FINAL REPORT
STRUCTURE FOUNDATION EXPLORATION
FOOTE ROAD RETAINING WALL #3
MED-18-12.99
MEDINA COUNTY, OHIO
PID#: 92953

Prepared For:

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NEAS PROJECT 15-0091

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EXECUTIVE SUMMARY

The Ohio Department of Transportation (ODOT) has proposed the construction of Retaining Wall #3 (RW #3) located along the east side of Foote Road starting from STA. 21+44.68 to STA. 22 +76.68, as part of the proposed SR-18 widening and improvement project (MED-18-12.99, PID 92953) in the City of Medina, Medina County, Ohio.

National Engineering & Architectural Services (NEAS). Inc. has been contracted to perform geotechnical engineering services for the project. The purpose of the geotechnical engineering services was to perform geotechnical explorations within the project limits to obtain information concerning the subsurface soil and groundwater conditions relevant to the design and construction of the project. Between March 31, 2020 and April 10, 2020, NEAS performed the site reconnaissance and exploration program for the project. The subsequent document presents the results of a structure foundation exploration with respect to the proposed RW #3, as a cast-in-place (CIP) wall type. As part of the exploration for RW #3, NEAS advanced 2 project borings and conducted laboratory testing to characterize the soils for engineering purposes. RW #3 will be approximately 132 ft in length and will have a maximum total height of approximately 10.3 ft at the beginning of wall (STA. 21+44.68).

The subsurface profile within the proposed project area generally consists of surficial materials comprised of topsoil generally underlain by natural stiff to hard cohesive soil. Bedrock was not encountered at the borings performed.

External stability (i.e., bearing resistance, sliding resistance, and eccentricity), and global stability analyses were performed for the proposed wall. For the analyses, factored bearing resistances ranging from 6.7 to 11.7 kips per square foot (ksf) were determined utilizing the provided RW #3 sections. Capacity to demand ratios (CDR) for bearing resistance, sliding, and eccentricity were calculated at the Strength Limit State. Based on the calculated CDR values, it was determined that the proposed CIP wall (RW #3) will provide adequate resistance to bearing, overturning and sliding.

Based on our global stability analyses for the referenced retaining wall section, the minimum slope stability safety factors for both short-term (Total Stress) and long-term (Effective Stress) conditions for RW #3 exceeded the desired value of 1.33. Therefore, it is our opinion that the subsurface conditions encountered at this location are generally satisfactory and the site can be considered to be stable at short-term and long-term condition.

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1. INTRODUCTION

1.1. General

NEAS presents our Structure Foundation Exploration Report for the proposed construction of Retaining Wall #3 (RW #3) located along the east side of Foote Road starting from STA. 21+44.68 to STA. 22+76.68, as part of the proposed SR-18 widening and improvement project (MED-18-12.99, PID 92953) in the City of Medina, Medina County, Ohio. This report presents a summary of the encountered surficial and subsurface conditions and our recommendations for retaining wall foundation design and construction in accordance with Load and Resistance Factor Design (LRFD) method as set forth in AASHTO's Publication *LRFD Bridge Design Specifications, 8th Edition* with 2017 interim revisions (BDS) (AASHTO, 2017) and *ODOT's 2020 LRFD Bridge Design Manual* (BDM) (ODOT, 2020).
The exploration was conducted in general accordance with National Engineering & Architectural Services Inc. (NEAS) proposal to GPD Group dated on February 28, 2020 and with the provisions of ODOT's *Specifications for Geotechnical Explorations* (SGE) (ODOT, 2020).

The scope of work performed by NEAS as part of the referenced project included: a review of published geotechnical information; performing 2 test borings; laboratory testing of soil samples in accordance with the SGE; performing geotechnical engineering analysis to assess foundation design and construction considerations; and development of this summary report.

1.2. Proposed Construction

RW #3 is proposed along the east side of Foote Road starting from approximate STA. 21+44.68 to approximate STA. 22+76.68.

Based on design information provided in the email dated June 5, 2020, and the site plan developed by GPD group, the proposed RW #3 will be a cast-in-place concrete retaining wall. It is our understanding that the wall will be approximately 132 ft in length and will have a maximum total height of approximately 10.3 ft at the beginning of wall.

2. GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1. Geology and Physiography

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

The project site is underlain by Wisconsinan-age till (unsorted mix of clay, silt, sand, gravel and boulders) over sandstone and shale deposited in Mississippian-age (ODNR, 2000). Bedrock topography maps indicated depth of bedrock ranging from elevation 850 ft to 900 ft, placing it between 80 ft and 130 ft deep (Schumacher, et al, 1996). It is mapped as Mississippian-age Cuyahoga Formation (Slucher, et al, 1996).

The soils at the project site have been mapped (Web Soil Survey) by the Natural Resources Conservation Service as being Mahoning silt loam with 2 to 6 percent slopes within the areas of the proposed project. Mahoning silt loam soils are somewhat poorly drained. The units of the Mahoning silt loam are classified as A-6a and A-6b soils according to the AASHTO method of soil classification.

2.2. Hydrology/Hydrogeology

The dominant hydraulic influences within the project area are West Branch Rocky River and Lake Medina, which are approximately 0.35 mile to the east of the proposed retaining wall. The flow line elevation is 925 ft and likely represents the local groundwater table.

The West Branch Rocky River and the area immediately adjacent to it are located in a special flood hazard zone subject to inundation by the 1% annual chance flood. However, the proposed retaining wall site is not located within a special flood hazard area based on available mapping by the Federal Emergency Management Agency's (FEMA) National Flood Hazard mapping program (FEMA, 2019).

2.3. Mining and Oil/Gas Production

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

No oil or gas wells are noted within the immediate vicinity of the proposed retaining wall site (ODNR [2], 2016).

2.4. Historical Records and Previous Phases of Project Exploration

A historic record search was performed through ODOT's Transportation Information Management System (TIMS); however, no report/plans were available for review within the limits of the retaining wall (RW #3) site. Therefore, historic borings are not referenced within this report nor within the project.

2.5. Site Reconnaissance

A field reconnaissance visit for the project area was conducted on March 31, 2020. The project site is located in Medina County, OH. Site conditions, including the existing pavement and embankment conditions were noted and photographed during the site visit. Photographs of notable pavement distress and a summary of our observations are provided below.

The land use of majority of project area consists of commercial properties to the east and west of the project site (i.e. businesses, shops).

The pavement conditions of Foote Road and the Foote Road and SR-18 intersection was observed to be in good condition (Photograph 1). Foote Road is a concrete road, and no signs of any distress, weathering, or cracking were observed. The embankment to the east of the Foote Road, was observed to be in good condition (Photograph 2). No signs of bulging or any other distress were observed. With respect to drainage, the pavement and the east embankment appeared to be well drained, with no observable signs of pounding or standing water (Photograph 3).

Photograph 1: Foote Road, north of intersection (looking north)

Photograph 2: East side Embankment (looking north)

Photograph 3: Drainage condition on East embankment

3. GEOTECHNICAL EXPLORATION

3.1. Field Exploration Program

The exploration for this wall was conducted by NEAS on April 10, 2020 and included 2 borings drilled to depths 26.5 ft bgs. The boring locations were selected by NEAS in general accordance with the guidelines contained in the SGE with the intent to evaluate subsurface soil and groundwater conditions. Borings were typically located along/near the proposed wall alignment in locations that were not restricted by maintenance of traffic, underground utilities or dictated by terrain (i.e. steep embankment slopes). Each as-drilled project boring location and corresponding ground surface elevation was surveyed in the field by NEAS following drilling. Each individual project boring log (included within Appendix B) includes the recorded boring latitude and longitude location (based on the surveyed Ohio State Plane North, NAD83, location) and the corresponding ground surface elevation. Latitude, longitude and elevations of the borings are shown on Table 1 below and the boring locations are depicted on the Target boring plan provided in Appendix A.

Table 1: Project Boring Summary

Boring Number	Location (Sta/Offset)	Latitude	Longitude	Elevation (NAVD 88) (ft)	Depth (ft)
B-039-1-17	21+47, 46' RT.	41.139397	-81.831404	985.9	26.5
B-039-2-17	22+57, 42' RT.	41.139701	-81.831424	984.5	26.5
Note: As-drilled boring Location (Sta/Offset) based on Proposed Foote Road alignment					

Borings were drilled using a CME 55T truck mounted drilling rig utilizing 3.25-inch diameter hollow stem augers. Soil samples were recovered at intervals of 2.5-ft to a depth of 30 ft bgs and at 5.0-ft intervals thereafter using a split spoon sampler (AASHTO T-206 "Standard Method for Penetration Test and Split Barrel Sampling of Soils."). The soil samples obtained from the exploration program were visually observed in the field by the NEAS field representative and preserved for review by a Geologist and possible laboratory testing. Standard penetration tests (SPT) were conducted using a CME auto hammer that has been calibrated to be 68.4% efficient as indicated on the boring logs.

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

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 on approximately 33% of the soil samples obtained. 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 in Appendix B.

3.2.2. Standard Penetration Test Results

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

4. GEOTECHNICAL FINDINGS

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

4.1. Subsurface Conditions

The subsurface profile within the proposed project area generally consists of surficial materials comprised of topsoil generally underlain by natural stiff to hard cohesive soil. Bedrock was not encountered at the borings performed.

4.1.1. Overburden Soil

At the proposed RW #3 site, the subsurface profile is very consistent. One soil stratum was encountered below the surficial material (generally topsoil). The stratum consisted of natural stiff to hard cohesive soils. These materials and the general profile are further described below.

The soil stratum encountered immediately beneath the topsoil consisted of cohesive soils and extended to end of boring (26.5 bgs). The soils in this stratum are classified on the borings logs as Silt and Clay (A-6a) and Silty Clay (A-6b). Those cohesive soils can be described as stiff to hard consistency correlating to converted SPT-N values (N_{60}) between 18 and 29 bpf. Natural moisture contents ranged from 15% to 19%. Based on Atterberg Limits test performed on representative samples of this material, the liquid limit is between 28 to 35 percent and plastic limit is between 16 to 20 percent.

4.1.2. Groundwater

Groundwater measurements were taken during the boring drilling procedures and immediately following the completion of each borehole. Groundwater was not observed during drilling and upon completion in either of the two borings performed as part of the referenced project.

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

5. ANALYSIS AND RECOMMENDATIONS

We understand that the construction of a retaining wall (RW #3) is planned along Foote Road, as part of the proposed as part of the SR-18 widening and improvement project (MED-18-12.99, PID 92953) in the City of Medina, Medina County, Ohio. The proposed retaining wall is proposed along the east side of Foote Road starting from STA. 21+44.68 to STA. 22+76.68.

Based on design information provided within the MED-18-12.99, Retaining Wall #3 Site Plan developed by GPD Group and the email dated on June 5, 2020, it is our understanding that the proposed RW #3 will be a cast-in-place concrete retaining wall and the wall will be approximately 132 ft in length and will have a maximum total height of approximately 10.3 ft at the beginning of wall (STA. 21+44.68).

A foundation review was completed for the foundations of the proposed RW #3. The analyses performed is based on the information presented in Section 5.1 of this report in addition to: 1) the soil characteristics gathered during the subsurface exploration (i.e., SPT results, laboratory test results, etc.); 2) the proposed Retaining Wall #3 plan developed by GPD Group; and, 3) other design assumptions presented in subsequent sections of this report. Geotechnical analyses consisting of external stability (i.e., bearing resistance, eccentricity, and sliding resistance), and global stability were performed for the proposed retaining wall.

The geotechnical engineering analyses were performed in accordance with ODOT's BDM (ODOT, 2020) and AASHTO's LRFD BDS (AASHTO, 2017).

5.1. Generalized Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were estimated based on the field (i.e., SPT N_{60} Values, hand penetrometer values, etc.) and laboratory (i.e., Atterberg Limits, grain size, etc.) test results using correlations provided in published engineering manuals, research reports and guidance documents. The developed soil profile and estimated engineering soil properties for use in analysis (with sited correlation/reference material) is summarized within Tables 2 through 3 below.

Table 2: Soil Profile and Estimated Engineering Properties - At Boring B-039-1-17

Foote Road Retaining Wall : Soil Profile, B-039-1-17						
Soil Description	Unit Weight(1) (pcf)	Moist Unit Weight(1) (pcf)	Saturated Unit Weight(1) (pcf)	Undrained Shear Strength(2) (psf)	Effective Cohesion(3) (psf)	Effective Friction Angle(3) (degrees)
Silty Clay Depth (985.9 ft - 981.4 ft)	115	115	125	3250	250	27
Silt and Clay Depth (981.4 ft - 973.9 ft)	115	115	125	3300	250	27
Silt and Clay Depth (973.9 ft - 966.9 ft)	115	115	125	3100	250	27
Silt and Clay Depth (966.9 ft - 959.4 ft)	115	115	125	3000	250	27

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

Table 3: Soil Profile and Estimated Engineering Properties - At Boring B-039-2-17

Foote Road Retaining Wall : Soil Profile, B-039-2-17						
Soil Description	Unit Weight(1) (pcf)	Moist Unit Weight(1) (pcf)	Saturated Unit Weight(1) (pcf)	Undrained Shear Strength(2) (psf)	Effective Cohesion(3) (psf)	Effective Friction Angle(3) (degrees)
Silt and Clay Depth (984.5 ft - 980 ft)	112	112	122	2350	200	25
Silt and Clay Depth (980 ft - 972.5 ft)	115	115	125	3100	250	27
Silt and Clay Depth (972.5 ft - 965.5 ft)	115	115	125	2900	250	26
Silt and Clay Depth (965.5 ft - 958 ft)	115	115	125	2500	250	26

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

5.2. Cast-In-Place Wall Design Assumptions

As the proposed RW #3 structure is proposed to be an approximate 10.3 ft tall, cast-in-place (CIP) wall, ODOT's BDM, AASHTO's LRFD BDS, and the project conditions dictate analysis parameters and design minimums/constraints to be used in the analysis and design process. The referenced parameters and design minimums/constraints that where significant to our analyses consist of the following:

- Embed the tops of footing founded on soil at least 1-ft from the nearest soil surface and embed the bottoms of footings founded on soil at least 4-ft from the nearest soil surface. Embed the bottoms of footings founded on embankment fill at least 5-ft from the nearest soil surface.
- In no case shall the bottom of the footings in existing soil or on embankment fills be above the frost line.
- The minimum wall stem thickness shall be 10-in. The minimum thickness shall be 1.5-ft at the top of the stem when concrete deflector parapets are cast directly on top of a retaining wall.
- Infill soils for CIP wall will meet the minimum design soil parameters ODOT's BDM Table 307-1 (ODOT, 2020) as shown in Table 4 below.

