REPORT
STRUCTURE FOUNDATION EXPLORATION
HUR-CR11-2.96 - BASELINE ROAD/BROADWAY ST
PID: 120504
HURON COUNTY, OHIO

Prepared For:

Richland Engineering Ltd.
29 North Park Street
Mansfield, OH 44902

Prepared by:

NATIONAL ENGINEERING AND ARCHITECTURAL SERVICES INC.
2800 Corporate Exchange Drive, Suite 240
Columbus, Ohio 43231

NEAS PROJECT 24-0031

January 19, 2026

National Engineering & Architectural Services Inc.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF APPENDICES

1. INTRODUCTION

1.1. General

National Engineering and Architectural Services Inc. (NEAS) presents our Structure Foundation Exploration Report for the proposed Baseline Road (Rd)/Broadway Street (St) bridge replacement project (HUR-CR11-2.96 Baseline Rd/Broadway St Bridge). The referenced bridge is located in the Village of Plymouth, Huron County, Ohio and carries Baseline Rd/Broadway St (CR11) over Ashland Railway. As part of the proposed project, the existing bridge will be replaced with a new structure and associated approach pavement. The report presents a summary of the encountered surficial and subsurface conditions as well as our recommendations for bridge foundation design and construction in accordance with AASHTO's Publication LRFD Bridge Design Specifications, 9th Edition (BDS) (AASHTO, 2020) and ODOT's 2020 LRFD Bridge Design Manual (BDM) (ODOT, 2025) and ODOT’s Geotechnical Design Manual (GDM) (ODOT [2], 2025).

The exploration was conducted in general accordance with NEAS, Inc.’s proposal to Wallace & Pancher, Inc. dated February 21, 2024, and with the provisions of ODOT’s *Specifications for Geotechnical Explorations* (SGE) (ODOT, 2024).

The scope of work performed included: 1) a review of published geotechnical information; 2) performing a total of 2 test borings; 3) laboratory testing of soil samples in accordance with the SGE; 4) performing geotechnical engineering analysis to assess foundation design and construction considerations; and 5) development of this summary report.

1.2. Proposed Construction

The existing bridge consists of a three-span, wooden beam bridge with a corrugated metal deck supported on timber piers and abutments originally built in 1950 and later rehabilitated in 1994. The referenced bridge over Ashland Railway is 58 ft in length (abutment to abutment) with an approximate roadway width of 32 ft (curb to curb) and 2.5-ft sidewalks on either side of the roadway.

Based on the proposed project Stage 2 plans prepared by Richland Engineering Ltd. (REL), dated August 25, 2025, the existing bridge is planned to be replaced with a three-span, concrete slab bridge on reinforced concrete semi-integral abutments with turned back mechanically stabilized earth (MSE) walls. The proposed structure will be 65 ft in length (abutment to abutment) with an approximate roadway width of 32 ft (curb to curb) and 6-ft sidewalks on either side of the roadway. The proposed substructures will be supported by a deep foundation system utilizing HP10x42 steel H-piles driven to refusal on bedrock.

2. GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1. Geology and Physiography

The project site is located on the boundary of two separate physiographic regions consisting of both the Central Ohio Clayey Till Plain on the western side of the project and the Galion Glaciated Low Plateau on the eastern side of the project. The Central Ohio Clayey Till Plain region is characterized as well-defined moraines with intervening flat-lying ground moraine and intermorainal lake basins. The region consists of clayey till at the surface and contains few large streams and limited sand and gravel outwash. Elevations of the region range from 700 to 1,150 ft above mean sea level (amsl), with moderate relief (100 ft). The geology within this region is described as clayey, high-lime Wisconsinan-age till and lacustrine materials over Lower Paleozoic-age carbonate rocks (i.e., limestone or dolostone) and, in the east, shales. Loess in this region is thin to absent. The Galion Glaciated Low Plateau region is characterized as rolling upland mantled with thin to thick drift and is transitional between the gently rolling Till Plain and the hilly Glaciated Allegheny Plateau with the overall area ranging in elevation from 800 ft to 1400 ft amsl, with moderate relief (100 ft). The geology is described as medium- to low-lime Wisconsinan-age till over Mississippian-age shales and sandstone (ODGS, 1998).

The geology at the bridge site is mapped as an average of 10 ft of Wisconsinan-age silt, present in some areas within this region, underlain by an average of 60 ft of Wisconsinan-age till atop and average of 20 ft of Wisconsinan-age sand and gravel all over variable sequences of sandstone, conglomerate, shale and siltstone varying in geological age (USGS & ODGS, 2008). Small areas of organic deposits were also mapped near the project site. The silt, which is noted as having a patchy or discontinuous distribution within this specific region, is present throughout the area as lowland surface deposits, terraces and thick, deltaic deposits of proglacial lakes and is characterized as commonly containing thin sand partings as well as possible localized clay, sand or gravel layers. The till is described as an unsorted mix of clay, silt, sand, gravel and boulders which may contain silt, sand and gravel lenses. The sand and gravel geologic unit is described as intermixed and interbedded sand and gravel commonly containing thin, discontinuous layers of silt, clay and till. The sand and gravel is characterized as having well to moderately sorted grain sizes that are moderately to well rounded, is finely to massively stratified and may locally contain organics.

Based on the Bedrock Geologic Units Map of Ohio (USGS & ODGS, 2006), bedrock within the project area consists of carbonaceous Shale of the Sunbury Shale group formation. This formation is comprised of Mississippian-age shale that is described as black to brownish-black and very thinly laminated. Based on the ODNR bedrock topography map of Ohio, bedrock elevations at the project site can be expected to be between 920 and 940 ft amsl, putting bedrock at a depth between 85 and 105 ft below ground surface (bgs).

The soils at the project site have been mapped (Web Soil Survey) by the Natural Resources Conservation Service (USDA, 2024) as Bennington silt loam (Huron and Richland County). The Bennington series consists of very deep, somewhat poorly drained soils formed in loamy till of medium lime content on ground moraines and end moraines. Bennington silt loam mapped at the bridge site is comprised of predominantly fine-grained soils classified as A-4, A-6 and A-7 type soils according to the AASHTO method of soil classification.

2.2. Hydrology/Hydrogeology

Groundwater at the project site can be expected at an elevation consistent with that of the nearby West Branch Huron River as it is the most dominant hydraulic influence in the vicinity of the project's boundaries. The water level of West Branch Huron River may be generally representative of the local groundwater table. However, it should be noted that perched groundwater systems may be existent in areas due to the presence of fine-grained soils making it difficult for groundwater to permeate to the phreatic surface.

The project 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, 2024).

2.3. Mining and Oil/Gas Production

No mines were noted on ODNR’s Abandoned Underground Mine Locator in the vicinity of the project site. (ODNR [1], 2024).

No oil or gas wells were noted on ODNR’s Oil and Gas Well Locator in the vicinity of the project site (ODNR [1], 2024).

2.4. Historical Records and Previous Phases of Project Exploration

A historic record search was performed through ODOT's Transportation Information Management System (TIMS). No historical borings were used in our analysis for this report.

2.5. Field Reconnaissance

A field reconnaissance visit for the Baseline Rd/Broadway St Bridge Replacement project was conducted on May 22, 2024, during which site conditions were noted and photographed. During our field reconnaissance, no geohazards were observed within the immediate vicinity of the project site. Land use immediately surrounding the bridge at this location can be described as predominantly wooded and railroad right-of-way. Residential property is located east of the bridge while institutional property consisting of a community center and a church are located west of the bridge. A summary of our site observations of the proposed bridge replacement site is provided below.

The existing bridge carrying Baseline Rd/Broadway St over the Ashland Railway is a three-span, wooden beam bridge on timber abutments and piers. Existing slopes including the existing spill-through slopes appeared to be stable and in good condition with the estimated inclinations from about 1.5 Horizontal to 1 Vertical (1.5H:1V) to 2H:1V. The slopes were heavily vegetated except for the spill through slopes (Photograph 1).

Overall, the bridge appeared to be in fair to poor condition with some minor to moderate structural distress observed. Some corrosion of the underside of the bridge deck was observed during the visit as well of some deterioration of the wooden beams (Photograph 2). No apparent signs of structural distress due to geotechnical concerns were observed during our field reconnaissance visit. Overall, the pavement at the site was observed to be in poor to fair condition with frequent low severity transverse cracking, longitudinal cracking and crack-seal deficiency as well as occasional moderate to high severity map cracking and pothole patching (Photograph 3). The bridge site appeared to be generally well drained, with the exception of standing water that was observed within the drainage swales on either side of the railroad at the base of the existing slopes (Photograph 1).

Photograph 1: Embankment slope and standing water

Photograph 2: Corrosion underside of bridge deck and deteriorated wooden beams

Photograph 3: Pavement condition along bridge deck

3. GEOTECHNICAL EXPLORATION

3.1. Field Exploration Program

The subsurface exploration of the project was conducted by NEAS between May 22, 2024, and June 20, 2024, and included 2 borings drilled to depths ranging from 91.5 ft to 97.0 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 within the planned project construction areas that were not restricted by underground utilities or dictated by terrain (e.g. steep embankment slopes). Project boring locations were located in the field after drilling by the project surveyor. Each individual project boring log (included within Appendix B) includes the recorded boring latitude and longitude location and the corresponding ground surface elevation (surveyed by the project surveyor). The boring locations are depicted on the Boring Plan provided in Appendix A. Latitude/Longitude, elevations and stationing and offsets of the borings are shown on Table 1 below.

Table 1: Project Boring Summary

Boring Number	Location (Sta/offset)(1)	Latitude	Longitude	Elevation (NAVD 88) (ft)	Depth (ft)	Structure
B-001-0-24	156+23, 7' LT.	40.995287	-82.668212	1023.1	97.0	Rear Abutment
B-002-0-24	157+49, 17' RT.	40.995217	-82.667756	1024.3	91.5	Forward Abutment

Notes:

1. *Stationing and Offset are in reference to centerline of Proposed Baseline Road.*

Project borings were drilled using a CME 75T truck-mounted drilling rig utilizing 3.25-inch (inner diameter) hollow stem auger. In general, soil samples were recovered at 2.5 ft interval to 40 ft bgs and at 5.0 ft thereafter, to the depth at which bedrock was encountered. Soil samples were collected using an 18-inch split spoon sampler (AASHTO T-206 "Standard Method for Penetration Test and Split Barrel Sampling of Soils."). The soil samples obtained from the exploration program were visually observed in the field by the NEAS field representative and preserved for review by a Geologist for possible laboratory testing. Standard penetration tests (SPT) were conducted using a CME auto hammer calibrated to be 84% efficient on March 8, 2024, 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 bentonite grout and patched with cold patch asphalt and/or quickset concrete where necessary and appropriate.

3.2. Laboratory Testing Program

The laboratory testing program consisted of classification testing, moisture content determinations and unconfined compressive strength of rock testing. Data from the laboratory testing program was incorporated onto the boring logs (Appendix B).

3.2.1. Classification Testing

Representative soil samples were selected for index properties (Atterberg Limits) and gradation testing for classification purposes on approximately 33% of the samples. At each boring location, samples were selected for testing with the intent of identification and classification of all significant soil units. Soils not selected for testing were compared to laboratory tested samples/strata and classified visually. Moisture content testing was conducted on all samples. The laboratory testing was performed in general accordance with applicable AASHTO specifications.

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

3.2.2. Standard Penetration Test Results

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

3.2.3. Unconfined Compressive Strength of Rock Testing

Unconfined Compressive Strength of Intact Rock Core tests were conducted in accordance with ASTM D 7012 "Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures" (Method C) on a rock core sample obtained from boring B-001-0-22. In general, the rock is classified as gray, slightly weathered, weak shale. The Unconfined Compressive Strength of a Rock Core Test results are summarized in Table 2 below and testing reports are provided in Appendix B.

Table 2: Bedrock Laboratory Testing Summary

Boring Number	Depth of Test Specimen (ft)	Rock Description	Moisture Content (%)	Dry Unit Weight (pcf)	Unconfined Compressive Strength (psi)	Strain (%)
B-001-0-22	92.9 - 93.2	Gray Shale	3.1	154.1	1,428	2.6

4. GEOTECHNICAL FINDINGS

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

4.1. Subsurface Conditions

The general subsurface profile is relatively uniform and consistent with the geological model for the project. The subsurface profile at the site of proposed bridge generally consists of surficial materials (i.e., pavement section) underlain by natural till soils followed by sands and gravels all over shale bedrock. The natural till encountered at the site can generally be described as medium stiff to hard fine-grained, cohesive soils except for occasional discontinuous strata of coarse-grained material. The natural sand and gravel encountered overlying bedrock at the bridge site can generally be described as dense to very dense coarse-grained material. These materials and the general profile are further described below.

4.1.1. Overburden Soil

Natural glacial till soils were encountered immediately below the pavement section in each of the borings performed at the bridge site and extended to a depth of 68.3 ft bgs (approximate elevations 654.8 to 656.0 ft amsl). Based on laboratory testing results, a visual review of the soil samples obtained, these soils are comprised of fine-grained, cohesive material and are classified on the boring logs as Sandy Silt (A-4a), Silt (A-4b), Silt and Clay (A-6a) and Silty Clay (A-6b). The exception being various strata encountered that consisted of coarse-grained material that classified on the logs as Gravel with Sand (A-1-b), Gravel with Sand and Silt (A-2-4), Gravel with Sand, Silt and Clay (A-2-6) and Coarse and Fine Sand (A-3a). With respect to the soil strength of the fine-grained till encountered, these soils can be described as having a consistency of medium stiff to hard correlating to converted SPT-N values (N_{60}) values between 7 and 35 blows per foot (bpf) and unconfined compressive strengths (estimated by means of hand penetrometer) between 1.5 and 4.5 tons per square foot (tsf). Natural moisture contents of the till ranged from 14 to 25 percent. Based on Atterberg Limits tests performed on representative samples of the till, the liquid and plastic limits ranged from 23 to 37 percent and from 15 to 21 percent, respectively. The layers of the coarse-grained material were at depths between 17.0 and 32.0 ft bgs (approximate elevations 991.1 and 1006.1 ft amsl) had thicknesses ranging from 7.5-ft to 11.3-ft. With respect to the soil strength of the coarse-grained till layers encountered within this stratum, these soils can be described as having a relative compactness of very loose to medium dense correlating to converted N_{60} values between 1 and 17 bpf. Natural moisture contents of the granular till ranged from 12 to 25 percent.

The soils encountered directly underlying the natural till soils consisted of sand and gravel which extended to bedrock at depths between 81.0 and 86.8 ft bgs (approximate elevations 936.3 and 943.3 ft amsl). Based on laboratory testing results, a visual review of the soil samples obtained, these soils are comprised of granular material and are classified on the boring logs as Gravel with Sand (A-1-b), Gravel and Stone Fragments with Sand, Silt and Clay (A-2-6) and Coarse and Fine Sand (A-3a). With respect to the soil strength, the natural sand and gravel can be described having a relative compactness of medium dense to very dense correlating to converted N_{60} values ranged from 11 to SPT-N refusal (i.e., less than 6 inches of penetration over 50 blows). Natural moisture contents of the sand and gravel ranged from 9 to 18 percent.

4.1.2. Groundwater

Groundwater measurements were taken during the drilling process and/or immediately after the completion of each borehole. Groundwater was encountered in one of the two borings performed at the bridge site. Groundwater was encountered at a depth of 20.0 ft bgs (elevation 1003.1 ft amsl) during drilling in boring B-001-0-22. It should be noted that groundwater is affected by many hydrologic characteristics in the area and may vary from those measured at the time of the exploration. The specific groundwater readings are included on the boring logs located within Appendix B.

4.1.3. Bedrock

Bedrock was encountered in each of the borings performed at the bridge replacement site and was classified as interbedded shale and sandstone with the predominant rock type of shale. The bedrock surface was present at a depth of 86.8 ft bgs (elevation 936.3 ft amsl) in B-001-0-22 and at a depth of 81.0 ft bgs (elevation 943.3 ft amsl) in B-002-0-22. Based on the elevation at which bedrock was encountered in the borings, it appears that the bedrock surface gradually slopes downward from east to west.

Rock coring was performed at each boring location once auger refusal was encountered. Recovery of the bedrock core samples ranged from 80 to 96 percent while rock quality designation (RQD) values ranged from 37 to 46 percent. A summary of the bedrock data is presented below in Table 3.

Table 3: Bedrock Summary

Boring Number	Depth to Bedrock (ft)	Depth to Top of Core Sample (ft)	Elevation of Top of Core Sample (ft)	Bedrock Recovery (%)	Bedrock RQD (%)
B-001-0-22	86.8	87.0	936.1	80	37
B-002-0-22	81.0	81.5	942.8	96	46

5. ANALYSES AND RECOMMENDATIONS

We understand that the existing 58-ft long, three-span bridge designated as HUR-CR11-0296 is proposed to be replaced with a new structure. Based on the proposed project Stage 1 plans prepared by REL, dated May 19, 2025, the existing bridge is planned to be replaced with a three-span, concrete slab bridge on reinforced concrete semi-integral abutments with turned back mechanically stabilized earth (MSE) walls. The proposed structure is planned to be supported by the natural soils through the use of a deep foundation system consisting of HP10x42 steel H-piles driven to refusal on bedrock. For analysis purposes, it has been assumed that: 1) the bottom of pile cap at the rear and forward abutments is at an elevation of 1016.5 and 1017.3 ft amsl, respectively, and is the elevation at which the piles will be driven; 2) the bottom of pile cap at the pier locations are at an elevation of 1000.0 ft amsl; and, 3) groundwater elevation is at the pile driving elevation.

The deep pile foundation will be designed according to LRFD BDS (AASHTO, 2020) and ODOT BDM (ODOT, 2025) criteria. The computer program *GRLWEAP 14* developed by GRL Engineers, Inc., was used to evaluate pile drivability and to estimate whether HP10x42 steel H-piles can be driven to the anticipated bedrock elevation without overstressing or encountering practical refusal within the overburden soils prior to bedrock. Input information for the *GRLWEAP 14* program was based on the soil characteristics gathered during the geotechnical exploration (i.e., SPT results, laboratory test results, etc.) and our geotechnical experience. Tables 3 and 4 in Section 5.1. of this report presents each soil strata and their engineering properties that were used in the review. Based on our evaluation of the subsurface conditions and our geotechnical engineering analysis of the proposed bridge replacement project, it is our opinion that the bridge foundations can be supported on HP10x42 driven to bedrock refusal as indicated in Section 5.2.1. of this report.

5.1. Soil Profile for Analysis

For analysis purposes, each substructure location (boring log) was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were estimated based on their field (i.e., SPT N_{60} Values, hand penetrometer values, etc.) and laboratory (i.e., Atterberg Limits, grain size, etc.) test results using correlations provided in published engineering manuals, research reports and guidance documents. The developed soil profile and estimated engineering soil properties for use in analysis (with cited correlation/reference material) is summarized per boring within Tables 3 and 4 below.

Table 4: Soil Profile and Estimated Soil Parameters – B-001-0-22

Bridge HUR-CR11-0296 over Ashland Railway: Rear Abutment / Pier 1, B-001-0-22					
Soil Description	**Unit Weight (pcf)**	**Undrained Shear Strength (psf)**	**Effective Cohesion (psf)**	**Effective Friction Angle (degrees)**	**Setup Factor (f_{su})**
Sandy Silt Elevation (1023.1 ft - 1006.1 ft)	120	1400	140	24	1.5
Gravel with Sand, Silt and Clay Elevation (1006.1 ft - 1003.6 ft)	110	-	-	31	1.2
Gravel with Sand and Silt Elevation (1003.6 ft - 996.1 ft)	120	-	-	31	1.2
Sandy Silt Elevation (996.1 ft - 991.1 ft)	120	1500	150	24	1.5
Coarse and Fine Sand Elevation (991.1 ft - 983.6 ft)	118	-	-	29	1.0
Gravel with Sand and Silt Elevation (983.6 ft - 979.8 ft)	122	-	-	33	1.2
Sandy Silt Elevation (979.8 ft - 974.8 ft)	122	2250	210	26	1.5
Silt Elevation (974.8 ft - 954.8 ft)	122	2300	220	26	1.5
Coarse and Fine Sand Elevation (954.8 ft - 944.8 ft)	128	-	-	35	1.0
Gravel with Sand Elevation (944.8 ft - 936.3 ft)	135	-	-	38	1.0
lues calculated per ODOT GDM Section 404/1304 and/or ODOT BDM Table 305-2.					

Table 5: Soil Profile and Estimated Soil Parameters – B-002-0-22

Bridge HUR-CR11-0296 over Ashland Railway: Forward Abutment / Pier 2, B-002-0-22					
Soil Description	**Unit Weight (pcf)**	**Undrained Shear Strength (psf)**	**Effective Cohesion (psf)**	**Effective Friction Angle (degrees)**	**Setup Factor (f_{su})**
Silt and Clay Depth (1024.3 ft - 1017.3 ft)	108	1100	110	22	1.5
Silt and Clay Depth (1017.3 ft - 1012.3 ft)	125	3500	280	26	1.5
Silty Clay Depth (1012.3 ft - 1004.8 ft)	125	3050	260	25	1.75
Silt and Clay Depth (1004.8 ft - 997.3 ft)	122	1900	190	24	1.5
Coarse and Fine Sand Depth (997.3 ft - 994.3 ft)	115	-	-	26	1.0
Coarse and Fine Sand Depth (994.3 ft - 989.8 ft)	125	-	-	33	1.0
Sandy Silt Depth (989.8 ft - 981 ft)	122	1800	180	25	1.5
Silt Depth (981 ft - 961 ft)	125	3000	255	26	1.5
Sandy Silt Elevation (961 ft - 956 ft)	125	3000	255	26	1.5
Gravel with Sand Elevation (956 ft - 951 ft)	122	-	-	33	1.0
Coarse and Fine Sand Elevation (951 ft - 946 ft)	128	-	-	34	1.0
Gravel with Sand, Silt and Clay Elevation (946 ft - 943.3 ft)	135	-	-	39	1.2
Values calculated per ODOT GDM Section 404/1304 and/or ODOT BDM Table 305-2.					

5.2. Bridge Foundation Analysis and Recommendations

5.2.1. Pile Drivability Analysis

Based on the determined soil profile and our estimated engineering soil properties, a wave equation drivability analysis was performed per ODOT BDM Section 305.3.1.2 utilizing the computer program *GRLWEAP 14* developed by GRL Engineers, Inc. (*GRLWEAP 14* results can be found in Appendix C). NEAS's pile drivability evaluation estimated a Delmag D19-42 diesel hammer to determine whether a HP10x42 steel H-pile would be either overstressed (i.e., compressive stresses experienced by pile during driving are greater than 90% of the yield strength of the steel) or encounter practical refusal (i.e., hammer blow counts higher than 100 blows per foot) at any time during pile installation prior to reaching the 'geotechnical pile length'. For the purposes of this report and our analysis the term 'geotechnical pile length' has been assumed to represent the length of pile from bottom of pile cap (assumed pile cap bearing elevation) to the elevation at which bedrock refusal is achieved.

The results of the evaluation indicated that the referenced steel H-pile size would not be overstressed or encounter practical refusal during the pile installation process at the proposed substructure locations based on: 1) the indicated steel section; 2) a pile hammer with a minimum rated energy of 42,000 ft-lbs; 3) the use of ASTM A572 Grade 50 steel piles; and, 4) the provided Soil Profile and Estimated Engineering Properties presented in Section 5.1 of this report. It should be noted; however, that driveability is difficult to assess quantitatively as the field test results (i.e., field SPT-N values, pocket penetrometer values, etc.), subsequent strength parameters of glacial till as well as weathered shale tend to be very high; therefore, the contractor should provide an analysis to demonstrate that the equipment planned for use is capable of pile installation without over-stressing the pile.

5.2.2. Pile Foundation Recommendations

We recommend that a driven pile foundation be used for support of the proposed bridge substructures, with the piles consisting of HP10X42 steel H-piles driven to bedrock refusal. Refusal is met during driving when the pile penetration is an inch or less after receiving at least 20 blows from the pile hammer. The factored resistance for piles driven to refusal on bedrock is typically governed by structural resistance as opposed to driving resistance for friction piles; therefore, the maximum total factored load for any single HP10x42 steel H-pile shall not exceed 310 kips (maximum factored structural resistance) (ODOT, 2025). This total factored load (single pile) may be used to support the proposed abutments under the following conditions: 1) piles are installed in accordance with Sections 507 and 523 of the ODOT's CMS; 2) the piles are axially loaded pile with negligible moment; 3) steel piles have a yield strength of 50 kips per square inch (ksi); 4) assumed no appreciable loss of section due to deterioration throughout the life of the structure; 5) steel H-piles are assumed to be subject to damage sue to severe driving conditions equating to a structural resistance factor of 0.5 and, 6) the piles are fully braced along their length.

Driven to bedrock refusal, pile tip elevations are estimated to range from 936.3 to 943.3 ft amsl. Pile lengths based on: 1) our Pile Drivability Analysis (presented in Section 5.2.1); and, 2) the "Estimated Length" and "Order Length" definitions and formulas presented in Section 305.3.5.2 of the BDM, are shown in Table 6.

Table 6: Estimated Pile Lengths

Pile Type	Bottom of Pile Cap Elevation (ft amsl)	Geotechnical Pile Length (ft)	Geotechnical Pile Tip Elevation (ft amsl)	Estimated Pile Length[(1)] (ft)	Order Length[(1)] (ft)
BUT-CR11-0296 Baseline Rd/Broadway St - Rear Abutment, B-001-0-22					
HP10x42	1016.5	80.2	936.3	85	90
BUT-CR11-0296 Baseline Rd/Broadway St - Pier 1, B-001-0-22					
HP10x42	1000.0	63.7	936.3	70	75
BUT-CR11-0296 Baseline Rd/Broadway St - Pier 2, B-002-0-22					
HP10x42	1000.0	56.7	943.3	60	65
BUT-CR11-0296 Baseline Rd/Broadway St - Forward Abutment, B-002-0-22					
HP10x42	1017.3	74.0	943.3	80	85

Notes:
1. *Based on definitions and formulas presented in Section 305.3.5.2 of the 2020 BDM.*

5.3. Retaining Walls 1 and 2 Analysis and Recommendations

5.3.1. *Retaining Wall Design Assumptions*

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

- Minimum reinforcement strap lengths of proposed MSE walls are to be 70% of the total wall height (as measured from proposed profile grade at the face of the wall to the top of the leveling pad) or 8 ft, whichever is greater, at the section of wall being analyzed, per ODOT's BDM section 307.4-A;
- Minimum MSE wall embedment depths (as measured from top of the leveling pad to the lowest point on the ground surface within 4-ft of the face of the wall) are to conform to Figure 201-5 presented in ODOT's BDM and be the larger of 3 ft or the local frost depth;
- Soils below the bottom of leveling pad will be undercut a minimum of 1 ft and replaced Granular Material Type C according to the requirements of ODOT Construction & Materials Specifications Section 204.07 (CMS 204.07);
- Maximum allowable differential settlement in the longitudinal direction is 1%. (BDM Section 307.1.6); and,
- Reinforced Zone and Retained Fill soils will meet the minimum design soil parameters per Table 840.04-1 of the ODOT Supplemental Specification 840 (SS-840) as shown in Table 7 below.

Table 7: Design Soil Parameters for Fill Materials

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

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

With respect to design constraints and assumptions specific to the RW-1 & RW-2 MSE walls, the geometry of the proposed walls (i.e., exposed wall heights, existing ground elevations, proposed final grade behind/at the toe of the wall, etc.) is assumed to be consistent with that shown in the proposed RW-1 & RW-2 plans developed by REL provided via email on September 4, 2025.

5.3.1.1. Retaining Wall - Soil Profile for Analysis

For retaining wall external and global stability analyses performed, the generalized material profile and estimated engineering properties developed for the proposed HUR-CR11-0296 bridge substructures, presented in Section 5.1. of this report, were assumed to be representative of the subsurface conditions at the proposed RW-1 and RW-2 locations. Specifically, soil properties estimated at the proposed rear abutment (B-001-0-22) were utilized for RW-1 analyses while soil properties estimated at the proposed forward abutment(B-002-0-22) were utilized for RW-2 analyses.

5.3.2. External Stability

Based on our estimated engineering soil properties, the developed generalized profile and the retaining wall design assumptions provided in Section 5.3.1. of this report, external stability analyses of the proposed RW-1 and RW-2 were performed. External stability was evaluated at each step along both the proposed RW-1 and RW-2 alignments. Each cross section was evaluated for resistance to bearing pressure, sliding forces and overturning at the Strength Limit State in accordance with Section 11.10.5 of the AASHTO's LRFD BDS. The capacity to demand ratios (CDRs) calculated for the referenced sections with respect to bearing, sliding and overturning, as well as the calculated bearing resistances and pressures, are presented in Table 8 below. (External Stability and Bearing Resistance Calculation Results can be found in Appendix D).

Table 8: External Stability Analysis Summary

Dimensions						
	Retaining Wall 1				**Retaining Wall 2**	
Design Wall Height (ft)	9.3	12.2	15.1	18.5	15.7	18.5
Length of Reinforcement (ft)	8.0	8.5	10.6	13.0	11.0	13.0
Length of Reinf. To Height Ratio	0.9	0.7	0.7	0.7	0.7	0.7
Approximate Station$^{(1)}$	0+00.0 - 0+68.3 1+84.3 - 2+52.6	0+68.3 - 0+77.3 1+75.3 - 1+84.3	0+77.3 - 0+86.3 1+66.3 - 1+75.3	0+86.3 - 1+66.3	20+14.8 - 20+21.9 21+00.4 - 21+05.2	20+21.9 - 21+00.4
Capacity Demand Ratio (CDR)						
Bearing Capacity	1.76	1.42	1.28	1.02	1.69	1.17
Overturning / Eccentricity	1.86	1.38	1.48	1.58	1.50	1.58
Sliding	1.13	1.00	1.06	1.11	1.07	1.11
Factored Bearing Resistance (ksf)	4.3	4.8	5.2	4.8	7.0	5.5
Service Bearing Pressure (ksf)	1.7	2.4	2.9	3.4	3.0	3.4
Factored Bearing Pressure (ksf)	2.4	3.4	4.0	4.7	4.1	4.7

Notes:
1. *Stationing in reference to Retaining Wall 1 or Retaining Wall 2 alignments.*

5.3.3. Global Stability

For purposes of evaluating the stability of the proposed RW-1 and RW-2, NEAS reviewed cross-sections along the length of the proposed retaining walls to determine the subsurface conditions that posed the greatest potential for slope instability. In general, cross-sections along the proposed wall alignments were reviewed to determine the sections that would represent a combination of existing subsurface conditions and planned site grading that would be most critical to slope stability (i.e., maximum total wall height, maximum embankment height measured from toe of slope to top of wall, proposed/existing grades behind and in front of the wall, weak and/or thick soil layer, etc.). Based on our review of the available information at the referenced location and the associated soil properties, one (1) cross-section for each retaining wall were estimated to be most "critical" and were analyzed for global stability. The cross-sections analyzed for global stability for each of the proposed walls consisted of the maximum wall-height sections at STA. 2+52.5 (RW-1 Alignment) for RW-1 and STA. 20+37 (RW-2 Alignment) for RW-2.

For the indicated cross-sections, NEAS developed a representative cross-sectional model to use as the basis for global stability analysis. The models were developed from NEAS's interpretation of the available information which included: 1) the RW 1 & RW-2 plans developed by REL provided via email on September 4, 2025; 2) a live load surcharge of 250 pounds per square foot (psf) accounting for traffic induced loads; and, 3) test borings and laboratory data developed as part of this project. With respect to the soil's engineering properties, the provided generalized soil profile and estimated engineering properties presented in Section 5.1. of this report were used in our analysis.

The above referenced global stability models were analyzed for long-term (Effective Stress) and short-term (Total Stress) slope stability utilizing the software entitled *Slide2* by Rocscience, Inc. Specifically, the Modified Bishop and Spencer analysis methods were used to calculate a factor of safety (FOS) for circular and translational 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 1.5 which equates to an AASHTO resistance factor less than 0.65 (per AASHTO's LRFD BDS, the specified resistance factors are essentially the inverse of the FOS that should be targeted in slope stability programs). For this analysis, a resistance factor of 0.65 or lower was targeted as the MSE wall abutments contain or support a structural element of the proposed bridge. Based on our slope stability analyses for the referenced MSE wall sections, the minimum slope stability factor was estimated to be about 1.679 (0.6 resistance factor). Graphical outputs of the slope stability program (cross-sectional model, calculated safety factor, and critical failure plane) are presented within Appendix E.

