern-Price	3.583



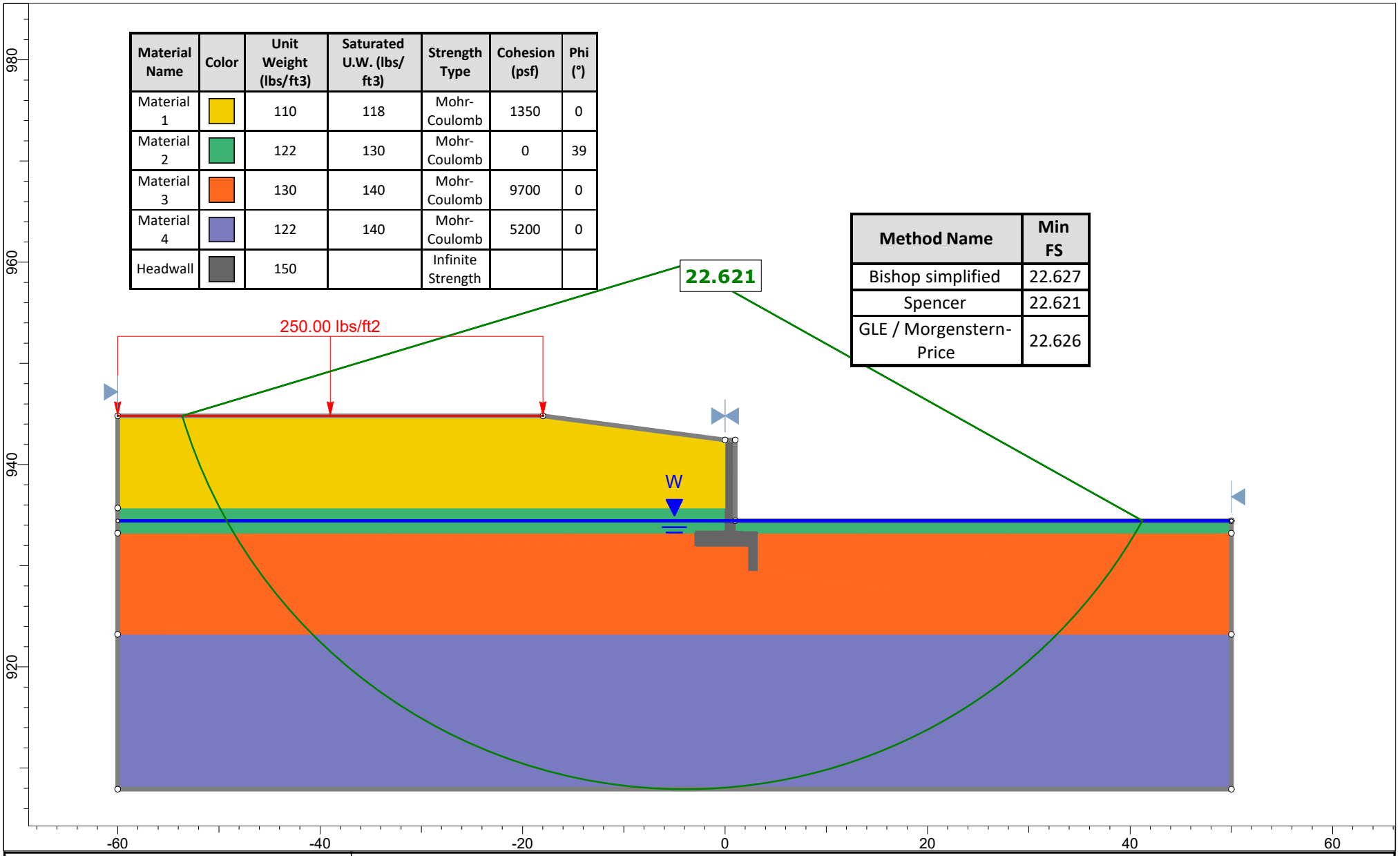
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SLIDEINTERPRET 9.036

Project	BRO-68-33.36		
Group	Group 1	Scenario	Master Scenario
Drawn By	ZM	Company	NEAS, inc.
Date	3/20/2025	File Name	BRO-68_Culvert Under Greenbush Rd_Inlet_Effective_B-05.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		110	118	Mohr-Coulomb	1350	0
Material 2		122	130	Mohr-Coulomb	0	39
Material 3		130	140	Mohr-Coulomb	9700	0
Material 4		122	140	Mohr-Coulomb	5200	0
Headwall		150		Infinite Strength		

Method Name	Min FS
Bishop simplified	22.627
Spencer	22.621
GLE / Morgenstern-Price	22.626