Table 4: Design Soil Parameters for Fill Materials

Fill Zone	Type of Soil	Design Soil Unit Weight	Friction Angle	Cohesion
CIP Wall Infill	Granular Embankment, per 703.16.B	120 lbs/ft^3	32 °	0 pcf

With respect to design constraints and assumptions specific to the proposed RW #3, the geometry of the proposed wall (i.e., exposed wall heights, existing ground elevations, proposed final grade behind/at the toe of the wall, etc.) is assumed to be consistent with that shown in the Retaining Wall #3 Plan and Elevation developed by GPD Group.

5.3. External Stability

Based on our estimated engineering soil properties and the RW #3 design assumptions provided in Sections 5.1. and 5.2. of this report, respectively, external stability analyses of the proposed CIP wall were performed. As the wall geometry is anticipated to change along the length of the wall, external stability was evaluated at two (2) separate cross-sections including one critical section. The one critical cross-section selected is STA 21+44.68 (maximum wall height). Each cross-section was evaluated for resistance to bearing pressure, sliding forces and overturning at the Strength Limit State in accordance with Section 11.6.1 – 11.6.3 of the AASHTO's LRFD BDS.

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 5 below. (External Stability Results can be found in Appendix C). The capacity to demand ratios (CDR) larger than 1.0 indicate a safe design. Based on our analyses, the CDRs for all bearing, sliding and overturning are larger than 1.0, therefore, it was determined that the proposed CIP wall (RW #3) will provide adequate resistance to bearing, overturning and sliding.

Table 5: External Stability Analysis Summary

Retaining Wall #3 External Analysis Summary		
Top of Wall (feet)	989.10	988.40
Bottom of Footing (feet)	978.80	978.80
Exposed Wall Height (feet)	3.7	5.4
Design Wall Height (feet)	10.3	9.6
Approximate Station[(1)]	21+44.68	22+00
Nearby Boring Log Used in Calculation	B-039-1-17	B-039-2-17
Capactiy Demand Ratio (CDR)		
Sliding (Undrained/Drained)	1.93 / 1.61	1.86 / 1.43
Overturning / Eccentricity	2.74	3.51
Bearing Capacity (Undrained/Drained)	4.74 / 6.18	4.09 / 3.96
Factored Bearing Resistance (ksf)[(2)] (Undrained/Drained)	9.0 / 11.7	7.0 / 6.7

Notes:

1. *Stationing in reference to the proposed Foote Road.*
2. *Bearing Resistance calculated in accordance to Section 10.6.3.1 of 2017 LRFD BDS and factored using Resistance Factor provided in Table 11.5.7-1 of 2014 LRFD BDS.*

5.4. Global Stability

For purposes of evaluating the stability of the proposed retaining wall (RW #3) site, NEAS reviewed one cross-section within the project limit that was interpreted to represent conditions that posed the greatest potential for slope instability. In general, cross-sections along the proposed wall alignment were reviewed to determine if the section would represent a combination of existing subsurface conditions and planned site grading that would be most critical to slope stability (i.e., maximum total wall height, maximum embankment height measured from toe of slope to top of wall coping, proposed cut into existing embankment slopes, weak or thick soil layer, etc.). Based on our review of the available information at the referenced locations and the associated soil properties, one (1) cross-section was estimated to be most "critical" and was analyzed for global stability. The one cross-section analyzed for global stability is at STA. 21+44.68 in reference to the proposed Foote Road alignment.

For the cross-section, NEAS developed a representative cross-sectional model to use as the basis for global stability analyses. The model was developed from NEAS's interpretation of the available information which included: 1) The proposed RW #3 Plan and Elevation developed by GPD Group; and, 2) test borings and laboratory data developed as part of this report. With respect to the soil's engineering properties, the provided Soil Profile and Estimated Engineering Properties presented in Section 5.1 of this report were used in our analyses.

The above referenced slope stability model was analyzed for long-term (Effective Stress) and short-term (Total Stress) slope stability utilizing the software entitled *Slide 7.0* by Rocscience, Inc. Specifically, the Bishop, Corrected Janbu, Spencer and GLE method analysis methods were used to calculate a factor of safety (FOS) for circular type slope failures, respectively. The FOS is the ratio of the resisting forces and the driving forces, with the desired safety factor being more than about 1.33 which equates to an AASHTO resistance factor less than 0.75 (per AASHTO's LRFD BDS the specified resistance factors are essentially the inverse of the FOS that should be targeted in slope stability programs). For this analysis, a resistance factor of 0.75 or lower is targeted as the slope contains or support a structural element.

Global stability analyses were performed for RW #3. The results of the analyses RW #3 are summarized in Table 6. The graphical output of the slope stability program (cross-sectional model, calculated safety factor, and critical failure plane) is presented in Appendix D. Based on our global stability analyses for the referenced retaining wall section, the minimum slope stability safety factors for both short-term (Total Stress) and long-term (Effective Stress) conditions exceeded the desired value of 1.33. Therefore, it is our opinion that the subsurface conditions encountered at this location are generally satisfactory and the site can be considered to be stable at short-term and long-term condition.

Table 6: Global Stability Analysis Summary

Station	Near Boring	Description	Minimum Factor of Safety	Equivalent Resistance Factor	Status (OK/NG)
STA. 21+44.68	B-039-1-17	Short Term	12.79	0.08	OK
		Long Term	2.62	0.38	OK

5.5. 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, 8th Edition* (AASHTO LRFD, 2017), we recommend a project site classification of D – Stiff Soil. Following seismic site classification, seismic design parameters for the site were developed using the web-based ATC Hazards by Location (ATC, 2019) which references the 2016 AASHTO Guide Specifications for LRFD Seismic Bridge Design. The ATC Hazards by Location Maps generated LRFD Seismic Design parameters as presented in Table 7. The ATC Hazards by Location Maps detailed report can be found in Appendix E.

Table 7: Seismic Design Parameters

Variable	Symbol (AASHTO 3.10)	Value
Latitude		41.139397
Longitude		-81.831404
Site Class		D
Peak Ground Acceleration	PGA	0.064g
Short Period Acceleration	S_s	0.124g
Long Period Acceleration	S_1	0.05g
Site Factor (zero period)	F_{PGA}	1.6
Site Factor (short period)	F_a	1.6
Site Factor (long period)	F_v	2.4
Zero period response seismic coefficient	$A_s = F_{PGA} * PGA$	0.1024g
Short period response seismic coefficient (0.2 seconds)	$S_{DS} = F_a * S_s$	0.132g
Long period response seismic coefficient (1.0 second)	$S_{D1} = F_v * S_1$	0.08g
Seismic Design Category	SDC	B

6. QUALIFICATIONS

This investigation was performed in accordance with accepted geotechnical engineering practice for the purpose of characterizing the subsurface conditions at the site of Retaining Wall #3 for the MED-18-12.99 (PID 92953) project. This report has been prepared for GPD Group, ODOT and their design consultants to be used solely in evaluating the soils underlying the retaining wall site and presenting geotechnical engineering recommendations specific to this project. The assessment of general site environmental conditions or the presence of pollutants in the soil, rock and groundwater of the site was beyond the scope of this geotechnical exploration. Our recommendations are based on the results of our field explorations, laboratory 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 retaining wall (RW #3) is made, the conclusions and recommendations contained in this report should not be considered valid until they are reviewed, and have been modified or verified in writing by a geotechnical engineer.

It has been a pleasure to be of service to GPD Group in performing this geotechnical exploration for the MED-18-12.99 project. Please call if there are any questions, or if we can be of further service.

Respectfully Submitted,

NEAS. Inc.

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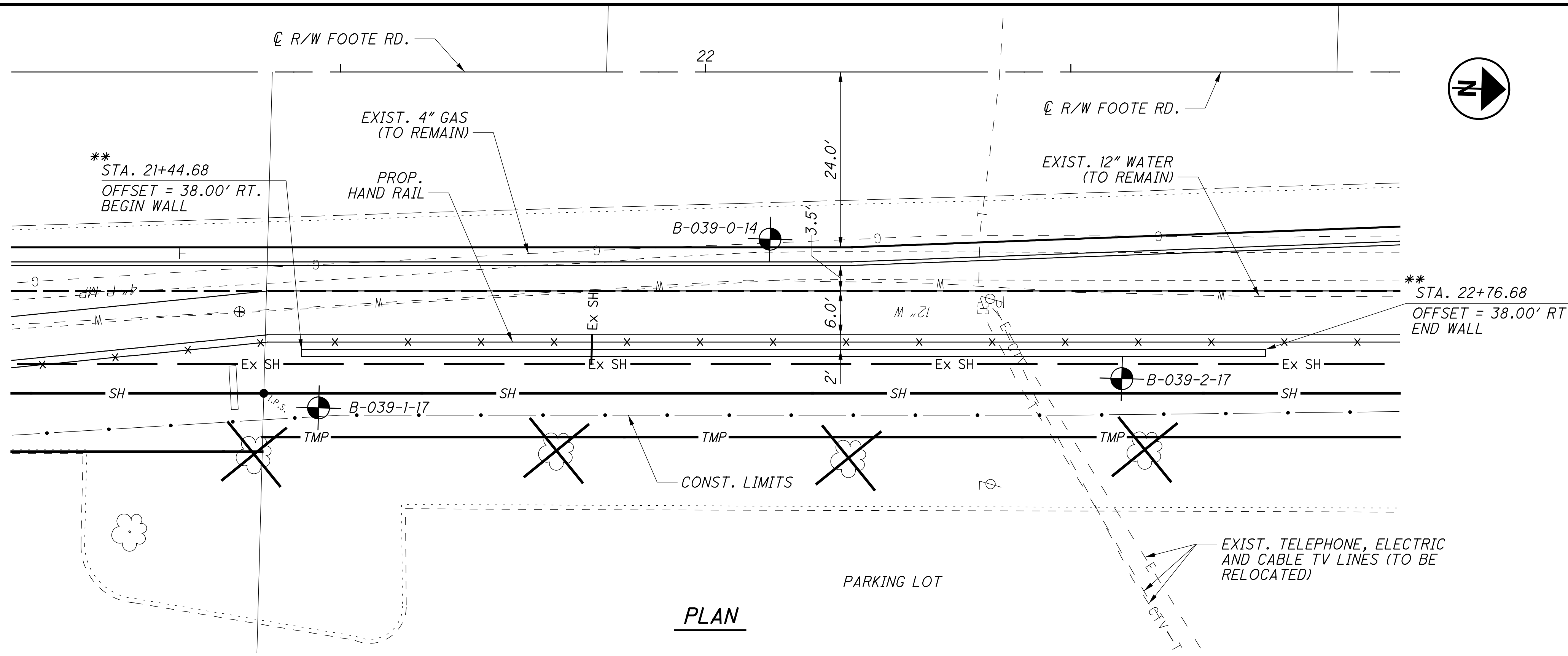
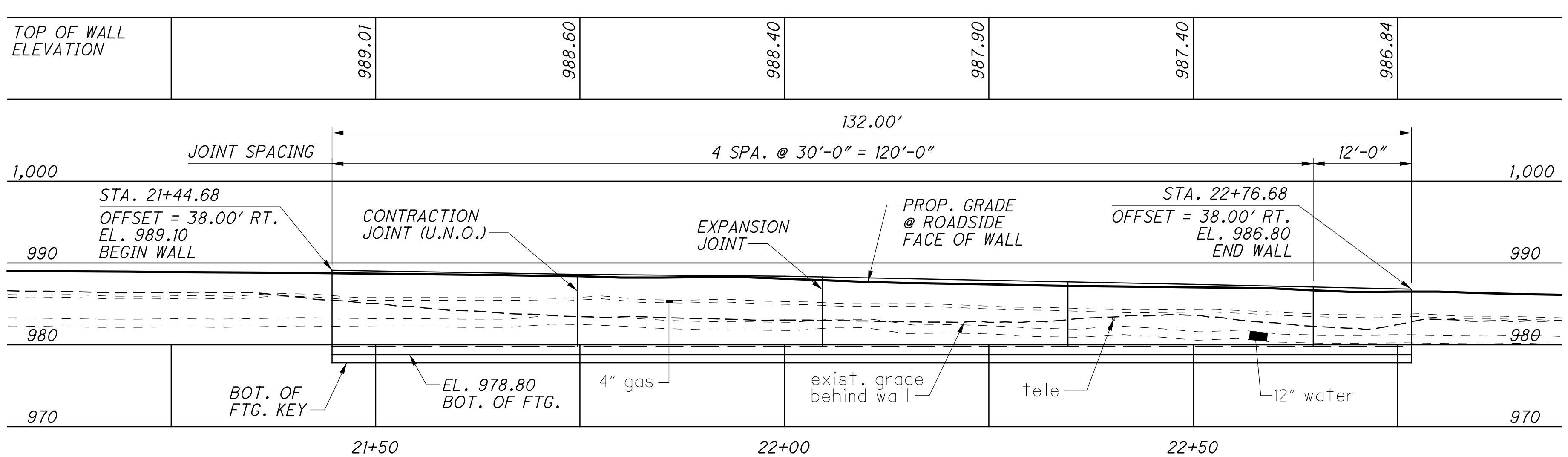
APPENDIX A

SITE PLAN



PLAN



PROFILE

LEGEND:

** STATIONS & OFFSETS ARE GIVEN TO THE ROADSIDE FACE OF THE WALL

PROJECT SOIL BORING

NOTE:

1. FOR WALL GENERAL NOTES, SEE SHT. NO. .

CALCULATED DJC
CHECKED DGN

PLAN AND PROFILE - RETAINING WALL NO. 5

MED-18-12.99

XX
XX

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APPENDIX B

SOIL BORING LOGS

PROJECT: MED-18-12.99	DRILLING FIRM / OPERATOR: NEAS / ASHBAUGH	DRILL RIG: CME 55T	STATION / OFFSET: 21+47, 46' RT.	EXPLORATION ID B-039-1-17
TYPE: RETAINING WALL	SAMPLING FIRM / LOGGER: NEAS / ASHBAUGH	HAMMER: CME AUTOMATIC	ALIGNMENT: FOOTE ROAD	
PID: 92953 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/5/19	ELEVATION: 985.9 (MSL) EOB: 26.5 ft.	PAGE 1 OF 1
START: 4/10/20 END: 4/10/20	SAMPLING METHOD: SPT	ENERGY RATIO (%): 68.4	LAT / LONG: 41.139397, -81.831404	