5.3.4. Settlement and Downdrag

Based on our review of the proposed RW-1 and RW-2 plans developed by REL provided via email on September 4, 2025 as well as field and laboratory test data, total settlement at both the rear and forward abutment locations is anticipated to be less than 0.4 inches. Therefore, downdrag loading of the piles is not anticipated and a waiting period does not need to be implemented during construction to facilitate settlement at the substructure locations. With respect to differential settlement along the proposed MSE wall alignments, these magnitudes are anticipated to be less than the tolerable limit of 1% per ODOT BDM Section 307.1.6. Due to: 1) the minimal amount of new fill proposed along the roadway and MSE wall alignments; 2) the relatively uniform thicknesses of the fill to be placed across the width of the roadway and MSE walls; and 3) the gradual transition of the proposed new fill from proposed fill areas to non-fill areas (i.e., proposed grade to existing grade), this settlement magnitude is not anticipated to be a concern for pavements or at-grade structures (i.e., guardrails, signage, etc.).

6. QUALIFICATIONS

This report has been prepared for Richland Engineering Ltd. and their design consultants to be used solely in evaluating the soils underlying the bridge 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 evaluation. Our recommendations are based on the results of the field explorations, laboratory tests results from representative soil samples, and geotechnical engineering analyses. The results of the field explorations and laboratory tests, which form the basis of our recommendations, are presented in the appendices as noted. This report does not reflect any variations that may occur between the borings or elsewhere on the site, or variations whose nature and extent may not become evident until a later stage of construction. In the event that any changes in the nature, design or location of the proposed bridge replacement project 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 Richland Engineering Ltd. in performing this geotechnical exploration for the Baseline Road Bridge Replacement project (HUR-CR11-2.96 Baseline Road/Broadway St). Please call if there are any questions, or if we can be of further service.

Respectfully Submitted,

Kevin C Arens

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

Kevin C. Arens, P.E.
Geotechnical Engineer

REFERENCES

AASHTO. (2020). *LRFD Bridge Design Specifications; 9th Edition.* Washington, D.C.: American Association of State Highway and Transportation Officials.

FEMA. (2024). *National Flood Hazard Layer kmz v3.0.* Federal Emergency Management Agency.

ODGS. (1998). Physiographic regions of Ohio: Ohio Department of Natural Resources, Division of Geological Survey. page-size map with text, 2p., scale 1:2,100,00.

ODNR [1]. (2024). Ohio Abandoned Mine Locator Interactive Map. *Mines of Ohio*. Ohio Department of Natural Resources, Division of Geological Survey & Division of Mineral Resources. Retrieved from https://gis.ohiodnr.gov/MapViewer/?config= OhioMines

ODOT [2]. (2025). *Geotechnical Design Manual.* Columbus, OH: Ohio Department of Transportation: Office of Geotechnical Engineering.

ODOT. (2024). *Specifications for Geotechnical Explorations.* Ohio Department of Transportation: Office of Geotechnical Engineering.

ODOT. (2025). *2020 Bridge Design Manual.* Columbus, OH: Ohio Department of Transportation: Office of Structural Engineering. Retrieved from http://www.dot.state.oh.us/Divisions/Engineering/Structures/standard/Bridges/Pages/BDM2004.aspx

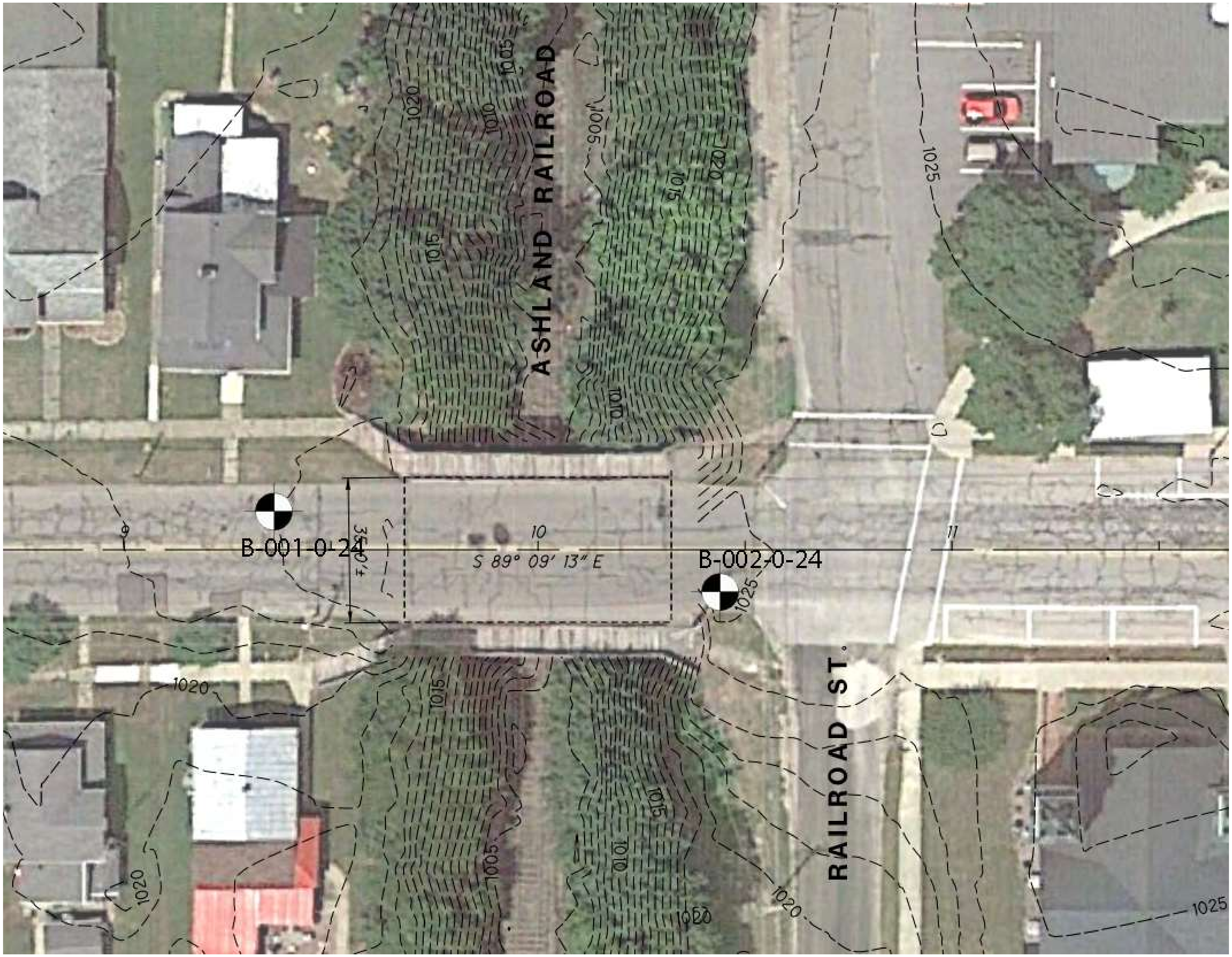
USDA. (2024, February). Web Soil Survey. Retrieved from http://websoilsurvey.nrcs.usda.gov

USGS & ODGS. (2006, June). Geologic Units of Ohio. *ohgeol.kmz*. United States Geologic Survey.

USGS & ODGS. (2008). Venteris, E.R., Shrake, D.L., Larsen, G.E., Pavey, R.R., and Schumacher, G.A., Surficial geology of the Mansfield 30 x 60-minute quadrangle: Ohio Division of Geological Survey Map SG-2. scale 1:100,000.

APPENDIX A

BORING LOCATION PLAN



Baseline Road Bridge Boring Location Plan

APPENDIX B

BORING LOGS AND LABORATORY TESTING

PROJECT: BASELINE RD BRIDGE
TYPE: BRIDGE
PID: 124504 SFN:
START: 6/19/24 END: 6/20/24

DRILLING FIRM / OPERATOR: NEAS / J. HODGES
SAMPLING FIRM / LOGGER: NEAS / J. HODGES
DRILLING METHOD: 3.25" HSA / NQ2
SAMPLING METHOD: SPT / NQ2

DRILL RIG: CME 75T
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 3/8/24
ENERGY RATIO (%): 84

STATION / OFFSET: 156+23, 7' LT.
ALIGNMENT: CR-11 (BASELINE RD)
ELEVATION: 1023.1 (MSL) EOB: 97.0 ft.
LAT / LONG: 40.995287, -82.668212

EXPLORATION ID
B-001-0-22

PAGE
1 OF 4

MATERIAL DESCRIPTION AND NOTES	ELEV. 1023.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI			
11.5" ASPHALT AND 3.0" BRICK AND 5.0" BASE (DRILLERS DESCRIPTION)	1021.5																	
VERY STIFF TO HARD, BROWN, **SANDY SILT**, LITTLE TO SOME GRAVEL, LITTLE CLAY, IRON STAINING, DAMP			5 4 4	11	39	SS-1	3.50	21	11	17	32	19	26	18	8	15	A-4a (3)	
			2 4 5	13	44	SS-2	4.50	-	-	-	-	-	-	-	-	15	A-4a (V)	
			7 5 7	17	72	SS-3	4.50	-	-	-	-	-	-	-	-	15	A-4a (V)	
			4 3 5	11	100	SS-4	2.50	13	28	18	26	15	28	19	9	16	A-4a (1)	
			2 3 4	10	100	SS-5	4.50	-	-	-	-	-	-	-	-	15	A-4a (V)	
	1006.1		4 2 3	7	100	SS-6	2.75	-	-	-	-	-	-	-	-	19	A-4a (V)	
LOOSE, BROWN, **GRAVEL WITH SAND, SILT, AND CLAY**, MOIST	1003.6		3 3 3	8	100	SS-7	-	20	33	15	19	13	33	21	12	21	A-2-6 (0)	
VERY LOOSE, BROWNISH GRAY, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, WET	1001.1	W 1003.1	1 1 2	4	100	SS-8	-	-	-	-	-	-	-	-	-	23	A-1-b (V)	
LOOSE, BROWNISH GRAY, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, MOIST			2 1 3	6	100	SS-9	-	-	-	-	-	-	-	-	-	17	A-2-4 (V)	
	996.1		2 3 3	8	94	SS-10	-	11	39	19	21	10	26	18	8	16	A-2-4 (0)	
VERY STIFF, GRAY, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, WET TO MOIST			2 3 7	14	100	SS-11	3.50	-	-	-	-	-	-	-	-	24	A-4a (V)	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 6/26/25 12:03 - P:\24-0031 BASELINE ROAD (VAR-STW MUNI BR) WALLACE-PANCHER\120504\GEOTECHNICAL\BRIDGES\HUR-CR

PID: 124504	SFN:	PROJECT: BASELINE RD BRIDGE	STATION / OFFSET: 156+23, 7' LT.	START: 6/19/24	END: 6/20/24	PG 2 OF 4	B-001-0-22

MATERIAL DESCRIPTION AND NOTES	ELEV. 993.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
VERY STIFF, GRAY, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, WET TO MOIST *(continued)*	991.1	31 32	2 2 5	10	67	SS-12	2.50	3	10	18	42	27	23	15	8	17	A-4a (7)	
VERY LOOSE TO LOOSE, GRAY, **COARSE AND FINE SAND**, LITTLE SILT, TRACE GRAVEL, TRACE CLAY, WET		33 34	1 1 2	4	100	SS-13	-	-	-	-	-	-	-	-	-	21	A-3a (V)	
	986.1	35 36	2 1 3	6	100	SS-14	-	-	-	-	-	-	-	-	-	19	A-3a (V)	
MEDIUM DENSE, GRAY, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, WET	983.6	37 38 39	3 3 7	14	89	SS-15	-	-	-	-	-	-	-	-	-	16	A-1-b (V)	
LOOSE, GRAY, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, WET	979.8	40 41 42 43	4 2 5	10	100	SS-16	-	-	-	-	-	-	-	-	-	12	A-2-4 (V)	
VERY STIFF, GRAY, **SANDY SILT**, "AND" CLAY, TRACE GRAVEL, DAMP	974.8	44 45 46 47 48	4 5 8	18	89	SS-17	3.75	2	5	12	44	37	26	16	10	16	A-4a (8)	
VERY STIFF TO HARD, GRAY, **SILT**, SOME CLAY, TRACE SAND, TRACE GRAVEL, DAMP TO MOIST		49 50 51	4 5 5	14	44	SS-18	4.50	-	-	-	-	-	-	-	-	16	A-4b (V)	
		52 53 54 55 56	4 6 7	18	100	SS-19	3.50	2	3	11	50	34	25	16	9	16	A-4b (8)	
		57 58 59 60 61	5 7 8	21	100	SS-20	4.00	-	-	-	-	-	-	-	-	17	A-4b (V)	

PID: 124504	SFN:	PROJECT: BASELINE RD BRIDGE	STATION / OFFSET: 156+23, 7' LT.	START: 6/19/24	END: 6/20/24	PG 3 OF 4	B-001-0-22

MATERIAL DESCRIPTION AND NOTES	ELEV. 961.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI			
VERY STIFF TO HARD, GRAY, **SILT**, SOME CLAY, TRACE SAND, TRACE GRAVEL, DAMP TO MOIST *(continued)*		63 64																
SS-21 BECOMES WET		65 66	4 6 10	22	100	SS-21	4.50	1	0	1	64	34	26	18	8	25	A-4b (8)	
	954.8	67 68																
DENSE, GRAY, **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, TRACE GRAVEL, WET		69																
		70 71	6 11 14	35	100	SS-22	-	-	-	-	-	-	-	-	-	18	A-3a (V)	
		72 73 74																
		75 76	10 11 11	31	100	SS-23	-	-	-	-	-	-	-	-	-	17	A-3a (V)	
	944.8	77 78																
DENSE, GRAY, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST		79																
		80 81	9 11 15	36	100	SS-24	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
	939.8	82 83																
VERY DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND, SILT, AND CLAY**, MOIST		84																
		85	27 50/4"	-	90	SS-25	-	-	-	-	-	-	-	-	-	10	A-2-6 (V)	
	936.3	86 TR	50/3"	-	67	SS-26	-	-	-	-	-	-	-	-	-	9	A-2-6 (V)	
INTERBEDDED SHALE (87%) AND SANDSTONE (13%), BEDDING DISCONTINUITIES: LOW ANGLE, JOINT DISCONTINUITIES: HIGH ANGLE, FRACTURED TO MODERATELY FRACTURED, OPEN TO NARROW, SLIGHTLY ROUGH SURFACE CONDITION, RQD 37%, REC. 80%; **SHALE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, WEAK TO SLIGHTLY STRONG, LAMINATED TO THIN BEDDED, FISSILE; **SANDSTONE**, LIGHT GRAY, SLIGHTLY WEATHERED, STRONG, MEDIUM GRAINED. @92.9'-93.2'; Qu = 1428 PSI @ 2.6%		87 88 89 90 91 92 93 94	37		80	NQ2-1											CORE	

PID: 124504 | SFN: | PROJECT: BASELINE RD BRIDGE | STATION / OFFSET: 156+23, 7' LT. | START: 6/19/24 | END: 6/20/24 | PG 4 OF 4 | B-001-0-22

MATERIAL DESCRIPTION AND NOTES	ELEV. 928.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI			
	926.1	95 96 EOB 97																

NOTES: GROUNDWATER ENCOUNTERED AT 20.0' DURING DRILLING. HOLE DID NOT CAVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; PUMPED 150 GAL. BENTONITE GROUT; POURED 5 BAGS HOLE PLUG

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 6/26/25 12:03 - P:\24-0031 BASELINE ROAD (VAR-STW MUNI BR) WALLACE-PANCHER\120504\GEOTECHNICAL\BRIDGES\HUR-CR

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

(Project: Baseline Rd Bridge Replacement, Boring Location: B-001-0-22, NQ2-1, Depth: 92.9-93.2ft)

Tested Date: 10/8/2024

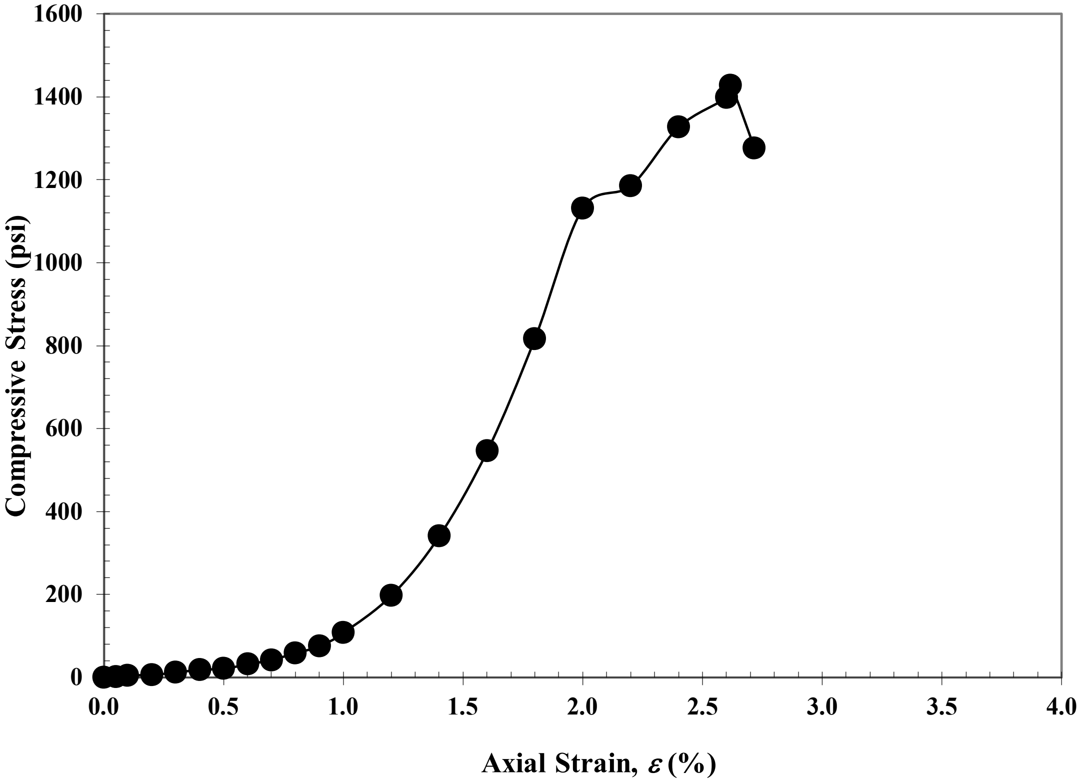
Specimen Properties

Average Dia., D_{avg} (in): 1.95
Average Height, H_{avg} (in): 4.05
Length to Diameter Ratio: 2.08
Area, A (in^2): 2.99
Volume, V (in^3): 12.09
Wet Mass of Specimen (lb): 1.1
Moisture Content (%): 3.1
Dry Mass of Specimen (lb): 1.1
Wet Unit Weight, γ (lb/ft^3): 158.9
Dry Unit Weight, γ_d (lb/ft^3): 154.1

Final Specimen Figure

Results

Unconfined Compressive Strength (psi): **1428** **10** (MPa)
Strain (%): **2.6**



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

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

PROJECT: BASELINE RD BRIDGE	DRILLING FIRM / OPERATOR: NEAS / J. HODGES	DRILL RIG: CME 75T	STATION / OFFSET: 157+49, 17' RT.	EXPLORATION ID B-002-0-22
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: NEAS / J. HODGES	HAMMER: CME AUTOMATIC	ALIGNMENT: CR-11 (BASELINE RD)	
PID: 124504 SFN:	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 3/8/24	ELEVATION: 1024.3 (MSL) EOB: 91.5 ft.	PAGE 1 OF 3
START: 5/22/24 END: 5/23/24	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 84	LAT / LONG: 40.995217, -82.667756	

MATERIAL DESCRIPTION AND NOTES	ELEV. 1024.3	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI			
8.0" ASPHALT AND 3.0" BRICK AND 5.5" BASE (DRILLERS DESCRIPTION)	1022.9	1																
STIFF, BROWN AND GRAY, **SILT AND CLAY**, SOME SAND, TRACE GRAVEL, IRON STAINING, SS-1 CONTAINS NO INTACT SOIL FOR HP READINGS, MOIST		2 3 4	7 5 2	10	28	SS-1	-	3	9	22	39	27	30	19	11	20	A-6a (7)	
	1017.3	5 6	2 3 3	8	100	SS-2	1.50	-	-	-	-	-	-	-	-	20	A-6a (V)	
HARD, GRAY, **SANDY SILT**, "AND" CLAY, TRACE GRAVEL, DAMP	1014.8	7 8 9	5 6 9	21	100	SS-3	4.50	2	5	10	47	36	25	17	8	16	A-4a (8)	
HARD, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP	1012.3	10 11	6 10 15	35	100	SS-4	4.50	6	4	12	36	42	32	19	13	16	A-6a (9)	
HARD, BROWN, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP		12 13 14	3 7 11	25	100	SS-5	4.50	-	-	-	-	-	-	-	-	18	A-6b (V)	
		15 16	5 8 12	28	100	SS-6	4.50	-	-	-	-	-	-	-	-	18	A-6b (V)	
	1004.8	17 18 19	2 7 8	21	100	SS-7	4.50	3	4	10	38	45	37	21	16	19	A-6b (10)	
VERY STIFF, GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP		20 21	3 5 8	18	100	SS-8	3.50	-	-	-	-	-	-	-	-	17	A-6a (V)	
		22 23 24	1 4 5	13	100	SS-9	3.75	2	4	9	38	47	31	19	12	19	A-6a (9)	
	997.3	25 26	3 4 7	15	100	SS-10	3.75	-	-	-	-	-	-	-	-	14	A-6a (V)	
VERY LOOSE TO MEDIUM DENSE, GRAY, **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, TRACE GRAVEL, WET		27 28 29	0 0 1	1	56	SS-11	-	-	-	-	-	-	-	-	-	25	A-3a (V)	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 6/26/25 12:03 - P:\24-0031 BASELINE ROAD (VAR-STW MUNI BR) WALLACE-PANCHER\120504\GEOTECHNICAL\BRIDGES\HUR-CR

PID: 124504	SFN:	PROJECT: BASELINE RD BRIDGE	STATION / OFFSET: 157+49, 17' RT.	START: 5/22/24	END: 5/23/24	PG 2 OF 3	B-002-0-22

MATERIAL DESCRIPTION AND NOTES	ELEV. 994.3	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
VERY LOOSE TO MEDIUM DENSE, GRAY, **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, TRACE GRAVEL, WET *(continued)*		31	3 4 8	17	100	SS-12	-	-	-	-	-	-	-	-	-	21	A-3a (V)	
		32																
		33	2 4 5	13	100	SS-13	-	-	-	-	-	-	-	-	-	18	A-3a (V)	
	989.8	34																
VERY STIFF, GRAY, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, DAMP		35 36	3 4 5	13	100	SS-14	3.25	4	8	16	39	33	24	17	7	16	A-4a (7)	
		37																
		38 39	2 4 6	14	61	SS-15	2.75	-	-	-	-	-	-	-	-	16	A-4a (V)	
		40 41	3 4 8	17	100	SS-16	3.75	3	4	14	47	32	24	15	9	15	A-4a (8)	
		42																
	981.0	43																
VERY STIFF TO HARD, GRAY, **SILT**, SOME CLAY, LITTLE SAND, TRACE GRAVEL, DAMP		44																
		45 46	4 9 14	32	100	SS-17	4.25	-	-	-	-	-	-	-	-	15	A-4b (V)	
		47 48 49																
		50 51	3 6 10	22	100	SS-18	3.50	-	-	-	-	-	-	-	-	16	A-4b (V)	
		52 53 54																
		55 56	4 5 9	20	100	SS-19	4.00	1	3	11	50	35	25	16	9	16	A-4b (8)	
		57 58 59																
		60 61	5 6 10	22	100	SS-20	3.75	-	-	-	-	-	-	-	-	16	A-4b (V)	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 6/26/25 12:03 - P:\24-0031 BASELINE ROAD (VAR-STW MUNI BR) WALLACE-PANCHER\120504\GEOTECHNICAL\BRIDGES\HUR-CR

PID: 124504 | SFN: ________ | PROJECT: BASELINE RD BRIDGE | STATION / OFFSET: 157+49, 17' RT. | START: 5/22/24 | END: 5/23/24 | PG 3 OF 3 | B-002-0-22

MATERIAL DESCRIPTION AND NOTES	ELEV. 962.2	DEPTHS	SPT/ RQD	N60	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI			
	961.0	63																
VERY STIFF, GRAY, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, DAMP	956.0	64 65 66 67 68	4 7 10	24	100	SS-21	3.00	10	14	15	37	24	23	15	8	14	A-4a (5)	
MEDIUM DENSE, GRAY, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, WET	951.0	69 70 71 72 73	4 4 4	11	100	SS-22	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
DENSE, GRAY, **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, TRACE GRAVEL, MOIST	946.0	74 75 76 77 78	6 9 13	31	100	SS-23	-	-	-	-	-	-	-	-	-	11	A-3a (V)	
VERY DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND, SILT, AND CLAY**, MOIST	943.3	79 80	23 50/4"	-	70	SS-24	-	-	-	-	-	-	-	-	-	12	A-2-6 (V)	
INTERBEDDED SHALE (91%) AND SANDSTONE (9%), BEDDING DISCONTINUITIES: LOW ANGLE, JOINT DISCONTINUITIES: HIGH ANGLE, HIGHLY FRACTURED TO MODERATELY FRACTURED, OPEN TO NARROW, SLIGHTLY ROUGH SURFACE CONDITION, RQD 46%, REC. 96%; **SHALE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, WEAK TO SLIGHTLY STRONG, LAMINATED TO THIN BEDDED, FISSILE; **SANDSTONE**, LIGHT GRAY, SLIGHTLY WEATHERED, STRONG, MEDIUM GRAINED.	932.8	TR 81	50	-	83	SS-25	-	-	-	-	-	-	-	-	-	5	Rock (V)	
		82 83 84 85 86 87 88 89 90 91 EOB	46		96	NQ2-1											CORE	

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

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 1 BAG ASPHALT PATCH; PUMPED 100 GAL. BENTONITE GROUT; POURED 10 BAGS GRAVEL

APPENDIX C

GRLWEAP ANALYSIS

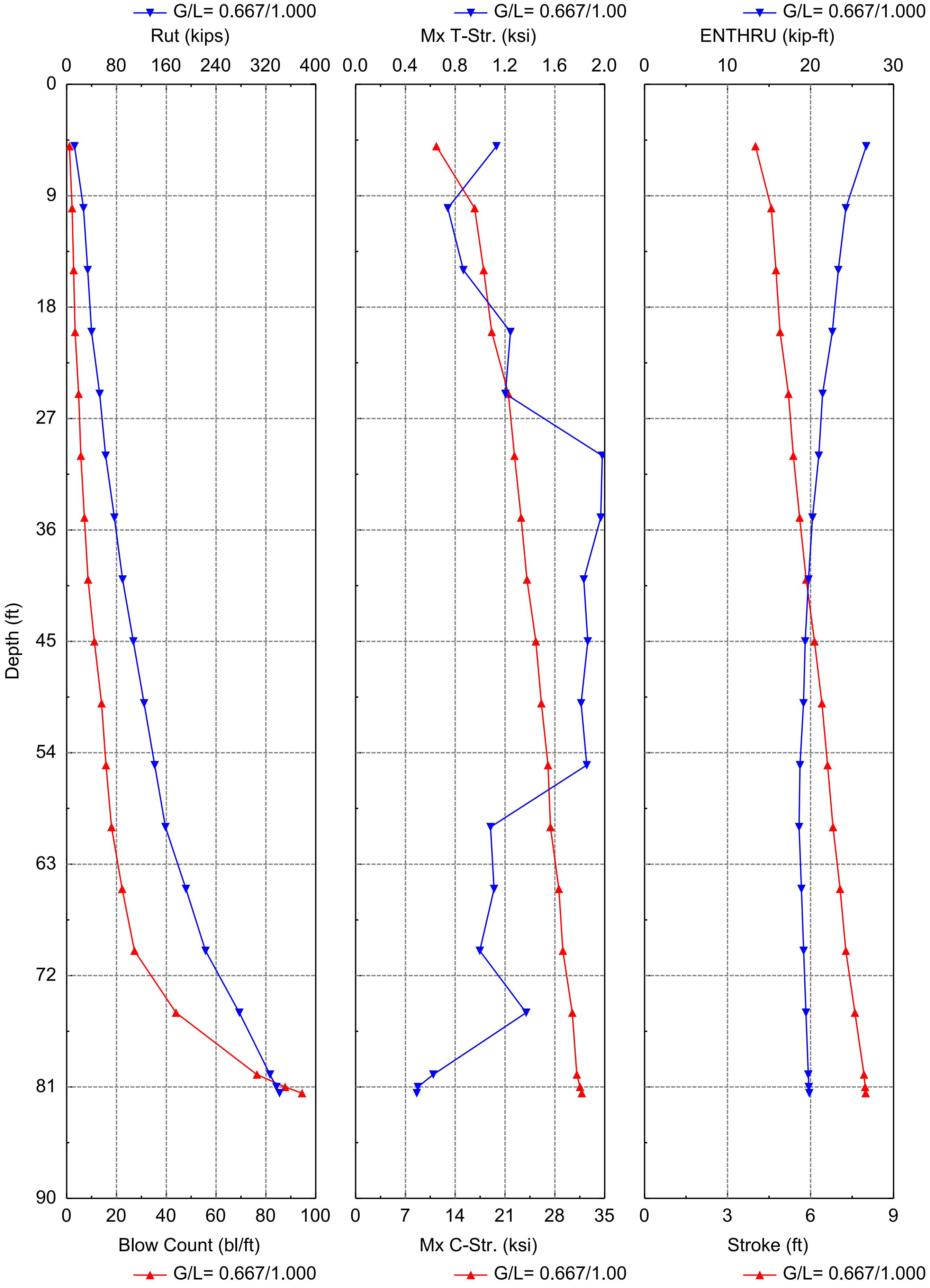
REAR ABUTMENT

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

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	12.6	11.5	1.1	1.1	11.351	1.132	4.01	26.7	D 19-42
10.0	27.1	26.0	1.1	2.2	16.715	0.740	4.58	24.2	D 19-42
15.0	33.8	32.0	1.8	2.8	18.001	0.866	4.75	23.3	D 19-42
20.0	40.1	38.3	1.8	3.4	19.138	1.244	4.90	22.6	D 19-42
25.0	53.0	51.9	1.2	4.8	21.459	1.202	5.20	21.4	D 19-42
30.0	62.5	61.4	1.1	5.7	22.320	1.981	5.38	21.0	D 19-42
35.0	76.7	72.4	4.3	7.1	23.254	1.969	5.61	20.2	D 19-42
40.0	89.8	88.1	1.7	8.6	24.086	1.833	5.84	19.7	D 19-42
45.0	107.1	105.3	1.8	11.1	25.322	1.865	6.14	19.3	D 19-42
50.0	124.3	122.5	1.8	14.0	26.098	1.811	6.41	19.1	D 19-42
55.0	141.5	139.7	1.8	15.8	27.034	1.856	6.61	18.7	D 19-42
60.0	158.7	156.9	1.8	18.0	27.373	1.084	6.81	18.6	D 19-42
65.0	191.5	182.3	9.3	22.3	28.556	1.112	7.07	18.9	D 19-42
70.0	223.0	213.7	9.3	27.2	29.118	0.997	7.27	19.2	D 19-42
75.0	277.3	255.6	21.7	43.9	30.439	1.370	7.60	19.4	D 19-42
80.0	326.6	304.9	21.7	76.4	31.080	0.624	7.93	19.7	D 19-42
81.0	336.9	315.2	21.7	87.7	31.539	0.500	7.97	19.8	D 19-42
81.5	341.8	320.1	21.7	94.5	31.790	0.491	7.99	19.8	D 19-42

Total driving time: 28 minutes; Total Number of Blows: 1255 (starting at penetration 5.0 ft)

0.0 ft @ Ele. 1016.52 ft amsl (Bottom of Pile Cap)


G/L= 0.667/1.000
Rut (kips)
0
80
160
240
320
400
G/L= 0.667/1.00
Mx T-Str. (ksi)
0.0
0.4
0.8
1.2
1.6
2.0
G/L= 0.667/1.000
ENTHRU (kip-ft)
0
10
20
30
Depth (ft)
0
9
18
27
36
45
54
63
72
81
90
0
20
40
60
80
100
Blow Count (bl/ft)
G/L= 0.667/1.000
0
7
14
21
28
35
Mx C-Str. (ksi)
G/L= 0.667/1.00
0
3
6
9
Stroke (ft)
G/L= 0.667/1.000