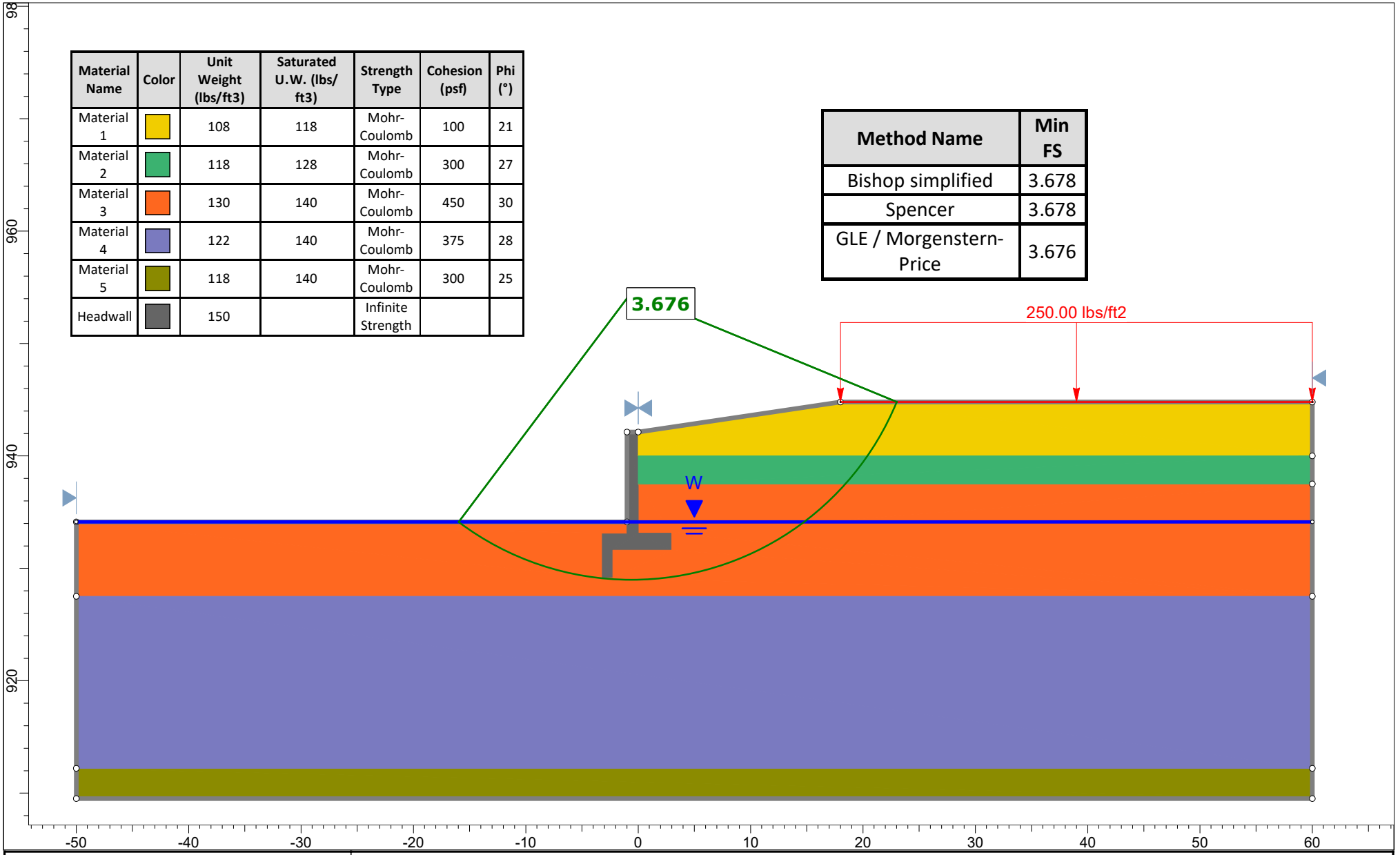
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SLIDEINTERPRET 9.036

Project	BRO-68-33.36		
Group	Group 1	Scenario	Master Scenario
Drawn By	ZM	Company	NEAS, inc.
Date	3/20/2025	File Name	BRO-68_Culvert Under Greenbush Rd_Inlet_Total_B-05.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		108	118	Mohr-Coulomb	100	21
Material 2		118	128	Mohr-Coulomb	300	27
Material 3		130	140	Mohr-Coulomb	450	30
Material 4		122	140	Mohr-Coulomb	375	28
Material 5		118	140	Mohr-Coulomb	300	25
Headwall		150		Infinite Strength		

Method Name	Min FS
Bishop simplified	3.678
Spencer	3.678
GLE / Morgenstern-Price	3.676