MATERIAL DESCRIPTION AND NOTES	ELEV. 985.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
6.0" TOPSOIL (DRILLERS DESCRIPTION)	985.4																	
HARD, BROWN MOTTLED WITH GRAY AND ORANGISH BROWN, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, CONTAINS TRACE IRON STAINING, DAMP		3	8 11 12	26	78	SS-1	4.50	4	10	11	36	39	35	19	16	17	A-6b (10)	
	981.4	4																
VERY STIFF TO HARD, BROWN MOTTLED WITH GRAY AND ORANGISH BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, SS-2 TO SS-7 CONTAIN IRON STAINING, DAMP TO MOIST		5	10 10 11	24	89	SS-2	4.50	2	7	11	38	42	31	19	12	15	A-6a (9)	
		8	9 11 13	27	100	SS-3	4.50	-	-	-	-	-	-	-	-	18	A-6a (V)	
		10	10 12 13	29	100	SS-4	4.00	-	-	-	-	-	-	-	-	19	A-6a (V)	
		13	8 8 12	23	100	SS-5	4.50	-	-	-	-	-	-	-	-	18	A-6a (V)	
		15	9 10 13	26	100	SS-6	4.50	-	-	-	-	-	-	-	-	19	A-6a (V)	
		18	11 10 13	26	100	SS-7	3.25	-	-	-	-	-	-	-	-	19	A-6a (V)	
@20.0'; BECOMES GRAY		20	10 10 12	25	100	SS-8	3.25	-	-	-	-	-	-	-	-	16	A-6a (V)	
		23	8 7 11	21	100	SS-9	4.50	-	-	-	-	-	-	-	-	17	A-6a (V)	
		25	9 11 13	27	100	SS-10	2.25	3	8	11	39	39	28	16	12	16	A-6a (9)	
	959.4	EOB																

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. BORING OFFSET 20' NORTHEAST DUE TO UTILITIES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: SHOVELED SOIL CUTTINGS

PROJECT: MED-18-12.99	DRILLING FIRM / OPERATOR: NEAS / ASHBAUGH	DRILL RIG: CME 55T	STATION / OFFSET: 22+57, 42' RT.	EXPLORATION ID B-039-2-17
TYPE: RETAINING WALL	SAMPLING FIRM / LOGGER: NEAS / ASHBAUGH	HAMMER: CME AUTOMATIC	ALIGNMENT: FOOTE ROAD	
PID: 92953 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/5/19	ELEVATION: 984.5 (MSL) EOB: 26.5 ft.	PAGE 1 OF 1
START: 4/10/20 END: 4/10/20	SAMPLING METHOD: SPT	ENERGY RATIO (%): 68.4	LAT / LONG: 41.139701, -81.831424	

MATERIAL DESCRIPTION AND NOTES	ELEV. 984.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
4.5" TOPSOIL (DRILLERS DESCRIPTION)	984.1																	
STIFF TO HARD, BROWN BECOMING BROWN MOTTLED WITH GRAY AND ORANGISH BROWN, **SILT AND CLAY**, LITTLE TO SOME SAND, TRACE GRAVEL, SS-3 TO SS-5 CONTAIN IRON STAINING, DAMP		3	8 7 10	19	78	SS-1	4.50	2	7	11	38	42	34	20	14	17	A-6a (10)	
		5–6	9 10 12	25	94	SS-2	4.00	-	-	-	-	-	-	-	-	18	A-6a (V)	
		8	10 9 14	26	100	SS-3	4.50	-	-	-	-	-	-	-	-	18	A-6a (V)	
		10–11	10 10 11	24	100	SS-4	4.50	-	-	-	-	-	-	-	-	18	A-6a (V)	
		13	9 11 10	24	100	SS-5	4.50	-	-	-	-	-	-	-	-	19	A-6a (V)	
@15.0'; BECOMES GRAY		15–16	7 9 11	23	100	SS-6	3.75	-	-	-	-	-	-	-	-	16	A-6a (V)	
		18	8 10 10	23	100	SS-7	2.75	2	8	12	41	37	28	17	11	16	A-6a (8)	
		20–21	7 7 9	18	100	SS-8	2.00	-	-	-	-	-	-	-	-	17	A-6a (V)	
		23	8 9 9	21	100	SS-9	1.50	-	-	-	-	-	-	-	-	16	A-6a (V)	
	958.0	25–26 EOB	9 9 10	22	100	SS-10	2.25	-	-	-	-	-	-	-	-	16	A-6a (V)	

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. BORING OFFSET 15' EAST DUE TO UTILITIES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: SHOVELED SOIL CUTTINGS

APPENDIX C

EXTERNAL STABILITY ANALYSIS

Objective: **To evaluate the external stability of CIP wall's with level backfill (no backslope).**
Method: **In accordance with ODOT Bridge Design Manual, 2020 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., Nov. 2017, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].**

Givens:

Backfill Soil Design Parameters:

$\phi'_f := 32\ deg$ — Effective angle of internal friction

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

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

$\delta := 0.67 \cdot \phi'_f$ $\quad \delta = 21.4\ deg$ — 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):

$\phi'_{fd} := 27\ deg$ — Effective angle of internal friction

$\gamma_{fd} := 115\ \frac{lbf}{ft^3}$ — Unit weight

$c'_{fd} := 250\ \frac{lbf}{ft^2}$ — Effective Cohesion

$\delta_{fd} := 0.67 \cdot \phi'_{fd}$ $\quad \delta_{fd} = 18.1\ deg$ — 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)

$\gamma_{fd} = 115\ \frac{lbf}{ft^3}$ — Unit weight

$Su_{fdu} := 3000\ \frac{lbf}{ft^2}$ — 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)**

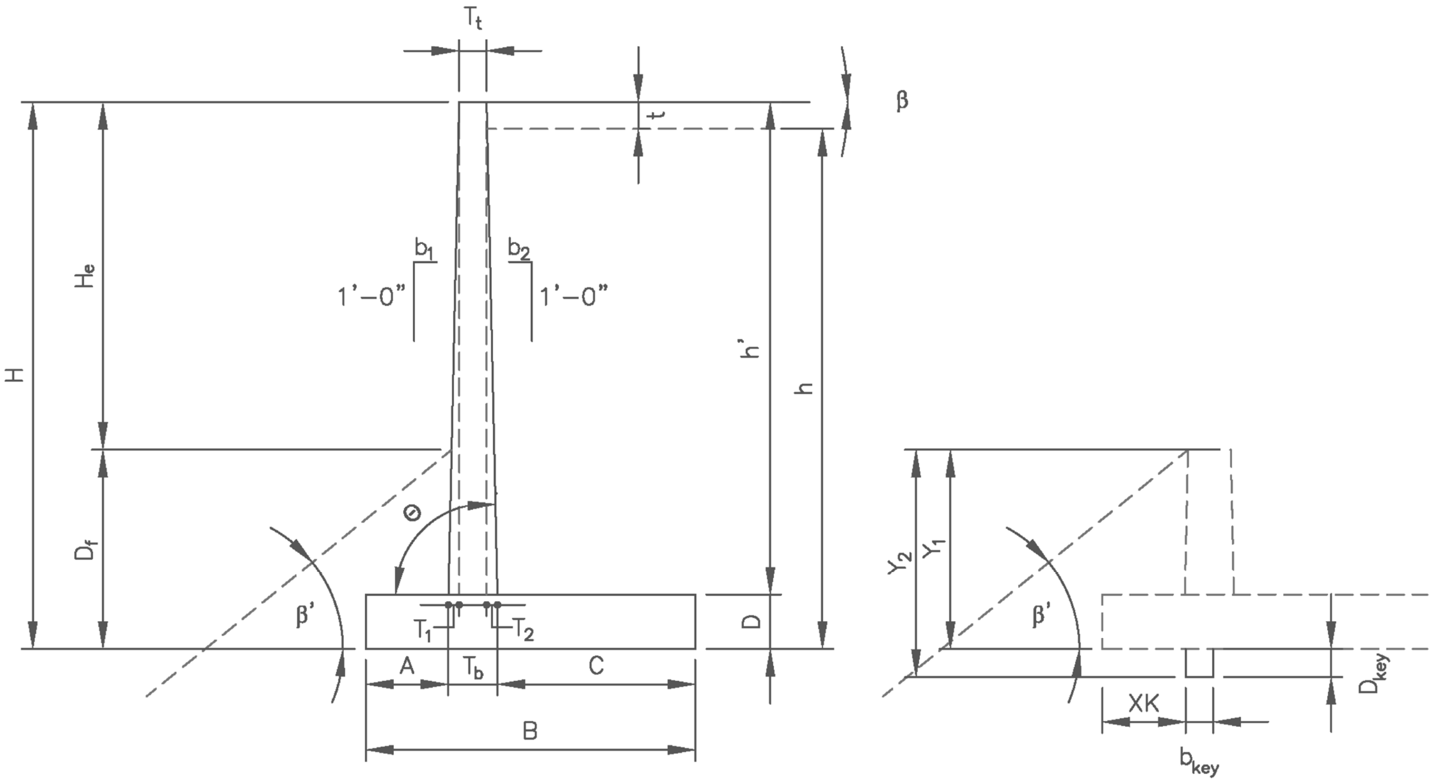
Foundation Surcharge Soil Parameters:

$\gamma_q := 120\ \frac{lbf}{ft^3}$ — 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

$\gamma_p := 150\ \frac{lbf}{ft^3}$ — Pavement Unit weight



Wall Geometry:

$H_e := 3.7\ ft$ — Exposed wall height

$D_f := 6.6\ ft$ — 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.**

$H := H_e + D_f$ $\quad H = 10.3\ ft$ — 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)

$\beta := 0\ deg$

Inclination of ground slope:

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

Inclination of ground slope behind face of wall. Horizontal backfill behind CIP wall, β = 0 deg

$\beta' := 0\ deg$

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.

$t := 0 \cdot ft$ — Pavement thickness

Preliminary Wall Dimensioning:

$B := 6.25\ ft$ $\quad \frac{2}{5} \cdot H = 4.12\ ft$ to $\frac{3}{5} \cdot H = 6.18\ ft$ Footing base width (2/5H to 3/5H)

$A := 1.75\ ft$ $\quad \frac{H}{8} = 1.29\ ft$ to $\frac{H}{5} = 2.06\ ft$ Toe projection (H/8 to H/5)

$D := 1\ ft$ $\quad \frac{H}{8} = 1.29\ ft$ to $\frac{H}{5} = 2.06\ ft$ Footing thickness (H/8 to H/5)

Shear Key Dimensioning:

$D_{key} := 1.0\ ft$ Depth of shear key from bottom of footing
Note: Footings on rock typically require shear key

$b_{key} := 1.0\ ft$ Width of shear key

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

Other Wall Dimensions:

$h' := H - D$ $\quad h' = 9.3\ ft$ 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

$C := B - A - T_b$ $\quad C = 3.5\ ft$ Heel projection

$\theta := \text{if}\left(b_2 > 0, \text{atan}\left(\frac{12}{b_2}\right), 90 \cdot deg\right)$ $\quad \theta = 90\ deg$ Angle of back face of wall to horizontal

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

$y_1 := D_f$ $\quad y_1 = 6.6\ ft$ Depth to where passive pressure may begin to be utilized in front of wall. (Typically Df)

$y_2 := D_f + D_{key}$ $\quad y_2 = 7.6\ ft$ Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.

$h := H - t$ $\quad h = 10.3\ ft$ Height of retained fill at back of heel

Live Load Surcharge Parameters:

$\lambda := 2\ ft$ Horizontal distance from the back of the wall to point of traffic surcharge load

$SUR := \text{if}\left(\lambda < \frac{H}{2}, 250\ \frac{lbf}{ft^2}, 100\ \frac{lbf}{ft^2}\right) = 250\ \frac{lbf}{ft^2}$

Live load surcharge (per LRFD BDS [3.11.6.4])
Note: If vehicular loading is within 1 ft of the backface of the wall and with a design height, H, less than 20 ft, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2 for adjusted surcharge load calculation.
Note: when λ < H/2, SUR equal 100 psf to account for construction loads

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{\left(\sin\left(\phi'_f + \delta\right) \cdot \sin\left(\phi'_f - \beta\right)\right)}{\left(\sin\left(\theta - \delta\right) \cdot \sin\left(\theta + \beta\right)\right)}}\right)^2 \qquad \Gamma = 2.81$$

$$k_{af} := \left(\frac{\left(\sin\left(\theta + \phi'_f\right)\right)^2}{\left(\Gamma \cdot \left(\sin\left(\theta\right)\right)^2 \cdot \sin\left(\theta - \delta\right)\right)}\right) \qquad k_{af} = 0.275$$

Active Earth Pressure Coefficient (**per LRFD Sect. 3.11.5.3**)

Foundation Soil Passive Earth:

Drained Conditions assuming($\phi'_{fd} > 0$):

Input Parameters for **LRFD Figure 3.11.5.4-2**, assumes θ = 90 degrees

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

$k'_p := 5.0$

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

Determine Reduction Factor (R) by interpolation:

$R_d := .789$

Reduction Factor

$k_{pd} := R_d \cdot k'_p \qquad k_{pd} = 3.945$

Passive Earth Pressure Coefficient for Drained Conditions

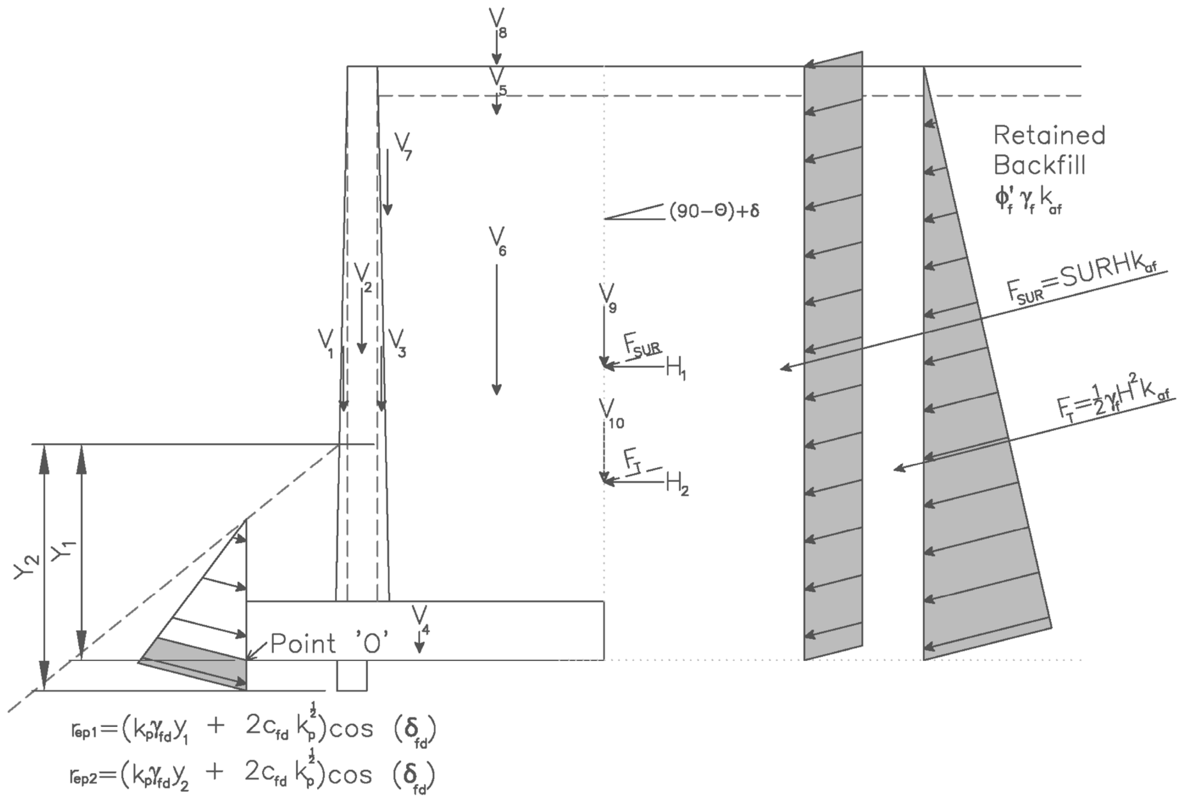
Undrained Conditions ($\phi_{fdu} > 0$): **Note:** Expand window below to complete calculation