SOIL PROFILE

Depth ft	Soil Type -	Spec. Wt lb/ft³	Su ksf	Phi °	Unit Rs ksf	Unit Rt ksf
0.0	Clay	120.0	1.4	0.0	1.19	12.60
10.4	Clay	120.0	1.4	0.0	1.19	12.60
10.4	Sand	110.0	0.0	31.0	0.34	20.57
12.9	Sand	110.0	0.0	31.0	0.38	20.71
12.9	Sand	120.0	0.0	31.0	0.38	20.71
20.4	Sand	120.0	0.0	31.0	0.51	20.71
20.4	Clay	120.0	1.5	0.0	1.29	13.50
25.4	Clay	120.0	1.5	0.0	1.29	13.50
25.4	Sand	118.0	0.0	29.0	0.52	13.32
32.9	Sand	118.0	0.0	29.0	0.63	13.32
32.9	Sand	122.0	0.0	33.0	0.87	50.11
36.7	Sand	122.0	0.0	33.0	0.95	50.11
36.7	Clay	122.0	2.2	0.0	1.57	20.25
41.7	Clay	122.0	2.2	0.0	1.57	20.25
41.7	Clay	122.0	2.3	0.0	1.57	20.70
61.7	Clay	122.0	2.3	0.0	1.57	20.70
61.7	Sand	128.0	0.0	35.0	1.75	107.60
71.7	Sand	128.0	0.0	35.0	2.03	107.60
71.7	Sand	135.0	0.0	38.0	2.75	251.83
80.2	Sand	135.0	0.0	38.0	3.11	251.83

GRLWEAP: Wave Equation Analysis of Pile Foundations

Baseline Rd Bridge Replacement over Ashalnd Railway + Rear Abutment - HP10x42 6/29/2025
NATIONAL ENGINEERING AND ARCHITECTURAL GRLWEAP 14.1.20.1

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

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

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

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

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

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

HAMMER DATA

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

Hammer and Drive System Segment Data

Segment -	Weight kips	Stiffness kips/in	COR -	C-Slack in	Damping kips/ft/s
1	0.800	140,084.4	1.000	0.000	
2	0.800	140,084.4	1.000	0.000	
3	0.800	140,084.4	1.000	0.000	
4	0.800	140,084.4	1.000	0.000	
5	0.800	70,754.7	0.900	0.120	
Imp Block	0.753	109,976.0	0.800	0.120	
Helmet	2.700				5.3

Ram Weight: (kips)	4.00	Ram Length: (ft)	10.76
Ram Area: (in²)	124.69		
Maximum (Eq) Stroke: (ft)	10.81	Actual (Eq) Stroke: (ft)	10.81
Efficiency:	0.800	Rated Energy: (kip-ft)	43.24
Maximum Pressure: (psi)	1,600.00	Actual Pressure: (psi)	1,600.00
Combustion Delay: (ms)	2.00	Ignition Duration: (ms)	2.00
Expansion Exponent:	1.25		

Hammer Cushion		Pile Cushion	
Cross Sect. Area: (in²)	415.00	Cross Sect. Area: (in²)	0.00
Elastic Modulus: (ksi)	530.0	Elastic Modulus: (ksi)	0.0
Thickness: (in)	2.00	Thickness: (in)	0.00
Coeff. of Restitution:	0.800	Coeff. of Restitution:	0.500
RoundOut: (in)	0.120	RoundOut: (in)	0.120
Stiffness: (kips/in)	109,976.0	Stiffness: (kips/in)	0.0
Helmet Weight: (kips)	2.700		

PILE INPUT

Uniform Pile		Pile Type:	H Pile
Pile Length: (ft)	90.000	Pile Penetration: (ft)	81.500
Pile Size: (ft)	0.84	Toe Area: (in²)	12.40

Table of Depths Analyzed with Driving System Modifiers

Depth ft	Temp Length ft	Wait Time Hr	Hammer -
5.00	85.0	0.0	DELMAG D 19-42
10.00	85.0	0.0	DELMAG D 19-42
15.00	85.0	0.0	DELMAG D 19-42
20.00	85.0	0.0	DELMAG D 19-42
25.00	85.0	0.0	DELMAG D 19-42
30.00	85.0	0.0	DELMAG D 19-42
35.00	85.0	0.0	DELMAG D 19-42
40.00	85.0	0.0	DELMAG D 19-42
45.00	85.0	0.0	DELMAG D 19-42
50.00	85.0	0.0	DELMAG D 19-42
55.00	85.0	0.0	DELMAG D 19-42
60.00	85.0	0.0	DELMAG D 19-42
65.00	85.0	0.0	DELMAG D 19-42
70.00	85.0	0.0	DELMAG D 19-42
75.00	85.0	0.0	DELMAG D 19-42
80.00	85.0	0.0	DELMAG D 19-42
81.00	85.0	0.0	DELMAG D 19-42
81.50	85.0	0.0	DELMAG D 19-42

Other Information for DELMAG D 19-42

Depth ft	Stroke ft	Diesel Pressure %	Efficiency -	P.C. Stiff. Fact. -	P.C. COR -
5.00	10.8	100.0	0.80	1.0	0.50
10.00	10.8	100.0	0.80	1.0	0.50
15.00	10.8	100.0	0.80	1.0	0.50
20.00	10.8	100.0	0.80	1.0	0.50
25.00	10.8	100.0	0.80	1.0	0.50
30.00	10.8	100.0	0.80	1.0	0.50
35.00	10.8	100.0	0.80	1.0	0.50
40.00	10.8	100.0	0.80	1.0	0.50
45.00	10.8	100.0	0.80	1.0	0.50
50.00	10.8	100.0	0.80	1.0	0.50
55.00	10.8	100.0	0.80	1.0	0.50
60.00	10.8	100.0	0.80	1.0	0.50
65.00	10.8	100.0	0.80	1.0	0.50
70.00	10.8	100.0	0.80	1.0	0.50
75.00	10.8	100.0	0.80	1.0	0.50
80.00	10.8	100.0	0.80	1.0	0.50
81.00	10.8	100.0	0.80	1.0	0.50

81.50	10.8	100.0	0.80	1.0	0.50

PILE, SOIL, ANALYSIS OPTIONS

Analysis type:	Driveability Analysis	Soil Damping Option:	Smith
Max No Analysis Iterations:	0	Time Increment/Critical:	160
Residual Stress Analysis:	0	Analysis Time-Input(ms):	0
Output Level:	Normal	Gravitational Acceleration (ft/s²):	32.169
Hammer Gravity (ft/s²):	32.169	Pile Gravity (ft/s²):	32.169

DRIVEABILITY ANALYSIS

Analysis Depth (ft)	81.50	Standard Soil Setup	
Hammer Name	DELMAG D 19-42	Hammer ID	41
Diesel Pressure: (psi)	230.40	Stroke (ft)	10.81
Efficiency	0.80		
Shaft Gain/Loss Factor	0.667	Toe Gain/Loss Factor	1.000
Shaft Gain/Loss Factor	1.000	Toe Gain/Loss Factor	1.000

SOIL RESISTANCE PARAMETERS

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Setup F. -	Limit D. ft	Setup T. Hours	EB Area in²
0.00	1.2	12.6	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
10.40	1.2	12.6	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
10.40	0.3	20.6	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
11.65	0.4	20.7	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
16.65	0.4	20.7	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
18.52	0.5	20.7	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
20.40	0.5	20.7	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
20.40	1.3	13.5	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
25.40	1.3	13.5	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
25.40	0.5	13.3	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
27.27	0.5	13.3	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
29.15	0.6	13.3	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
32.90	0.6	13.3	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
32.90	0.9	50.1	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
34.80	0.9	50.1	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
36.70	1.0	50.1	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
36.70	1.6	20.2	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
41.70	1.6	20.2	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
41.70	1.6	20.7	0.10	0.100	0.150	0.1	1.5	6.56	24.0	12.40
61.70	1.6	20.7	0.10	0.100	0.150	0.1	1.5	6.56	24.0	12.40
61.70	1.7	107.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
63.37	1.8	107.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
65.03	1.8	107.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
66.70	1.9	107.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
68.37	1.9	107.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
70.03	2.0	107.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
71.70	2.0	107.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
71.70	2.7	251.8	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
73.40	2.8	251.8	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
75.10	2.9	251.8	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
76.80	3.0	251.8	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40

78.50	3.0	251.8	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
81.50	3.1	251.8	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40

PILE PROFILE

Lb Top ft	X-Area in²	E-Mod ksi	Spec. Wt lb/ft³	Perim. ft	C-Index -	Wave Sp ft/s	Impedance kips/ft/s
0.00	12.4	30,000	492.00	3.296	0	16,806.4	22.1
85.00	12.4	30,000	492.00	3.296	0	16,806.4	22.1

PILE AND SOIL MODEL Total Capacity Rut (kips): 341.822

Seg. -	Weight kips	Stiffn. kips/in	C-Slk in	T-Slk in	COR -	Ru kips	Js/Jt s/ft	Qs/Qt in	LbTop ft	Perim. ft	X-Area in²
1	0.14	9,482	0.12	0.00	0.85	0.0	0.000	0.10	3.27	3.30	12.4
2	0.14	9,482	0.00	0.00	1.00	8.0	0.150	0.10	6.54	3.30	12.4
3	0.14	9,482	0.00	0.00	1.00	8.6	0.150	0.10	9.81	3.30	12.4
4	0.14	9,482	0.00	0.00	1.00	8.6	0.150	0.10	13.08	3.30	12.4
5	0.14	9,482	0.00	0.00	1.00	4.5	0.127	0.10	16.35	3.30	12.4
6	0.14	9,482	0.00	0.00	1.00	3.6	0.100	0.10	19.62	3.30	12.4
7	0.14	9,482	0.00	0.00	1.00	4.2	0.100	0.10	22.88	3.30	12.4
8	0.14	9,482	0.00	0.00	1.00	7.8	0.143	0.10	26.15	3.30	12.4
9	0.14	9,482	0.00	0.00	1.00	8.7	0.143	0.10	29.42	3.30	12.4
10	0.14	9,482	0.00	0.00	1.00	5.9	0.050	0.10	32.69	3.30	12.4
11	0.14	9,482	0.00	0.00	1.00	6.5	0.050	0.10	35.96	3.30	12.4
12	0.14	9,482	0.00	0.00	1.00	7.9	0.095	0.10	39.23	3.30	12.4
13	0.14	9,482	0.00	0.00	1.00	10.4	0.140	0.10	42.50	3.30	12.4
14	0.14	9,482	0.00	0.00	1.00	11.3	0.150	0.10	45.77	3.30	12.4
15	0.14	9,482	0.00	0.00	1.00	11.2	0.150	0.10	49.04	3.30	12.4
19	0.14	9,482	0.00	0.00	1.00	11.2	0.150	0.10	62.12	3.30	12.4
20	0.14	9,482	0.00	0.00	1.00	11.7	0.144	0.10	65.38	3.30	12.4
21	0.14	9,482	0.00	0.00	1.00	19.4	0.050	0.10	68.65	3.30	12.4
22	0.14	9,482	0.00	0.00	1.00	20.4	0.050	0.10	71.92	3.30	12.4
23	0.14	9,482	0.00	0.00	1.00	21.4	0.050	0.10	75.19	3.30	12.4
24	0.14	9,482	0.00	0.00	1.00	30.3	0.050	0.10	78.46	3.30	12.4
25	0.14	9,482	0.00	0.00	1.00	31.8	0.050	0.10	81.73	3.30	12.4
26	0.14	9,482	0.00	0.00	1.00	33.1	0.050	0.10	85.00	3.30	12.4
Toe						21.7	0.149	0.10	85.00		

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

OTHER OPTIONS

Pile Damping (%):	1	Pile Damping Fact. (kips/ft/s):	0.443

EXTREMA TABLE at 81.5 FT; HAMMER: D 19-42

Shaft/Toe Gain/Loss Factor = 0.667/1.000

Rut = 341.8 kips Rtoe = 21.7 kips Time Inc. = 0.076 ms

Hammer DELMAG D 19-42 Efficiency 0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	383.1	0.00	30.89	14.43	0.786	19.84
6.5	6.1	394.2	0.49	31.79	13.94	0.755	18.95
9.8	1.8	379.5	0.15	30.61	13.51	0.726	17.54
13.1	0.0	360.1	0.00	29.04	13.25	0.699	16.21
16.3	0.0	339.6	0.00	27.39	13.01	0.673	15.19
19.6	0.0	332.9	0.00	26.85	12.73	0.646	14.50
22.9	0.0	332.7	0.00	26.83	12.31	0.618	13.87
26.2	0.0	331.8	0.00	26.76	11.92	0.590	13.04
29.4	0.0	317.2	0.00	25.58	11.67	0.563	12.02
32.7	0.0	299.2	0.00	24.13	11.38	0.535	11.18
36.0	0.0	297.2	0.00	23.97	11.00	0.507	10.55
39.2	0.0	297.7	0.00	24.01	10.52	0.480	9.87
42.5	0.0	294.0	0.00	23.71	9.96	0.453	9.04
45.8	0.0	281.4	0.00	22.69	9.41	0.426	8.12
49.0	0.0	266.7	0.00	21.50	8.95	0.402	7.26
52.3	0.0	254.1	0.00	20.49	8.53	0.380	6.49
55.6	0.0	238.7	0.00	19.25	8.14	0.358	5.79
58.8	0.0	224.8	0.00	18.13	7.78	0.337	5.14
62.1	0.0	217.2	0.00	17.52	7.45	0.317	4.55
65.4	0.0	205.6	0.00	16.58	7.13	0.299	4.02
68.7	0.0	194.4	0.00	15.68	6.81	0.282	3.50
71.9	0.0	175.1	0.00	14.12	6.53	0.267	2.97
75.2	0.0	163.1	0.00	13.15	6.35	0.253	2.47
78.5	0.0	147.1	0.00	11.87	6.72	0.241	1.92
81.7	0.0	115.1	0.00	9.28	7.75	0.232	1.34
85.0	0.0	70.6	0.00	5.69	8.31	0.227	1.05

Converged Stroke (ft) 7.99 Fixed Combustion Pressure (psi) 1,600.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81 7.80 8.02 7.99

Shaft/Toe Gain/Loss Factor = 1.000/1.000

Rut = 409.8 kips Rtoe = 21.7 kips Time Inc. = 0.076 ms

Hammer DELMAG D 19-42 Efficiency 0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft

3.3	0.0	402.3	0.00	32.45	14.66	0.722	19.87
6.5	6.0	418.8	0.48	33.77	13.92	0.688	18.67
9.8	0.0	396.7	0.00	31.99	13.30	0.653	16.75
13.1	0.0	367.8	0.00	29.66	12.95	0.620	14.90
16.3	0.0	336.5	0.00	27.13	12.67	0.588	13.54
19.6	0.0	327.5	0.00	26.41	12.30	0.557	12.71
22.9	0.0	328.9	0.00	26.53	11.73	0.526	12.02
26.2	0.0	329.6	0.00	26.58	11.20	0.498	11.10
29.4	0.0	315.7	0.00	25.46	10.90	0.469	9.93
32.7	0.0	303.9	0.00	24.50	10.57	0.440	9.04
36.0	0.0	305.4	0.00	24.63	10.10	0.410	8.42
39.2	0.0	305.7	0.00	24.66	9.59	0.380	7.73
42.5	0.0	300.3	0.00	24.22	8.99	0.351	6.88
45.8	0.0	284.0	0.00	22.91	8.42	0.323	5.92
49.0	0.0	264.2	0.00	21.31	7.91	0.297	5.01
52.3	0.0	250.2	0.00	20.18	7.45	0.270	4.19
55.6	0.0	240.8	0.00	19.42	7.01	0.245	3.46
58.8	0.0	224.7	0.00	18.12	6.64	0.222	2.83
62.1	0.0	210.1	0.00	16.95	6.30	0.201	2.31
65.4	0.0	194.6	0.00	15.70	5.98	0.181	1.86
68.7	0.0	177.5	0.00	14.32	5.75	0.163	1.48
71.9	0.0	161.4	0.00	13.02	5.51	0.147	1.18
75.2	0.0	143.0	0.00	11.54	5.37	0.132	0.91
78.5	0.0	124.3	0.00	10.03	5.62	0.119	0.67
81.7	0.0	97.8	0.00	7.89	6.60	0.111	0.44
85.0	0.0	59.8	0.00	4.82	7.28	0.106	0.34

Converged Stroke (ft) 8.35 Fixed Combustion Pressure (psi) 1,600.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81	8.23	8.35	8.35

SUMMARY TABLE at 81.5 FT; HAMMER: D 19-42

Rut kips	Bl Ct b/ft	Stk Dn ft	Stk Up ft	Mx T-Str ksi	LTop ft	Mx C-Str ksi	LTop ft	ENTHRU kip-ft	Bl Rt b/min	ActRes kips
341.8	94.5	7.99	0.00	0.49	6.5	31.79	6.5	19.8	41.7	341.8
409.8	2,164.4	8.35	0.00	0.48	6.5	33.77	6.5	19.9	40.9	409.8

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.667/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	12.6	11.5	1.1	1.1	11.35	1.13	4.01	26.7	D 19-42
10.0	27.1	26.0	1.1	2.2	16.71	0.74	4.58	24.2	D 19-42
15.0	33.8	32.0	1.8	2.8	18.00	0.87	4.75	23.3	D 19-42
20.0	40.1	38.3	1.8	3.4	19.14	1.24	4.90	22.6	D 19-42
25.0	53.0	51.9	1.2	4.8	21.46	1.20	5.20	21.4	D 19-42
30.0	62.5	61.4	1.1	5.7	22.32	1.98	5.38	21.0	D 19-42
35.0	76.7	72.4	4.3	7.1	23.25	1.97	5.61	20.2	D 19-42
40.0	89.8	88.1	1.7	8.6	24.09	1.83	5.84	19.7	D 19-42
45.0	107.1	105.3	1.8	11.1	25.32	1.86	6.14	19.3	D 19-42
50.0	124.3	122.5	1.8	14.0	26.10	1.81	6.41	19.1	D 19-42
55.0	141.5	139.7	1.8	15.8	27.03	1.86	6.61	18.7	D 19-42
60.0	158.7	156.9	1.8	18.0	27.37	1.08	6.81	18.6	D 19-42
65.0	191.5	182.3	9.3	22.3	28.56	1.11	7.07	18.9	D 19-42
70.0	223.0	213.7	9.3	27.2	29.12	1.00	7.27	19.2	D 19-42
75.0	277.3	255.6	21.7	43.9	30.44	1.37	7.60	19.4	D 19-42
80.0	326.6	304.9	21.7	76.4	31.08	0.62	7.93	19.7	D 19-42
81.0	336.9	315.2	21.7	87.7	31.54	0.50	7.97	19.8	D 19-42
81.5	341.8	320.1	21.7	94.5	31.79	0.49	7.99	19.8	D 19-42

G/L at Shaft and Toe: 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	18.3	17.2	1.1	1.4	14.17	0.72	4.26	26.1	D 19-42
10.0	40.0	39.0	1.1	3.6	18.94	0.80	4.92	22.5	D 19-42
15.0	48.3	46.6	1.8	4.4	20.80	1.16	5.10	21.7	D 19-42
20.0	55.9	54.1	1.8	5.1	22.11	1.46	5.26	21.3	D 19-42
25.0	75.5	74.3	1.2	7.5	23.94	2.16	5.65	20.2	D 19-42
30.0	85.6	84.4	1.1	8.3	24.54	2.25	5.79	19.9	D 19-42
35.0	100.7	96.4	4.3	10.0	25.53	2.12	6.01	19.5	D 19-42
40.0	120.5	118.7	1.7	13.1	26.53	2.02	6.34	19.3	D 19-42
45.0	146.3	144.5	1.8	16.2	27.78	2.10	6.63	18.9	D 19-42
50.0	172.1	170.3	1.8	19.5	28.34	1.26	6.91	19.1	D 19-42
55.0	197.9	196.1	1.8	23.6	29.64	2.34	7.16	19.4	D 19-42
60.0	223.7	221.9	1.8	29.1	29.81	1.81	7.37	19.5	D 19-42
65.0	259.5	250.2	9.3	39.0	31.23	2.79	7.66	19.7	D 19-42
70.0	291.0	281.7	9.3	52.1	31.35	2.04	7.87	19.7	D 19-42
75.0	345.2	323.6	21.7	106.6	32.89	1.42	8.19	20.1	D 19-42
80.0	394.6	372.9	21.7	476.5	32.98	0.51	8.33	20.1	D 19-42

81.0	404.8	383.1	21.7	1,155.3	33.51	0.49	8.35	19.9	D 19-42
81.5	409.8	388.1	21.7	2,164.4	33.77	0.48	8.35	19.9	D 19-42

PIER 1

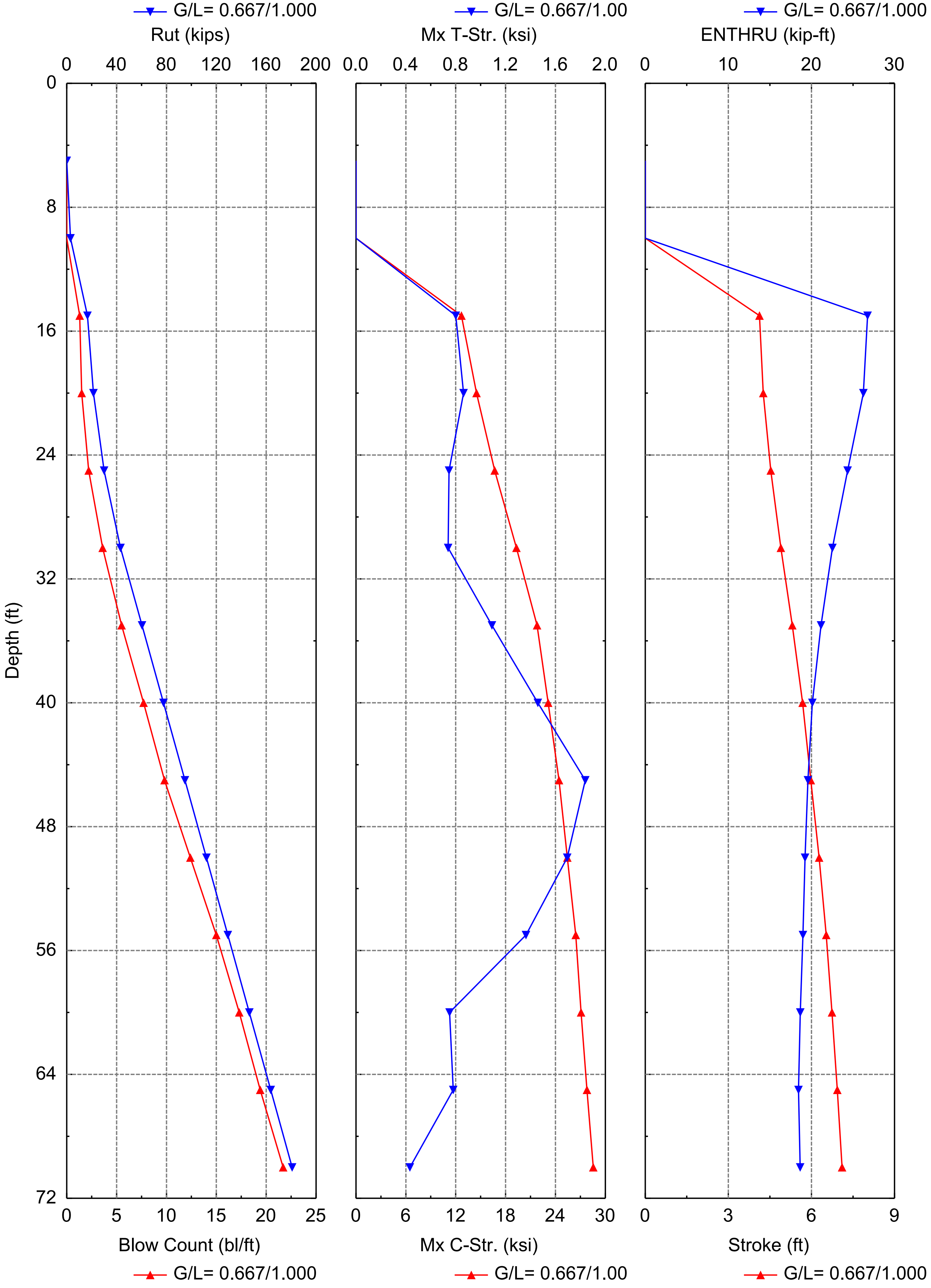
Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 0.667/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.3	0.000	0.000	10.81	0.0	D 19-42
10.0	3.0	1.8	1.2	0.3	0.000	0.000	10.81	0.0	D 19-42
15.0	16.7	15.8	0.9	1.3	12.695	0.800	4.12	26.7	D 19-42
20.0	21.4	20.3	1.1	1.5	14.499	0.863	4.26	26.2	D 19-42
25.0	30.0	26.5	3.5	2.2	16.696	0.746	4.53	24.4	D 19-42
30.0	43.2	41.4	1.7	3.6	19.307	0.739	4.90	22.5	D 19-42
35.0	60.4	58.6	1.8	5.5	21.792	1.090	5.31	21.2	D 19-42
40.0	77.6	75.8	1.8	7.7	23.126	1.459	5.68	20.1	D 19-42
45.0	94.8	93.0	1.8	9.8	24.431	1.840	5.98	19.6	D 19-42
50.0	112.0	110.3	1.8	12.4	25.431	1.695	6.28	19.2	D 19-42
55.0	129.2	127.5	1.8	15.0	26.437	1.363	6.53	19.0	D 19-42
60.0	146.4	144.7	1.8	17.3	27.088	0.751	6.74	18.7	D 19-42
65.0	163.6	161.9	1.8	19.4	27.794	0.780	6.93	18.4	D 19-42
70.0	180.8	179.1	1.8	21.7	28.556	0.432	7.11	18.6	D 19-42

Total driving time: 11 minutes; Total Number of Blows: 535 (starting at penetration 5.0 ft)

0.0 ft @ Ele. 1006 ft amsl (Approximate existing ground surface elevation)
Pile Cap Ele. = 1000.0 ft amsl
6 ft of overburden above driving elevation (i.e., for pier cap in cast excavation)


G/L= 0.667/1.000
Rut (kips)
0
40
80
120
160
200
G/L= 0.667/1.00
Mx T-Str. (ksi)
0.0
0.4
0.8
1.2
1.6
2.0
G/L= 0.667/1.000
ENTHRU (kip-ft)
0
10
20
30
Depth (ft)
0
8
16
24
32
40
48
56
64
72
0
5
10
15
20
25
Blow Count (bl/ft)
G/L= 0.667/1.000
0
6
12
18
24
30
Mx C-Str. (ksi)
G/L= 0.667/1.00
0
3
6
9
Stroke (ft)
G/L= 0.667/1.000

SOIL PROFILE

Depth ft	Soil Type -	Spec. Wt lb/ft³	Su ksf	Phi °	Unit Rs ksf	Unit Rt ksf
0.0	Sand	120.0	0.0	0.0	0.00	0.00
6.0	Sand	120.0	0.0	0.0	0.00	0.61
6.0	Sand	120.0	0.0	31.0	0.11	7.34
9.9	Sand	120.0	0.0	31.0	0.18	11.13
9.9	Clay	120.0	1.5	0.0	1.29	13.50
14.9	Clay	120.0	1.5	0.0	1.29	13.50
14.9	Sand	118.0	0.0	29.0	0.23	10.31
22.4	Sand	118.0	0.0	29.0	0.35	13.32
22.4	Sand	122.0	0.0	33.0	0.48	35.94
26.2	Sand	122.0	0.0	33.0	0.56	42.32
26.2	Clay	122.0	2.2	0.0	1.57	20.25
31.2	Clay	122.0	2.2	0.0	1.57	20.25
31.2	Clay	122.0	2.3	0.0	1.57	20.70
51.2	Clay	122.0	2.3	0.0	1.57	20.70
51.2	Clay	122.0	2.3	0.0	1.57	20.70
61.2	Clay	122.0	2.3	0.0	1.57	20.70
61.2	Clay	122.0	2.3	0.0	1.57	20.70
69.7	Clay	122.0	2.3	0.0	1.57	20.70

Baseline Rd Bridge Replacement over Ashalnd Railway + Pier 1 - HP10x42 6/30/2025
NATIONAL ENGINEERING AND ARCHITECTURAL GRLWEAP 14.1.20.1

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

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

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

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

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

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

HAMMER DATA

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

Hammer and Drive System Segment Data

Segment -	Weight kips	Stiffness kips/in	COR -	C-Slack in	Damping kips/ft/s
1	0.800	140,084.4	1.000	0.000	
2	0.800	140,084.4	1.000	0.000	
3	0.800	140,084.4	1.000	0.000	
4	0.800	140,084.4	1.000	0.000	
5	0.800	70,754.7	0.900	0.120	
Imp Block	0.753	109,976.0	0.800	0.120	
Helmet	2.700				5.3

Ram Weight: (kips)	4.00	Ram Length: (ft)	10.76
Ram Area: (in²)	124.69		
Maximum (Eq) Stroke: (ft)	10.81	Actual (Eq) Stroke: (ft)	10.81
Efficiency:	0.800	Rated Energy: (kip-ft)	43.24
Maximum Pressure: (psi)	1,600.00	Actual Pressure: (psi)	1,600.00
Combustion Delay: (ms)	2.00	Ignition Duration: (ms)	2.00
Expansion Exponent:	1.25		

Hammer Cushion		Pile Cushion	
Cross Sect. Area: (in²)	415.00	Cross Sect. Area: (in²)	0.00
Elastic Modulus: (ksi)	530.0	Elastic Modulus: (ksi)	0.0
Thickness: (in)	2.00	Thickness: (in)	0.00
Coeff. of Restitution:	0.800	Coeff. of Restitution:	0.500
RoundOut: (in)	0.120	RoundOut: (in)	0.120
Stiffness: (kips/in)	109,976.0	Stiffness: (kips/in)	0.0
Helmet Weight: (kips)	2.700		

PILE INPUT

Uniform Pile		Pile Type:	H Pile
Pile Length: (ft)	75.000	Pile Penetration: (ft)	70.000
Pile Size: (ft)	0.84	Toe Area: (in²)	12.40

Table of Depths Analyzed with Driving System Modifiers

Depth ft	Temp Length ft	Wait Time Hr	Hammer -
5.00	75.0	0.0	DELMAG D 19-42
10.00	75.0	0.0	DELMAG D 19-42
15.00	75.0	0.0	DELMAG D 19-42
20.00	75.0	0.0	DELMAG D 19-42
25.00	75.0	0.0	DELMAG D 19-42
30.00	75.0	0.0	DELMAG D 19-42
35.00	75.0	0.0	DELMAG D 19-42
40.00	75.0	0.0	DELMAG D 19-42
45.00	75.0	0.0	DELMAG D 19-42
50.00	75.0	0.0	DELMAG D 19-42
55.00	75.0	0.0	DELMAG D 19-42
60.00	75.0	0.0	DELMAG D 19-42
65.00	75.0	0.0	DELMAG D 19-42
70.00	75.0	0.0	DELMAG D 19-42

Other Information for DELMAG D 19-42

Depth ft	Stroke ft	Diesel Pressure %	Efficiency -	P.C. Stiff. Fact. -	P.C. COR -
5.00	10.8	100.0	0.80	1.0	0.50
10.00	10.8	100.0	0.80	1.0	0.50
15.00	10.8	100.0	0.80	1.0	0.50
20.00	10.8	100.0	0.80	1.0	0.50
25.00	10.8	100.0	0.80	1.0	0.50
30.00	10.8	100.0	0.80	1.0	0.50
35.00	10.8	100.0	0.80	1.0	0.50
40.00	10.8	100.0	0.80	1.0	0.50
45.00	10.8	100.0	0.80	1.0	0.50
50.00	10.8	100.0	0.80	1.0	0.50
55.00	10.8	100.0	0.80	1.0	0.50
60.00	10.8	100.0	0.80	1.0	0.50
65.00	10.8	100.0	0.80	1.0	0.50
70.00	10.8	100.0	0.80	1.0	0.50

PILE, SOIL, ANALYSIS OPTIONS

Analysis type:	Driveability Analysis	Soil Damping Option:	Smith
Max No Analysis Iterations:	0	Time Increment/Critical:	160
Residual Stress Analysis:	0	Analysis Time-Input(ms):	0
Output Level:	Normal	Gravitational Acceleration (ft/s²):	32.169
Hammer Gravity (ft/s²):	32.169	Pile Gravity (ft/s²):	32.169

DRIVEABILITY ANALYSIS

Analysis Depth (ft)	70.00	Standard Soil Setup	
Hammer Name	DELMAG D 19-42	Hammer ID	41
Diesel Pressure: (psi)	230.40	Stroke (ft)	10.81
Efficiency	0.80		
Shaft Gain/Loss Factor	0.667	Toe Gain/Loss Factor	1.000
Shaft Gain/Loss Factor	1.000	Toe Gain/Loss Factor	1.000