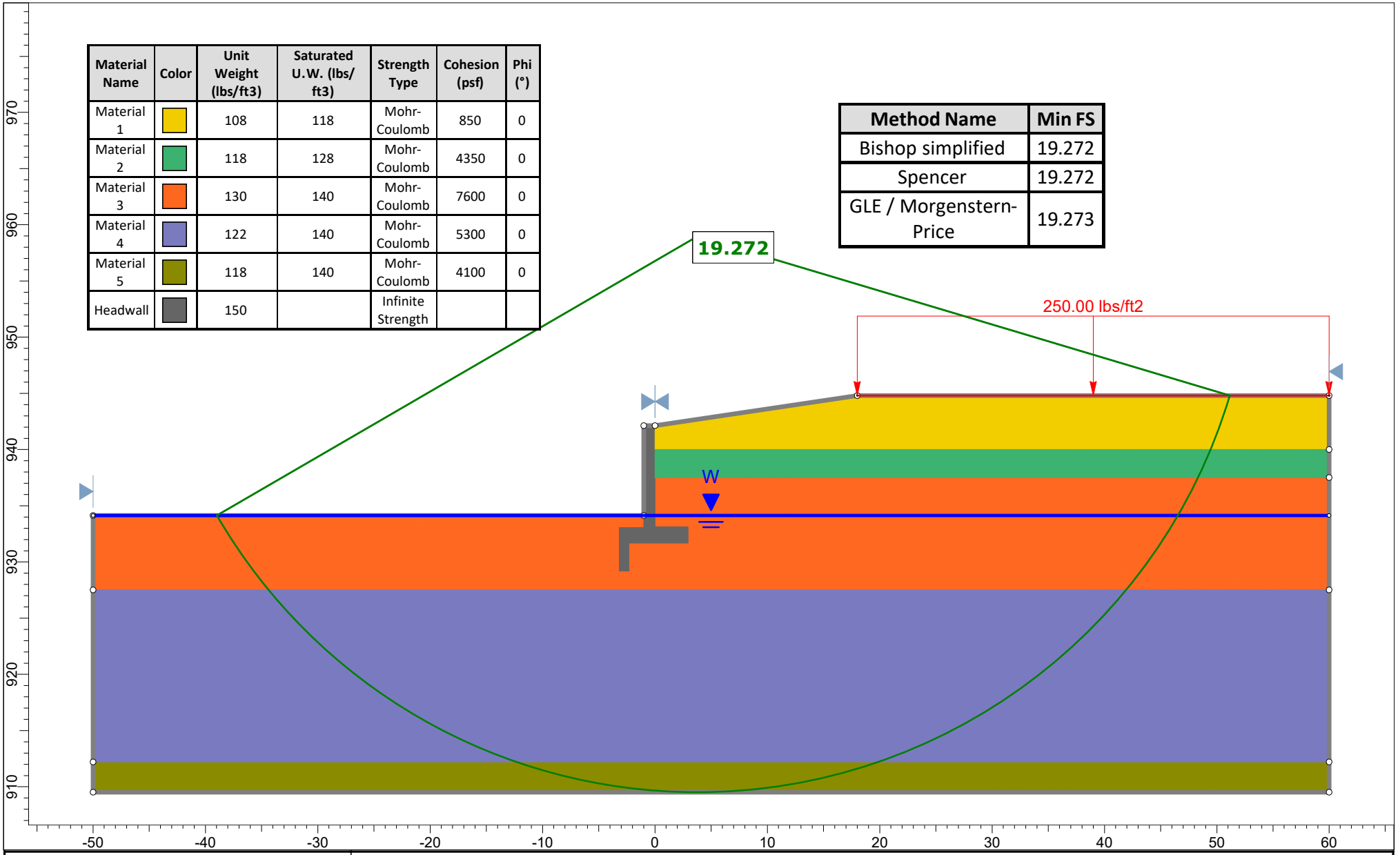
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SLIDEINTERPRET 9.036

Project	BRO-68-33.36		
Group	Group 1	Scenario	Master Scenario
Drawn By	ZM	Company	NEAS, inc.
Date	3/20/2025	File Name	BRO-68_Culvert Under Greenbush_Outlet_Effective_B-02.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		108	118	Mohr-Coulomb	850	0
Material 2		118	128	Mohr-Coulomb	4350	0
Material 3		130	140	Mohr-Coulomb	7600	0
Material 4		122	140	Mohr-Coulomb	5300	0
Material 5		118	140	Mohr-Coulomb	4100	0
Headwall		150		Infinite Strength		

Method Name	Min FS
Bishop simplified	19.272
Spencer	19.272
GLE / Morgenstern-Price	19.273



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SLIDEINTERPRET 9.036

Project	BRO-68-33.36		
Group	Group 1	Scenario	Master Scenario
Drawn By	ZM	Company	NEAS, inc.
Date	3/20/2025	File Name	BRO-68_Culvert Under Greenbush_Outlet_Total_B-02.slmd