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

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



$F_T := \frac{1}{2} \cdot \gamma_f \cdot H^2 \cdot k_{af}$	$F_T = 1750.4\ \frac{lbf}{ft}$	Active Earth Force Resultant (EH)
$F_{SUR} := SUR \cdot H \cdot k_{af}$	$F_{SUR} = 708.1\ \frac{lbf}{ft}$	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)
$V_2 := T_t \cdot h' \cdot \gamma_c$	$V_2 = 1395\ \frac{lbf}{ft}$	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)
$V_4 := D \cdot B \cdot \gamma_c$	$V_4 = 937.5\ \frac{lbf}{ft}$	Wall Footing (DC)
$V_5 := t \cdot (T_2 + C) \cdot \gamma_p$	$V_5 = 0\ \frac{lbf}{ft}$	Pavement (DC)
$V_6 := C \cdot (h' - t) \cdot \gamma_f$	$V_6 = 3906\ \frac{lbf}{ft}$	Soil Backfill - Heel (EV)
$V_7 := \frac{1}{2} \cdot b_2 \cdot (h' - t)^2 \cdot \gamma_f$	$V_7 = 0\ \frac{lbf}{ft}$	Soil Backfill - Batter (EV)
$V_8 := SUR \cdot (T_2 + C)$	$V_8 = 875\ \frac{lbf}{ft}$	Live Load Surcharge above Heel- (LS) - Strength Ib
$V_9 := F_{SUR} \cdot \sin(90 \cdot deg - \theta + \delta)$	$V_9 = 258.8\ \frac{lbf}{ft}$	Live Load Surcharge Resultant (vertical comp. - LS) - Strength Ia
$V_{10} := F_T \cdot \sin(90 \cdot deg - \theta + \delta)$	$V_{10} = 639.8\ \frac{lbf}{ft}$	Active earth force resultant (vertical component - EH)

Moment Arm: Moment:

Moments produced from vertical loads about Point 'O'

$d_{v1} := A + \frac{2}{3} \cdot T_1 = 1.8\ ft$ $MV_1 := V_1 \cdot d_{v1} = 0\ lbf$

$d_{v2} := A + T_1 + \frac{T_t}{2} = 2.3\ ft$ $MV_2 := V_2 \cdot d_{v2} = 3138.8\ lbf$

$d_{v3} := A + T_1 + T_t + \frac{T_2}{3} = 2.8\ ft$ $MV_3 := V_3 \cdot d_{v3} = 0\ lbf$

$d_{v4} := \frac{B}{2} = 3.1\ ft$ $MV_4 := V_4 \cdot d_{v4} = 2929.7\ lbf$

$d_{v5} := B - \frac{T_2 + C}{2} = 4.5\ ft$ $MV_5 := V_5 \cdot d_{v5} = 0\ lbf$

$d_{v6} := B - \frac{C}{2} = 4.5\ ft$ $MV_6 := V_6 \cdot d_{v6} = 17577\ lbf$

$d_{v7} := A + T_1 + T_t + \left(\frac{2}{3} \cdot b_2 \cdot (h' - t)\right) = 2.8\ ft$ $MV_7 := V_7 \cdot d_{v7} = 0\ lbf$

$d_{v8} := B - \frac{T_2 + C}{2} = 4.5\ ft$ $MV_8 := V_8 \cdot d_{v8} = 3937.5\ lbf$

$d_{v9} := B = 6.3\ ft$ $MV_9 := V_9 \cdot d_{v9} = 1617.7\ lbf$

$d_{v10} := B = 6.3\ ft$ $MV_{10} := V_{10} \cdot d_{v10} = 3998.9\ lbf$

Horizontal Loads:

$H_1 := F_{SUR} \cdot \cos(90 \cdot deg - \theta + \delta)$ $H_1 = 659.1\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (horizontal comp. - LS)

$H_2 := F_T \cdot \cos(90 \cdot deg - \theta + \delta)$ $H_2 = 1629.3\ \frac{lbf}{ft}$ Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm: Moment:

$d_{h1} := \frac{H}{2}$ $d_{h1} = 5.2\ ft$ $MH_1 := H_1 \cdot d_{h1}$ $MH_1 = 3394.3\ \frac{lbf \cdot ft}{ft}$

$d_{h2} := \frac{H}{3}$ $d_{h2} = 3.4\ ft$ $MH_2 := H_2 \cdot d_{h2}$ $MH_2 = 5593.9\ \frac{lbf \cdot ft}{ft}$

Unfactored Loads by Load Type:

$V_{DC} := V_1 + V_2 + V_3 + V_4 + V_5$ $V_{DC} = 2332.5\ \frac{lbf}{ft}$ $V_{EV} := V_6 + V_7$ $V_{EV} = 3906\ \frac{lbf}{ft}$

$V_{LS\_Ia} := V_9$ $V_{LS\_Ia} = 258.8\ \frac{lbf}{ft}$ $V_{LS\_Ib} := V_8 + V_9$ $V_{LS\_Ib} = 1133.8\ \frac{lbf}{ft}$

$V_{EH} := V_{10}$ $V_{EH} = 639.8\ \frac{lbf}{ft}$ $H_{LS} := H_1$ $H_{LS} = 659.1\ \frac{lbf}{ft}$

$H_{EH} := H_2$ $H_{EH} = 1629.3\ \frac{lbf}{ft}$

Unfactored Moments by Load Type

$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4 + MV_5$ $\quad M_{DC} = 6068.4\ \frac{lbf \cdot ft}{ft}$

$M_{EV} := MV_6 + MV_7$ $\quad M_{EV} = 17577\ \frac{lbf \cdot ft}{ft}$

$M_{LSV\_Ia} := MV_9$ $\quad M_{LSV\_Ia} = 1617.7\ \frac{lbf \cdot ft}{ft}$

$M_{LSV\_Ib} := MV_8 + MV_9$ $\quad M_{LSV\_Ib} = 5555.2\ \frac{lbf \cdot ft}{ft}$

$M_{EH1} := MV_{10}$ $\quad M_{EH1} = 3998.9\ \frac{lbf \cdot ft}{ft}$

$M_{LSH} := MH_1$ $\quad M_{LSH} = 3394.3\ \frac{lbf \cdot ft}{ft}$

$M_{EH2} := MH_2$ $\quad M_{EH2} = 5593.9\ \frac{lbf \cdot ft}{ft}$

Load Combination Limit States:

$\eta := 1$ LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia: (Sliding and Eccentricity) $\quad Ia_{DC} := 0.9$ $\quad Ia_{EV} := 1$ $\quad Ia_{EH} := 1.5$ $\quad Ia_{LS} := 1.75$

Strength Limit State Ib: (Bearing Capacity) $\quad Ib_{DC} := 1.25$ $\quad Ib_{EV} := 1.35$ $\quad Ib_{EH} := 1.5$ $\quad Ib_{LS} := 1.75$

Factored Vertical Loads by Limit State:

$V_{Ia} := \eta \cdot \left(\left(Ia_{DC} \cdot V_{DC} \right) + \left(Ia_{EV} \cdot V_{EV} \right) + \left(Ia_{EH} \cdot V_{EH} \right) + \left(Ia_{LS} \cdot V_{LS\_Ia} \right) \right)$ $\quad V_{Ia} = 7417.9\ \frac{lbf}{ft}$

$V_{Ib} := \eta \cdot \left(\left(Ib_{DC} \cdot V_{DC} \right) + \left(Ib_{EV} \cdot V_{EV} \right) + \left(Ib_{EH} \cdot V_{EH} \right) + \left(Ib_{LS} \cdot V_{LS\_Ib} \right) \right)$ $\quad V_{Ib} = 11132.7\ \frac{lbf}{ft}$

Factored Horizontal Loads by Limit State:

$H_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot H_{LS} \right) + \left(Ia_{EH} \cdot H_{EH} \right) \right)$ $\quad H_{Ia} = 3597.3\ \frac{lbf}{ft}$

$H_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot H_{LS} \right) + \left(Ib_{EH} \cdot H_{EH} \right) \right)$ $\quad H_{Ib} = 3597.3\ \frac{lbf}{ft}$

Factored Moments Produced by Vertical Loads by Limit State:

$MV_{Ia} := \eta \cdot \left(\left(Ia_{DC} \cdot M_{DC} \right) + \left(Ia_{EV} \cdot M_{EV} \right) + \left(Ia_{EH} \cdot M_{EH1} \right) + \left(Ia_{LS} \cdot M_{LSV\_Ia} \right) \right)$ $\quad MV_{Ia} = 31867.9\ \frac{lbf \cdot ft}{ft}$

$MV_{Ib} := \eta \cdot \left(\left(Ib_{DC} \cdot M_{DC} \right) + \left(Ib_{EV} \cdot M_{EV} \right) + \left(Ib_{EH} \cdot M_{EH1} \right) + \left(Ib_{LS} \cdot M_{LSV\_Ib} \right) \right)$ $\quad MV_{Ib} = 47034.4\ \frac{lbf \cdot ft}{ft}$

Factored Moments Produced by Horizontal Loads by Limit State:

$MH_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot M_{LSH} \right) + \left(Ia_{EH} \cdot M_{EH2} \right) \right)$ $\quad MH_{Ia} = 14330.9\ \frac{lbf \cdot ft}{ft}$

$MH_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot M_{LSH} \right) + \left(Ib_{EH} \cdot M_{EH2} \right) \right)$ $\quad MH_{Ib} = 14330.9\ \frac{lbf \cdot ft}{ft}$

Compute Bearing Resistance:

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

$\Sigma M_R := MV_{Ib}$ $\Sigma M_R = 47034.4\ \frac{lbf \cdot ft}{ft}$ Sum of Resisting Moments (Strength Ib)

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 14330.9\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 11132.7\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$ $x = 2.9\ ft$ Distance from Point "O" the resultant intersects the base

$e := \left| \frac{B}{2} - x \right|$ $e = 0.19\ ft$ Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e. When the foundation eccentricity is negative the absolute value is used.

Foundation Layout:

$B' := B - 2 \cdot e$ $B' = 5.9\ ft$ Effective Footing Width

$L' := 132\ ft$ Effective Footing Length (Assumed)

$H' := H_{Ib}$ $H' = 3597.3\ \frac{lbf}{ft}$ Summation of Horizontal Loads (Strength Ib)

$V' := V_{Ib}$ $V' = 11132.7\ \frac{lbf}{ft}$ Summation of Vertical Loads (Strength Ib)

$D_f = 6.6\ ft$ Footing embedment

$d_w := D_f$ Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan\left(45\ deg + \frac{\phi'_{fd}}{2}\right)^2, 1.0\right)$ $N_q = 13.2$

$N_c := \text{if}\left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14\right)$ $N_c = 23.94$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$ $N_\gamma = 14.5$

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

$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L'}\right)\right)$ $s_c = 1.025$

$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd})\right), 1\right)$ $s_q = 1.023$

$s_\gamma := \text{if}\left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'}\right), 1\right)$ $s_\gamma = 0.982$

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]**:

$C_{wq} := \text{if}\left(d_w \ge D_f, 1.0, 0.5\right)$ $\qquad C_{wq} = 1$

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

Depth Correction Factor per Hanson (1970):

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

$d_q = 1.25$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\qquad N_{cm} = 24.53$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\qquad N_{qm} = 13.498$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\qquad N_{\gamma m} = 14.212$

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

$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_{fd} \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\qquad q_{nd} = 21307.7 \frac{lbf}{ft^2}$

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

$\phi_b := .55$

Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Rd} := \phi_b \cdot q_{nd}$ $\qquad q_{Rd} = 11.7\ ksf$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

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

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

$$s_c := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'}{L'}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L'}\right)\right)$$ $s_c = 1.009$

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

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

Load inclination factors:

$i_q := 1$

$i_\gamma := 1$

$i_c := 1$

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

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $N_{cm} = 5.186$

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

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

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B} \le 1, 1 + 2 \cdot \tan\left(\phi_{fdu}\right) \cdot \left(1 - \sin\left(\phi_{fdu}\right)\right)^2 \cdot \frac{D_f}{B}, 1 + 2 \cdot \tan\left(\phi_{fdu}\right) \cdot \left(1 - \sin\left(\phi_{fdu}\right)\right)^2 \cdot \operatorname{atan}\left(\frac{D_f}{B}\right)\right)$$

$d_q = 1$

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

$$q_{nu} := Su_{fdu} \cdot N_{cm} + \gamma_{fd} \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$ $q_{nu} = 16316.3 \, \frac{lbf}{ft^2}$

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

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

$q_{Ru} := \phi_b \cdot q_{nu}$ $q_{Ru} = 9 \, ksf$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 11.7 \, ksf$

Undrained Conditions: $q_{Ru} = 9 \, ksf$

Evaluate External Stability of Wall:

Compute the ultimate bearing stress :

$e = 0.19\ ft$

$\sigma_V := \dfrac{\Sigma V}{B - 2 \cdot e}$ $\qquad$ $\sigma_V = 1.895\ ksf$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $\qquad CDR_{Bearing\_D} := \dfrac{q_{Rd}}{\sigma_V}$ $\qquad$ Is the CDR > or = to 1.0? $\qquad CDR_{Bearing\_D} = 6.18$

Undrained Conditions: $\qquad CDR_{Bearing\_U} := \dfrac{q_{Ru}}{\sigma_V}$ $\qquad$ Is the CDR > or = to 1.0? $\qquad CDR_{Bearing\_U} = 4.74$