SOIL RESISTANCE PARAMETERS

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Setup F. -	Limit D. ft	Setup T. Hours	EB Area in²
0.00	0.0	0.0	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
2.00	0.0	0.2	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
4.00	0.0	0.4	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
6.00	0.0	0.6	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
6.00	0.1	7.3	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
7.95	0.1	9.3	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
9.90	0.2	11.1	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
9.90	1.3	13.5	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
14.90	1.3	13.5	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
14.90	0.2	10.3	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
16.77	0.3	11.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
18.65	0.3	12.8	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
20.52	0.3	13.3	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
22.40	0.3	13.3	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
22.40	0.5	35.9	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
24.30	0.5	39.1	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
26.20	0.6	42.3	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
26.20	1.6	20.2	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
31.20	1.6	20.2	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
31.20	1.6	20.7	0.10	0.100	0.150	0.1	1.5	6.56	24.0	12.40
70.00	1.6	20.7	0.10	0.100	0.150	0.1	1.5	6.56	24.0	12.40

PILE PROFILE

Lb Top ft	X-Area in²	E-Mod ksi	Spec. Wt lb/ft³	Perim. ft	C-Index -	Wave Sp ft/s	Impedance kips/ft/s
0.00	12.4	30,000	492.00	3.296	0	16,806.4	22.1
75.00	12.4	30,000	492.00	3.296	0	16,806.4	22.1

PILE AND SOIL MODEL

Total Capacity Rut (kips): 180.844

Seg. -	Weight kips	Stiffn. kips/in	C-Slk in	T-Slk in	COR -	Ru kips	Js/Jt s/ft	Qs/Qt in	LbTop ft	Perim. ft	X-Area in²

1	0.14	9,507	0.12	0.00	0.85	0.0	0.000	0.10	3.26	3.30	12.4
2	0.14	9,507	0.00	0.00	1.00	0.0	0.000	0.10	6.52	3.30	12.4
3	0.14	9,507	0.00	0.00	1.00	0.0	0.000	0.10	9.78	3.30	12.4
4	0.14	9,507	0.00	0.00	1.00	0.7	0.100	0.10	13.04	3.30	12.4
5	0.14	9,507	0.00	0.00	1.00	4.8	0.143	0.10	16.30	3.30	12.4
6	0.14	9,507	0.00	0.00	1.00	9.2	0.150	0.10	19.57	3.30	12.4
7	0.14	9,507	0.00	0.00	1.00	3.4	0.086	0.10	22.83	3.30	12.4
8	0.14	9,507	0.00	0.00	1.00	3.3	0.050	0.10	26.09	3.30	12.4
9	0.14	9,507	0.00	0.00	1.00	4.1	0.084	0.10	29.35	3.30	12.4
10	0.14	9,507	0.00	0.00	1.00	7.6	0.134	0.10	32.61	3.30	12.4
11	0.14	9,507	0.00	0.00	1.00	11.3	0.150	0.10	35.87	3.30	12.4
12	0.14	9,507	0.00	0.00	1.00	11.2	0.150	0.10	39.13	3.30	12.4
23	0.14	9,507	0.00	0.00	1.00	11.2	0.150	0.10	75.00	3.30	12.4
Toe						1.8	0.149	0.10	75.00		

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

OTHER OPTIONS

Pile Damping (%):	1	Pile Damping Fact. (kips/ft/s):	0.443

EXTREMA TABLE at 70.0 FT; HAMMER: D 19-42

Shaft/Toe Gain/Loss Factor = 0.667/1.000

Rut = 180.8 kips Rtoe = 1.8 kips Time Inc. = 0.076 ms

Hammer DELMAG D 19-42 Efficiency 0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	339.0	0.00	27.34	13.64	0.852	18.65
6.5	2.0	340.3	0.16	27.44	13.55	0.836	18.56
9.8	3.8	342.6	0.31	27.63	13.35	0.819	18.46
13.0	5.2	348.4	0.42	28.10	13.01	0.801	18.30
16.3	5.4	354.1	0.43	28.56	12.64	0.782	17.84
19.6	1.2	344.9	0.10	27.81	12.46	0.764	16.86
22.8	0.0	325.2	0.00	26.22	12.29	0.749	16.05
26.1	0.0	324.0	0.00	26.13	12.05	0.735	15.70
29.3	0.0	322.3	0.00	25.99	11.67	0.721	15.33
32.6	0.0	325.5	0.00	26.25	11.16	0.710	14.73
35.9	0.0	318.6	0.00	25.69	10.62	0.700	13.72
39.1	0.0	301.6	0.00	24.33	10.08	0.691	12.55
42.4	0.0	285.1	0.00	23.00	9.53	0.683	11.40
45.7	0.0	269.2	0.00	21.71	9.01	0.676	10.29
48.9	0.0	253.6	0.00	20.45	8.49	0.669	9.20
52.2	0.0	238.4	0.00	19.22	8.17	0.664	8.13
55.4	0.0	223.4	0.00	18.01	8.21	0.659	7.07
58.7	0.0	208.1	0.00	16.78	7.79	0.655	6.02
62.0	0.0	189.7	0.00	15.30	7.86	0.653	4.97
65.2	2.1	169.7	0.17	13.69	8.49	0.653	3.92
68.5	3.4	148.5	0.27	11.98	9.55	0.652	2.86
71.7	3.2	115.2	0.26	9.29	9.94	0.653	1.79
75.0	1.7	64.6	0.14	5.21	10.55	0.653	1.26

Converged Stroke (ft) 7.11 Fixed Combustion Pressure (psi) 1,600.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81	6.73	7.17	7.11

Shaft/Toe Gain/Loss Factor = 1.000/1.000

Rut = 264.7 kips Rtoe = 1.8 kips Time Inc. = 0.076 ms

Hammer DELMAG D 19-42 Efficiency 0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	362.3	0.00	29.22	14.65	0.800	19.45
6.5	6.3	370.1	0.51	29.85	14.53	0.771	19.14
9.8	12.0	377.9	0.97	30.48	14.30	0.742	18.81

13.0	16.6	385.1	1.34	31.05	13.80	0.711	18.41
16.3	19.4	392.0	1.56	31.61	13.23	0.680	17.60
19.6	15.2	389.1	1.22	31.38	12.99	0.650	16.12
22.8	4.2	373.0	0.34	30.08	12.80	0.620	14.90
26.1	4.8	371.8	0.39	29.99	12.48	0.592	14.34
29.3	6.4	367.8	0.51	29.66	11.96	0.565	13.81
32.6	5.8	363.4	0.47	29.31	11.20	0.540	13.02
35.9	0.0	353.7	0.00	28.52	10.34	0.516	11.79
39.1	0.0	331.8	0.00	26.76	9.51	0.494	10.44
42.4	0.0	308.6	0.00	24.89	8.83	0.475	9.23
45.7	0.0	280.3	0.00	22.61	8.24	0.460	8.14
48.9	0.0	245.0	0.00	19.76	7.72	0.446	7.12
52.2	0.0	224.0	0.00	18.07	7.25	0.434	6.17
55.4	0.0	203.8	0.00	16.43	6.84	0.424	5.28
58.7	0.0	183.3	0.00	14.78	6.46	0.417	4.43
62.0	0.0	164.0	0.00	13.23	6.12	0.410	3.62
65.2	0.0	150.6	0.00	12.14	6.18	0.405	2.82
68.5	0.0	133.1	0.00	10.73	6.67	0.401	2.04
71.7	0.0	104.0	0.00	8.39	7.83	0.399	1.26
75.0	0.0	58.7	0.00	4.73	8.85	0.397	0.87

Converged Stroke (ft) 7.81 Fixed Combustion Pressure (psi) 1,600.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81 7.67 7.84 7.81

SUMMARY TABLE at 70.0 FT; HAMMER: D 19-42

Rut	Bl Ct	Stk Dn	Stk Up	Mx T-Str	LTop	Mx C-Str	LTop	ENTHRU	Bl Rt	ActRes
kips	b/ft	ft	ft	ksi	ft	ksi	ft	kip-ft	b/min	kips
180.8	21.7	7.11	0.00	0.43	16.3	28.56	16.3	18.6	44.1	180.8
264.7	40.3	7.81	0.00	1.56	16.3	31.61	16.3	19.5	42.2	264.7

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.667/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.0	0.00	0.00	10.81	0.0	D 19-42
10.0	3.0	1.8	1.2	0.0	0.00	0.00	10.81	0.0	D 19-42
15.0	16.7	15.8	0.9	1.3	12.70	0.80	4.12	26.7	D 19-42
20.0	21.4	20.3	1.1	1.5	14.50	0.86	4.26	26.2	D 19-42
25.0	30.0	26.5	3.5	2.2	16.70	0.75	4.53	24.4	D 19-42
30.0	43.2	41.4	1.7	3.6	19.31	0.74	4.90	22.5	D 19-42
35.0	60.4	58.6	1.8	5.5	21.79	1.09	5.31	21.2	D 19-42
40.0	77.6	75.8	1.8	7.7	23.13	1.46	5.68	20.1	D 19-42
45.0	94.8	93.0	1.8	9.8	24.43	1.84	5.98	19.6	D 19-42
50.0	112.0	110.3	1.8	12.4	25.43	1.69	6.28	19.2	D 19-42
55.0	129.2	127.5	1.8	15.0	26.44	1.36	6.53	19.0	D 19-42
60.0	146.4	144.7	1.8	17.3	27.09	0.75	6.74	18.7	D 19-42
65.0	163.6	161.9	1.8	19.4	27.79	0.78	6.93	18.4	D 19-42
70.0	180.8	179.1	1.8	21.7	28.56	0.43	7.11	18.6	D 19-42

G/L at Shaft and Toe: 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.0	0.00	0.00	10.81	0.0	D 19-42
10.0	3.4	2.3	1.2	0.0	0.00	0.00	10.81	0.0	D 19-42
15.0	24.1	23.2	0.9	1.9	15.40	0.72	4.40	25.2	D 19-42
20.0	28.8	27.7	1.1	2.2	16.58	0.88	4.53	24.4	D 19-42
25.0	38.1	34.6	3.5	3.0	18.27	1.12	4.76	23.2	D 19-42
30.0	58.2	56.5	1.7	5.2	21.73	1.17	5.25	21.3	D 19-42
35.0	84.1	82.3	1.8	8.4	24.08	2.21	5.78	19.9	D 19-42
40.0	109.9	108.1	1.8	11.9	25.59	1.70	6.21	19.4	D 19-42
45.0	135.7	133.9	1.8	15.4	27.02	1.69	6.56	19.1	D 19-42
50.0	161.5	159.7	1.8	18.7	28.05	1.45	6.85	19.2	D 19-42
55.0	187.3	185.5	1.8	22.3	29.05	1.86	7.13	19.3	D 19-42
60.0	213.1	211.3	1.8	26.7	29.61	1.92	7.38	19.5	D 19-42
65.0	238.9	237.1	1.8	32.6	30.41	2.04	7.59	19.4	D 19-42
70.0	264.7	262.9	1.8	40.3	31.61	1.56	7.81	19.5	D 19-42

PIER 2

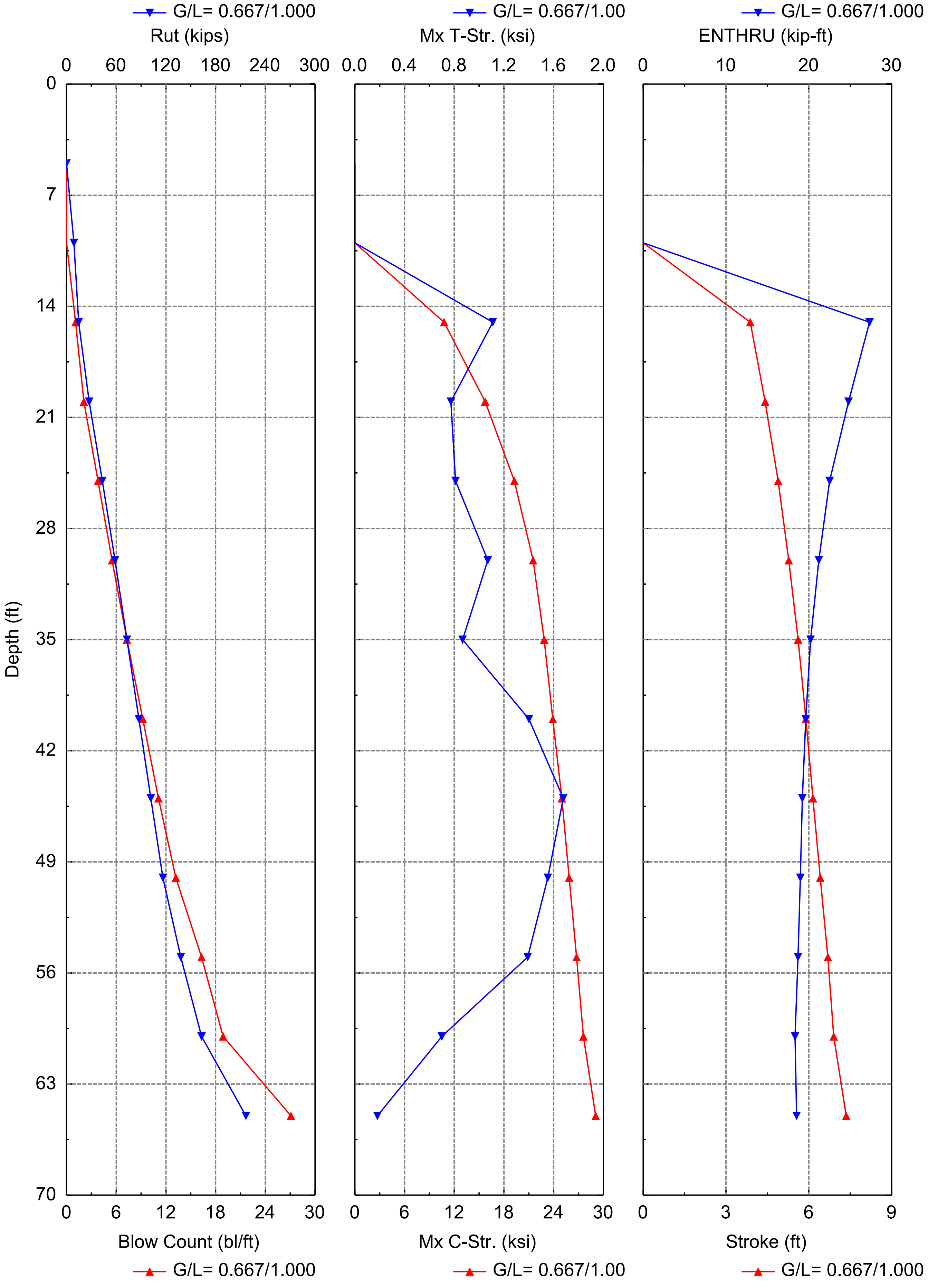
Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 0.667/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.3	0.000	0.000	10.81	0.0	D 19-42
10.0	9.0	8.7	0.3	0.0	0.000	0.000	0.00	0.0	D 19-42
15.0	14.7	12.6	2.1	1.1	10.768	1.110	3.88	27.3	D 19-42
20.0	27.3	25.9	1.4	2.1	15.731	0.773	4.42	24.8	D 19-42
25.0	43.2	41.8	1.4	3.8	19.249	0.810	4.89	22.5	D 19-42
30.0	58.5	56.2	2.3	5.5	21.518	1.070	5.27	21.2	D 19-42
35.0	73.0	70.6	2.3	7.3	22.861	0.868	5.61	20.2	D 19-42
40.0	87.4	85.1	2.3	9.2	23.900	1.401	5.89	19.6	D 19-42
45.0	101.8	99.5	2.3	11.1	24.993	1.678	6.15	19.2	D 19-42
50.0	116.3	113.9	2.3	13.2	25.870	1.553	6.41	19.0	D 19-42
55.0	137.9	133.6	4.3	16.3	26.758	1.391	6.69	18.7	D 19-42
60.0	163.0	156.6	6.3	18.9	27.596	0.701	6.90	18.3	D 19-42
65.0	216.6	189.6	27.0	27.1	29.072	0.181	7.35	18.5	D 19-42

Total driving time: 11 minutes; Total Number of Blows: 510 (starting at penetration 5.0 ft)

0.0 ft @ Ele. 1006 ft amsl (Approximate existing ground surface elevation)
Pile Cap Ele. = 1000.0 ft amsl
6 ft of overburden above driving elevation (i.e., for pier cap in cast excavation)


G/L= 0.667/1.000
Rut (kips)
0
60
120
180
240
300
G/L= 0.667/1.00
Mx T-Str. (ksi)
0.0
0.4
0.8
1.2
1.6
2.0
G/L= 0.667/1.000
ENTHRU (kip-ft)
0
10
20
30
Depth (ft)
0
7
14
21
28
35
42
49
56
63
70
0
6
12
18
24
30
Blow Count (bl/ft)
G/L= 0.667/1.000
0
6
12
18
24
30
Mx C-Str. (ksi)
G/L= 0.667/1.00
0
3
6
9
Stroke (ft)
G/L= 0.667/1.000

SOIL PROFILE

Depth ft	Soil Type -	Spec. Wt lb/ft³	Su ksf	Phi °	Unit Rs ksf	Unit Rt ksf
0.0	Sand	120.0	0.0	0.0	0.00	0.00
6.0	Sand	120.0	0.0	0.0	0.00	0.61
6.0	Clay	122.0	1.9	0.0	1.37	17.10
8.7	Clay	122.0	1.9	0.0	1.37	17.10
8.7	Sand	115.0	0.0	26.0	0.11	3.72
11.7	Sand	115.0	0.0	26.0	0.15	4.34
11.7	Sand	125.0	0.0	33.0	0.25	19.00
16.2	Sand	125.0	0.0	33.0	0.35	26.66
16.2	Clay	122.0	1.8	0.0	1.44	16.20
25.0	Clay	122.0	1.8	0.0	1.44	16.20
25.0	Clay	125.0	3.0	0.0	1.31	27.00
45.0	Clay	125.0	3.0	0.0	1.31	27.00
45.0	Clay	125.0	3.0	0.0	1.31	27.00
50.0	Clay	125.0	3.0	0.0	1.31	27.00
50.0	Sand	122.0	0.0	33.0	1.13	50.11
55.0	Sand	122.0	0.0	33.0	1.25	50.11
55.0	Sand	120.0	0.0	34.0	1.34	73.64
60.0	Sand	120.0	0.0	34.0	1.46	73.64
60.0	Sand	120.0	0.0	39.0	2.31	313.69
66.0	Sand	120.0	0.0	39.0	2.53	313.69

GRLWEAP: Wave Equation Analysis of Pile Foundations

Baseline Rd Bridge Replacement over Ashalnd Railway + Forward Abutment - HP10x42 6/30/2025
NATIONAL ENGINEERING AND ARCHITECTURAL GRLWEAP 14.1.20.1

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

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

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

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

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

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

HAMMER DATA

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

Hammer and Drive System Segment Data

Segment -	Weight kips	Stiffness kips/in	COR -	C-Slack in	Damping kips/ft/s
1	0.800	140,084.4	1.000	0.000	
2	0.800	140,084.4	1.000	0.000	
3	0.800	140,084.4	1.000	0.000	
4	0.800	140,084.4	1.000	0.000	
5	0.800	70,754.7	0.900	0.120	
Imp Block	0.753	109,976.0	0.800	0.120	
Helmet	2.700				5.3

Ram Weight: (kips)	4.00	Ram Length: (ft)	10.76
Ram Area: (in²)	124.69		
Maximum (Eq) Stroke: (ft)	10.81	Actual (Eq) Stroke: (ft)	10.81
Efficiency:	0.800	Rated Energy: (kip-ft)	43.24
Maximum Pressure: (psi)	1,600.00	Actual Pressure: (psi)	1,600.00
Combustion Delay: (ms)	2.00	Ignition Duration: (ms)	2.00
Expansion Exponent:	1.25		

Hammer Cushion		Pile Cushion	
Cross Sect. Area: (in²)	415.00	Cross Sect. Area: (in²)	0.00
Elastic Modulus: (ksi)	530.0	Elastic Modulus: (ksi)	0.0
Thickness: (in)	2.00	Thickness: (in)	0.00
Coeff. of Restitution:	0.800	Coeff. of Restitution:	0.500
RoundOut: (in)	0.120	RoundOut: (in)	0.120
Stiffness: (kips/in)	109,976.0	Stiffness: (kips/in)	0.0
Helmet Weight: (kips)	2.700		

PILE INPUT

Uniform Pile		Pile Type:	H Pile
Pile Length: (ft)	65.000	Pile Penetration: (ft)	65.000
Pile Size: (ft)	0.84	Toe Area: (in²)	12.40

Table of Depths Analyzed with Driving System Modifiers

Depth ft	Temp Length ft	Wait Time Hr	Hammer -
5.00	65.0	0.0	DELMAG D 19-42
10.00	65.0	0.0	DELMAG D 19-42
15.00	65.0	0.0	DELMAG D 19-42
20.00	65.0	0.0	DELMAG D 19-42
25.00	65.0	0.0	DELMAG D 19-42
30.00	65.0	0.0	DELMAG D 19-42
35.00	65.0	0.0	DELMAG D 19-42
40.00	65.0	0.0	DELMAG D 19-42
45.00	65.0	0.0	DELMAG D 19-42
50.00	65.0	0.0	DELMAG D 19-42
55.00	65.0	0.0	DELMAG D 19-42
60.00	65.0	0.0	DELMAG D 19-42
65.00	65.0	0.0	DELMAG D 19-42

Other Information for DELMAG D 19-42

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

PILE, SOIL, ANALYSIS OPTIONS

Analysis type:	Driveability Analysis	Soil Damping Option:	Smith
Max No Analysis Iterations:	0	Time Increment/Critical:	160
Residual Stress Analysis:	0	Analysis Time-Input(ms):	0
Output Level:	Normal	Gravitational Acceleration (ft/s²):	32.169
Hammer Gravity (ft/s²):	32.169	Pile Gravity (ft/s²):	32.169

DRIVEABILITY ANALYSIS

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

SOIL RESISTANCE PARAMETERS

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Setup F. -	Limit D. ft	Setup T. Hours	EB Area in²
0.00	0.0	0.0	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
2.00	0.0	0.2	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
4.00	0.0	0.4	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
6.00	0.0	0.6	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
6.00	1.4	17.1	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
8.70	1.4	17.1	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
8.70	0.1	3.7	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
10.20	0.1	4.1	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
11.70	0.1	4.3	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
11.70	0.2	19.0	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
13.95	0.3	22.7	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
16.20	0.4	26.7	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
16.20	1.4	16.2	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
25.00	1.4	16.2	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
25.00	1.3	27.0	0.10	0.100	0.150	0.1	1.5	6.56	24.0	12.40
45.00	1.3	27.0	0.10	0.100	0.150	0.1	1.5	6.56	24.0	12.40
45.00	1.3	27.0	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
50.00	1.3	27.0	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
50.00	1.1	50.1	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
51.67	1.2	50.1	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
55.00	1.2	50.1	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
55.00	1.3	73.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
56.67	1.4	73.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
58.33	1.4	73.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
60.00	1.5	73.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
60.00	2.3	313.7	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
62.00	2.4	313.7	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
64.00	2.5	313.7	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
65.00	2.5	313.7	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40

PILE PROFILE

Lb Top ft	X-Area in²	E-Mod ksi	Spec. Wt lb/ft³	Perim. ft	C-Index -	Wave Sp ft/s	Impedance kips/ft/s
0.00	12.4	30,000	492.00	3.296	0	16,806.4	22.1
65.00	12.4	30,000	492.00	3.296	0	16,806.4	22.1

PILE AND SOIL MODEL Total Capacity Rut (kips): 216.572

Seg. -	Weight kips	Stiffn. kips/in	C-Slk in	T-Slk in	COR -	Ru kips	Js/Jt s/ft	Qs/Qt in	LbTop ft	Perim. ft	X-Area in²
1	0.14	9,539	0.12	0.00	0.85	0.0	0.000	0.10	3.25	3.30	12.4
2	0.14	9,539	0.00	0.00	1.00	1.5	0.150	0.10	6.50	3.30	12.4
3	0.14	9,539	0.00	0.00	1.00	7.0	0.146	0.10	9.75	3.30	12.4
4	0.14	9,539	0.00	0.00	1.00	2.0	0.050	0.10	13.00	3.30	12.4
5	0.14	9,539	0.00	0.00	1.00	3.5	0.057	0.10	16.25	3.30	12.4
6	0.14	9,539	0.00	0.00	1.00	10.3	0.150	0.10	19.50	3.30	12.4
7	0.14	9,539	0.00	0.00	1.00	10.3	0.150	0.10	22.75	3.30	12.4
8	0.14	9,539	0.00	0.00	1.00	10.0	0.150	0.10	26.00	3.30	12.4
9	0.14	9,539	0.00	0.00	1.00	9.4	0.150	0.10	29.25	3.30	12.4
15	0.14	9,539	0.00	0.00	1.00	9.4	0.150	0.10	48.75	3.30	12.4
16	0.14	9,539	0.00	0.00	1.00	11.2	0.092	0.10	52.00	3.30	12.4
17	0.14	9,539	0.00	0.00	1.00	13.1	0.050	0.10	55.25	3.30	12.4
18	0.14	9,539	0.00	0.00	1.00	14.8	0.050	0.10	58.50	3.30	12.4
19	0.14	9,539	0.00	0.00	1.00	18.4	0.083	0.10	61.75	3.30	12.4
20	0.14	9,539	0.00	0.00	1.00	21.7	0.100	0.10	65.00	3.30	12.4
Toe						27.0	0.149	0.10	65.00		

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

OTHER OPTIONS

Pile Damping (%):	1	Pile Damping Fact. (kips/ft/s):	0.443

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

Shaft/Toe Gain/Loss Factor = 0.667/1.000

Rut = 216.6 kips   Rtoe = 27.0 kips   Time Inc. = 0.076 ms

Hammer   DELMAG D 19-42   Efficiency   0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	354.1	0.00	28.56	13.81	0.782	18.51
6.5	2.1	360.5	0.17	29.07	13.51	0.760	18.25
9.8	2.2	359.6	0.18	29.00	13.33	0.739	17.55
13.0	0.0	343.7	0.00	27.72	13.02	0.720	16.91
16.3	0.0	350.9	0.00	28.30	12.50	0.702	16.59
19.5	0.0	356.3	0.00	28.73	11.97	0.685	15.79
22.8	0.0	339.0	0.00	27.34	11.47	0.670	14.62
26.0	0.0	321.8	0.00	25.95	10.97	0.656	13.51
29.3	0.0	306.0	0.00	24.68	10.49	0.642	12.49
32.5	0.0	292.1	0.00	23.56	10.01	0.629	11.55
35.8	0.0	278.4	0.00	22.45	9.53	0.617	10.64
39.0	0.0	265.2	0.00	21.39	9.06	0.606	9.77
42.3	0.0	252.3	0.00	20.35	8.61	0.596	8.93
45.5	0.0	239.4	0.00	19.31	8.27	0.587	8.11
48.8	0.0	225.8	0.00	18.21	7.96	0.578	7.31
52.0	0.0	209.5	0.00	16.90	7.73	0.569	6.54
55.3	0.0	191.3	0.00	15.43	8.61	0.561	5.79
58.5	0.0	174.4	0.00	14.06	9.19	0.554	5.02
61.8	0.0	143.7	0.00	11.59	9.49	0.547	4.05
65.0	0.0	97.3	0.00	7.84	9.72	0.543	3.48

Converged Stroke (ft)   7.35   Fixed Combustion Pressure (psi)   1,600.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81   7.01   7.40   7.35

Shaft/Toe Gain/Loss Factor = 1.000/1.000

Rut = 277.3 kips   Rtoe = 27.0 kips   Time Inc. = 0.076 ms

Hammer   DELMAG D 19-42   Efficiency   0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	384.0	0.00	30.97	14.45	0.715	18.78
6.5	5.9	392.9	0.47	31.69	14.00	0.687	18.35
9.8	9.0	398.8	0.73	32.16	13.79	0.658	17.31
13.0	3.2	385.1	0.25	31.05	13.40	0.629	16.34
16.3	3.9	390.3	0.31	31.47	12.67	0.600	15.83
19.5	2.8	394.2	0.22	31.79	11.87	0.572	14.72

22.8	0.0	372.1	0.00	30.01	11.09	0.545	13.12
26.0	0.0	350.1	0.00	28.23	10.35	0.521	11.69
29.3	0.0	327.8	0.00	26.44	9.63	0.501	10.47
32.5	0.0	305.1	0.00	24.60	8.94	0.482	9.39
35.8	0.0	277.0	0.00	22.34	8.45	0.466	8.38
39.0	0.0	248.8	0.00	20.06	8.01	0.451	7.45
42.3	0.0	230.8	0.00	18.61	7.61	0.439	6.59
45.5	0.0	212.3	0.00	17.12	7.24	0.427	5.78
48.8	0.0	192.3	0.00	15.51	6.93	0.417	5.01
52.0	0.0	172.6	0.00	13.92	6.71	0.408	4.34
55.3	0.0	163.3	0.00	13.17	7.07	0.400	3.82
58.5	0.0	151.6	0.00	12.23	7.28	0.393	3.32
61.8	0.0	127.0	0.00	10.25	8.25	0.387	2.66
65.0	0.0	90.5	0.00	7.30	8.72	0.383	2.26

Converged Stroke (ft) 7.87 Fixed Combustion Pressure (psi) 1,600.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81 7.64 7.89 7.87

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

Rut kips	Bl Ct b/ft	Stk Dn ft	Stk Up ft	Mx T-Str ksi	LTop ft	Mx C-Str ksi	LTop ft	ENTHRU kip-ft	Bl Rt b/min	ActRes kips
216.6	27.1	7.35	0.00	0.18	9.8	29.07	6.5	18.5	43.4	216.6
277.3	42.4	7.87	0.00	0.73	9.8	32.16	9.8	18.8	42.1	277.3

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.667/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.0	0.00	0.00	10.81	0.0	D 19-42
10.0	9.0	8.7	0.3	0.0	0.00	0.00	0.00	0.0	D 19-42
15.0	14.7	12.6	2.1	1.1	10.77	1.11	3.88	27.3	D 19-42
20.0	27.3	25.9	1.4	2.1	15.73	0.77	4.42	24.8	D 19-42
25.0	43.2	41.8	1.4	3.8	19.25	0.81	4.89	22.5	D 19-42
30.0	58.5	56.2	2.3	5.5	21.52	1.07	5.27	21.2	D 19-42
35.0	73.0	70.6	2.3	7.3	22.86	0.87	5.61	20.2	D 19-42
40.0	87.4	85.1	2.3	9.2	23.90	1.40	5.89	19.6	D 19-42
45.0	101.8	99.5	2.3	11.1	24.99	1.68	6.15	19.2	D 19-42
50.0	116.3	113.9	2.3	13.2	25.87	1.55	6.41	19.0	D 19-42
55.0	137.9	133.6	4.3	16.3	26.76	1.39	6.69	18.7	D 19-42
60.0	163.0	156.6	6.3	18.9	27.60	0.70	6.90	18.3	D 19-42
65.0	216.6	189.6	27.0	27.1	29.07	0.18	7.35	18.5	D 19-42

G/L at Shaft and Toe: 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.0	0.00	0.00	10.81	0.0	D 19-42
10.0	13.1	12.7	0.3	1.1	10.40	0.54	3.85	27.2	D 19-42
15.0	18.7	16.6	2.1	1.4	12.58	0.94	4.09	26.8	D 19-42
20.0	37.4	36.0	1.4	3.1	18.17	0.75	4.73	23.2	D 19-42
25.0	61.2	59.8	1.4	5.8	22.11	1.30	5.33	21.1	D 19-42
30.0	83.8	81.4	2.3	8.6	23.81	1.57	5.80	19.9	D 19-42
35.0	105.4	103.1	2.3	11.5	25.41	1.70	6.18	19.3	D 19-42
40.0	127.1	124.7	2.3	14.8	26.50	1.73	6.53	18.9	D 19-42
45.0	148.7	146.4	2.3	17.8	27.61	1.47	6.81	18.6	D 19-42
50.0	170.4	168.0	2.3	20.9	28.48	0.75	7.04	18.6	D 19-42
55.0	192.0	187.7	4.3	23.2	30.31	0.85	7.22	18.5	D 19-42
60.0	217.1	210.7	6.3	26.3	31.26	0.58	7.41	18.4	D 19-42
65.0	277.3	250.2	27.0	42.4	32.16	0.73	7.87	18.8	D 19-42

FORWARD ABUTMENT

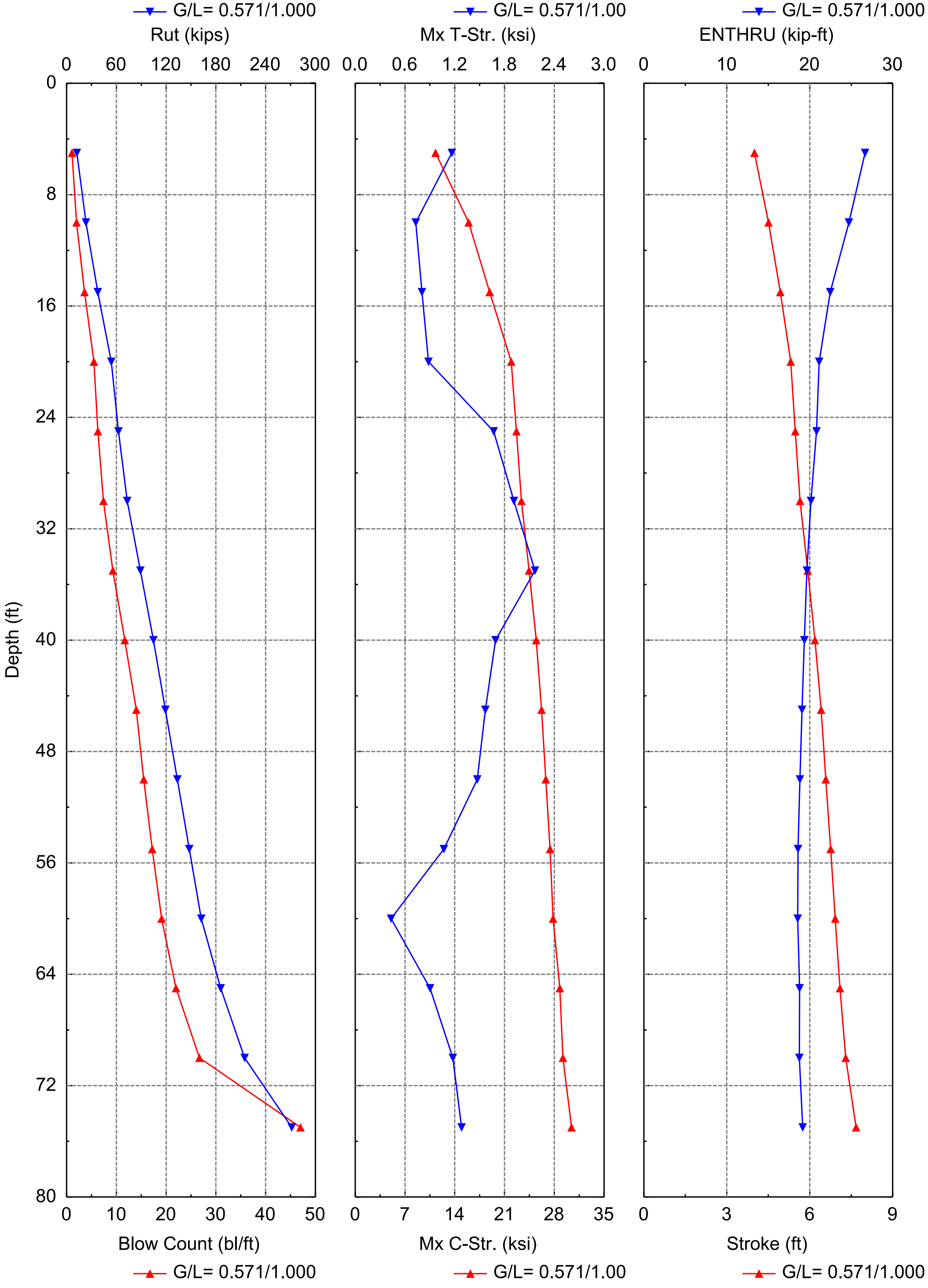
Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 0.571/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	12.5	9.8	2.7	1.1	11.303	1.164	4.00	26.7	D 19-42
10.0	23.3	21.0	2.4	2.0	15.956	0.730	4.51	24.7	D 19-42
15.0	37.6	36.1	1.5	3.6	18.907	0.804	4.93	22.5	D 19-42
20.0	53.9	52.4	1.5	5.5	21.944	0.883	5.31	21.1	D 19-42
25.0	62.6	58.9	3.7	6.3	22.677	1.669	5.47	20.8	D 19-42
30.0	73.1	71.7	1.4	7.4	23.376	1.914	5.65	20.2	D 19-42
35.0	88.9	87.5	1.4	9.3	24.449	2.167	5.93	19.7	D 19-42
40.0	104.7	102.3	2.3	11.7	25.458	1.692	6.19	19.3	D 19-42
45.0	119.1	116.8	2.3	14.0	26.214	1.569	6.41	19.1	D 19-42
50.0	133.5	131.2	2.3	15.5	26.802	1.471	6.58	18.8	D 19-42
55.0	148.0	145.7	2.3	17.2	27.409	1.069	6.76	18.6	D 19-42
60.0	162.4	160.1	2.3	19.1	27.834	0.432	6.92	18.6	D 19-42
65.0	185.8	181.5	4.3	22.0	28.786	0.902	7.09	18.8	D 19-42
70.0	214.7	208.4	6.3	26.7	29.217	1.177	7.30	18.8	D 19-42
75.0	271.5	244.5	27.0	47.0	30.434	1.283	7.68	19.1	D 19-42

Total driving time: 20 minutes; Total Number of Blows: 922 (starting at penetration 5.0 ft)

0.0 ft @ Ele. 1017.25 ft amsl (Bottom of Pile Cap)


G/L= 0.571/1.000
Rut (kips)
0
60
120
180
240
300
G/L= 0.571/1.00
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 0.571/1.000
ENTHRU (kip-ft)
0
10
20
30
Depth (ft)
0
8
16
24
32
40
48
56
64
72
80
0
10
20
30
40
50
Blow Count (bl/ft)
G/L= 0.571/1.000
0
7
14
21
28
35
Mx C-Str. (ksi)
G/L= 0.571/1.00
0
3
6
9
Stroke (ft)
G/L= 0.571/1.000

SOIL PROFILE

Depth ft	Soil Type -	Spec. Wt lb/ft³	Su ksf	Phi °	Unit Rs ksf	Unit Rt ksf
0.0	Clay	125.0	3.5	0.0	0.89	31.50
5.0	Clay	125.0	3.5	0.0	0.89	31.50
5.0	Clay	125.0	3.0	0.0	1.29	27.45
12.5	Clay	125.0	3.0	0.0	1.29	27.45
12.5	Clay	122.0	1.9	31.0	1.48	17.10
20.0	Clay	122.0	1.9	31.0	1.48	17.10
20.0	Sand	115.0	0.0	26.0	0.27	7.55
23.0	Sand	115.0	0.0	26.0	0.31	8.52
23.0	Sand	125.0	0.0	33.0	0.52	39.10
27.5	Sand	125.0	0.0	33.0	0.62	47.04
27.5	Clay	122.0	1.8	0.0	1.44	16.20
36.3	Clay	122.0	1.8	0.0	1.44	16.20
36.3	Clay	125.0	3.0	31.0	1.31	27.00
56.3	Clay	125.0	3.0	31.0	1.31	27.00
56.3	Clay	125.0	3.0	31.0	1.31	27.00
61.3	Clay	125.0	3.0	31.0	1.31	27.00
61.3	Sand	122.0	0.0	33.0	1.40	50.11
66.3	Sand	122.0	0.0	33.0	1.52	50.11
66.3	Sand	120.0	0.0	34.0	1.63	73.64
71.3	Sand	120.0	0.0	34.0	1.75	73.64
71.3	Sand	120.0	0.0	39.0	2.77	313.69
74.0	Sand	120.0	0.0	39.0	2.87	313.69

GRLWEAP: Wave Equation Analysis of Pile Foundations

Baseline Rd Bridge Replacement over Ashalnd Railway + Forward Abutment - TUR01x42 6/29/2025
NATIONAL ENGINEERING AND ARCHITECTURAL GRLWEAP 14.1.20.1

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

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

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

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

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

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

HAMMER DATA

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

Hammer and Drive System Segment Data

Segment -	Weight kips	Stiffness kips/in	COR -	C-Slack in	Damping kips/ft/s
1	0.800	140,084.4	1.000	0.000	
2	0.800	140,084.4	1.000	0.000	
3	0.800	140,084.4	1.000	0.000	
4	0.800	140,084.4	1.000	0.000	
5	0.800	70,754.7	0.900	0.120	
Imp Block	0.753	109,976.0	0.800	0.120	
Helmet	2.700				5.3

Ram Weight: (kips)	4.00	Ram Length: (ft)	10.76
Ram Area: (in²)	124.69		
Maximum (Eq) Stroke: (ft)	10.81	Actual (Eq) Stroke: (ft)	10.81
Efficiency:	0.800	Rated Energy: (kip-ft)	43.24
Maximum Pressure: (psi)	1,600.00	Actual Pressure: (psi)	1,600.00
Combustion Delay: (ms)	2.00	Ignition Duration: (ms)	2.00
Expansion Exponent:	1.25		

Hammer Cushion		Pile Cushion	
Cross Sect. Area: (in²)	415.00	Cross Sect. Area: (in²)	0.00
Elastic Modulus: (ksi)	530.0	Elastic Modulus: (ksi)	0.0
Thickness: (in)	2.00	Thickness: (in)	0.00
Coeff. of Restitution:	0.800	Coeff. of Restitution:	0.500
RoundOut: (in)	0.120	RoundOut: (in)	0.120
Stiffness: (kips/in)	109,976.0	Stiffness: (kips/in)	0.0
Helmet Weight: (kips)	2.700		

PILE INPUT

Uniform Pile		Pile Type:	H Pile
Pile Length: (ft)	85.000	Pile Penetration: (ft)	75.000
Pile Size: (ft)	0.84	Toe Area: (in²)	12.40

Table of Depths Analyzed with Driving System Modifiers

Depth ft	Temp Length ft	Wait Time Hr	Hammer -
5.00	85.0	0.0	DELMAG D 19-42
10.00	85.0	0.0	DELMAG D 19-42
15.00	85.0	0.0	DELMAG D 19-42
20.00	85.0	0.0	DELMAG D 19-42
25.00	85.0	0.0	DELMAG D 19-42
30.00	85.0	0.0	DELMAG D 19-42
35.00	85.0	0.0	DELMAG D 19-42
40.00	85.0	0.0	DELMAG D 19-42
45.00	85.0	0.0	DELMAG D 19-42
50.00	85.0	0.0	DELMAG D 19-42
55.00	85.0	0.0	DELMAG D 19-42
60.00	85.0	0.0	DELMAG D 19-42
65.00	85.0	0.0	DELMAG D 19-42
70.00	85.0	0.0	DELMAG D 19-42
75.00	85.0	0.0	DELMAG D 19-42

Other Information for DELMAG D 19-42

Depth ft	Stroke ft	Diesel Pressure %	Efficiency -	P.C. Stiff. Fact. -	P.C. COR -
5.00	10.8	100.0	0.80	1.0	0.50
10.00	10.8	100.0	0.80	1.0	0.50
15.00	10.8	100.0	0.80	1.0	0.50
20.00	10.8	100.0	0.80	1.0	0.50
25.00	10.8	100.0	0.80	1.0	0.50
30.00	10.8	100.0	0.80	1.0	0.50
35.00	10.8	100.0	0.80	1.0	0.50
40.00	10.8	100.0	0.80	1.0	0.50
45.00	10.8	100.0	0.80	1.0	0.50
50.00	10.8	100.0	0.80	1.0	0.50
55.00	10.8	100.0	0.80	1.0	0.50
60.00	10.8	100.0	0.80	1.0	0.50
65.00	10.8	100.0	0.80	1.0	0.50
70.00	10.8	100.0	0.80	1.0	0.50
75.00	10.8	100.0	0.80	1.0	0.50

PILE, SOIL, ANALYSIS OPTIONS

Analysis type:	Driveability Analysis	Soil Damping Option:	Smith
Max No Analysis Iterations:	0	Time Increment/Critical:	160
Residual Stress Analysis:	0	Analysis Time-Input(ms):	0

Output Level:	Normal	Gravitational Acceleration (ft/s²):	32.169
Hammer Gravity (ft/s²):	32.169	Pile Gravity (ft/s²):	32.169

DRIVEABILITY ANALYSIS

Analysis Depth (ft)	75.00	Standard Soil Setup	
Hammer Name	DELMAG D 19-42	Hammer ID	41
Diesel Pressure: (psi)	230.40	Stroke (ft)	10.81
Efficiency	0.80		
Shaft Gain/Loss Factor	0.571	Toe Gain/Loss Factor	1.000
Shaft Gain/Loss Factor	1.000	Toe Gain/Loss Factor	1.000

SOIL RESISTANCE PARAMETERS

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Setup F. -	Limit D. ft	Setup T. Hours	EB Area in²
0.00	0.9	31.5	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
5.00	0.9	31.5	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
5.00	1.3	27.4	0.10	0.100	0.200	0.1	1.8	6.56	168.0	12.40
12.50	1.3	27.4	0.10	0.100	0.200	0.1	1.8	6.56	168.0	12.40
12.50	1.5	17.1	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
20.00	1.5	17.1	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
20.00	0.3	7.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
21.50	0.3	8.0	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
23.00	0.3	8.5	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
23.00	0.5	39.1	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
25.25	0.6	43.1	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
27.50	0.6	47.0	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
27.50	1.4	16.2	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
36.30	1.4	16.2	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
36.30	1.3	27.0	0.10	0.100	0.150	0.1	1.5	6.56	24.0	12.40
56.30	1.3	27.0	0.10	0.100	0.150	0.1	1.5	6.56	24.0	12.40
56.30	1.3	27.0	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
61.30	1.3	27.0	0.10	0.100	0.150	0.1	1.5	6.56	168.0	12.40
61.30	1.4	50.1	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
62.97	1.4	50.1	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
64.63	1.5	50.1	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
66.30	1.5	50.1	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
66.30	1.6	73.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
67.97	1.7	73.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
71.30	1.7	73.6	0.10	0.100	0.050	0.1	1.0	6.56	1.0	12.40
71.30	2.8	313.7	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
72.65	2.8	313.7	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40
75.00	2.9	313.7	0.10	0.100	0.100	0.1	1.2	6.56	24.0	12.40

PILE PROFILE

Lb Top	X-Area	E-Mod	Spec. Wt	Perim.	C-Index	Wave Sp	Impedance

ft	in²	ksi	lb/ft³	ft	-	ft/s	kips/ft/s
0.00	12.4	30,000	492.00	3.296	0	16,806.4	22.1
85.00	12.4	30,000	492.00	3.296	0	16,806.4	22.1

PILE AND SOIL MODEL Total Capacity Rut (kips): 271.476

Seg.	Weight	Stiffn.	C-Slk	T-Slk	COR	Ru	Js/Jt	Qs/Qt	LbTop	Perim.	X-Area
-	kips	kips/in	in	in	-	kips	s/ft	in	ft	ft	in²
1	0.14	9,482	0.12	0.00	0.85	0.0	0.000	0.10	3.27	3.30	12.4
2	0.14	9,482	0.00	0.00	1.00	0.0	0.000	0.10	6.54	3.30	12.4
3	0.14	9,482	0.00	0.00	1.00	0.0	0.000	0.10	9.81	3.30	12.4
4	0.14	9,482	0.00	0.00	1.00	6.0	0.150	0.10	13.08	3.30	12.4
5	0.14	9,482	0.00	0.00	1.00	7.0	0.175	0.10	16.35	3.30	12.4
6	0.14	9,482	0.00	0.00	1.00	8.0	0.200	0.10	19.62	3.30	12.4
7	0.14	9,482	0.00	0.00	1.00	8.3	0.193	0.10	22.88	3.30	12.4
8	0.14	9,482	0.00	0.00	1.00	10.7	0.150	0.10	26.15	3.30	12.4
9	0.14	9,482	0.00	0.00	1.00	10.7	0.150	0.10	29.42	3.30	12.4
10	0.14	9,482	0.00	0.00	1.00	4.4	0.102	0.10	32.69	3.30	12.4
11	0.14	9,482	0.00	0.00	1.00	5.7	0.050	0.10	35.96	3.30	12.4
12	0.14	9,482	0.00	0.00	1.00	8.6	0.123	0.10	39.23	3.30	12.4
13	0.14	9,482	0.00	0.00	1.00	10.4	0.150	0.10	42.50	3.30	12.4
14	0.14	9,482	0.00	0.00	1.00	10.4	0.150	0.10	45.77	3.30	12.4
15	0.14	9,482	0.00	0.00	1.00	9.6	0.150	0.10	49.04	3.30	12.4
16	0.14	9,482	0.00	0.00	1.00	9.4	0.150	0.10	52.31	3.30	12.4
21	0.14	9,482	0.00	0.00	1.00	9.4	0.150	0.10	68.65	3.30	12.4
22	0.14	9,482	0.00	0.00	1.00	10.5	0.130	0.10	71.92	3.30	12.4
23	0.14	9,482	0.00	0.00	1.00	15.7	0.050	0.10	75.19	3.30	12.4
24	0.14	9,482	0.00	0.00	1.00	17.3	0.050	0.10	78.46	3.30	12.4
25	0.14	9,482	0.00	0.00	1.00	19.3	0.060	0.10	81.73	3.30	12.4
26	0.14	9,482	0.00	0.00	1.00	25.4	0.100	0.10	85.00	3.30	12.4
Toe						27.0	0.149	0.10	85.00		

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

OTHER OPTIONS

Pile Damping (%):	1	Pile Damping Fact. (kips/ft/s):	0.443

EXTREMA TABLE at 75.0 FT; HAMMER: D 19-42

Shaft/Toe Gain/Loss Factor = 0.571/1.000

Rut = 271.5 kips Rtoe = 27.0 kips Time Inc. = 0.076 ms

Hammer DELMAG D 19-42 Efficiency 0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft
3.3	0.0	358.3	0.00	28.90	14.41	0.795	19.14
6.5	5.9	360.7	0.48	29.09	14.22	0.767	18.85
9.8	11.4	367.8	0.92	29.66	13.80	0.742	18.61
13.1	15.9	377.4	1.28	30.43	13.31	0.717	18.02
16.3	12.9	370.7	1.04	29.89	12.76	0.692	16.98
19.6	8.1	359.8	0.65	29.02	12.23	0.666	15.79
22.9	1.6	344.1	0.13	27.75	11.69	0.641	14.56
26.2	0.0	327.9	0.00	26.44	11.23	0.616	13.34
29.4	0.0	308.0	0.00	24.83	10.95	0.592	12.13
32.7	0.0	292.5	0.00	23.58	10.68	0.569	11.30
36.0	0.0	291.1	0.00	23.47	10.29	0.548	10.82
39.2	0.0	289.9	0.00	23.38	9.81	0.529	10.23
42.5	0.0	281.3	0.00	22.69	9.30	0.510	9.45
45.8	0.0	268.3	0.00	21.64	8.84	0.492	8.62
49.0	0.0	255.2	0.00	20.58	8.47	0.475	7.87
52.3	0.0	241.7	0.00	19.49	8.13	0.460	7.21
55.6	0.0	223.5	0.00	18.02	7.81	0.446	6.59
58.8	0.0	208.1	0.00	16.78	7.51	0.432	6.00
62.1	0.0	196.6	0.00	15.85	7.24	0.419	5.45
65.4	0.0	185.4	0.00	14.95	6.97	0.407	4.93
68.7	0.0	176.2	0.00	14.21	6.73	0.397	4.44
71.9	0.0	168.8	0.00	13.61	6.51	0.386	3.96
75.2	0.0	159.7	0.00	12.88	6.56	0.377	3.45
78.5	0.0	144.4	0.00	11.64	7.07	0.368	2.89
81.7	0.0	117.4	0.00	9.47	8.02	0.361	2.30
85.0	0.0	84.8	0.00	6.84	8.30	0.355	1.98

Converged Stroke (ft) 7.68 Fixed Combustion Pressure (psi) 1,600.0

(Eq) Strokes Analyzed and Last Return (ft)

10.81 7.48 7.69 7.68

Shaft/Toe Gain/Loss Factor = 1.000/1.000

Rut = 358.0 kips Rtoe = 27.0 kips Time Inc. = 0.076 ms

Hammer DELMAG D 19-42 Efficiency 0.800

Lb Top ft	Mx.T-For. kips	Mx.C-For kips	Mx.T-Str. ksi	Mx.C-Str. ksi	Mx Vel. ft/s	Mx Dis. in	ENTHRU kip-ft

3.3	0.0	411.8	0.00	33.21	15.30	0.736	19.88
6.5	6.2	421.1	0.50	33.96	15.04	0.701	19.42
9.8	12.1	425.3	0.97	34.30	14.43	0.666	18.94
13.1	17.3	427.2	1.39	34.45	13.67	0.631	17.96
16.3	11.1	412.3	0.89	33.25	12.78	0.596	16.40
19.6	0.8	395.7	0.07	31.91	11.89	0.563	14.63
22.9	0.0	373.1	0.00	30.09	11.06	0.534	12.89
26.2	0.0	350.9	0.00	28.30	10.36	0.506	11.34
29.4	0.0	329.0	0.00	26.53	9.98	0.480	9.91
32.7	0.0	306.1	0.00	24.69	9.61	0.453	8.93
36.0	0.0	305.0	0.00	24.60	9.17	0.425	8.37
39.2	0.0	305.9	0.00	24.67	8.72	0.396	7.66
42.5	0.0	295.7	0.00	23.85	8.22	0.368	6.74
45.8	0.0	276.7	0.00	22.32	7.76	0.342	5.79
49.0	0.0	257.5	0.00	20.77	7.36	0.318	4.98
52.3	0.0	237.9	0.00	19.18	7.02	0.296	4.30
55.6	0.0	224.0	0.00	18.06	6.68	0.278	3.72
58.8	0.0	208.3	0.00	16.80	6.37	0.260	3.20
62.1	0.0	194.4	0.00	15.67	6.09	0.244	2.74
65.4	0.0	178.5	0.00	14.40	5.81	0.229	2.33
68.7	0.0	170.4	0.00	13.74	5.58	0.216	1.98
71.9	0.0	154.9	0.00	12.49	5.37	0.204	1.66
75.2	0.0	141.4	0.00	11.40	5.31	0.193	1.39
78.5	0.0	125.3	0.00	10.10	5.69	0.184	1.14
81.7	0.0	104.2	0.00	8.40	6.64	0.176	0.89
85.0	0.0	77.7	0.00	6.26	7.12	0.170	0.76

Converged Stroke (ft) 8.31 Fixed Combustion Pressure (psi) 1,600.0
(Eq) Strokes Analyzed and Last Return (ft)
10.81 8.16 8.32 8.31

SUMMARY TABLE at 75.0 FT; HAMMER: D 19-42

Rut kips	Bl Ct b/ft	Stk Dn ft	Stk Up ft	Mx T-Str ksi	LTop ft	Mx C-Str ksi	LTop ft	ENTHRU kip-ft	Bl Rt b/min	ActRes kips
271.5	47.0	7.68	0.00	1.28	13.1	30.43	13.1	19.1	42.6	271.5
358.0	170.9	8.31	0.00	1.39	13.1	34.45	13.1	19.9	41.0	358.0

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 0.571/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	12.5	9.8	2.7	1.1	11.30	1.16	4.00	26.7	D 19-42
10.0	23.3	21.0	2.4	2.0	15.96	0.73	4.51	24.7	D 19-42
15.0	37.6	36.1	1.5	3.6	18.91	0.80	4.93	22.5	D 19-42
20.0	53.9	52.4	1.5	5.5	21.94	0.88	5.31	21.1	D 19-42
25.0	62.6	58.9	3.7	6.3	22.68	1.67	5.47	20.8	D 19-42
30.0	73.1	71.7	1.4	7.4	23.38	1.91	5.65	20.2	D 19-42
35.0	88.9	87.5	1.4	9.3	24.45	2.17	5.93	19.7	D 19-42
40.0	104.7	102.3	2.3	11.7	25.46	1.69	6.19	19.3	D 19-42
45.0	119.1	116.8	2.3	14.0	26.21	1.57	6.41	19.1	D 19-42
50.0	133.5	131.2	2.3	15.5	26.80	1.47	6.58	18.8	D 19-42
55.0	148.0	145.7	2.3	17.2	27.41	1.07	6.76	18.6	D 19-42
60.0	162.4	160.1	2.3	19.1	27.83	0.43	6.92	18.6	D 19-42
65.0	185.8	181.5	4.3	22.0	28.79	0.90	7.09	18.8	D 19-42
70.0	214.7	208.4	6.3	26.7	29.22	1.18	7.30	18.8	D 19-42
75.0	271.5	244.5	27.0	47.0	30.43	1.28	7.68	19.1	D 19-42

G/L at Shaft and Toe: 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Bl Ct b/ft	Mx C-Str ksi	Mx T-Str ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	17.3	14.6	2.7	1.4	13.74	0.76	4.23	26.3	D 19-42
10.0	36.6	34.3	2.4	3.5	18.72	0.72	4.91	22.5	D 19-42
15.0	60.3	58.8	1.5	6.5	22.76	1.49	5.50	20.6	D 19-42
20.0	84.7	83.2	1.5	9.4	25.17	2.36	5.90	19.8	D 19-42
25.0	93.3	89.7	3.7	10.3	25.43	2.98	6.02	19.6	D 19-42
30.0	107.8	106.5	1.4	12.3	26.47	2.57	6.23	19.3	D 19-42
35.0	131.6	130.2	1.4	15.3	27.34	2.59	6.53	19.1	D 19-42
40.0	154.7	152.4	2.3	17.9	28.34	1.54	6.79	19.2	D 19-42
45.0	176.4	174.1	2.3	20.8	29.06	2.43	7.03	19.5	D 19-42
50.0	198.0	195.7	2.3	24.5	29.79	2.57	7.23	19.5	D 19-42
55.0	219.7	217.4	2.3	29.0	30.62	2.44	7.45	19.6	D 19-42
60.0	241.4	239.0	2.3	34.6	31.90	3.02	7.65	19.7	D 19-42
65.0	266.6	262.3	4.3	43.0	32.61	2.89	7.85	19.6	D 19-42
70.0	295.5	289.2	6.3	56.5	32.38	1.59	8.00	19.6	D 19-42
75.0	358.0	331.0	27.0	170.9	34.45	1.39	8.31	19.9	D 19-42

APPENDIX D

EXTERNAL STABILTY ANALYSIS

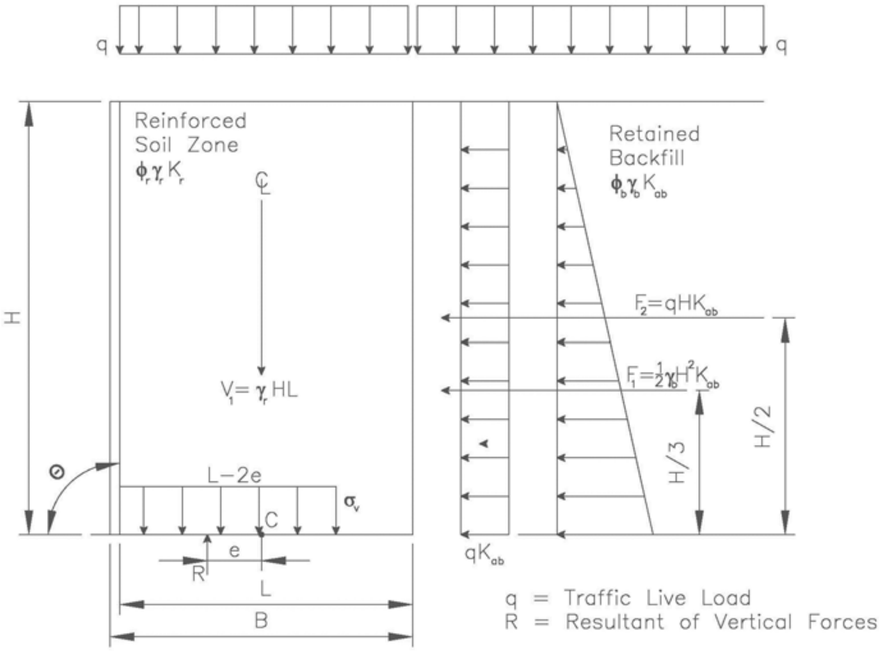
RETAINING WALL 1

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.

Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- **Horizontal backfill behind MSE wall on granular (drained) soils.**
- **For battered or vertical walls with a back face of wall angle of θ to horizontal.**
- **Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ.**
- **MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall**
- **Load combinations and wall configuration are as shown below:**



Givens:

Wall Geometry:

$H_e := 6.3 \cdot ft$	Exposed wall height
$\theta := 90 \cdot deg$	Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Reinforced Backfill Soil Design Parameters:

$\phi'_r := 34 \cdot deg$	Effective angle of internal friction (Per BDM [Table 307-1])
$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$	Unit weight (Per BDM [Table 307-1])
$c'_r := 0 \cdot \frac{lbf}{ft^2}$	Effective Cohesion

Retained Backfill Soil Design Parameters:

$\phi'_b := 30 \cdot deg$	Effective angle of internal friction (Per BDM [Table 307-1])
$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$	Unit weight (Per BDM [Table 307-1])
$c'_b := 0 \cdot \frac{lbf}{ft^2}$	Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$\phi'_f := 24 \cdot deg$ Effective angle of internal friction

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

$c'_f := 140 \cdot \frac{lbf}{ft^2}$ Cohesion

Undrained Conditions (Total Stress):

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

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

$c_f := 1400 \cdot \frac{lbf}{ft^2}$ Cohesion (Use Su if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

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

Depth of Embedment Check:

$d_{frost} := 3\ ft$ $d_{user} := 0\ ft$ Local Frost Depth

$Slope_{fw} := 0\ deg$

Inclination of ground slope in front of wall :

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

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

$H_{est} := d_{est} + \left(4\ ft \cdot \tan\left(Slope_{fw}\right)\right) + H_e$ $H_{est} = 9.3\ ft$

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

$d_{eSlope} = 0.5\ ft$ Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$d_e := \max\left(d_{est}, d_{eSlope}\right)$ $d_e = 3\ ft$ Minimum Required Embedment Depth used in analysis.