Limiting Eccentricity at Base of Wall (Strength Ia):

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

$e_{max} := \dfrac{B}{3}$ $\qquad e_{max} = 2.1\ ft$ $\qquad$ Maximum Eccentricity **LRFD [11.6.3.3.] Equals B/3 for soil.**

$\Sigma M_R := MV_{Ia}$ $\qquad \Sigma M_R = 31867.9\ \dfrac{lbf \cdot ft}{ft}$ $\qquad$ Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$ $\qquad \Sigma M_O = 14330.9\ \dfrac{lbf \cdot ft}{ft}$ $\qquad$ Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$ $\qquad \Sigma V = 7417.9\ \dfrac{lbf}{ft}$ $\qquad$ Sum of Vertical Loads (Strength Ia)

$x := \dfrac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$ $\qquad x = 2.4\ ft$ $\qquad$ Distance from Point "O" the resultant intersects the base

$e := \text{abs}\left(\dfrac{B}{2} - x\right)$ $\qquad e = 0.76\ ft$ $\qquad$ 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)

$CDR_{Eccentricity} := \dfrac{e_{max}}{e}$ $\qquad$ Is the CDR > or = to 1.0? $\qquad CDR_{Eccentricity} = 2.74$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$ $\qquad R_u = 3597.3 \ \frac{lbf}{ft}$

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

$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot \left(y_2 - y_1\right)$ $\qquad R_{ep} = 4005.9 \ \frac{lbf}{ft}$ 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:

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

$\Sigma V := V_{Ia}$ $\qquad \Sigma V = 7417.9 \ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$R_\tau := c \cdot \Sigma V \cdot \tan\left(\phi'_{fd}\right)$ $\qquad R_\tau = 3779.6 \ \frac{lbf}{ft}$ Nominal sliding resistance Cohesionless Soils

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

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

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

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

Factored Sliding Resistance to be used in CDR Calculations: $R_R = 5782.568 \ \frac{lbf}{ft}$

Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{R_u}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 1.61$

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

$$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot \left(y_2 - y_1\right) \qquad R_{ep} = 6479.6 \ \frac{lbf}{ft}$$ 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:

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

$$\Sigma V := V_{Ia} \qquad \Sigma V = 7417.9 \ \frac{lbf}{ft}$$ Sum of Vertical Loads (Strength Ia)

$$e = 0.76 \ ft$$ Wall eccentricity, Calculated in above Limiting Eccentricity at Base of Wall (Strength Ia) Section.

$$B = 6.3 \ ft$$ Footing base width

$$\frac{B}{6} = 1 \ ft$$ If e < B/6 the resultant is in the middle one-third

$$\sigma_{vmax} := \frac{\Sigma V}{B} \cdot \left(1 + 6 \cdot \frac{e}{B}\right) \qquad \sigma_{vmax} = 2053.8 \ \frac{lbf}{ft^2}$$ Max vertical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$$\sigma_{vmin} := \frac{\Sigma V}{B} \cdot \left(1 - 6 \cdot \frac{e}{B}\right) \qquad \sigma_{vmin} = 319.9 \ \frac{lbf}{ft^2}$$ Max verical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$$q_{max} := \frac{1}{2} \cdot \sigma_{vmax} \qquad q_{max} = 1026.9 \ \frac{lbf}{ft^2}$$ Max unit shear resistance as 1/2 max vertical stress **LRFD [10.6.3.4]**.

$$q_{min} := \frac{1}{2} \cdot \sigma_{vmin} \qquad q_{min} = 160 \ \frac{lbf}{ft^2}$$ Minimum unit shear resistance as 1/2 minimum vertical stress **LRFD [10.6.3.4]**.

Determine which Cohesive Soil Resistance Case is Present:

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

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

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

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

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

Unit Shear Resistance for Case 1:

$S_1 := Su_{fdu} - q_{min} = 2840\ \frac{lbf}{ft^2}$ $\qquad S_2 := q_{min} = 160\ \frac{lbf}{ft^2}$

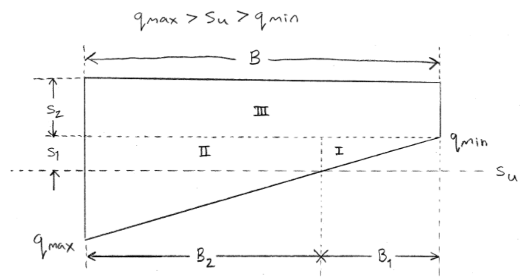
$B_1 := \frac{B \cdot \left(Su_{fdu} - q_{min}\right)}{q_{max} - q_{min}} = 20.5\ ft$ $\qquad B_2 := \frac{B \cdot \left(q_{max} - Su_{fdu}\right)}{q_{max} - q_{min}} = -14.2\ ft$

$B_3 := B = 6.3\ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 29074.4\ \frac{lbf}{ft}$ $\qquad II := S_1 \cdot B_2 = -40398.6\ \frac{lbf}{ft}$

$III := S_2 \cdot B_3 = 999.8\ \frac{lbf}{ft}$

$R_{\tau\_case1} := I + II + III = -10324.4\ \frac{lbf}{ft}$



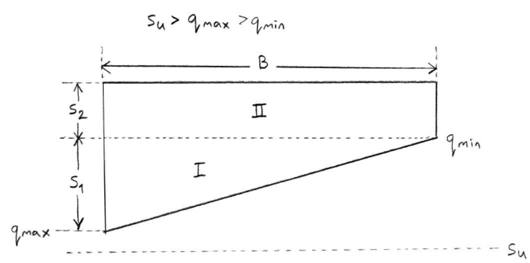
Unit Shear Resistance for Case 2:

$S_1 := q_{max} - q_{min} = 866.9\ \frac{lbf}{ft^2}$ $\qquad S_2 := q_{min} = 160\ \frac{lbf}{ft^2}$

$B = 6.3\ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B = 2709.2\ \frac{lbf}{ft}$ $\qquad II := S_2 \cdot B = 999.8\ \frac{lbf}{ft}$

$R_{\tau\_case2} := I + II = 3709\ \frac{lbf}{ft}$



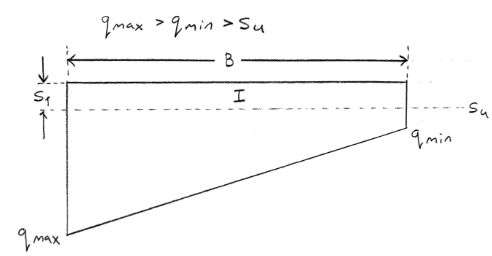
Unit Shear Resistance for Case 3:

$S_1 := Su_{fdu} = 3000\ \frac{lbf}{ft^2}$

$B = 6.3\ ft$

$I := S_1 \cdot B = 18750\ \frac{lbf}{ft}$

$R_{\tau\_case3} := I = 18750\ \frac{lbf}{ft}$



Unit Shear Resistance for Case 4:

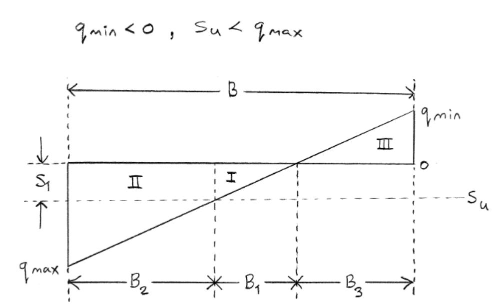
$S_1 := Su_{fdu} = 3000\ \frac{lbf}{ft^2}$

$B_3 := \frac{B \cdot \left(-q_{min}\right)}{q_{max} - q_{min}} = -1.2\ ft$ $\qquad B_1 := \left(\frac{Su_{fdu}}{q_{max}}\right) \cdot \left(B - B_3\right) = 21.6\ ft$

$B_2 := B - \left(B_1 + B_3\right) = -14.2\ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 32441.9\ \frac{lbf}{ft}$ $\qquad II := S_1 \cdot B_2 = -42674.1\ \frac{lbf}{ft}$

$R_{\tau\_case4} := I + II = -10232.2\ \frac{lbf}{ft}$



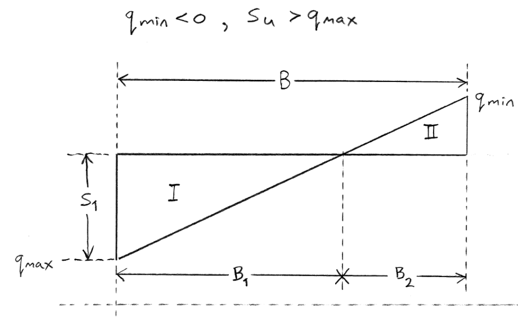
Unit Shear Resistance for Case 5:

$$S_1 := q_{max} = 1026.9\ \frac{lbf}{ft^2}$$

$$B_1 := \frac{B \cdot q_{max}}{q_{max} - q_{min}} = 7.4\ ft \qquad B_2 := B - B_1 = -1.2\ ft$$

$$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 3801.2\ \frac{lbf}{ft}$$

$$R_{\tau\_case5} := I = 3801.2\ \frac{lbf}{ft}$$



Define the Applicable Case:

$R_\tau := R_{\tau\_case2}$ $\qquad R_\tau = 3709\ \frac{lbf}{ft}$ Nominal sliding resistance Cohesive Soils

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

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

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

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

Factored Sliding Resistance to be used in CDR Calculations: $R_R = 6948.746\ \frac{lbf}{ft}$

Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{R_u}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 1.93$

Objective: **To evaluate the external stability of CIP wall's with level backfill (no backslope).**
Method: **In accordance with ODOT Bridge Design Manual, 2020 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., Nov. 2017, [Sect. 11.6.1, Sect. 11.6.2, and Sect. 11.6.3].**

Givens:

Backfill Soil Design Parameters:

$\phi'_f := 32\ deg$ Effective angle of internal friction

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

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

$\delta := 0.67 \cdot \phi'_f$ $\delta = 21.4\ deg$ 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):

$\phi'_{fd} := 25\ deg$ Effective angle of internal friction

$\gamma_{fd} := 112\ \frac{lbf}{ft^3}$ Unit weight

$c'_{fd} := 200\ \frac{lbf}{ft^2}$ Effective Cohesion

$\delta_{fd} := 0.67 \cdot \phi'_{fd}$ $\delta_{fd} = 16.8\ deg$ 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)

$\gamma_{fd} = 112\ \frac{lbf}{ft^3}$ Unit weight

$Su_{fdu} := 2350\ \frac{lbf}{ft^2}$ Undrained Shear Strength

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

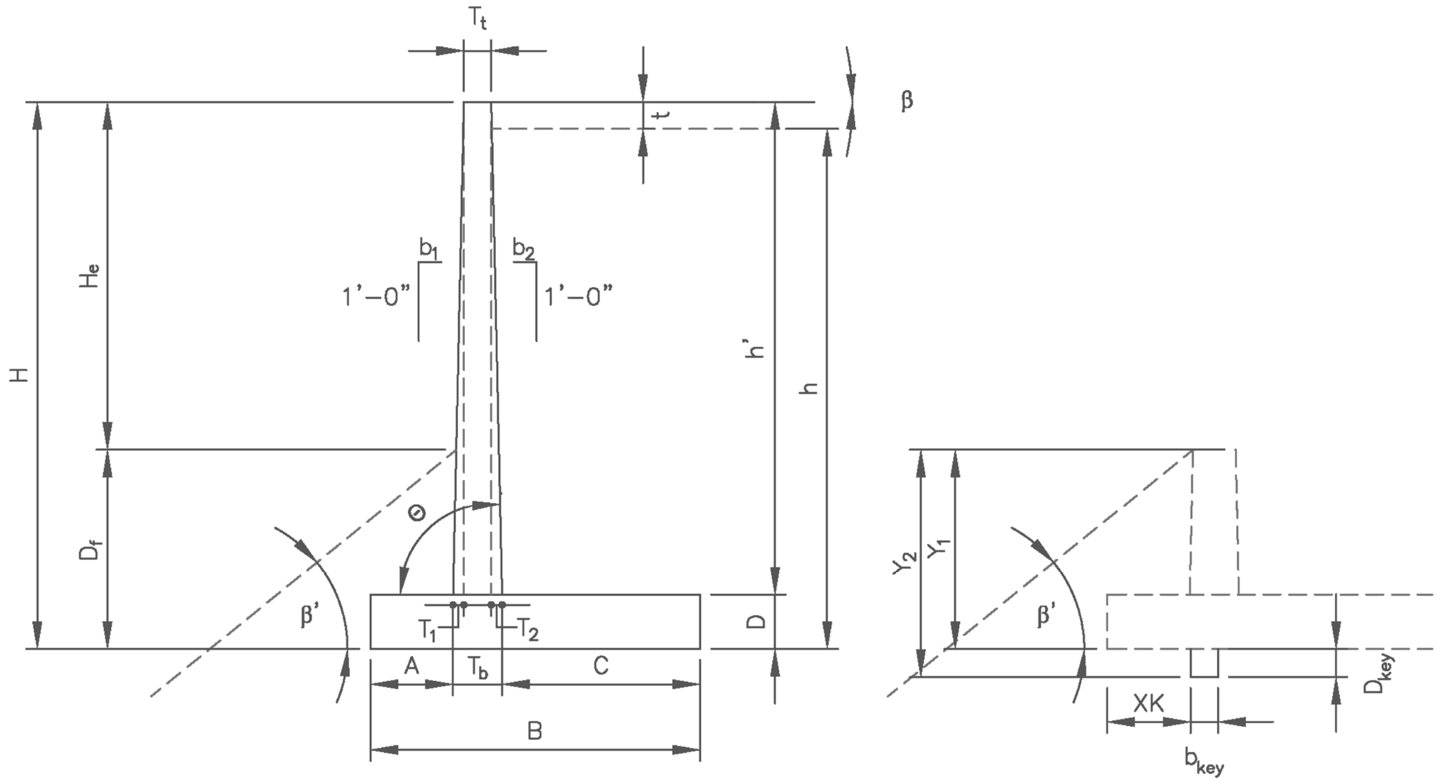
Foundation Surcharge Soil Parameters:

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

$\gamma_p := 150\ \frac{lbf}{ft^3}$ Pavement Unit weight



Wall Geometry:

$H_e := 5.4\ ft$ — Exposed wall height

$D_f := 4.2\ ft$ — 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.**

$H := H_e + D_f$ $\quad H = 9.6\ ft$ — 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)

$\beta := 0\ deg$ — 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.

$t := 0 \cdot ft$ — Pavement thickness

Preliminary Wall Dimensioning:

$B := 6.25\ ft$	$\frac{2}{5} \cdot H = 3.84\ ft$	to	$\frac{3}{5} \cdot H = 5.76\ ft$	Footing base width (2/5H to 3/5H)
$A := 1.75\ ft$	$\frac{H}{8} = 1.2\ ft$	to	$\frac{H}{5} = 1.92\ ft$	Toe projection (H/8 to H/5)
$D := 1\ ft$	$\frac{H}{8} = 1.2\ ft$	to	$\frac{H}{5} = 1.92\ ft$	Footing thickness (H/8 to H/5)

Shear Key Dimensioning:

$D_{key} := 1.0\ ft$ — Depth of shear key from bottom of footing
Note: Footings on rock typically require shear key

$b_{key} := 1.0\ ft$ — Width of shear key

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

Other Wall Dimensions:

$h' := H - D$	$h' = 8.6\ ft$	Stem height
$T_1 := b_1 \cdot h'$	$T_1 = 0\ ft$	Stem front batter width
$T_2 := b_2 \cdot h'$	$T_2 = 0\ ft$	Stem back batter width
$T_b := T_1 + T_2 + T_t$	$T_b = 1\ ft$	Stem thickness at bottom of wall
$C := B - A - T_b$	$C = 3.5\ ft$	Heel projection
$\theta := \text{if}\left(b_2 > 0, \text{atan}\left(\frac{12}{b_2}\right), 90 \cdot deg\right)$	$\theta = 90\ deg$	Angle of back face of wall to horizontal
$b := 12\ in$	$b = 1\ ft$	Concrete strip width (for design)
$y_1 := D_f$	$y_1 = 4.2\ ft$	Depth to where passive pressure may begin to be utilized in front of wall. (Typically Df)
$y_2 := D_f + D_{key}$	$y_2 = 5.2\ ft$	Bottom of shear key/footing depth i.e. depth to where passive pressure may no longer be utilized.
$h := H - t$	$h = 9.6\ ft$	Height of retained fill at back of heel

Live Load Surcharge Parameters:

$\lambda := 2\ ft$ — Horizontal distance from the back of the wall to point of traffic surcharge load

$SUR := \text{if}\left(\lambda < \frac{H}{2}, 250\ \frac{lbf}{ft^2}, 100\ \frac{lbf}{ft^2}\right) = 250\ \frac{lbf}{ft^2}$

Live load surcharge (per LRFD BDS [3.11.6.4])
Note: If vehicular loading is within 1 ft of the backface of the wall and with a design height, H, less than 20 ft, see LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2 for adjusted surcharge load calculation.
Note: when λ< H/2, SUR equal 100 psf to account for construction loads

Calculations:

Earth Pressure Coefficients:

Backfill Active Earth:

$$\Gamma := \left(1 + \sqrt{\frac{\left(\sin\left(\phi'_f + \delta\right) \cdot \sin\left(\phi'_f - \beta\right)\right)}{\left(\sin\left(\theta - \delta\right) \cdot \sin\left(\theta + \beta\right)\right)}}\right)^2 \qquad \Gamma = 2.81$$

$$k_{af} := \left(\frac{\left(\sin\left(\theta + \phi'_f\right)\right)^2}{\left(\Gamma \cdot \left(\sin\left(\theta\right)\right)^2 \cdot \sin\left(\theta - \delta\right)\right)}\right) \qquad k_{af} = 0.275$$

Active Earth Pressure Coefficient (**per LRFD Sect. 3.11.5.3**)

Foundation Soil Passive Earth:

Drained Conditions assuming($\phi'_{fd} > 0$):
Input Parameters for **LRFD Figure 3.11.5.4-2**, assumes θ = 90 degrees

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

$k'_p := 5.0$ Passive Earth Pressure Coefficient from **LRFD Figure 3.11.5.4-2**

Determine Reduction Factor (R) by interpolation:

$R_d := .789$ Reduction Factor

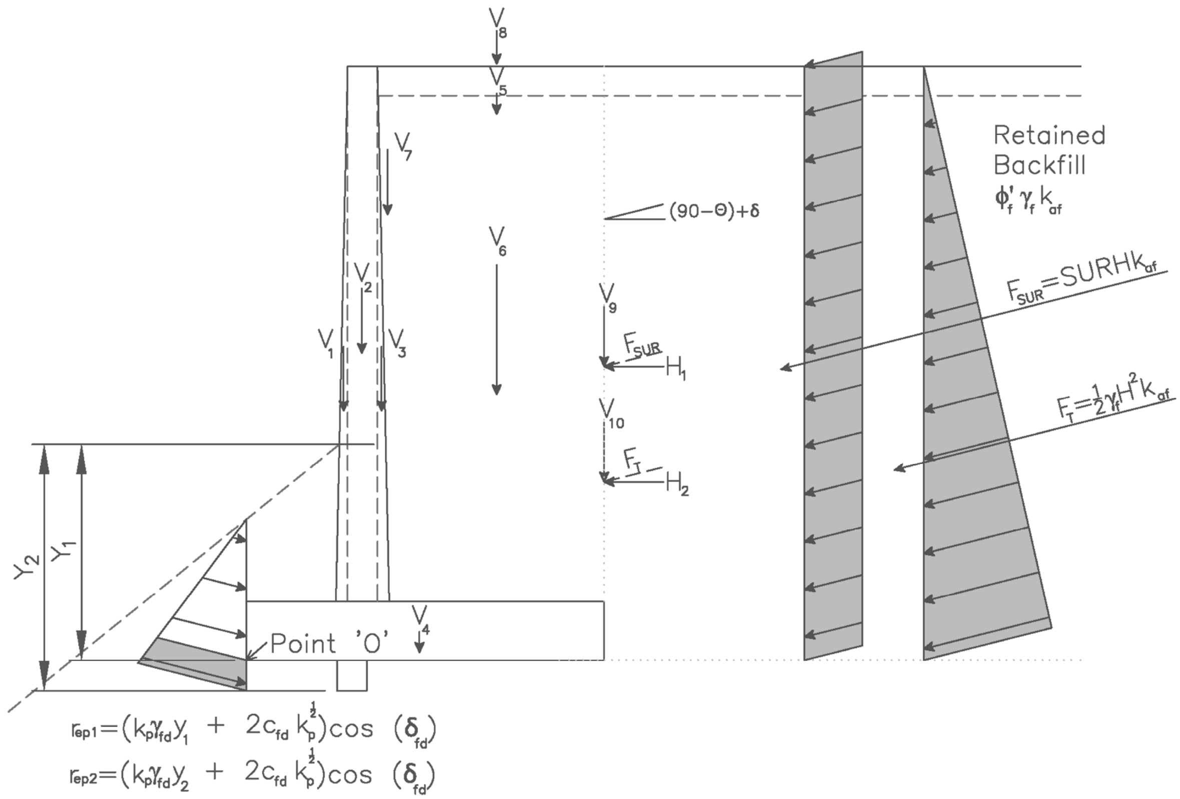
$k_{pd} := R_d \cdot k'_p \qquad k_{pd} = 3.945$ Passive Earth Pressure Coefficient for Drained Conditions

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

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



$F_T := \frac{1}{2} \cdot \gamma_f \cdot H^2 \cdot k_{af}$	$F_T = 1520.6\ \frac{lbf}{ft}$	Active Earth Force Resultant (EH)
$F_{SUR} := SUR \cdot H \cdot k_{af}$	$F_{SUR} = 660\ \frac{lbf}{ft}$	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)
$V_2 := T_t \cdot h' \cdot \gamma_c$	$V_2 = 1290\ \frac{lbf}{ft}$	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)
$V_4 := D \cdot B \cdot \gamma_c$	$V_4 = 937.5\ \frac{lbf}{ft}$	Wall Footing (DC)
$V_5 := t \cdot \left(T_2 + C\right) \cdot \gamma_p$	$V_5 = 0\ \frac{lbf}{ft}$	Pavement (DC)
$V_6 := C \cdot \left(h' - t\right) \cdot \gamma_f$	$V_6 = 3612\ \frac{lbf}{ft}$	Soil Backfill - Heel (EV)
$V_7 := \frac{1}{2} \cdot b_2 \cdot \left(h' - t\right)^2 \cdot \gamma_f$	$V_7 = 0\ \frac{lbf}{ft}$	Soil Backfill - Batter (EV)
$V_8 := SUR \cdot \left(T_2 + C\right)$	$V_8 = 875\ \frac{lbf}{ft}$	Live Load Surcharge above Heel- (LS) - Strength Ib
$V_9 := F_{SUR} \cdot \sin\left(90 \cdot deg - \theta + \delta\right)$	$V_9 = 241.2\ \frac{lbf}{ft}$	Live Load Surcharge Resultant (vertical comp. - LS) - Strength Ia
$V_{10} := F_T \cdot \sin\left(90 \cdot deg - \theta + \delta\right)$	$V_{10} = 555.8\ \frac{lbf}{ft}$	Active earth force resultant (vertical component - EH)

Moment Arm:

Moment:

Moments produced from vertical loads about Point 'O'

$d_{v1} := A + \frac{2}{3} \cdot T_1 = 1.8\ ft$ $\qquad MV_1 := V_1 \cdot d_{v1} = 0\ lbf$

$d_{v2} := A + T_1 + \frac{T_t}{2} = 2.3\ ft$ $\qquad MV_2 := V_2 \cdot d_{v2} = 2902.5\ lbf$

$d_{v3} := A + T_1 + T_t + \frac{T_2}{3} = 2.8\ ft$ $\qquad MV_3 := V_3 \cdot d_{v3} = 0\ lbf$

$d_{v4} := \frac{B}{2} = 3.1\ ft$ $\qquad MV_4 := V_4 \cdot d_{v4} = 2929.7\ lbf$

$d_{v5} := B - \frac{T_2 + C}{2} = 4.5\ ft$ $\qquad MV_5 := V_5 \cdot d_{v5} = 0\ lbf$

$d_{v6} := B - \frac{C}{2} = 4.5\ ft$ $\qquad MV_6 := V_6 \cdot d_{v6} = 16254\ lbf$

$d_{v7} := A + T_1 + T_t + \left(\frac{2}{3} \cdot b_2 \cdot (h' - t)\right) = 2.8\ ft$ $\qquad MV_7 := V_7 \cdot d_{v7} = 0\ lbf$

$d_{v8} := B - \frac{T_2 + C}{2} = 4.5\ ft$ $\qquad MV_8 := V_8 \cdot d_{v8} = 3937.5\ lbf$

$d_{v9} := B = 6.3\ ft$ $\qquad MV_9 := V_9 \cdot d_{v9} = 1507.7\ lbf$

$d_{v10} := B = 6.3\ ft$ $\qquad MV_{10} := V_{10} \cdot d_{v10} = 3473.8\ lbf$

Horizontal Loads:

$H_1 := F_{SUR} \cdot \cos(90 \cdot deg - \theta + \delta)$ $\qquad H_1 = 614.3\ \frac{lbf}{ft}$ $\qquad$ Live Load Surcharge Resultant (horizontal comp. - LS)

$H_2 := F_T \cdot \cos(90 \cdot deg - \theta + \delta)$ $\qquad H_2 = 1415.4\ \frac{lbf}{ft}$ $\qquad$ Active Earth Force Resultant (horizontal comp. - EH)

Moment Arm:

Moment:

$d_{h1} := \frac{H}{2}$ $\qquad d_{h1} = 4.8\ ft$ $\qquad MH_1 := H_1 \cdot d_{h1}$ $\qquad MH_1 = 2948.7\ \frac{lbf \cdot ft}{ft}$

$d_{h2} := \frac{H}{3}$ $\qquad d_{h2} = 3.2\ ft$ $\qquad MH_2 := H_2 \cdot d_{h2}$ $\qquad MH_2 = 4529.1\ \frac{lbf \cdot ft}{ft}$

Unfactored Loads by Load Type:

$V_{DC} := V_1 + V_2 + V_3 + V_4 + V_5$ $\qquad V_{DC} = 2227.5\ \frac{lbf}{ft}$ $\qquad V_{EV} := V_6 + V_7$ $\qquad V_{EV} = 3612\ \frac{lbf}{ft}$

$V_{LS\_Ia} := V_9$ $\qquad V_{LS\_Ia} = 241.2\ \frac{lbf}{ft}$ $\qquad V_{LS\_Ib} := V_8 + V_9$ $\qquad V_{LS\_Ib} = 1116.2\ \frac{lbf}{ft}$

$V_{EH} := V_{10}$ $\qquad V_{EH} = 555.8\ \frac{lbf}{ft}$ $\qquad H_{LS} := H_1$ $\qquad H_{LS} = 614.3\ \frac{lbf}{ft}$

$H_{EH} := H_2$ $\qquad H_{EH} = 1415.4\ \frac{lbf}{ft}$

Unfactored Moments by Load Type

$M_{DC} := MV_1 + MV_2 + MV_3 + MV_4 + MV_5$ $\qquad M_{DC} = 5832.2\ \frac{lbf \cdot ft}{ft}$

$M_{EV} := MV_6 + MV_7$ $\qquad M_{EV} = 16254\ \frac{lbf \cdot ft}{ft}$

$M_{LSV\_Ia} := MV_9$ $\qquad M_{LSV\_Ia} = 1507.7\ \frac{lbf \cdot ft}{ft}$

$M_{LSV\_Ib} := MV_8 + MV_9$ $\qquad M_{LSV\_Ib} = 5445.2\ \frac{lbf \cdot ft}{ft}$

$M_{EH1} := MV_{10}$ $\qquad M_{EH1} = 3473.8\ \frac{lbf \cdot ft}{ft}$

$M_{LSH} := MH_1$ $\qquad M_{LSH} = 2948.7\ \frac{lbf \cdot ft}{ft}$

$M_{EH2} := MH_2$ $\qquad M_{EH2} = 4529.1\ \frac{lbf \cdot ft}{ft}$

Load Combination Limit States:

$\eta := 1$ LRFD Load Modifier

Strength Limit State I: EV(min) = 1.00 EV(max) = 1.35
EH(min) = 0.90 EH(max) = 1.50
LS = 1.75

Strength Limit State Ia: (Sliding and Eccentricity) $\quad Ia_{DC} := 0.9 \quad Ia_{EV} := 1 \quad Ia_{EH} := 1.5 \quad Ia_{LS} := 1.75$

Strength Limit State Ib: (Bearing Capacity) $\quad Ib_{DC} := 1.25 \quad Ib_{EV} := 1.35 \quad Ib_{EH} := 1.5 \quad Ib_{LS} := 1.75$

Factored Vertical Loads by Limit State:

$V_{Ia} := \eta \cdot \left(\left(Ia_{DC} \cdot V_{DC} \right) + \left(Ia_{EV} \cdot V_{EV} \right) + \left(Ia_{EH} \cdot V_{EH} \right) + \left(Ia_{LS} \cdot V_{LS\_Ia} \right) \right)$ $\qquad V_{Ia} = 6872.6\ \frac{lbf}{ft}$