$H := d_e + \left(4\ ft \cdot \tan\left(Slope_{fw}\right)\right) + H_e$ $H = 9.3\ ft$ Design Wall Height

Estimate Length of Reinforcement:

$L_{user} := 0 \cdot ft$ User inputted value (if changes need to be made to satisfy other requirements)

$L := \max\left(8 \cdot ft, 0.7\ H, L_{user}\right)$ $L = 8\ ft$ Length of Reinforcement

Live Load Surcharge Parameters:

$SUR := 250 \cdot \frac{lbf}{ft^2}$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)
Note: If vehicular loading is within 1 ft of the backface of the wall and with a design height, H, less than 20 ft, see **LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2** for adjusted surcharge load calculation.
Note: When traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$\beta := 0\ deg$ $\quad \delta := 0.67 \cdot \phi'_b$

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

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

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

Active Earth Pressure Coefficient

$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$ $\quad F_T = 1542.6\ \frac{lbf}{ft}$ Active Earth Force Resultant (EH)

$F_{SUR} := SUR \cdot H \cdot k_{af}$ $\quad F_{SUR} = 691.1\ \frac{lbf}{ft}$ Live Load Surcharge (LS)

Vertical Loads:

$V_1 := \gamma_r \cdot H \cdot L$ $\quad V_1 = 8928\ \frac{lbf}{ft}$ Soil backfill - reinforced soil (EV)

$V_2 := SUR \cdot L$ $\quad V_2 = 2000\ \frac{lbf}{ft}$ Live Load Surcharge - (LS)

Moment Arm: Moment:

$d_{v1} := 0 \cdot ft$ $\quad d_{v1} = 0\ ft$ $\quad MV_1 := V_1 \cdot d_{v1}$ $\quad MV_1 = 0\ \frac{lbf \cdot ft}{ft}$

$d_{v2} := 0\ ft$ $\quad d_{v2} = 0\ ft$ $\quad MV_2 := V_2 \cdot d_{v2}$ $\quad MV_2 = 0\ \frac{lbf \cdot ft}{ft}$

Horizontal Loads:

$H_1 := F_T = 1542.6\ \frac{lbf}{ft}$ Active Earth Force Resultant (horizontal comp. - EH)

$H_2 := F_{SUR} = 691.1\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm: Moment:

$d_{h1} := \frac{H}{3}$ $\quad d_{h1} = 3.1\ ft$ $\quad MH_1 := H_1 \cdot d_{h1}$ $\quad MH_1 = 4782.1\ \frac{lbf \cdot ft}{ft}$

$d_{h2} := \frac{H}{2}$ $\quad d_{h2} = 4.7\ ft$ $\quad MH_2 := H_2 \cdot d_{h2}$ $\quad MH_2 = 3213.8\ \frac{lbf \cdot ft}{ft}$

Unfactored Loads by Load Type

$V_{EV} := V_1$ $\qquad V_{EV} = 8928\ \dfrac{lbf}{ft}$

$V_{LS} := V_2$ $\qquad V_{LS} = 2000\ \dfrac{lbf}{ft}$

$H_{EH} := H_1$ $\qquad H_{EH} = 1542.6\ \dfrac{lbf}{ft}$

$H_{LS} := H_2$ $\qquad H_{LS} = 691.1\ \dfrac{lbf}{ft}$

Unfactored Moments by Load Type

$M_{EV} := MV_1$ $\qquad M_{EV} = 0\ \dfrac{lbf \cdot ft}{ft}$

$M_{LS} := MV_2$ $\qquad M_{LS} = 0\ \dfrac{lbf \cdot ft}{ft}$

$M_{EH2} := MH_1$ $\qquad M_{EH2} = 4782.1\ \dfrac{lbf \cdot ft}{ft}$

$M_{LS2} := MH_2$ $\qquad M_{LS2} = 3213.8\ \dfrac{lbf \cdot ft}{ft}$

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) $\qquad Ia_{EV} := 1 \qquad Ia_{EH} := 1.5 \qquad Ia_{LS} := 1.75$

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

Factored Vertical Loads by Limit State:

$V_{Ia} := \eta \cdot \left(Ia_{EV} \cdot V_{EV}\right)$ $\qquad V_{Ia} = 8928\ \dfrac{lbf}{ft}$

$V_{Ib} := \eta \cdot \left(\left(Ib_{EV} \cdot V_{EV}\right) + \left(Ib_{LS} \cdot V_{LS}\right)\right)$ $\qquad V_{Ib} = 15552.8\ \dfrac{lbf}{ft}$

Factored Horizontal Loads by Limit State:

$H_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot H_{LS}\right) + \left(Ia_{EH} \cdot H_{EH}\right)\right)$ $\qquad H_{Ia} = 3523.4\ \dfrac{lbf}{ft}$

$H_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot H_{LS}\right) + \left(Ib_{EH} \cdot H_{EH}\right)\right)$ $\qquad H_{Ib} = 3523.4\ \dfrac{lbf}{ft}$

Factored Moments Produced by Vertical Loads by Limit State:

$MV_{Ia} := \eta \cdot \left(Ia_{EV} \cdot M_{EV}\right)$ $\qquad MV_{Ia} = 0\ \dfrac{lbf \cdot ft}{ft}$

$MV_{Ib} := \eta \cdot \left(\left(Ib_{EV} \cdot M_{EV}\right) + \left(Ib_{LS} \cdot M_{LS}\right)\right)$ $\qquad MV_{Ib} = 0\ \dfrac{lbf \cdot ft}{ft}$

Factored Moments Produced by Horizontal Loads by Limit State:

$MH_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot M_{LS2}\right) + \left(Ia_{EH} \cdot M_{EH2}\right)\right)$ $\qquad MH_{Ia} = 12797.2\ \dfrac{lbf \cdot ft}{ft}$

$MH_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot M_{LS2}\right) + \left(Ib_{EH} \cdot M_{EH2}\right)\right)$ $\qquad MH_{Ib} = 12797.2\ \dfrac{lbf \cdot ft}{ft}$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength Ib):

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

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 12797.2 \ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 15552.8 \ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$ $e_{wall} = 0.8 \ ft$ Wall Eccentricity

$B' := \text{if}\left(e_{wall} > 0, L - 2 \cdot e_{wall}, L\right)$ $B' = 6.4 \ ft$ Effective Bearing Width

Foundation Layout:

$L_{Wall} := 68.3 \cdot ft$ Assumed Footing Length (Wall Section Length)

$H' := H_{Ib}$ $H' = 3523.4 \ \frac{lbf}{ft}$ Summation of Horizontal Loads (Strength Ib)

$V' := V_{Ib}$ $V' = 15552.8 \ \frac{lbf}{ft}$ Summation of Vertical Loads (Strength Ib)

$D_f := d_e$ $D_f = 3 \ ft$ Footing embedment

$d_w := -0.5 \cdot ft$ Depth of Groundwater below Bearing Grade

$\theta' := 90 \cdot deg$ Direction of H' and V' resultant measured from wall back face **LRFD [Figure C10.6.3.1.2a-1]**

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \ deg + \frac{\phi'_f}{2}\right)^2, 1.0\right)$ $N_q = 9.6$

$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$ $N_c = 19.32$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$ $N_\gamma = 9.4$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right)$ $s_c = 1.046$

$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \cdot \tan(\phi'_f)\right), 1\right)$ $s_q = 1.041$

$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{Wall}}\right), 1\right)$ $s_\gamma = 0.963$

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

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

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

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

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

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

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

Depth Correction Factor per **Hanson (1970):**

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

$d_q = 1.1$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\qquad N_{cm} = 20.217$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\qquad N_{qm} = 10.001$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\qquad N_{\gamma m} = 9.09$

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

$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\qquad q_{nd} = 6.6\ \mathbf{ksf}$

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

$\phi_b := 0.65$ $\qquad$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Rd} := \phi_b \cdot q_{nd}$ $\qquad q_{Rd} = 4.3\ \mathbf{ksf}$ $\qquad$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

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

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

$$s_c := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right) \qquad s_c = 1.019$$

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

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

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

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

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

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

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\qquad N_{cm} = 5.236$

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

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

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

$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\qquad q_{nu} = 7.5$ ***ksf***

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

$\phi_b := 0.65$ $\qquad$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Ru} := \phi_b \cdot q_{nu}$ $\qquad q_{Ru} = 4.9$ ***ksf*** $\qquad$ Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 4.3$ ***ksf***

Undrained Conditions: $q_{Ru} = 4.9$ ***ksf***

Factored Bearing Resistance to be used in CDR Calculations:

$q_R := q_{Rd}$

$q_R = 4.3$ ***ksf***

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

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

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

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 12797.2\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 15552.8\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$e_{wall} := \frac{\left(\Sigma M_O - \Sigma M_R\right)}{\Sigma V}$ $e_{wall} = 0.8\ ft$ Wall Eccentricity

$B' := \text{if}\left(e_{wall} > 0, L - 2 \cdot e_{wall}, L\right)$ $B' = 6.4\ ft$ Effective Bearing Width

Compute the bearing stress:

$\sigma := \frac{V_1 + V_2}{B'}$ $\sigma = 1.7\ ksf$ Nominal Bearing Stress

$\sigma_v := \frac{\Sigma V}{B'}$ $\sigma_v = 2.4\ ksf$ Factored Bearing Stress

Bearing Capacity:Demand Ratio (CDR)

$CDR_{Bearing} := \frac{q_R}{\sigma_v}$ Is the CDR > or = to 1.0? $CDR_{Bearing} = 1.76$

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

$e_{max} := \frac{L}{3}$ $e_{max} = 2.7\ ft$ Maximum Eccentricity **LRFD [C11.6.3.3.]**

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

$\Sigma M_O := MH_{Ia}$ $\Sigma M_O = 12797.2\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$ $\Sigma V = 8928\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$e_{wall} := \frac{\left(\Sigma M_O - \Sigma M_R\right)}{\Sigma V}$ $e_{wall} = 1.4\ ft$

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}}$ Is the CDR > or = to 1.0? $CDR_{Eccentricity} = 1.86$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$F_{\tau} := H_{Ia}$ $F_{\tau} = 3523.4\ \frac{lbf}{ft}$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$\Sigma V := V_{Ia}$ $\Sigma V = 8928\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$R_{\tau d} := \Sigma V \cdot \tan\left(\phi'_{f}\right)$ $R_{\tau d} = 3975\ \frac{lbf}{ft}$ Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

Drained Conditions: $R_{\tau d} = 3.975\ \frac{kip}{ft}$

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

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

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

$\phi R_{n} := \phi_{\tau} \cdot R_{\tau}$ $R_{R} := \phi R_{n}$

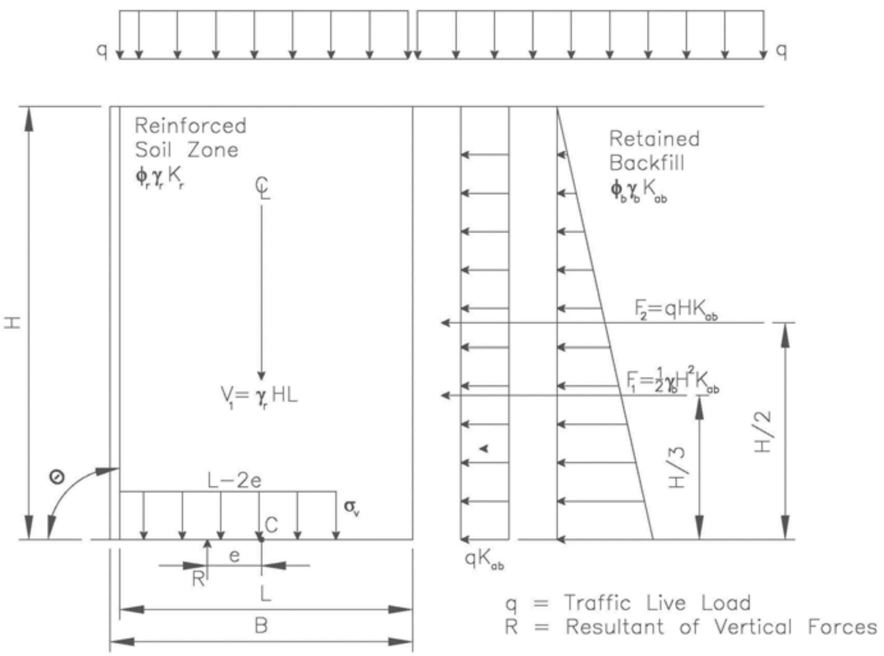
$R_{R} = 4\ \frac{kip}{ft}$

Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_{R}}{F_{\tau}}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 1.13$

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

- **Horizontal backfill behind MSE wall on granular (drained) soils.**
- **For battered or vertical walls with a back face of wall angle of θ to horizontal.**
- **Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_{μ}.**
- **MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall**
- **Load combinations and wall configuration are as shown below:**



Givens:

Wall Geometry:

$H_e := 9.2 \cdot ft$	Exposed wall height
$\theta := 90 \cdot deg$	Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Reinforced Backfill Soil Design Parameters:

$\phi'_r := 34 \cdot deg$	Effective angle of internal friction (Per BDM [Table 307-1])
$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$	Unit weight (Per BDM [Table 307-1])
$c'_r := 0 \cdot \frac{lbf}{ft^2}$	Effective Cohesion

Retained Backfill Soil Design Parameters:

$\phi'_b := 30 \cdot deg$	Effective angle of internal friction (Per BDM [Table 307-1])
$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$	Unit weight (Per BDM [Table 307-1])
$c'_b := 0 \cdot \frac{lbf}{ft^2}$	Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$\phi'_f := 24 \cdot deg$ Effective angle of internal friction

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

$c'_f := 140 \cdot \frac{lbf}{ft^2}$ Cohesion

Undrained Conditions (Total Stress):

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

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

$c_f := 1400 \cdot \frac{lbf}{ft^2}$ Cohesion (Use Su if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

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

Depth of Embedment Check:

$d_{frost} := 3 \ ft$ $d_{user} := 0 \ ft$ Local Frost Depth

$Slope_{fw} := 0 \ deg$

Inclination of ground slope in front of wall :

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

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

$H_{est} := d_{est} + \left(4 \ ft \cdot \tan\left(Slope_{fw}\right)\right) + H_e$ $H_{est} = 12.2 \ ft$

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

$d_{eSlope} = 0.6 \ ft$ Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$d_e := \max\left(d_{est}, d_{eSlope}\right)$ $d_e = 3 \ ft$ Minimum Required Embedment Depth used in analysis.

$H := d_e + \left(4 \ ft \cdot \tan\left(Slope_{fw}\right)\right) + H_e$ $H = 12.2 \ ft$ Design Wall Height

Estimate Length of Reinforcement:

$L_{user} := 0 \cdot ft$ User inputted value (if changes need to be made to satisfy other requirements)

$L := \max\left(8 \cdot ft, 0.7 \ H, L_{user}\right)$ $L = 8.5 \ ft$ Length of Reinforcement

Live Load Surcharge Parameters:

$SUR := 250 \cdot \frac{lbf}{ft^2}$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)
Note: If vehicular loading is within 1 ft of the backface of the wall and with a design height, H, less than 20 ft, see **LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2** for adjusted surcharge load calculation.
Note: When traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$\beta := 0\ deg$ $\qquad \delta := 0.67 \cdot \phi'_b$

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

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

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

Active Earth Pressure Coefficient

$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$ $\qquad F_T = 2654.7\ \frac{lbf}{ft}$ — Active Earth Force Resultant (EH)

$F_{SUR} := SUR \cdot H \cdot k_{af}$ $\qquad F_{SUR} = 906.6\ \frac{lbf}{ft}$ — Live Load Surcharge (LS)

Vertical Loads:

$V_1 := \gamma_r \cdot H \cdot L$ $\qquad V_1 = 12502.6\ \frac{lbf}{ft}$ — Soil backfill - reinforced soil (EV)

$V_2 := SUR \cdot L$ $\qquad V_2 = 2135\ \frac{lbf}{ft}$ — Live Load Surcharge - (LS)

Moment Arm: / Moment:

$d_{v1} := 0 \cdot ft$ $\qquad d_{v1} = 0\ ft$ $\qquad MV_1 := V_1 \cdot d_{v1}$ $\qquad MV_1 = 0\ \frac{lbf \cdot ft}{ft}$

$d_{v2} := 0\ ft$ $\qquad d_{v2} = 0\ ft$ $\qquad MV_2 := V_2 \cdot d_{v2}$ $\qquad MV_2 = 0\ \frac{lbf \cdot ft}{ft}$

Horizontal Loads:

$H_1 := F_T = 2654.7\ \frac{lbf}{ft}$ — Active Earth Force Resultant (horizontal comp. - EH)

$H_2 := F_{SUR} = 906.6\ \frac{lbf}{ft}$ — Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm: / Moment:

$d_{h1} := \frac{H}{3}$ $\qquad d_{h1} = 4.1\ ft$ $\qquad MH_1 := H_1 \cdot d_{h1}$ $\qquad MH_1 = 10795.6\ \frac{lbf \cdot ft}{ft}$

$d_{h2} := \frac{H}{2}$ $\qquad d_{h2} = 6.1\ ft$ $\qquad MH_2 := H_2 \cdot d_{h2}$ $\qquad MH_2 = 5530.5\ \frac{lbf \cdot ft}{ft}$

Unfactored Loads by Load Type

$V_{EV} := V_1$ $\quad V_{EV} = 12502.6\ \dfrac{lbf}{ft}$

$V_{LS} := V_2$ $\quad V_{LS} = 2135\ \dfrac{lbf}{ft}$

$H_{EH} := H_1$ $\quad H_{EH} = 2654.7\ \dfrac{lbf}{ft}$

$H_{LS} := H_2$ $\quad H_{LS} = 906.6\ \dfrac{lbf}{ft}$

Unfactored Moments by Load Type

$M_{EV} := MV_1$ $\quad M_{EV} = 0\ \dfrac{lbf \cdot ft}{ft}$

$M_{LS} := MV_2$ $\quad M_{LS} = 0\ \dfrac{lbf \cdot ft}{ft}$

$M_{EH2} := MH_1$ $\quad M_{EH2} = 10795.6\ \dfrac{lbf \cdot ft}{ft}$

$M_{LS2} := MH_2$ $\quad M_{LS2} = 5530.5\ \dfrac{lbf \cdot ft}{ft}$

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) $\quad Ia_{EV} := 1 \quad Ia_{EH} := 1.5 \quad Ia_{LS} := 1.75$

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

Factored Vertical Loads by Limit State:

$V_{Ia} := \eta \cdot \left(Ia_{EV} \cdot V_{EV}\right)$ $\quad V_{Ia} = 12502.6\ \dfrac{lbf}{ft}$

$V_{Ib} := \eta \cdot \left(\left(Ib_{EV} \cdot V_{EV}\right) + \left(Ib_{LS} \cdot V_{LS}\right)\right)$ $\quad V_{Ib} = 20614.7\ \dfrac{lbf}{ft}$

Factored Horizontal Loads by Limit State:

$H_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot H_{LS}\right) + \left(Ia_{EH} \cdot H_{EH}\right)\right)$ $\quad H_{Ia} = 5568.6\ \dfrac{lbf}{ft}$

$H_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot H_{LS}\right) + \left(Ib_{EH} \cdot H_{EH}\right)\right)$ $\quad H_{Ib} = 5568.6\ \dfrac{lbf}{ft}$

Factored Moments Produced by Vertical Loads by Limit State:

$MV_{Ia} := \eta \cdot \left(Ia_{EV} \cdot M_{EV}\right)$ $\quad MV_{Ia} = 0\ \dfrac{lbf \cdot ft}{ft}$

$MV_{Ib} := \eta \cdot \left(\left(Ib_{EV} \cdot M_{EV}\right) + \left(Ib_{LS} \cdot M_{LS}\right)\right)$ $\quad MV_{Ib} = 0\ \dfrac{lbf \cdot ft}{ft}$

Factored Moments Produced by Horizontal Loads by Limit State:

$MH_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot M_{LS2}\right) + \left(Ia_{EH} \cdot M_{EH2}\right)\right)$ $\quad MH_{Ia} = 25871.8\ \dfrac{lbf \cdot ft}{ft}$

$MH_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot M_{LS2}\right) + \left(Ib_{EH} \cdot M_{EH2}\right)\right)$ $\quad MH_{Ib} = 25871.8\ \dfrac{lbf \cdot ft}{ft}$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength Ib):

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

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 25871.8 \ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 20614.7 \ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$e_{wall} := \frac{\left(\Sigma M_O - \Sigma M_R\right)}{\Sigma V}$ $e_{wall} = 1.3 \ ft$ Wall Eccentricity

$B' := \text{if}\left(e_{wall} > 0, L - 2 \cdot e_{wall}, L\right)$ $B' = 6 \ ft$ Effective Bearing Width

Foundation Layout:

$L_{Wall} := 9 \cdot ft$ Assumed Footing Length (Wall Section Length)

$H' := H_{Ib}$ $H' = 5568.6 \ \frac{lbf}{ft}$ Summation of Horizontal Loads (Strength Ib)

$V' := V_{Ib}$ $V' = 20614.7 \ \frac{lbf}{ft}$ Summation of Vertical Loads (Strength Ib)

$D_f := d_e$ $D_f = 3 \ ft$ Footing embedment

$d_w := -0.5 \cdot ft$ Depth of Groundwater below Bearing Grade

$\theta' := 90 \cdot deg$ Direction of H' and V' resultant measured from wall back face **LRFD [Figure C10.6.3.1.2a-1]**

Drained Conditions (Effective Stress):

$$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan\left(\phi'_f\right)} \cdot \tan\left(45 \ deg + \frac{\phi'_f}{2}\right)^2, 1.0\right) \qquad N_q = 9.6$$

$$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan\left(\phi'_f\right)}, 5.14\right) \qquad N_c = 19.32$$

$$N_\gamma := 2 \cdot \left(N_q + 1\right) \cdot \tan\left(\phi'_f\right) \qquad N_\gamma = 9.4$$

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

$$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right) \qquad s_c = 1.333$$

$$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \cdot \tan\left(\phi'_f\right)\right), 1\right) \qquad s_q = 1.298$$

$$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{Wall}}\right), 1\right) \qquad s_\gamma = 0.732$$

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

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

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

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

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

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

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

Depth Correction Factor per **Hanson (1970):**

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

$d_q = 1.2$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\qquad$ $N_{cm} = 25.758$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\qquad$ $N_{qm} = 12.468$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\qquad$ $N_{\gamma m} = 6.911$

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

$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\qquad$ $q_{nd} = 7.5\ \textbf{ksf}$

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

$\phi_b := 0.65$ $\qquad$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Rd} := \phi_b \cdot q_{nd}$ $\qquad$ $q_{Rd} = 4.8\ \textbf{ksf}$ $\qquad$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

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

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

$$s_c := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right)$$ $s_c = 1.134$

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

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

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

$i_q := 1$ $i_q = 1$

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

$i_c := 1$ $i_c = 1$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $N_{cm} = 5.829$

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

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

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

$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $q_{nu} = 8.4$ ***ksf***

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

$\phi_b := 0.65$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Ru} := \phi_b \cdot q_{nu}$ $q_{Ru} = 5.4$ ***ksf*** Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 4.8$ ***ksf***

Undrained Conditions: $q_{Ru} = 5.4$ ***ksf***

Factored Bearing Resistance to be used in CDR Calculations:

$q_R := q_{Rd}$

$q_R = 4.8$ ***ksf***

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

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

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

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 25871.8 \ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 20614.7 \ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

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

$B' := \text{if}(e_{wall} > 0, L - 2 \cdot e_{wall}, L)$ $B' = 6 \ ft$ Effective Bearing Width

Compute the bearing stress:

$\sigma := \frac{V_1 + V_2}{B'}$ $\sigma = 2.4 \ ksf$ Nominal Bearing Stress

$\sigma_v := \frac{\Sigma V}{B'}$ $\sigma_v = 3.4 \ ksf$ Factored Bearing Stress

Bearing Capacity:Demand Ratio (CDR)

$CDR_{Bearing} := \frac{q_R}{\sigma_v}$ Is the CDR > or = to 1.0? $CDR_{Bearing} = 1.42$

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

$e_{max} := \frac{L}{3}$ $e_{max} = 2.8 \ ft$ Maximum Eccentricity **LRFD [C11.6.3.3.]**

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

$\Sigma M_O := MH_{Ia}$ $\Sigma M_O = 25871.8 \ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$ $\Sigma V = 12502.6 \ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$ $e_{wall} = 2.1 \ ft$

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}}$ Is the CDR > or = to 1.0? $CDR_{Eccentricity} = 1.38$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$F_\tau := H_{Ia}$ $\quad F_\tau = 5568.6\ \frac{lbf}{ft}$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$\Sigma V := V_{Ia}$ $\quad \Sigma V = 12502.6\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$R_{\tau d} := \Sigma V \cdot \tan\left(\phi'_f\right)$ $\quad R_{\tau d} = 5566.5\ \frac{lbf}{ft}$ Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

Drained Conditions: $R_{\tau d} = 5.566\ \frac{kip}{ft}$

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

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

$\phi_\tau := 1.0$

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

$\phi R_n := \phi_\tau \cdot R_\tau$ $\quad R_R := \phi R_n$

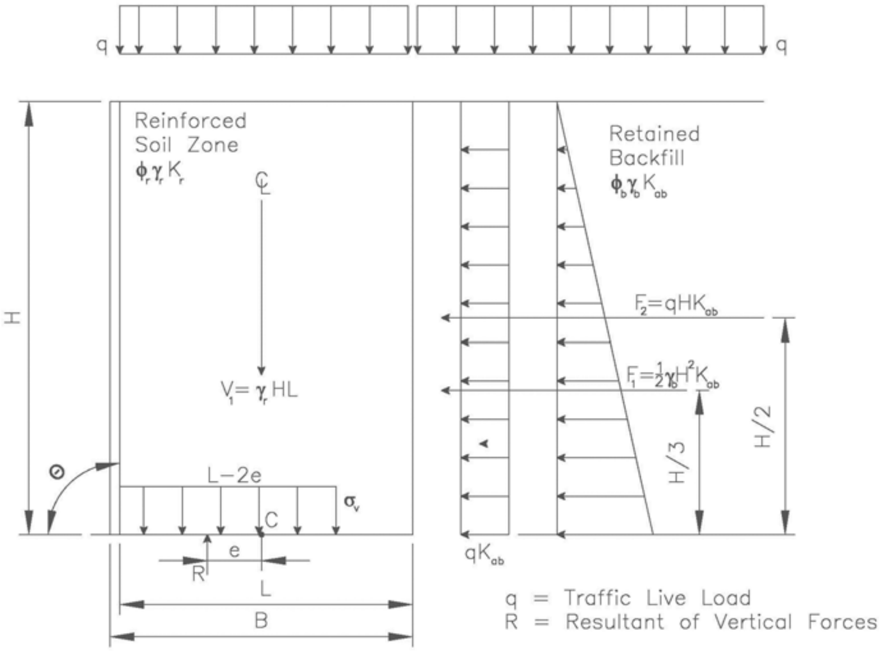
$R_R = 5.6\ \frac{kip}{ft}$

Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{F_\tau}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 1.00$

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

- **Horizontal backfill behind MSE wall on granular (drained) soils.**
- **For battered or vertical walls with a back face of wall angle of θ to horizontal.**
- **Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ.**
- **MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall**
- **Load combinations and wall configuration are as shown below:**



Givens:

Wall Geometry:

$H_e := 12.1 \cdot ft$ Exposed wall height

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

Reinforced Backfill Soil Design Parameters:

$\phi'_r := 34 \cdot deg$ Effective angle of internal friction (Per BDM [Table 307-1])

$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$ Unit weight (Per BDM [Table 307-1])

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

Retained Backfill Soil Design Parameters:

$\phi'_b := 30 \cdot deg$ Effective angle of internal friction (Per BDM [Table 307-1])

$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$ Unit weight (Per BDM [Table 307-1])

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

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$\phi'_f := 24 \cdot deg$ Effective angle of internal friction

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

$c'_f := 140 \cdot \dfrac{lbf}{ft^2}$ Cohesion

Undrained Conditions (Total Stress):

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

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

$c_f := 1400 \cdot \dfrac{lbf}{ft^2}$ Cohesion (Use Su if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

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

Depth of Embedment Check:

$d_{frost} := 3\ ft$ $\quad d_{user} := 0\ ft$ Local Frost Depth

$Slope_{fw} := 0\ deg$

Inclination of ground slope in front of wall :

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

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

$H_{est} := d_{est} + \left(4\ ft \cdot \tan\left(Slope_{fw}\right)\right) + H_e$ $\quad H_{est} = 15.1\ ft$

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

$d_{eSlope} = 0.8\ ft$ Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$d_e := \max\left(d_{est}, d_{eSlope}\right)$ $\quad d_e = 3\ ft$ Minimum Required Embedment Depth used in analysis.

$H := d_e + \left(4\ ft \cdot \tan\left(Slope_{fw}\right)\right) + H_e$ $\quad H = 15.1\ ft$ Design Wall Height

Estimate Length of Reinforcement:

$L_{user} := 0 \cdot ft$ User inputted value (if changes need to be made to satisfy other requirements)

$L := \max\left(8 \cdot ft, 0.7\ H, L_{user}\right)$ $\quad L = 10.6\ ft$ Length of Reinforcement

Live Load Surcharge Parameters:

$SUR := 250 \cdot \frac{lbf}{ft^2}$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)
Note: If vehicular loading is within 1 ft of the backface of the wall and with a design height, H, less than 20 ft, see **LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2** for adjusted surcharge load calculation.
Note: When traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$\beta := 0\ deg \qquad \delta := 0.67 \cdot \phi'_b$

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

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

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

Active Earth Pressure Coefficient

$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af} \qquad F_T = 4066.7\ \frac{lbf}{ft}$ Active Earth Force Resultant (EH)

$F_{SUR} := SUR \cdot H \cdot k_{af} \qquad F_{SUR} = 1122.2\ \frac{lbf}{ft}$ Live Load Surcharge (LS)

Vertical Loads:

$V_1 := \gamma_r \cdot H \cdot L \qquad V_1 = 19152.8\ \frac{lbf}{ft}$ Soil backfill - reinforced soil (EV)

$V_2 := SUR \cdot L \qquad V_2 = 2642.5\ \frac{lbf}{ft}$ Live Load Surcharge - (LS)

Moment Arm: Moment:

$d_{v1} := 0 \cdot ft \qquad d_{v1} = 0\ ft \qquad MV_1 := V_1 \cdot d_{v1} \qquad MV_1 = 0\ \frac{lbf \cdot ft}{ft}$

$d_{v2} := 0\ ft \qquad d_{v2} = 0\ ft \qquad MV_2 := V_2 \cdot d_{v2} \qquad MV_2 = 0\ \frac{lbf \cdot ft}{ft}$

Horizontal Loads:

$H_1 := F_T = 4066.7\ \frac{lbf}{ft}$ Active Earth Force Resultant (horizontal comp. - EH)

$H_2 := F_{SUR} = 1122.2\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm: Moment:

$d_{h1} := \frac{H}{3} \qquad d_{h1} = 5\ ft \qquad MH_1 := H_1 \cdot d_{h1} \qquad MH_1 = 20469\ \frac{lbf \cdot ft}{ft}$

$d_{h2} := \frac{H}{2} \qquad d_{h2} = 7.6\ ft \qquad MH_2 := H_2 \cdot d_{h2} \qquad MH_2 = 8472.3\ \frac{lbf \cdot ft}{ft}$

Unfactored Loads by Load Type

$V_{EV} := V_1$ $\qquad V_{EV} = 19152.8\ \frac{lbf}{ft}$

$V_{LS} := V_2$ $\qquad V_{LS} = 2642.5\ \frac{lbf}{ft}$

$H_{EH} := H_1$ $\qquad H_{EH} = 4066.7\ \frac{lbf}{ft}$

$H_{LS} := H_2$ $\qquad H_{LS} = 1122.2\ \frac{lbf}{ft}$

Unfactored Moments by Load Type

$M_{EV} := MV_1$ $\qquad M_{EV} = 0\ \frac{lbf \cdot ft}{ft}$

$M_{LS} := MV_2$ $\qquad M_{LS} = 0\ \frac{lbf \cdot ft}{ft}$

$M_{EH2} := MH_1$ $\qquad M_{EH2} = 20469\ \frac{lbf \cdot ft}{ft}$

$M_{LS2} := MH_2$ $\qquad M_{LS2} = 8472.3\ \frac{lbf \cdot ft}{ft}$

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) $\quad Ia_{EV} := 1 \quad Ia_{EH} := 1.5 \quad Ia_{LS} := 1.75$

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

Factored Vertical Loads by Limit State:

$V_{Ia} := \eta \cdot \left(Ia_{EV} \cdot V_{EV}\right)$ $\qquad V_{Ia} = 19152.8\ \frac{lbf}{ft}$

$V_{Ib} := \eta \cdot \left(\left(Ib_{EV} \cdot V_{EV}\right) + \left(Ib_{LS} \cdot V_{LS}\right)\right)$ $\qquad V_{Ib} = 30480.7\ \frac{lbf}{ft}$

Factored Horizontal Loads by Limit State:

$H_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot H_{LS}\right) + \left(Ia_{EH} \cdot H_{EH}\right)\right)$ $\qquad H_{Ia} = 8063.8\ \frac{lbf}{ft}$

$H_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot H_{LS}\right) + \left(Ib_{EH} \cdot H_{EH}\right)\right)$ $\qquad H_{Ib} = 8063.8\ \frac{lbf}{ft}$

Factored Moments Produced by Vertical Loads by Limit State:

$MV_{Ia} := \eta \cdot \left(Ia_{EV} \cdot M_{EV}\right)$ $\qquad MV_{Ia} = 0\ \frac{lbf \cdot ft}{ft}$

$MV_{Ib} := \eta \cdot \left(\left(Ib_{EV} \cdot M_{EV}\right) + \left(Ib_{LS} \cdot M_{LS}\right)\right)$ $\qquad MV_{Ib} = 0\ \frac{lbf \cdot ft}{ft}$

Factored Moments Produced by Horizontal Loads by Limit State:

$MH_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot M_{LS2}\right) + \left(Ia_{EH} \cdot M_{EH2}\right)\right)$ $\qquad MH_{Ia} = 45530\ \frac{lbf \cdot ft}{ft}$

$MH_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot M_{LS2}\right) + \left(Ib_{EH} \cdot M_{EH2}\right)\right)$ $\qquad MH_{Ib} = 45530\ \frac{lbf \cdot ft}{ft}$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength Ib):

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

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 45530 \ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 30480.7 \ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$ $e_{wall} = 1.5 \ ft$ Wall Eccentricity

$B' := \text{if}\left(e_{wall} > 0, L - 2 \cdot e_{wall}, L\right)$ $B' = 7.6 \ ft$ Effective Bearing Width

Foundation Layout:

$L_{Wall} := 9 \cdot ft$ Assumed Footing Length (Wall Section Length)

$H' := H_{Ib}$ $H' = 8063.8 \ \frac{lbf}{ft}$ Summation of Horizontal Loads (Strength Ib)

$V' := V_{Ib}$ $V' = 30480.7 \ \frac{lbf}{ft}$ Summation of Vertical Loads (Strength Ib)

$D_f := d_e$ $D_f = 3 \ ft$ Footing embedment

$d_w := -0.5 \cdot ft$ Depth of Groundwater below Bearing Grade

$\theta' := 90 \cdot deg$ Direction of H' and V' resultant measured from wall back face **LRFD [Figure C10.6.3.1.2a-1]**

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45 \ deg + \frac{\phi'_f}{2}\right)^2, 1.0\right)$ $N_q = 9.6$

$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$ $N_c = 19.32$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$ $N_\gamma = 9.4$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right)$ $s_c = 1.419$

$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \cdot \tan(\phi'_f)\right), 1\right)$ $s_q = 1.375$