$V_{Ib} := \eta \cdot \left(\left(Ib_{DC} \cdot V_{DC} \right) + \left(Ib_{EV} \cdot V_{EV} \right) + \left(Ib_{EH} \cdot V_{EH} \right) + \left(Ib_{LS} \cdot V_{LS\_Ib} \right) \right)$ $\qquad V_{Ib} = 10447.7\ \frac{lbf}{ft}$

Factored Horizontal Loads by Limit State:

$H_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot H_{LS} \right) + \left(Ia_{EH} \cdot H_{EH} \right) \right)$ $\qquad H_{Ia} = 3198.1\ \frac{lbf}{ft}$

$H_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot H_{LS} \right) + \left(Ib_{EH} \cdot H_{EH} \right) \right)$ $\qquad H_{Ib} = 3198.1\ \frac{lbf}{ft}$

Factored Moments Produced by Vertical Loads by Limit State:

$MV_{Ia} := \eta \cdot \left(\left(Ia_{DC} \cdot M_{DC} \right) + \left(Ia_{EV} \cdot M_{EV} \right) + \left(Ia_{EH} \cdot M_{EH1} \right) + \left(Ia_{LS} \cdot M_{LSV\_Ia} \right) \right)$ $\qquad MV_{Ia} = 29352.2\ \frac{lbf \cdot ft}{ft}$

$MV_{Ib} := \eta \cdot \left(\left(Ib_{DC} \cdot M_{DC} \right) + \left(Ib_{EV} \cdot M_{EV} \right) + \left(Ib_{EH} \cdot M_{EH1} \right) + \left(Ib_{LS} \cdot M_{LSV\_Ib} \right) \right)$ $\qquad MV_{Ib} = 43973\ \frac{lbf \cdot ft}{ft}$

Factored Moments Produced by Horizontal Loads by Limit State:

$MH_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot M_{LSH} \right) + \left(Ia_{EH} \cdot M_{EH2} \right) \right)$ $\qquad MH_{Ia} = 11953.8\ \frac{lbf \cdot ft}{ft}$

$MH_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot M_{LSH} \right) + \left(Ib_{EH} \cdot M_{EH2} \right) \right)$ $\qquad MH_{Ib} = 11953.8\ \frac{lbf \cdot ft}{ft}$

Compute Bearing Resistance:

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

$\Sigma M_R := MV_{Ib}$ $\quad \Sigma M_R = 43973\ \frac{lbf \cdot ft}{ft}$ Sum of Resisting Moments (Strength Ib)

$\Sigma M_O := MH_{Ib}$ $\quad \Sigma M_O = 11953.8\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\quad \Sigma V = 10447.7\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$x := \frac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$ $\quad x = 3.1\ ft$ Distance from Point "O" the resultant intersects the base

$e := \left|\frac{B}{2} - x\right|$ $\quad e = 0.06\ ft$ Wall eccentricity, **Note:** The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2**]. The effective bearing width is equal to B-2e. When the foundation eccentricity is negative the absolute value is used.

Foundation Layout:

$B' := B - 2 \cdot e$ $\quad B' = 6.1\ ft$ Effective Footing Width

$L' := 132\ ft$ Effective Footing Length (Assumed)

$H' := H_{Ib}$ $\quad H' = 3198.1\ \frac{lbf}{ft}$ Summation of Horizontal Loads (Strength Ib)

$V' := V_{Ib}$ $\quad V' = 10447.7\ \frac{lbf}{ft}$ Summation of Vertical Loads (Strength Ib)

$D_f = 4.2\ ft$ Footing embedment

$d_w := D_f$ Depth of Groundwater below ground surface at front of wall.

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan\left(45\ deg + \frac{\phi'_{fd}}{2}\right)^2, 1.0\right)$ $\quad N_q = 10.66$

$N_c := \text{if}\left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14\right)$ $\quad N_c = 20.72$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd})$ $\quad N_\gamma = 10.9$

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

$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L'}\right)\right)$ $\quad s_c = 1.024$

$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd})\right), 1\right)$ $\quad s_q = 1.022$

$s_\gamma := \text{if}\left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'}\right), 1\right)$ $\quad s_\gamma = 0.981$

Load inclination factors:

$i_q := 1$

$i_\gamma := 1$

$i_c := 1$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**. "Most geotechnical engineers do not 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]**:

$C_{wq} := \text{if}\left(d_w \geq D_f, 1.0, 0.5\right)$ $\quad C_{wq} = 1$

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

Depth Correction Factor per Hanson (1970):

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

$d_q = 1.21$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\quad N_{cm} = 21.216$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\quad N_{qm} = 10.893$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\quad N_{\gamma m} = 10.674$

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

$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_{fd} \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\quad q_{nd} = 12269.7 \ \frac{lbf}{ft^2}$

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

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

$q_{Rd} := \phi_b \cdot q_{nd}$ $\quad q_{Rd} = 6.7 \ ksf$ — Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

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

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

$$s_c := \text{if}\left(\phi_{fdu} > 0, 1 + \left(\frac{B'}{L'}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L'}\right)\right) \qquad s_c = 1.009$$

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

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

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].

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

$$N_{cm} := N_c \cdot s_c \cdot i_c \qquad N_{cm} = 5.188$$

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

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

Depth Correction Factor per Hanson (1970):

$$d_q := \text{if}\left(\frac{D_f}{B} \le 1, 1 + 2 \cdot \tan\left(\phi_{fdu}\right) \cdot \left(1 - \sin\left(\phi_{fdu}\right)\right)^2 \cdot \frac{D_f}{B}, 1 + 2 \cdot \tan\left(\phi_{fdu}\right) \cdot \left(1 - \sin\left(\phi_{fdu}\right)\right)^2 \cdot \text{atan}\left(\frac{D_f}{B}\right)\right)$$

$$d_q = 1$$

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

$$q_{nu} := Su_{fdu} \cdot N_{cm} + \gamma_{fd} \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \qquad q_{nu} = 12661.6 \ \frac{lbf}{ft^2}$$

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

$\phi_b := .55$

Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Ru} := \phi_b \cdot q_{nu} \qquad q_{Ru} = 7\ ksf$

Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 6.7\ ksf$

Undrained Conditions: $q_{Ru} = 7\ ksf$

Evaluate External Stability of Wall:

Compute the ultimate bearing stress :

$e = 0.06\ ft$

$\sigma_V := \dfrac{\Sigma V}{B - 2 \cdot e}$ $\quad\quad \sigma_V = 1.705\ ksf$

Bearing Capacity:Demand Ratio (CDR)

Drained Conditions: $CDR_{Bearing\_D} := \dfrac{q_{Rd}}{\sigma_V}$ Is the CDR > or = to 1.0? $CDR_{Bearing\_D} = 3.96$

Undrained Conditions: $CDR_{Bearing\_U} := \dfrac{q_{Ru}}{\sigma_V}$ Is the CDR > or = to 1.0? $CDR_{Bearing\_U} = 4.09$

Limiting Eccentricity at Base of Wall (Strength Ia):

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

$e_{max} := \dfrac{B}{3}$ $\quad e_{max} = 2.1\ ft$ $\quad$ Maximum Eccentricity **LRFD [11.6.3.3.] Equals B/3 for soil.**

$\Sigma M_R := MV_{Ia}$ $\quad \Sigma M_R = 29352.2\ \dfrac{lbf \cdot ft}{ft}$ $\quad$ Sum of Resisting Moments (Strength Ia)

$\Sigma M_O := MH_{Ia}$ $\quad \Sigma M_O = 11953.8\ \dfrac{lbf \cdot ft}{ft}$ $\quad$ Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$ $\quad \Sigma V = 6872.6\ \dfrac{lbf}{ft}$ $\quad$ Sum of Vertical Loads (Strength Ia)

$x := \dfrac{(\Sigma M_R - \Sigma M_O)}{\Sigma V}$ $\quad x = 2.5\ ft$ $\quad$ Distance from Point "O" the resultant intersects the base

$e := \text{abs}\left(\dfrac{B}{2} - x\right)$ $\quad e = 0.59\ ft$ $\quad$ 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)

$CDR_{Eccentricity} := \dfrac{e_{max}}{e}$ Is the CDR > or = to 1.0? $CDR_{Eccentricity} = 3.51$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$R_u := H_{Ia}$ $\qquad R_u = 3198.1\ \frac{lbf}{ft}$

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

$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot \left(y_2 - y_1\right)$ $\qquad R_{ep} = 2749.3\ \frac{lbf}{ft}$ 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:

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

$\Sigma V := V_{Ia}$ $\qquad \Sigma V = 6872.6\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$R_\tau := c \cdot \Sigma V \cdot \tan\left(\phi'_{fd}\right)$ $\qquad R_\tau = 3204.8\ \frac{lbf}{ft}$ Nominal sliding resistance Cohesionless Soils

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

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

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

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

Factored Sliding Resistance to be used in CDR Calculations: $R_R = 4579.417\ \frac{lbf}{ft}$

Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{R_u}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 1.43$

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

$R_{ep} := \frac{r_{ep1} + r_{ep2}}{2} \cdot \left(y_2 - y_1\right)$ $\quad R_{ep} = 5004.7\ \frac{lbf}{ft}$ — 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:

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

$\Sigma V := V_{Ia}$ $\quad \Sigma V = 6872.6\ \frac{lbf}{ft}$ — Sum of Vertical Loads (Strength Ia)

$e = 0.59\ ft$ — Wall eccentricity, Calculated in above Limiting Eccentricity at Base of Wall (Strength Ia) Section.

$B = 6.3\ ft$ — Footing base width

$\frac{B}{6} = 1\ ft$ — If e < B/6 the resultant is in the middle one-third

$\sigma_{vmax} := \frac{\Sigma V}{B} \cdot \left(1 + 6 \cdot \frac{e}{B}\right)$ $\quad \sigma_{vmax} = 1726.1\ \frac{lbf}{ft^2}$ — Max vertical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$\sigma_{vmin} := \frac{\Sigma V}{B} \cdot \left(1 - 6 \cdot \frac{e}{B}\right)$ $\quad \sigma_{vmin} = 473.2\ \frac{lbf}{ft^2}$ — Max verical stress (if resultant is in the middle one-third of base) **LRFD [11.6.3.2-2]**.

$q_{max} := \frac{1}{2} \cdot \sigma_{vmax}$ $\quad q_{max} = 863\ \frac{lbf}{ft^2}$ — Max unit shear resistance as 1/2 max vertical stress **LRFD [10.6.3.4]**.

$q_{min} := \frac{1}{2} \cdot \sigma_{vmin}$ $\quad q_{min} = 236.6\ \frac{lbf}{ft^2}$ — Minimum unit shear resistance as 1/2 minimum vertical stress **LRFD [10.6.3.4]**.

Determine which Cohesive Soil Resistance Case is Present:

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

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

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

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

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

Unit Shear Resistance for Case 1:

$$S_1 := Su_{fdu} - q_{min} = 2113.4\ \frac{lbf}{ft^2} \qquad S_2 := q_{min} = 236.6\ \frac{lbf}{ft^2}$$

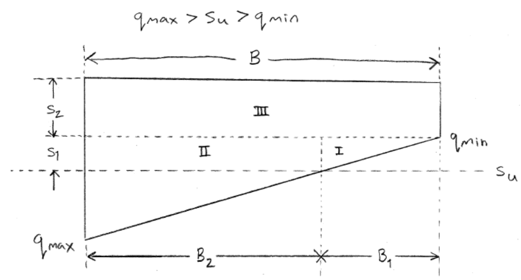
$$B_1 := \frac{B \cdot (Su_{fdu} - q_{min})}{q_{max} - q_{min}} = 21.1\ ft \qquad B_2 := \frac{B \cdot (q_{max} - Su_{fdu})}{q_{max} - q_{min}} = -14.8\ ft$$

$$B_3 := B = 6.3\ ft$$

$$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 22280.4\ \frac{lbf}{ft} \qquad II := S_1 \cdot B_2 = -31351.9\ \frac{lbf}{ft}$$

$$III := S_2 \cdot B_3 = 1478.6\ \frac{lbf}{ft}$$

$$R_{\tau\_case1} := I + II + III = -7592.9\ \frac{lbf}{ft}$$



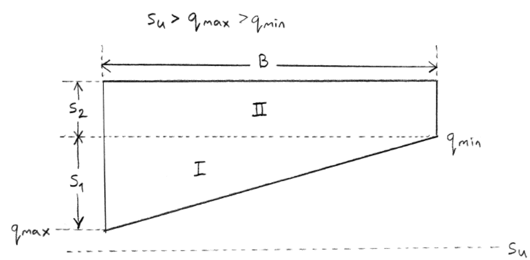
Unit Shear Resistance for Case 2:

$$S_1 := q_{max} - q_{min} = 626.5\ \frac{lbf}{ft^2} \qquad S_2 := q_{min} = 236.6\ \frac{lbf}{ft^2}$$

$$B = 6.3\ ft$$

$$I := \frac{1}{2} \cdot S_1 \cdot B = 1957.7\ \frac{lbf}{ft} \qquad II := S_2 \cdot B = 1478.6\ \frac{lbf}{ft}$$

$$R_{\tau\_case2} := I + II = 3436.3\ \frac{lbf}{ft}$$



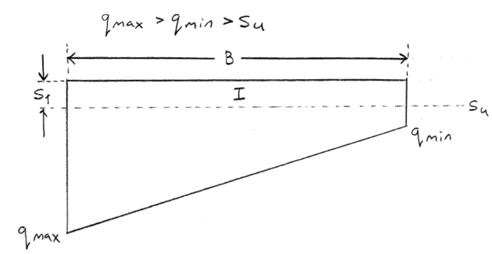
Unit Shear Resistance for Case 3:

$$S_1 := Su_{fdu} = 2350\ \frac{lbf}{ft^2}$$

$$B = 6.3\ ft$$

$$I := S_1 \cdot B = 14687.5\ \frac{lbf}{ft}$$

$$R_{\tau\_case3} := I = 14687.5\ \frac{lbf}{ft}$$



Unit Shear Resistance for Case 4:

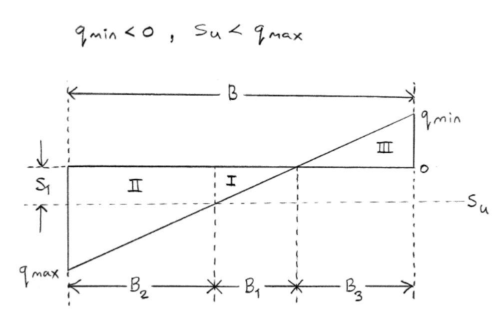
$$S_1 := Su_{fdu} = 2350\ \frac{lbf}{ft^2}$$

$$B_3 := \frac{B \cdot (-q_{min})}{q_{max} - q_{min}} = -2.4\ ft \qquad B_1 := \left(\frac{Su_{fdu}}{q_{max}}\right) \cdot (B - B_3) = 23.4\ ft$$

$$B_2 := B - (B_1 + B_3) = -14.8\ ft$$

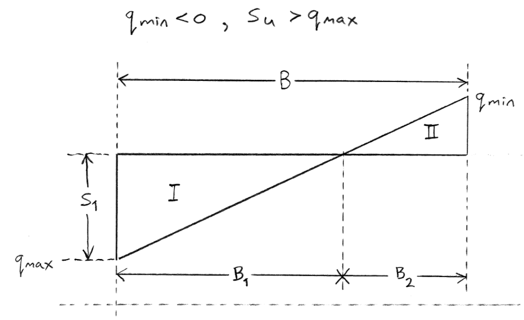
$$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 27547.7\ \frac{lbf}{ft} \qquad II := S_1 \cdot B_2 = -34861.4\ \frac{lbf}{ft}$$

$$R_{\tau\_case4} := I + II = -7313.7\ \frac{lbf}{ft}$$



Unit Shear Resistance for Case 5:

$S_1 := q_{max} = 863 \frac{lbf}{ft^2}$



$B_1 := \frac{B \cdot q_{max}}{q_{max} - q_{min}} = 8.6\ ft$

$B_2 := B - B_1 = -2.4\ ft$

$I := \frac{1}{2} \cdot S_1 \cdot B_1 = 3715.5 \frac{lbf}{ft}$

$R_{\tau\_case5} := I = 3715.5 \frac{lbf}{ft}$

Define the Applicable Case:

$R_\tau := R_{\tau\_case2}$

$R_\tau = 3436.3 \frac{lbf}{ft}$

Nominal sliding resistance Cohesive Soils

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

$\phi_{ep} := 0.5$

Resistance factor for passive resistance specified in **LRFD Table 10.5.5.2.2-1**

$\phi_\tau := 1.0$

Resistance factor for sliding resistance specified in **LRFD Table 11.5.7-1.**

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

$R_R := \phi R_n$

Factored Sliding Resistance to be used in CDR Calculations: $R_R = 5938.642 \frac{lbf}{ft}$

Sliding Capacity:Demand Ratio (CDR)

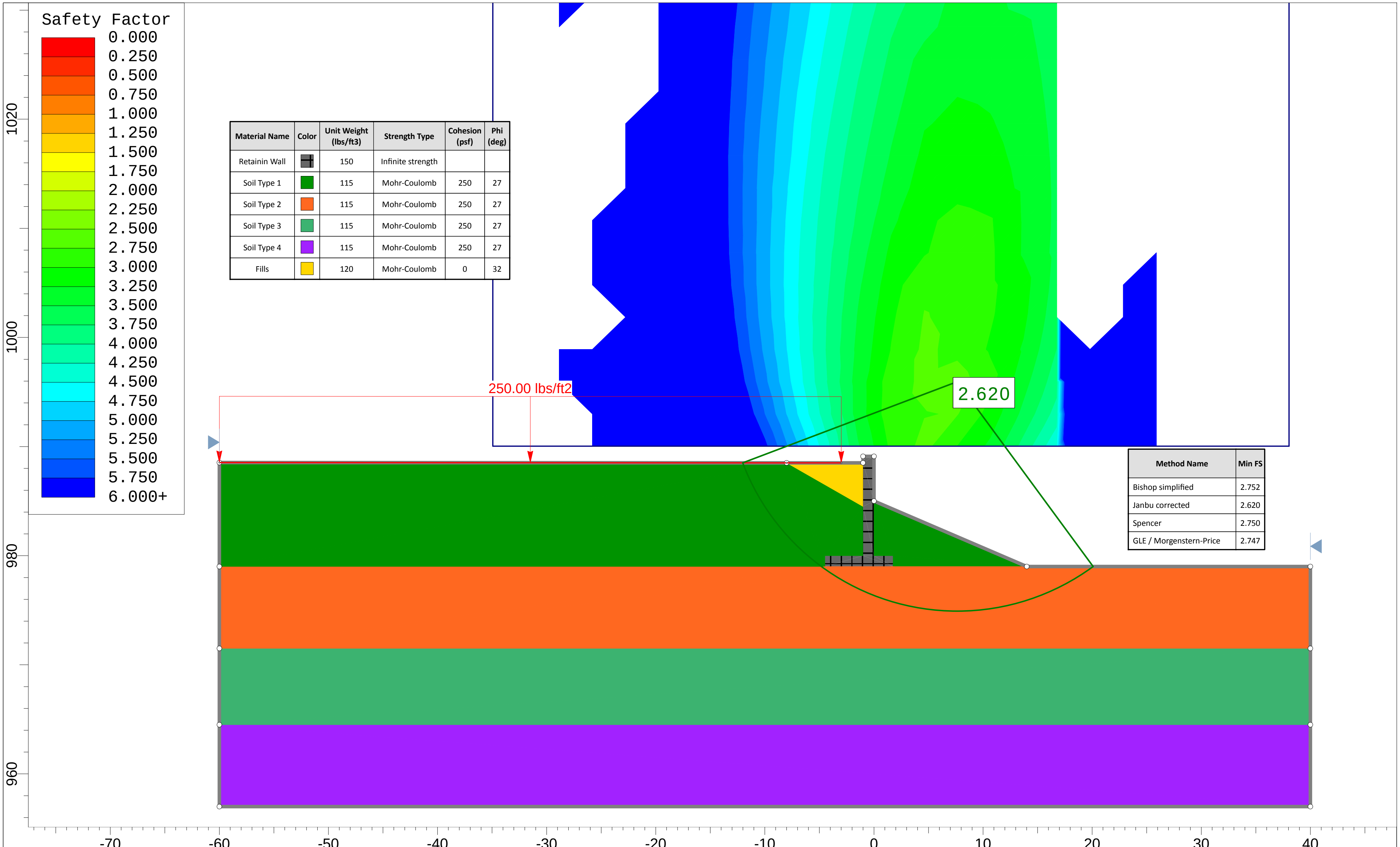
$CDR_{Sliding} := \frac{R_R}{R_u}$

Is the CDR > or = to 1.0?

$CDR_{Sliding} = 1.86$

APPENDIX D

GLOBAL STABILITY ANALYSIS

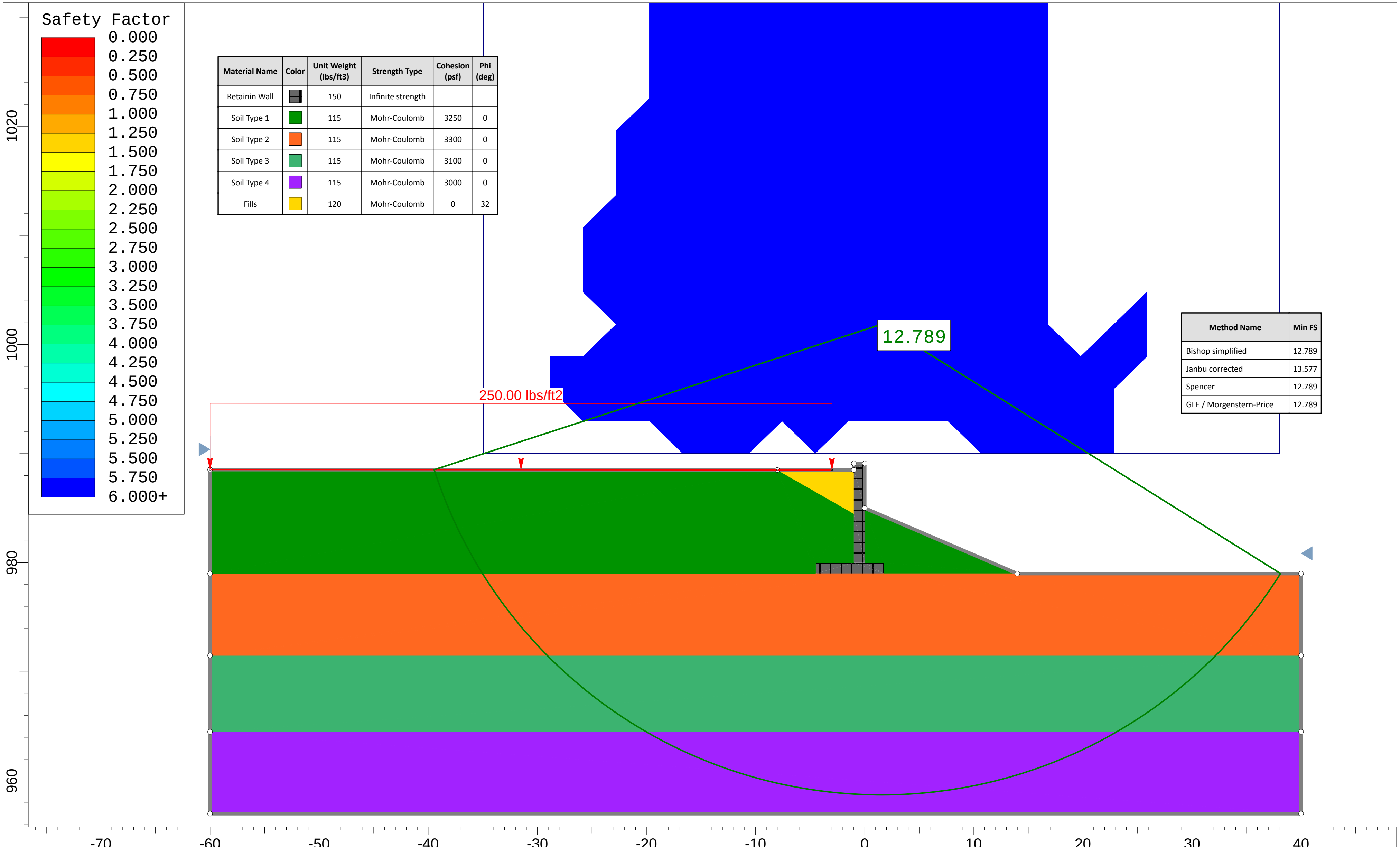


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Retainin Wall		150	Infinite strength		
Soil Type 1		115	Mohr-Coulomb	250	27
Soil Type 2		115	Mohr-Coulomb	250	27
Soil Type 3		115	Mohr-Coulomb	250	27
Soil Type 4		115	Mohr-Coulomb	250	27
Fills		120	Mohr-Coulomb	0	32

Method Name	Min FS
Bishop simplified	2.752
Janbu corrected	2.620
Spencer	2.750
GLE / Morgenstern-Price	2.747

Project	Foote Road Retaining Wall		
Analysis Description	Retaining Wall 5		
Drawn By	ZM	Scale	1:146
Company	NEAS, Inc.		
Date	06/07/2020	File Name	RW5_STA.21+44.68_Effective_B-039_1.slim

SLIDEINTERPRET 7.038



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Retainin Wall		150	Infinite strength		
Soil Type 1		115	Mohr-Coulomb	3250	0
Soil Type 2		115	Mohr-Coulomb	3300	0
Soil Type 3		115	Mohr-Coulomb	3100	0
Soil Type 4		115	Mohr-Coulomb	3000	0
Fills		120	Mohr-Coulomb	0	32

Method Name	Min FS
Bishop simplified	12.789
Janbu corrected	13.577
Spencer	12.789
GLE / Morgenstern-Price	12.789

rocscience

SLIDEINTERPRET 7.038

Project	Foote Road Retaining Wall
Analysis Description	Retaining Wall 5
Drawn By	ZM
Scale	1:146
Company	NEAS, Inc.
Date	06/07/2020
File Name	RW5_STA.21+44.68_Total_B-039_1.slim

APPENDIX E

SEISMIC PARAMETERS

ATC Hazards by Location

Search Information

Coordinates: 41.1393971, -81.8314039

Elevation: 988 ft

Timestamp: 2020-04-30T18:33:16.100Z

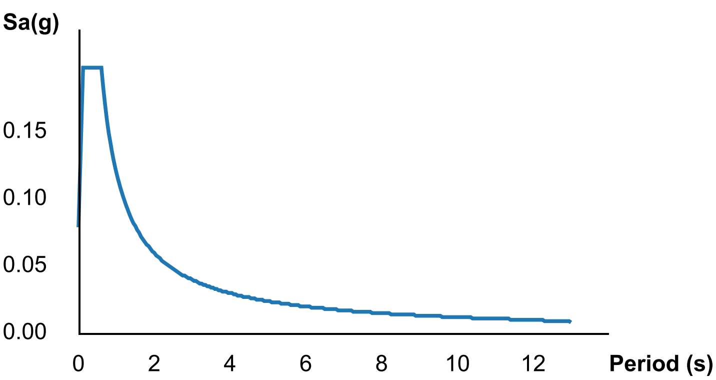
Hazard Type: Seismic

Reference Document: ASCE7-16

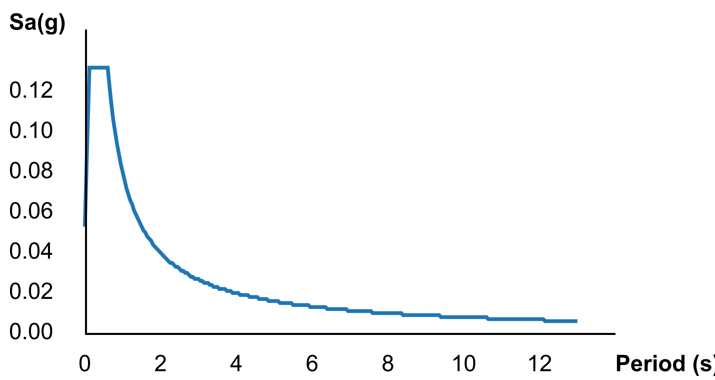
Risk Category: II

Site Class: D-default

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.124	MCE_R ground motion (period=0.2s)
S_1	0.05	MCE_R ground motion (period=1.0s)
S_{MS}	0.198	Site-modified spectral acceleration value
S_{M1}	0.12	Site-modified spectral acceleration value
S_{DS}	0.132	Numeric seismic design value at 0.2s SA
S_{D1}	0.08	Numeric seismic design value at 1.0s SA

▾Additional Information

Name	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2s
F_v	2.4	Site amplification factor at 1.0s
CR_S	0.949	Coefficient of risk (0.2s)

CR_1	0.915	Coefficient of risk (1.0s)
PGA	0.064	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.103	Site modified peak ground acceleration
T_L	12	Long-period transition period (s)
SsRT	0.124	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.13	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.05	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.054	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.