$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{Wall}}\right), 1\right)$ $s_\gamma = 0.663$

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

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

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

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

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

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

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

Depth Correction Factor per **Hanson (1970):**

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

$d_q = 1.1$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\qquad N_{cm} = 27.414$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\qquad N_{qm} = 13.206$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\qquad N_{\gamma m} = 6.26$

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

$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\qquad q_{nd} = 7.9\ \textit{ksf}$

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

$\phi_b := 0.65$ $\qquad$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Rd} := \phi_b \cdot q_{nd}$ $\qquad q_{Rd} = 5.2\ \textit{ksf}$ $\qquad$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

$N_q = 1$

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

$N_c = 5.14$

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

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

$s_c := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right)$ $s_c = 1.169$

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

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

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

$i_q := 1$ $i_q = 1$

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

$i_c := 1$ $i_c = 1$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $N_{cm} = 6.006$

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

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

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

$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $q_{nu} = 8.6\ \textbf{ksf}$

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

$\phi_b := 0.65$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Ru} := \phi_b \cdot q_{nu}$ $q_{Ru} = 5.6\ \textbf{ksf}$ Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 5.2\ \textbf{ksf}$

Undrained Conditions: $q_{Ru} = 5.6\ \textbf{ksf}$

Factored Bearing Resistance to be used in CDR Calculations:

$q_R := q_{Rd}$

$q_R = 5.2\ \textbf{ksf}$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

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

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

$\Sigma M_O := MH_{Ib}$ $\qquad$ $\Sigma M_O = 45530 \ \frac{lbf \cdot ft}{ft}$ $\qquad$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\qquad$ $\Sigma V = 30480.7 \ \frac{lbf}{ft}$ $\qquad$ Sum of Vertical Loads (Strength Ib)

$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$ $\qquad$ $e_{wall} = 1.5 \ ft$ $\qquad$ Wall Eccentricity

$B' := \text{if}\left(e_{wall} > 0, L - 2 \cdot e_{wall}, L\right)$ $\qquad$ $B' = 7.6 \ ft$ $\qquad$ Effective Bearing Width

Compute the bearing stress:

$\sigma := \frac{V_1 + V_2}{B'}$ $\qquad$ $\sigma = 2.9 \ ksf$ $\qquad$ Nominal Bearing Stress

$\sigma_v := \frac{\Sigma V}{B'}$ $\qquad$ $\sigma_v = 4.0 \ ksf$ $\qquad$ Factored Bearing Stress

Bearing Capacity:Demand Ratio (CDR)

$CDR_{Bearing} := \frac{q_R}{\sigma_v}$ $\qquad$ Is the CDR > or = to 1.0? $\qquad$ $CDR_{Bearing} = 1.28$

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

$e_{max} := \frac{L}{3}$ $\qquad$ $e_{max} = 3.5 \ ft$ $\qquad$ Maximum Eccentricity **LRFD [C11.6.3.3.]**

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

$\Sigma M_O := MH_{Ia}$ $\qquad$ $\Sigma M_O = 45530 \ \frac{lbf \cdot ft}{ft}$ $\qquad$ Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$ $\qquad$ $\Sigma V = 19152.8 \ \frac{lbf}{ft}$ $\qquad$ Sum of Vertical Loads (Strength Ia)

$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$ $\qquad$ $e_{wall} = 2.4 \ ft$

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}}$ $\qquad$ Is the CDR > or = to 1.0? $\qquad$ $CDR_{Eccentricity} = 1.48$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$F_\tau := H_{Ia}$ $F_\tau = 8063.8 \dfrac{lbf}{ft}$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$\Sigma V := V_{Ia}$ $\Sigma V = 19152.8 \dfrac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$R_{\tau d} := \Sigma V \cdot \tan\left(\phi'_f\right)$ $R_{\tau d} = 8527.4 \dfrac{lbf}{ft}$ Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

Drained Conditions: $R_{\tau d} = 8.527 \dfrac{kip}{ft}$

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

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

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

$\phi R_n := \phi_\tau \cdot R_\tau$ $R_R := \phi R_n$

$R_R = 8.5 \dfrac{kip}{ft}$

Sliding Capacity:Demand Ratio (CDR)

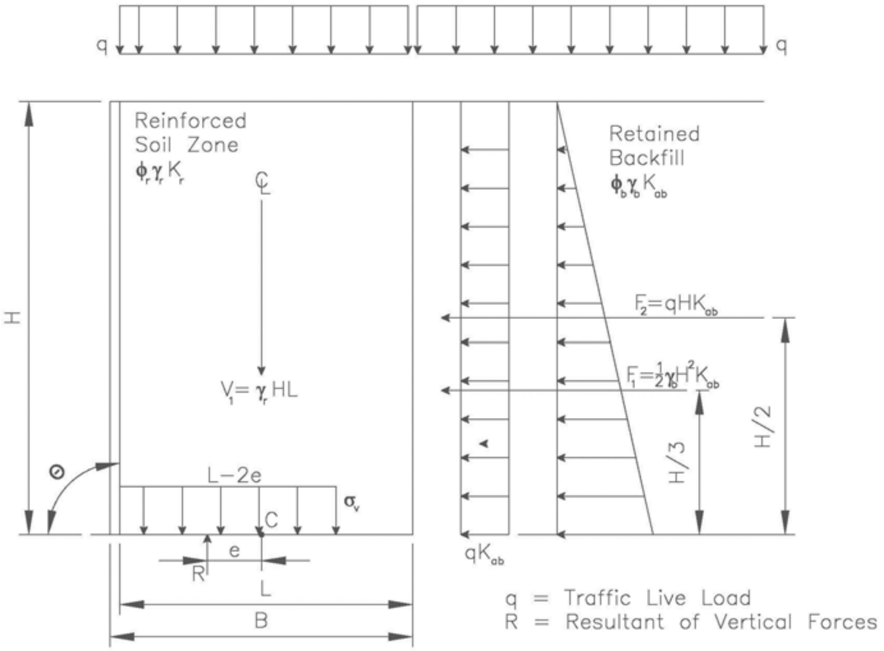
$CDR_{Sliding} := \dfrac{R_R}{F_\tau}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 1.06$

Objective: **To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.**

Method: **In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].**

Assumptions:

- **Horizontal backfill behind MSE wall on granular (drained) soils.**
- **For battered or vertical walls with a back face of wall angle of θ to horizontal.**
- **Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ.**
- **MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall**
- **Load combinations and wall configuration are as shown below:**



Givens:

Wall Geometry:

$H_e := 15.5 \cdot ft$	Exposed wall height
$\theta := 90 \cdot deg$	Angle of back face of wall to horizontal: 90 deg for vertical or near vertical walls (per Berg et al., 2009; near vertical = 80 deg < θ < 100 deg)

Reinforced Backfill Soil Design Parameters:

$\phi'_r := 34 \cdot deg$	Effective angle of internal friction (Per BDM [Table 307-1])
$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$	Unit weight (Per BDM [Table 307-1])
$c'_r := 0 \cdot \frac{lbf}{ft^2}$	Effective Cohesion

Retained Backfill Soil Design Parameters:

$\phi'_b := 30 \cdot deg$	Effective angle of internal friction (Per BDM [Table 307-1])
$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$	Unit weight (Per BDM [Table 307-1])
$c'_b := 0 \cdot \frac{lbf}{ft^2}$	Effective Cohesion

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$\phi'_f := 24 \cdot deg$ Effective angle of internal friction

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

$c'_f := 140 \cdot \frac{lbf}{ft^2}$ Cohesion

Undrained Conditions (Total Stress):

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

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

$c_f := 1400 \cdot \frac{lbf}{ft^2}$ Cohesion (Use Su if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

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

Depth of Embedment Check:

$d_{frost} := 3\ ft$ $\quad d_{user} := 0\ ft$ Local Frost Depth

$Slope_{fw} := 0\ deg$

Inclination of ground slope in front of wall :

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

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

$H_{est} := d_{est} + \left(4\ ft \cdot \tan\left(Slope_{fw}\right)\right) + H_e$ $\quad H_{est} = 18.5\ ft$

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

$d_{eSlope} = 0.9\ ft$ Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$d_e := \max\left(d_{est}, d_{eSlope}\right)$ $\quad d_e = 3\ ft$ Minimum Required Embedment Depth used in analysis.

$H := d_e + \left(4\ ft \cdot \tan\left(Slope_{fw}\right)\right) + H_e$ $\quad H = 18.5\ ft$ Design Wall Height

Estimate Length of Reinforcement:

$L_{user} := 0 \cdot ft$ User inputted value (if changes need to be made to satisfy other requirements)

$L := \max\left(8 \cdot ft, 0.7\ H, L_{user}\right)$ $\quad L = 13\ ft$ Length of Reinforcement

Live Load Surcharge Parameters:

$SUR := 250 \cdot \frac{lbf}{ft^2}$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)
Note: If vehicular loading is within 1 ft of the backface of the wall and with a design height, H, less than 20 ft, see **LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2** for adjusted surcharge load calculation.
Note: When traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$\beta := 0\ deg$ $\delta := 0.67 \cdot \phi'_b$

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

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

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

$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$ $F_T = 6104.2\ \frac{lbf}{ft}$ Active Earth Force Resultant (EH)

$F_{SUR} := SUR \cdot H \cdot k_{af}$ $F_{SUR} = 1374.8\ \frac{lbf}{ft}$ Live Load Surcharge (LS)

Vertical Loads:

$V_1 := \gamma_r \cdot H \cdot L$ $V_1 = 28749\ \frac{lbf}{ft}$ Soil backfill - reinforced soil (EV)

$V_2 := SUR \cdot L$ $V_2 = 3237.5\ \frac{lbf}{ft}$ Live Load Surcharge - (LS)

Moment Arm: Moment:

$d_{v1} := 0 \cdot ft$ $d_{v1} = 0\ ft$ $MV_1 := V_1 \cdot d_{v1}$ $MV_1 = 0\ \frac{lbf \cdot ft}{ft}$

$d_{v2} := 0\ ft$ $d_{v2} = 0\ ft$ $MV_2 := V_2 \cdot d_{v2}$ $MV_2 = 0\ \frac{lbf \cdot ft}{ft}$

Horizontal Loads:

$H_1 := F_T = 6104.2\ \frac{lbf}{ft}$ Active Earth Force Resultant (horizontal comp. - EH)

$H_2 := F_{SUR} = 1374.8\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm: Moment:

$d_{h1} := \frac{H}{3}$ $d_{h1} = 6.2\ ft$ $MH_1 := H_1 \cdot d_{h1}$ $MH_1 = 37642.8\ \frac{lbf \cdot ft}{ft}$

$d_{h2} := \frac{H}{2}$ $d_{h2} = 9.3\ ft$ $MH_2 := H_2 \cdot d_{h2}$ $MH_2 = 12717.2\ \frac{lbf \cdot ft}{ft}$

Unfactored Loads by Load Type

$V_{EV} := V_1$ $\qquad$ $V_{EV} = 28749\ \frac{lbf}{ft}$

$V_{LS} := V_2$ $\qquad$ $V_{LS} = 3237.5\ \frac{lbf}{ft}$

$H_{EH} := H_1$ $\qquad$ $H_{EH} = 6104.2\ \frac{lbf}{ft}$

$H_{LS} := H_2$ $\qquad$ $H_{LS} = 1374.8\ \frac{lbf}{ft}$

Unfactored Moments by Load Type

$M_{EV} := MV_1$ $\qquad$ $M_{EV} = 0\ \frac{lbf \cdot ft}{ft}$

$M_{LS} := MV_2$ $\qquad$ $M_{LS} = 0\ \frac{lbf \cdot ft}{ft}$

$M_{EH2} := MH_1$ $\qquad$ $M_{EH2} = 37642.8\ \frac{lbf \cdot ft}{ft}$

$M_{LS2} := MH_2$ $\qquad$ $M_{LS2} = 12717.2\ \frac{lbf \cdot ft}{ft}$

Load Combination Limit States:

$\eta := 1$ $\qquad$ LRFD Load Modifier

Strength Limit State I: $\qquad$ 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) $\qquad$ $Ia_{EV} := 1$ $\qquad$ $Ia_{EH} := 1.5$ $\qquad$ $Ia_{LS} := 1.75$

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

Factored Vertical Loads by Limit State:

$V_{Ia} := \eta \cdot \left(Ia_{EV} \cdot V_{EV}\right)$ $\qquad$ $V_{Ia} = 28749\ \frac{lbf}{ft}$

$V_{Ib} := \eta \cdot \left(\left(Ib_{EV} \cdot V_{EV}\right) + \left(Ib_{LS} \cdot V_{LS}\right)\right)$ $\qquad$ $V_{Ib} = 44476.8\ \frac{lbf}{ft}$

Factored Horizontal Loads by Limit State:

$H_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot H_{LS}\right) + \left(Ia_{EH} \cdot H_{EH}\right)\right)$ $\qquad$ $H_{Ia} = 11562.3\ \frac{lbf}{ft}$

$H_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot H_{LS}\right) + \left(Ib_{EH} \cdot H_{EH}\right)\right)$ $\qquad$ $H_{Ib} = 11562.3\ \frac{lbf}{ft}$

Factored Moments Produced by Vertical Loads by Limit State:

$MV_{Ia} := \eta \cdot \left(Ia_{EV} \cdot M_{EV}\right)$ $\qquad$ $MV_{Ia} = 0\ \frac{lbf \cdot ft}{ft}$

$MV_{Ib} := \eta \cdot \left(\left(Ib_{EV} \cdot M_{EV}\right) + \left(Ib_{LS} \cdot M_{LS}\right)\right)$ $\qquad$ $MV_{Ib} = 0\ \frac{lbf \cdot ft}{ft}$

Factored Moments Produced by Horizontal Loads by Limit State:

$MH_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot M_{LS2}\right) + \left(Ia_{EH} \cdot M_{EH2}\right)\right)$ $\qquad$ $MH_{Ia} = 78719.2\ \frac{lbf \cdot ft}{ft}$

$MH_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot M_{LS2}\right) + \left(Ib_{EH} \cdot M_{EH2}\right)\right)$ $\qquad$ $MH_{Ib} = 78719.2\ \frac{lbf \cdot ft}{ft}$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength Ib):

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

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 78719.2\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 44476.8\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$ $e_{wall} = 1.8\ ft$ Wall Eccentricity

$B' := \text{if}\left(e_{wall} > 0, L - 2 \cdot e_{wall}, L\right)$ $B' = 9.4\ ft$ Effective Bearing Width

Foundation Layout:

$L_{Wall} := 62.5 \cdot ft$ Assumed Footing Length (Wall Section Length)

$H' := H_{Ib}$ $H' = 11562.3\ \frac{lbf}{ft}$ Summation of Horizontal Loads (Strength Ib)

$V' := V_{Ib}$ $V' = 44476.8\ \frac{lbf}{ft}$ Summation of Vertical Loads (Strength Ib)

$D_f := d_e$ $D_f = 3\ ft$ Footing embedment

$d_w := -0.5 \cdot ft$ Depth of Groundwater below Bearing Grade

$\theta' := 90 \cdot deg$ Direction of H' and V' resultant measured from wall back face **LRFD [Figure C10.6.3.1.2a-1]**

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45\ deg + \frac{\phi'_f}{2}\right)^2, 1.0\right)$ $N_q = 9.6$

$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$ $N_c = 19.32$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$ $N_\gamma = 9.4$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right)$ $s_c = 1.075$

$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \cdot \tan(\phi'_f)\right), 1\right)$ $s_q = 1.067$

$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{Wall}}\right), 1\right)$ $s_\gamma = 0.94$

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

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

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

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

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

$C_{wq} := \text{if}\left(d_w \ge 0, 1, 0.5\right)$ $\qquad$ $C_{wq} = 0.5$

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

Depth Correction Factor per **Hanson (1970):**

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

$d_q = 1.1$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\qquad$ $N_{cm} = 20.769$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\qquad$ $N_{qm} = 10.247$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\qquad$ $N_{\gamma m} = 8.873$

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

$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\qquad$ $q_{nd} = 7.4\ \mathbf{ksf}$

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

$\phi_b := 0.65$ $\qquad$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Rd} := \phi_b \cdot q_{nd}$ $\qquad$ $q_{Rd} = 4.8\ \mathbf{ksf}$ $\qquad$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

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

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

$s_c := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right)$ $s_c = 1.03$

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

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

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

$i_q := 1$ $i_q = 1$

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

$i_c := 1$ $i_c = 1$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $N_{cm} = 5.295$

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

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

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

$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $q_{nu} = 7.6$ ***ksf***

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

$\phi_b := 0.65$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Ru} := \phi_b \cdot q_{nu}$ $q_{Ru} = 4.9$ ***ksf*** Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 4.8$ ***ksf***

Undrained Conditions: $q_{Ru} = 4.9$ ***ksf***

Factored Bearing Resistance to be used in CDR Calculations:

$q_R := q_{Rd}$

$q_R = 4.8$ ***ksf***

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

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

$\Sigma M_R := MV_{Ib}$	$\Sigma M_R = 0\ \dfrac{lbf \cdot ft}{ft}$	Sum of Resisting Moments (Strength Ib)
$\Sigma M_O := MH_{Ib}$	$\Sigma M_O = 78719.2\ \dfrac{lbf \cdot ft}{ft}$	Sum of Overturning Moments (Strength Ib)
$\Sigma V := V_{Ib}$	$\Sigma V = 44476.8\ \dfrac{lbf}{ft}$	Sum of Vertical Loads (Strength Ib)
$e_{wall} := \dfrac{\left(\Sigma M_O - \Sigma M_R\right)}{\Sigma V}$	$e_{wall} = 1.8\ ft$	Wall Eccentricity
$B' := \text{if}\left(e_{wall} > 0, L - 2 \cdot e_{wall}, L\right)$	$B' = 9.4\ ft$	Effective Bearing Width

Compute the bearing stress:

$\sigma := \dfrac{V_1 + V_2}{B'}$	$\sigma = 3.4\ ksf$	Nominal Bearing Stress
$\sigma_v := \dfrac{\Sigma V}{B'}$	$\sigma_v = 4.7\ ksf$	Factored Bearing Stress

Bearing Capacity:Demand Ratio (CDR)

$CDR_{Bearing} := \dfrac{q_R}{\sigma_v}$ Is the CDR > or = to 1.0? $CDR_{Bearing} = 1.02$

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

$e_{max} := \dfrac{L}{3}$	$e_{max} = 4.3\ ft$	Maximum Eccentricity **LRFD [C11.6.3.3.]**
$\Sigma M_R := MV_{Ia}$	$\Sigma M_R = 0\ \dfrac{lbf \cdot ft}{ft}$	Sum of Resisting Moments (Strength Ia)
$\Sigma M_O := MH_{Ia}$	$\Sigma M_O = 78719.2\ \dfrac{lbf \cdot ft}{ft}$	Sum of Overturning Moments (Strength Ia)
$\Sigma V := V_{Ia}$	$\Sigma V = 28749\ \dfrac{lbf}{ft}$	Sum of Vertical Loads (Strength Ia)
$e_{wall} := \dfrac{\left(\Sigma M_O - \Sigma M_R\right)}{\Sigma V}$	$e_{wall} = 2.7\ ft$	

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \dfrac{e_{max}}{e_{wall}}$ Is the CDR > or = to 1.0? $CDR_{Eccentricity} = 1.58$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$F_\tau := H_{Ia}$ $\quad$ $F_\tau = 11562.3\ \frac{lbf}{ft}$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$\Sigma V := V_{Ia}$ $\quad$ $\Sigma V = 28749\ \frac{lbf}{ft}$ $\quad$ Sum of Vertical Loads (Strength Ia)

$R_{\tau d} := \Sigma V \cdot \tan\left(\phi'_f\right)$ $\quad$ $R_{\tau d} = 12799.9\ \frac{lbf}{ft}$ $\quad$ Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

Drained Conditions: $R_{\tau d} = 12.8\ \frac{kip}{ft}$

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

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

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

$\phi R_n := \phi_\tau \cdot R_\tau$ $\quad$ $R_R := \phi R_n$

$R_R = 12.8\ \frac{kip}{ft}$

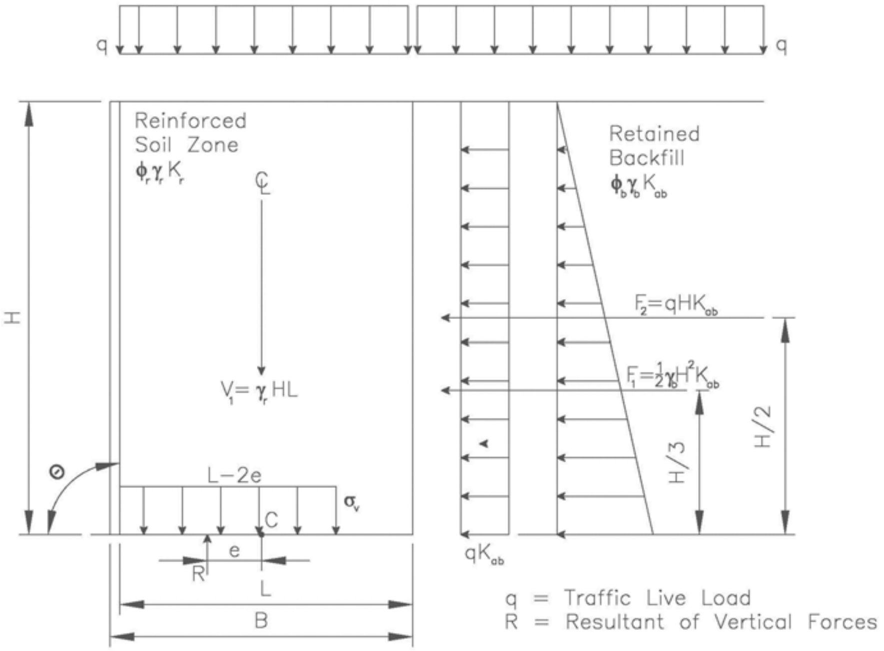
Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{F_\tau}$ $\quad$ Is the CDR > or = to 1.0? $\quad$ $CDR_{Sliding} = 1.11$

RETAINING WALL 2

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

- **Horizontal backfill behind MSE wall on granular (drained) soils.**
- **For battered or vertical walls with a back face of wall angle of θ to horizontal.**
- **Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_{μ}.**
- **MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall**
- **Load combinations and wall configuration are as shown below:**



Givens:

Wall Geometry:

$H_e := 12.7 \cdot ft$ Exposed wall height

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

Reinforced Backfill Soil Design Parameters:

$\phi'_r := 34 \cdot deg$ Effective angle of internal friction (Per BDM [Table 307-1])

$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$ Unit weight (Per BDM [Table 307-1])

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

Retained Backfill Soil Design Parameters:

$\phi'_b := 30 \cdot deg$ Effective angle of internal friction (Per BDM [Table 307-1])

$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$ Unit weight (Per BDM [Table 307-1])

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

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$\phi'_f := 24 \cdot deg$ Effective angle of internal friction

$\gamma_f := 122 \cdot \frac{lbf}{ft^3}$ Unit weight

$c'_f := 190 \cdot \frac{lbf}{ft^2}$ Cohesion

Undrained Conditions (Total Stress):

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

$\gamma_f = 122 \ \frac{lbf}{ft^3}$ Unit weight

$c_f := 1900 \cdot \frac{lbf}{ft^2}$ Cohesion (Use Su if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

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

Depth of Embedment Check:

$d_{frost} := 3\ ft$ $d_{user} := 0\ ft$ Local Frost Depth

$Slope_{fw} := 0\ deg$

Inclination of ground slope in front of wall :

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

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

$H_{est} := d_{est} + \left(4\ ft \cdot \tan\left(Slope_{fw}\right)\right) + H_e$ $H_{est} = 15.7\ ft$

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

$d_{eSlope} = 0.8\ ft$ Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$d_e := \max\left(d_{est}, d_{eSlope}\right)$ $d_e = 3\ ft$ Minimum Required Embedment Depth used in analysis.

$H := d_e + \left(4\ ft \cdot \tan\left(Slope_{fw}\right)\right) + H_e$ $H = 15.7\ ft$ Design Wall Height

Estimate Length of Reinforcement:

$L_{user} := 0 \cdot ft$ User inputted value (if changes need to be made to satisfy other requirements)

$L := \max\left(8 \cdot ft, 0.7\ H, L_{user}\right)$ $L = 11\ ft$ Length of Reinforcement

Live Load Surcharge Parameters:

$SUR := 250 \cdot \frac{lbf}{ft^2}$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)
Note: If vehicular loading is within 1 ft of the backface of the wall and with a design height, H, less than 20 ft, see **LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2** for adjusted surcharge load calculation.
Note: When traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$\beta := 0\ deg$ $\quad \delta := 0.67 \cdot \phi'_b$ — Inclination of ground slope behind face of wall and interface angle of friction between retained backfill and reinforced soil

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

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

Active Earth Pressure Coefficient

$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$ $\qquad F_T = 4396.3\ \frac{lbf}{ft}$ — Active Earth Force Resultant (EH)

$F_{SUR} := SUR \cdot H \cdot k_{af}$ $\qquad F_{SUR} = 1166.7\ \frac{lbf}{ft}$ — Live Load Surcharge (LS)

Vertical Loads:

$V_1 := \gamma_r \cdot H \cdot L$ $\qquad V_1 = 20705.2\ \frac{lbf}{ft}$ — Soil backfill - reinforced soil (EV)

$V_2 := SUR \cdot L$ $\qquad V_2 = 2747.5\ \frac{lbf}{ft}$ — Live Load Surcharge - (LS)

Moment Arm: Moment:

$d_{v1} := 0 \cdot ft$ $\qquad d_{v1} = 0\ ft$ $\qquad MV_1 := V_1 \cdot d_{v1}$ $\qquad MV_1 = 0\ \frac{lbf \cdot ft}{ft}$

$d_{v2} := 0\ ft$ $\qquad d_{v2} = 0\ ft$ $\qquad MV_2 := V_2 \cdot d_{v2}$ $\qquad MV_2 = 0\ \frac{lbf \cdot ft}{ft}$

Horizontal Loads:

$H_1 := F_T = 4396.3\ \frac{lbf}{ft}$ — Active Earth Force Resultant (horizontal comp. - EH)

$H_2 := F_{SUR} = 1166.7\ \frac{lbf}{ft}$ — Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm: Moment:

$d_{h1} := \frac{H}{3}$ $\qquad d_{h1} = 5.2\ ft$ $\qquad MH_1 := H_1 \cdot d_{h1}$ $\qquad MH_1 = 23007.3\ \frac{lbf \cdot ft}{ft}$

$d_{h2} := \frac{H}{2}$ $\qquad d_{h2} = 7.9\ ft$ $\qquad MH_2 := H_2 \cdot d_{h2}$ $\qquad MH_2 = 9159\ \frac{lbf \cdot ft}{ft}$

Unfactored Loads by Load Type

$V_{EV} := V_1$ $\qquad$ $V_{EV} = 20705.2\ \frac{lbf}{ft}$

$V_{LS} := V_2$ $\qquad$ $V_{LS} = 2747.5\ \frac{lbf}{ft}$

$H_{EH} := H_1$ $\qquad$ $H_{EH} = 4396.3\ \frac{lbf}{ft}$

$H_{LS} := H_2$ $\qquad$ $H_{LS} = 1166.7\ \frac{lbf}{ft}$

Unfactored Moments by Load Type

$M_{EV} := MV_1$ $\qquad$ $M_{EV} = 0\ \frac{lbf \cdot ft}{ft}$

$M_{LS} := MV_2$ $\qquad$ $M_{LS} = 0\ \frac{lbf \cdot ft}{ft}$

$M_{EH2} := MH_1$ $\qquad$ $M_{EH2} = 23007.3\ \frac{lbf \cdot ft}{ft}$

$M_{LS2} := MH_2$ $\qquad$ $M_{LS2} = 9159\ \frac{lbf \cdot ft}{ft}$

Load Combination Limit States:

$\eta := 1$ $\qquad$ LRFD Load Modifier

Strength Limit State I: $\qquad$ 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) $\qquad$ $Ia_{EV} := 1$ $\qquad$ $Ia_{EH} := 1.5$ $\qquad$ $Ia_{LS} := 1.75$

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

Factored Vertical Loads by Limit State:

$V_{Ia} := \eta \cdot \left(Ia_{EV} \cdot V_{EV}\right)$ $\qquad$ $V_{Ia} = 20705.2\ \frac{lbf}{ft}$

$V_{Ib} := \eta \cdot \left(\left(Ib_{EV} \cdot V_{EV}\right) + \left(Ib_{LS} \cdot V_{LS}\right)\right)$ $\qquad$ $V_{Ib} = 32760.1\ \frac{lbf}{ft}$

Factored Horizontal Loads by Limit State:

$H_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot H_{LS}\right) + \left(Ia_{EH} \cdot H_{EH}\right)\right)$ $\qquad$ $H_{Ia} = 8636.3\ \frac{lbf}{ft}$

$H_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot H_{LS}\right) + \left(Ib_{EH} \cdot H_{EH}\right)\right)$ $\qquad$ $H_{Ib} = 8636.3\ \frac{lbf}{ft}$

Factored Moments Produced by Vertical Loads by Limit State:

$MV_{Ia} := \eta \cdot \left(Ia_{EV} \cdot M_{EV}\right)$ $\qquad$ $MV_{Ia} = 0\ \frac{lbf \cdot ft}{ft}$

$MV_{Ib} := \eta \cdot \left(\left(Ib_{EV} \cdot M_{EV}\right) + \left(Ib_{LS} \cdot M_{LS}\right)\right)$ $\qquad$ $MV_{Ib} = 0\ \frac{lbf \cdot ft}{ft}$

Factored Moments Produced by Horizontal Loads by Limit State:

$MH_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot M_{LS2}\right) + \left(Ia_{EH} \cdot M_{EH2}\right)\right)$ $\qquad$ $MH_{Ia} = 50539.1\ \frac{lbf \cdot ft}{ft}$

$MH_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot M_{LS2}\right) + \left(Ib_{EH} \cdot M_{EH2}\right)\right)$ $\qquad$ $MH_{Ib} = 50539.1\ \frac{lbf \cdot ft}{ft}$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength Ib):

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

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 50539.1\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 32760.1\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$e_{wall} := \frac{\left(\Sigma M_O - \Sigma M_R\right)}{\Sigma V}$ $e_{wall} = 1.5\ ft$ Wall Eccentricity

$B' := \text{if}\left(e_{wall} > 0, L - 2 \cdot e_{wall}, L\right)$ $B' = 7.9\ ft$ Effective Bearing Width

Foundation Layout:

$L_{Wall} := 4.8 \cdot ft$ Assumed Footing Length (Wall Section Length)

$H' := H_{Ib}$ $H' = 8636.3\ \frac{lbf}{ft}$ Summation of Horizontal Loads (Strength Ib)

$V' := V_{Ib}$ $V' = 32760.1\ \frac{lbf}{ft}$ Summation of Vertical Loads (Strength Ib)

$D_f := d_e$ $D_f = 3\ ft$ Footing embedment

$d_w := -0.5 \cdot ft$ Depth of Groundwater below Bearing Grade

$\theta' := 90 \cdot deg$ Direction of H' and V' resultant measured from wall back face **LRFD [Figure C10.6.3.1.2a-1]**

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan\left(\phi'_f\right)} \cdot \tan\left(45\ deg + \frac{\phi'_f}{2}\right)^2, 1.0\right)$ $N_q = 9.6$

$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan\left(\phi'_f\right)}, 5.14\right)$ $N_c = 19.32$

$N_\gamma := 2 \cdot \left(N_q + 1\right) \cdot \tan\left(\phi'_f\right)$ $N_\gamma = 9.4$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right)$ $s_c = 1.818$

$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \cdot \tan\left(\phi'_f\right)\right), 1\right)$ $s_q = 1.733$

$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{Wall}}\right), 1\right)$ $s_\gamma = 0.341$

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

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

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

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

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

$C_{wq} := \text{if}\left(d_w \ge 0, 1, 0.5\right)$ $\qquad$ $C_{wq} = 0.5$

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

Depth Correction Factor per **Hanson (1970):**

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

$d_q = 1.1$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\qquad$ $N_{cm} = 35.138$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\qquad$ $N_{qm} = 16.645$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\qquad$ $N_{\gamma m} = 3.222$

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

$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\qquad$ $q_{nd} = 10.8\ \mathbf{ksf}$

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

$\phi_b := 0.65$ $\qquad$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Rd} := \phi_b \cdot q_{nd}$ $\qquad$ $q_{Rd} = 7\ \mathbf{ksf}$ $\qquad$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

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

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

$s_c := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right)$ $s_c = 1.329$

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

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

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

$i_q := 1$ $i_q = 1$

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

$i_c := 1$ $i_c = 1$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $N_{cm} = 6.833$

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

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

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

$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $q_{nu} = 13.2$ ***ksf***

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

$\phi_b := 0.65$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Ru} := \phi_b \cdot q_{nu}$ $q_{Ru} = 8.6$ ***ksf*** Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 7$ ***ksf***

Undrained Conditions: $q_{Ru} = 8.6$ ***ksf***

Factored Bearing Resistance to be used in CDR Calculations:

$q_R := q_{Rd}$

$q_R = 7$ ***ksf***

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

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

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

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 50539.1\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 32760.1\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$ $e_{wall} = 1.5\ ft$ Wall Eccentricity

$B' := \text{if}\left(e_{wall} > 0, L - 2 \cdot e_{wall}, L\right)$ $B' = 7.9\ ft$ Effective Bearing Width

Compute the bearing stress:

$\sigma := \frac{V_1 + V_2}{B'}$ $\sigma = 3.0\ ksf$ Nominal Bearing Stress

$\sigma_v := \frac{\Sigma V}{B'}$ $\sigma_v = 4.1\ ksf$ Factored Bearing Stress

Bearing Capacity:Demand Ratio (CDR)

$CDR_{Bearing} := \frac{q_R}{\sigma_v}$ Is the CDR > or = to 1.0? $CDR_{Bearing} = 1.69$

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

$e_{max} := \frac{L}{3}$ $e_{max} = 3.7\ ft$ Maximum Eccentricity **LRFD [C11.6.3.3.]**

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

$\Sigma M_O := MH_{Ia}$ $\Sigma M_O = 50539.1\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$ $\Sigma V = 20705.2\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$ $e_{wall} = 2.4\ ft$

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}}$ Is the CDR > or = to 1.0? $CDR_{Eccentricity} = 1.50$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$F_\tau := H_{Ia}$ $F_\tau = 8636.3\ \frac{lbf}{ft}$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$\Sigma V := V_{Ia}$ $\Sigma V = 20705.2\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$R_{\tau d} := \Sigma V \cdot \tan\left(\phi'_f\right)$ $R_{\tau d} = 9218.5\ \frac{lbf}{ft}$ Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

Drained Conditions: $R_{\tau d} = 9.219\ \frac{kip}{ft}$

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

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

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

$\phi R_n := \phi_\tau \cdot R_\tau$ $R_R := \phi R_n$

$R_R = 9.2\ \frac{kip}{ft}$

Sliding Capacity:Demand Ratio (CDR)

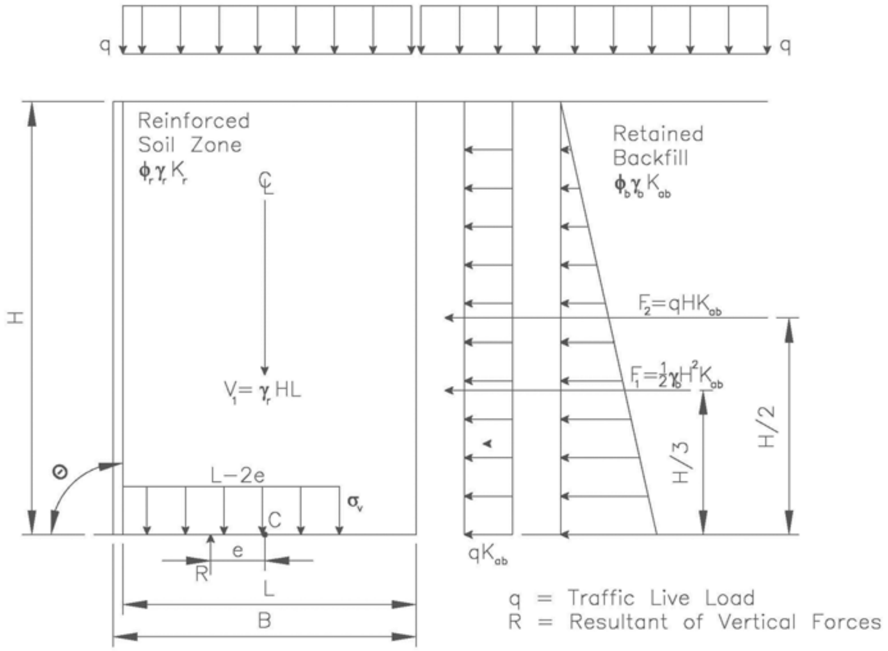
$CDR_{Sliding} := \frac{R_R}{F_\tau}$ Is the CDR > or = to 1.0? $CDR_{Sliding} = 1.07$

Objective: To evaluate the external stability of MSE wall design with vertical wall face and horizontal backfill.

Method: In accordance with ODOT Bridge Design Manual, 2013 [Sect. 307] LRFD Bridge Design Specifications, 8th Ed., 2018, [Sect. 11.10.5].

Assumptions:

- **Horizontal backfill behind MSE wall on granular (drained) soils.**
- **For battered or vertical walls with a back face of wall angle of θ to horizontal.**
- **Not for sheet type reinforcement. If so, use different assessment for Sliding parameter ϕ_μ.**
- **MSE wall not acting as abutment, if so must meet minimum embedment depth of H/10 if no slope in front of wall**
- **Load combinations and wall configuration are as shown below:**



Givens:

Wall Geometry:

$H_e := 15.5 \cdot ft$ Exposed wall height

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

Reinforced Backfill Soil Design Parameters:

$\phi'_r := 34 \cdot deg$ Effective angle of internal friction (Per BDM [Table 307-1])

$\gamma_r := 120 \cdot \frac{lbf}{ft^3}$ Unit weight (Per BDM [Table 307-1])

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

Retained Backfill Soil Design Parameters:

$\phi'_b := 30 \cdot deg$ Effective angle of internal friction (Per BDM [Table 307-1])

$\gamma_b := 120 \cdot \frac{lbf}{ft^3}$ Unit weight (Per BDM [Table 307-1])

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

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$\phi'_f := 24 \cdot deg$ Effective angle of internal friction

$\gamma_f := 122 \cdot \frac{lbf}{ft^3}$ Unit weight

$c'_f := 190 \cdot \frac{lbf}{ft^2}$ Cohesion

Undrained Conditions (Total Stress):

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

$\gamma_f = 122 \ \frac{lbf}{ft^3}$ Unit weight

$c_f := 1900 \cdot \frac{lbf}{ft^2}$ Cohesion (Use Su if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

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

Depth of Embedment Check:

$d_{frost} := 3 \ ft$ $d_{user} := 0 \ ft$ Local Frost Depth

$Slope_{fw} := 0 \ deg$

Inclination of ground slope in front of wall :

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

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

$H_{est} := d_{est} + \left(4 \ ft \cdot \tan\left(Slope_{fw}\right)\right) + H_e$ $H_{est} = 18.5 \ ft$

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

$d_{eSlope} = 0.9 \ ft$ Minimum Embedment Depth per Table C11.10.2.2-1 of LRFD BDS

$d_e := \max\left(d_{est}, d_{eSlope}\right)$ $d_e = 3 \ ft$ Minimum Required Embedment Depth used in analysis.

$H := d_e + \left(4 \ ft \cdot \tan\left(Slope_{fw}\right)\right) + H_e$ $H = 18.5 \ ft$ Design Wall Height

Estimate Length of Reinforcement:

$L_{user} := 0 \cdot ft$ User inputted value (if changes need to be made to satisfy other requirements)

$L := \max\left(8 \cdot ft, 0.7 \ H, L_{user}\right)$ $L = 13 \ ft$ Length of Reinforcement

Live Load Surcharge Parameters:

$SUR := 250 \cdot \frac{lbf}{ft^2}$

Live load surcharge (per **LRFD BDS [3.11.6.4]** & **BDM [307.1.1]**)
Note: If vehicular loading is within 1 ft of the backface of the wall and with a design height, H, less than 20 ft, see **LRFD BDS Section 3.11.6.4 and Table 3.11.6.4-2** for adjusted surcharge load calculation.
Note: When traffic vehicular live loads are not present within 0.5*H from the back of the reinforced zone let SUR equal 100 psf to account for construction loads.

Calculations:

Active Earth Pressure:

$\beta := 0\ deg$ $\delta := 0.67 \cdot \phi'_b$

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

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

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

$k_{af} = 0.2973$ Active Earth Pressure Coefficient

$F_T := \frac{1}{2} \cdot \gamma_b \cdot H^2 \cdot k_{af}$ $F_T = 6104.2\ \frac{lbf}{ft}$ Active Earth Force Resultant (EH)

$F_{SUR} := SUR \cdot H \cdot k_{af}$ $F_{SUR} = 1374.8\ \frac{lbf}{ft}$ Live Load Surcharge (LS)

Vertical Loads:

$V_1 := \gamma_r \cdot H \cdot L$ $V_1 = 28749\ \frac{lbf}{ft}$ Soil backfill - reinforced soil (EV)

$V_2 := SUR \cdot L$ $V_2 = 3237.5\ \frac{lbf}{ft}$ Live Load Surcharge - (LS)

Moment Arm: Moment:

$d_{v1} := 0 \cdot ft$ $d_{v1} = 0\ ft$ $MV_1 := V_1 \cdot d_{v1}$ $MV_1 = 0\ \frac{lbf \cdot ft}{ft}$

$d_{v2} := 0\ ft$ $d_{v2} = 0\ ft$ $MV_2 := V_2 \cdot d_{v2}$ $MV_2 = 0\ \frac{lbf \cdot ft}{ft}$

Horizontal Loads:

$H_1 := F_T = 6104.2\ \frac{lbf}{ft}$ Active Earth Force Resultant (horizontal comp. - EH)

$H_2 := F_{SUR} = 1374.8\ \frac{lbf}{ft}$ Live Load Surcharge Resultant (horizontal comp. - LS)

Moment Arm: Moment:

$d_{h1} := \frac{H}{3}$ $d_{h1} = 6.2\ ft$ $MH_1 := H_1 \cdot d_{h1}$ $MH_1 = 37642.8\ \frac{lbf \cdot ft}{ft}$

$d_{h2} := \frac{H}{2}$ $d_{h2} = 9.3\ ft$ $MH_2 := H_2 \cdot d_{h2}$ $MH_2 = 12717.2\ \frac{lbf \cdot ft}{ft}$

Unfactored Loads by Load Type

$V_{EV} := V_1$ $\quad V_{EV} = 28749\ \frac{lbf}{ft}$

$V_{LS} := V_2$ $\quad V_{LS} = 3237.5\ \frac{lbf}{ft}$

$H_{EH} := H_1$ $\quad H_{EH} = 6104.2\ \frac{lbf}{ft}$

$H_{LS} := H_2$ $\quad H_{LS} = 1374.8\ \frac{lbf}{ft}$

Unfactored Moments by Load Type

$M_{EV} := MV_1$ $\quad M_{EV} = 0\ \frac{lbf \cdot ft}{ft}$

$M_{LS} := MV_2$ $\quad M_{LS} = 0\ \frac{lbf \cdot ft}{ft}$

$M_{EH2} := MH_1$ $\quad M_{EH2} = 37642.8\ \frac{lbf \cdot ft}{ft}$

$M_{LS2} := MH_2$ $\quad M_{LS2} = 12717.2\ \frac{lbf \cdot ft}{ft}$

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) $\quad Ia_{EV} := 1$ $\quad Ia_{EH} := 1.5$ $\quad Ia_{LS} := 1.75$

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

Factored Vertical Loads by Limit State:

$V_{Ia} := \eta \cdot \left(Ia_{EV} \cdot V_{EV}\right)$ $\quad V_{Ia} = 28749\ \frac{lbf}{ft}$

$V_{Ib} := \eta \cdot \left(\left(Ib_{EV} \cdot V_{EV}\right) + \left(Ib_{LS} \cdot V_{LS}\right)\right)$ $\quad V_{Ib} = 44476.8\ \frac{lbf}{ft}$

Factored Horizontal Loads by Limit State:

$H_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot H_{LS}\right) + \left(Ia_{EH} \cdot H_{EH}\right)\right)$ $\quad H_{Ia} = 11562.3\ \frac{lbf}{ft}$

$H_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot H_{LS}\right) + \left(Ib_{EH} \cdot H_{EH}\right)\right)$ $\quad H_{Ib} = 11562.3\ \frac{lbf}{ft}$

Factored Moments Produced by Vertical Loads by Limit State:

$MV_{Ia} := \eta \cdot \left(Ia_{EV} \cdot M_{EV}\right)$ $\quad MV_{Ia} = 0\ \frac{lbf \cdot ft}{ft}$

$MV_{Ib} := \eta \cdot \left(\left(Ib_{EV} \cdot M_{EV}\right) + \left(Ib_{LS} \cdot M_{LS}\right)\right)$ $\quad MV_{Ib} = 0\ \frac{lbf \cdot ft}{ft}$

Factored Moments Produced by Horizontal Loads by Limit State:

$MH_{Ia} := \eta \cdot \left(\left(Ia_{LS} \cdot M_{LS2}\right) + \left(Ia_{EH} \cdot M_{EH2}\right)\right)$ $\quad MH_{Ia} = 78719.2\ \frac{lbf \cdot ft}{ft}$

$MH_{Ib} := \eta \cdot \left(\left(Ib_{LS} \cdot M_{LS2}\right) + \left(Ib_{EH} \cdot M_{EH2}\right)\right)$ $\quad MH_{Ib} = 78719.2\ \frac{lbf \cdot ft}{ft}$

Compute Bearing Resistance:

Compute the Effective Bearing Length (Strength Ib):

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

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 78719.2\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 44476.8\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$ $e_{wall} = 1.8\ ft$ Wall Eccentricity

$B' := \text{if}\left(e_{wall} > 0, L - 2 \cdot e_{wall}, L\right)$ $B' = 9.4\ ft$ Effective Bearing Width

Foundation Layout:

$L_{Wall} := 61 \cdot ft$ Assumed Footing Length (Wall Section Length)

$H' := H_{Ib}$ $H' = 11562.3\ \frac{lbf}{ft}$ Summation of Horizontal Loads (Strength Ib)

$V' := V_{Ib}$ $V' = 44476.8\ \frac{lbf}{ft}$ Summation of Vertical Loads (Strength Ib)

$D_f := d_e$ $D_f = 3\ ft$ Footing embedment

$d_w := -0.5 \cdot ft$ Depth of Groundwater below Bearing Grade

$\theta' := 90 \cdot deg$ Direction of H' and V' resultant measured from wall back face **LRFD [Figure C10.6.3.1.2a-1]**

Drained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45\ deg + \frac{\phi'_f}{2}\right)^2, 1.0\right)$ $N_q = 9.6$

$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right)$ $N_c = 19.32$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f)$ $N_\gamma = 9.4$

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

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right)$ $s_c = 1.077$

$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B'}{L_{Wall}} \cdot \tan(\phi'_f)\right), 1\right)$ $s_q = 1.069$

$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B'}{L_{Wall}}\right), 1\right)$ $s_\gamma = 0.938$

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

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

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

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

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

$C_{wq} := \text{if}\left(d_w \ge 0, 1, 0.5\right)$ $\qquad$ $C_{wq} = 0.5$

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

Depth Correction Factor per **Hanson (1970):**

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

$d_q = 1.1$

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\qquad$ $N_{cm} = 20.805$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\qquad$ $N_{qm} = 10.263$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\qquad$ $N_{\gamma m} = 8.859$

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

$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\qquad$ $q_{nd} = 8.5\ \mathbf{ksf}$

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

$\phi_b := 0.65$ $\qquad$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Rd} := \phi_b \cdot q_{nd}$ $\qquad$ $q_{Rd} = 5.5\ \mathbf{ksf}$ $\qquad$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

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

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

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

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

$$s_c := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B'}{L_{Wall}}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L_{Wall}}\right)\right) \qquad s_c = 1.031$$

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

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

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

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

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

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

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

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\qquad N_{cm} = 5.299$

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

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

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

$$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma} \qquad q_{nu} = 10.3\ \textbf{ksf}$$

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

$\phi_b := 0.65$ $\qquad$ Bearing resistance factor **LRFD Table 11.5.7-1.**

$q_{Ru} := \phi_b \cdot q_{nu}$ $\qquad q_{Ru} = 6.7\ \textbf{ksf}$ $\qquad$ Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 5.5\ \textbf{ksf}$

Undrained Conditions: $q_{Ru} = 6.7\ \textbf{ksf}$

Factored Bearing Resistance to be used in CDR Calculations:

$q_R := q_{Rd}$

$q_R = 5.5\ \textbf{ksf}$

Evaluate External Stability of Wall:

Bearing Resistance at Base of the Wall:

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

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

$\Sigma M_O := MH_{Ib}$ $\Sigma M_O = 78719.2\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ib)

$\Sigma V := V_{Ib}$ $\Sigma V = 44476.8\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ib)

$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$ $e_{wall} = 1.8\ ft$ Wall Eccentricity

$B' := \text{if}\left(e_{wall} > 0, L - 2 \cdot e_{wall}, L\right)$ $B' = 9.4\ ft$ Effective Bearing Width

Compute the bearing stress:

$\sigma := \frac{V_1 + V_2}{B'}$ $\sigma = 3.4\ ksf$ Nominal Bearing Stress

$\sigma_v := \frac{\Sigma V}{B'}$ $\sigma_v = 4.7\ ksf$ Factored Bearing Stress

Bearing Capacity:Demand Ratio (CDR)

$CDR_{Bearing} := \frac{q_R}{\sigma_v}$ Is the CDR > or = to 1.0? $CDR_{Bearing} = 1.17$

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

$e_{max} := \frac{L}{3}$ $e_{max} = 4.3\ ft$ Maximum Eccentricity **LRFD [C11.6.3.3.]**

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

$\Sigma M_O := MH_{Ia}$ $\Sigma M_O = 78719.2\ \frac{lbf \cdot ft}{ft}$ Sum of Overturning Moments (Strength Ia)

$\Sigma V := V_{Ia}$ $\Sigma V = 28749\ \frac{lbf}{ft}$ Sum of Vertical Loads (Strength Ia)

$e_{wall} := \frac{(\Sigma M_O - \Sigma M_R)}{\Sigma V}$ $e_{wall} = 2.7\ ft$

Eccentricity Capacity:Demand Ratio (CDR)

$CDR_{Eccentricity} := \frac{e_{max}}{e_{wall}}$ Is the CDR > or = to 1.0? $CDR_{Eccentricity} = 1.58$

Sliding Resistance at Base of Wall LRFD [10.6.3.4]:

Factored Sliding Force (Strength Ia):

$F_\tau := H_{Ia}$ $\qquad$ $F_\tau = 11562.3 \ \frac{lbf}{ft}$

Compute sliding resistance between soil and foundation:

Drained Conditions:

$\Sigma V := V_{Ia}$ $\qquad$ $\Sigma V = 28749 \ \frac{lbf}{ft}$ $\qquad$ Sum of Vertical Loads (Strength Ia)

$R_{\tau d} := \Sigma V \cdot \tan\left(\phi'_f\right)$ $\qquad$ $R_{\tau d} = 12799.9 \ \frac{lbf}{ft}$ $\qquad$ Nominal sliding resistance Drained Conditions

Nominal Sliding Resistance Drained Conditions:

Drained Conditions: $R_{\tau d} = 12.8 \ \frac{kip}{ft}$

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

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

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

$\phi R_n := \phi_\tau \cdot R_\tau$ $\qquad$ $R_R := \phi R_n$

$R_R = 12.8 \ \frac{kip}{ft}$

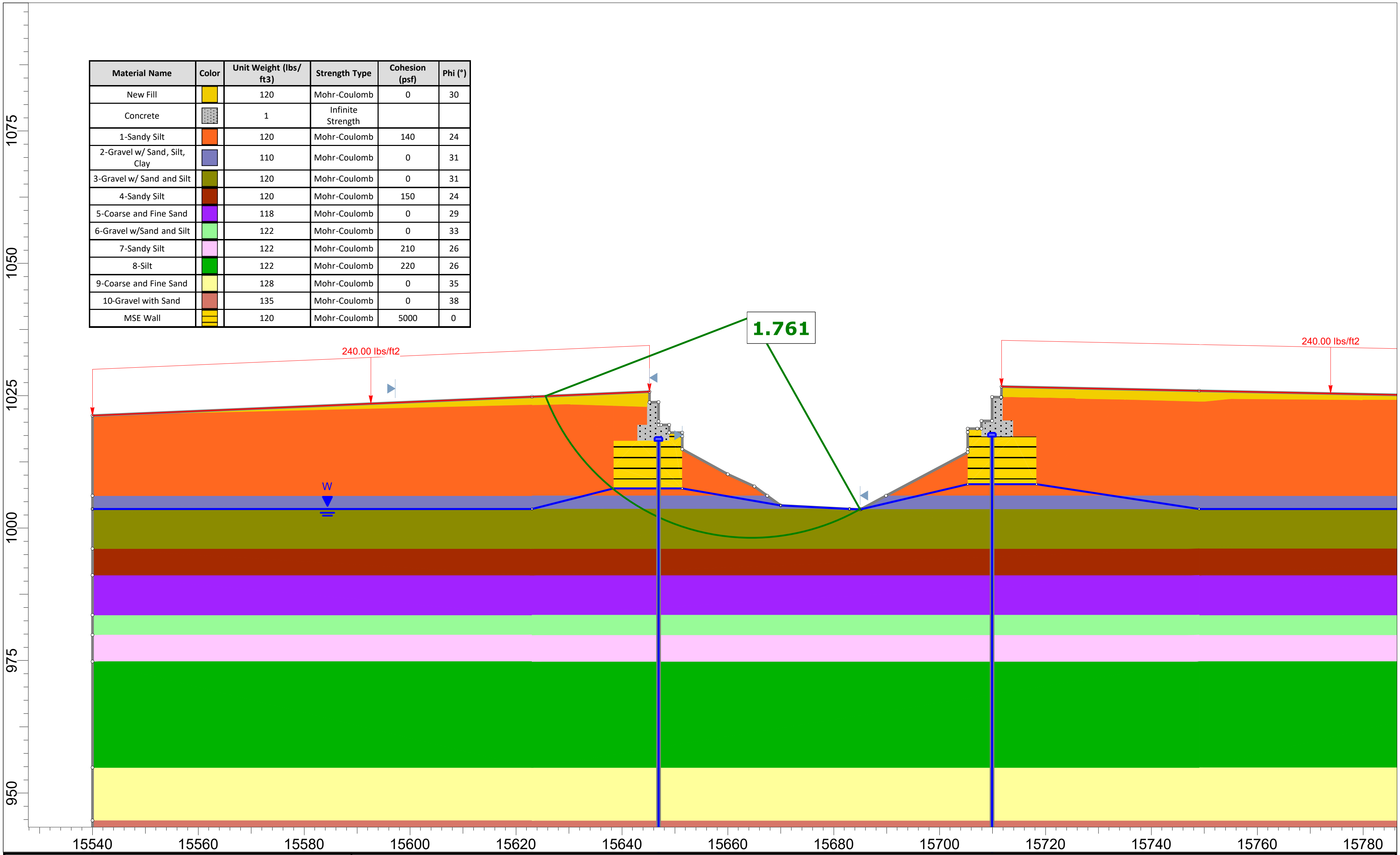
Sliding Capacity:Demand Ratio (CDR)

$CDR_{Sliding} := \frac{R_R}{F_\tau}$ $\qquad$ Is the CDR > or = to 1.0? $\qquad$ $CDR_{Sliding} = 1.11$

APPENDIX E

GLOBAL STABILTY ANALYSIS

RETAINING WALL 1

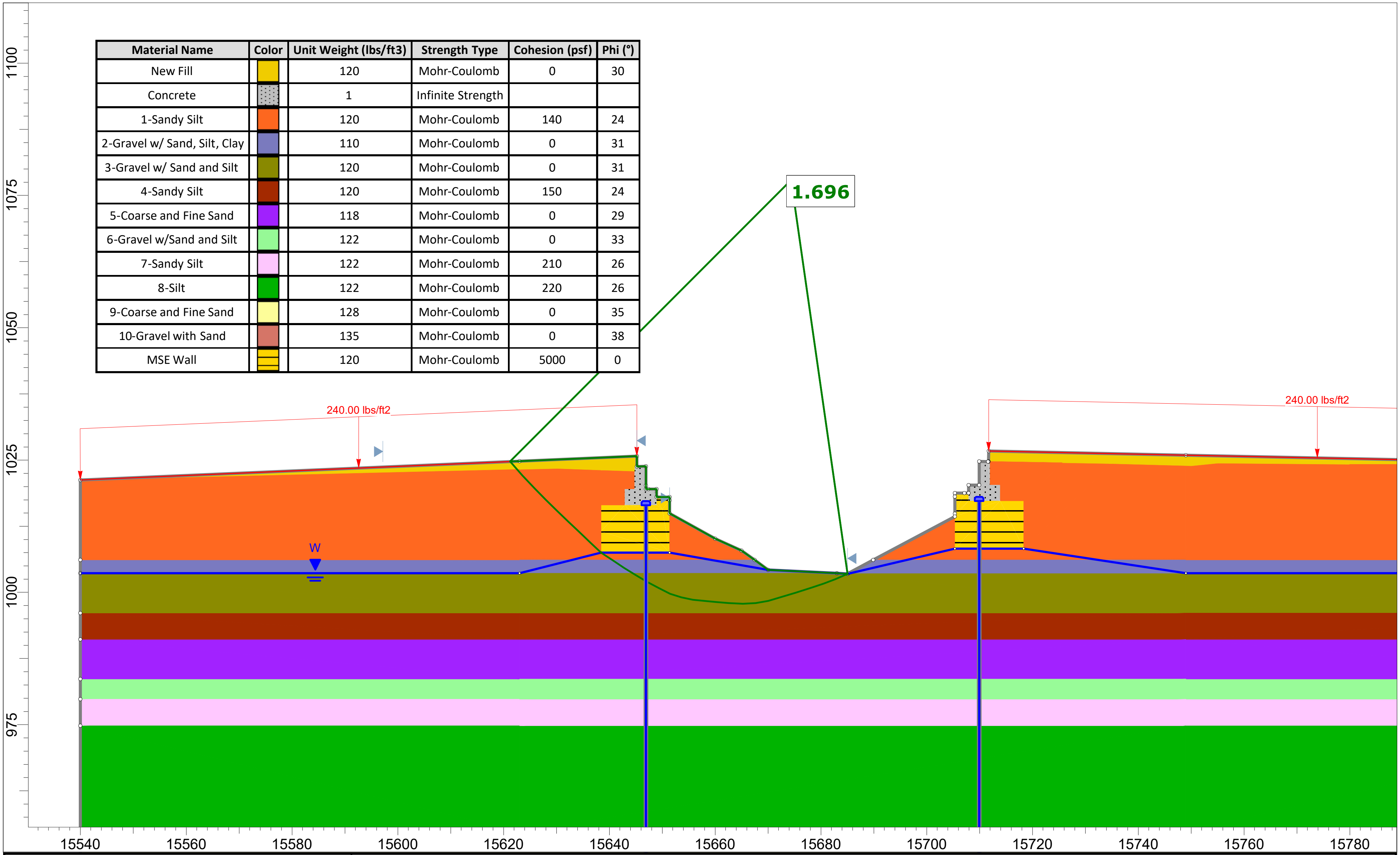


Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Fill	🟨	120	Mohr-Coulomb	0	30
Concrete	▨	1	Infinite Strength		
1-Sandy Silt	🟧	120	Mohr-Coulomb	140	24
2-Gravel w/ Sand, Silt, Clay	🟦	110	Mohr-Coulomb	0	31
3-Gravel w/ Sand and Silt	🟫	120	Mohr-Coulomb	0	31
4-Sandy Silt	🟥	120	Mohr-Coulomb	150	24
5-Coarse and Fine Sand	🟪	118	Mohr-Coulomb	0	29
6-Gravel w/Sand and Silt	🟩	122	Mohr-Coulomb	0	33
7-Sandy Silt	🟪	122	Mohr-Coulomb	210	26
8-Silt	🟩	122	Mohr-Coulomb	220	26
9-Coarse and Fine Sand	🟨	128	Mohr-Coulomb	0	35
10-Gravel with Sand	🟥	135	Mohr-Coulomb	0	38
MSE Wall	▤	120	Mohr-Coulomb	5000	0

ROCSCIENCE

SLIDEINTERPRET 9.040

Project	HUR-CR 11-2.96 - BASELINE ROAD/BROADWAY ST		
Group	Effective - Circular	*Scenario*	Master Scenario
Drawn By	BPA	*Company*	NEAS
Date	1/14/2026, 4:18:09 PM	*File Name*	RA_RW_011526_pile.slmd



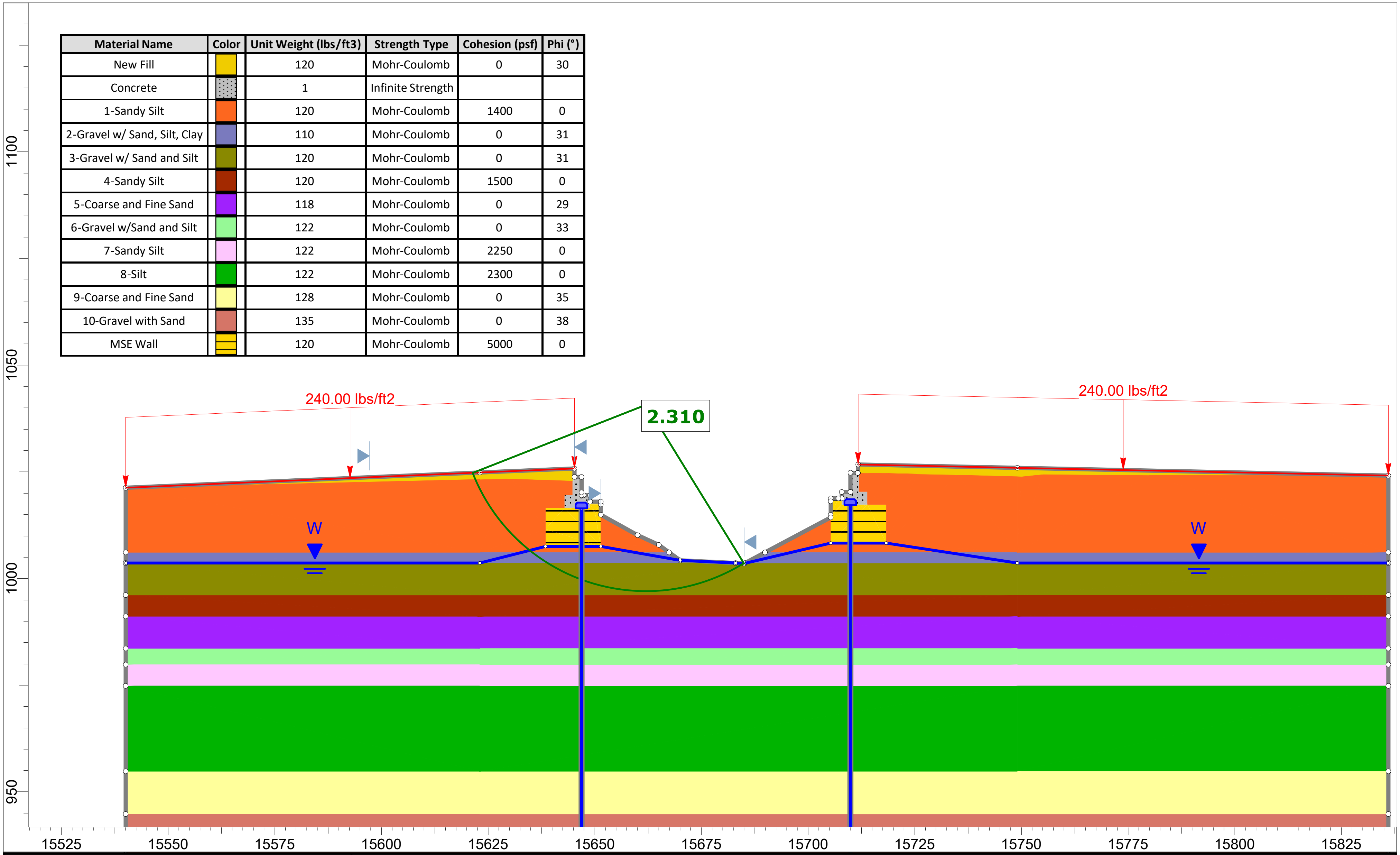
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Fill	🟨	120	Mohr-Coulomb	0	30
Concrete	▨	1	Infinite Strength		
1-Sandy Silt	🟧	120	Mohr-Coulomb	140	24
2-Gravel w/ Sand, Silt, Clay	🟦	110	Mohr-Coulomb	0	31
3-Gravel w/ Sand and Silt	🟫	120	Mohr-Coulomb	0	31
4-Sandy Silt	🟥	120	Mohr-Coulomb	150	24
5-Coarse and Fine Sand	🟪	118	Mohr-Coulomb	0	29
6-Gravel w/Sand and Silt	🟩	122	Mohr-Coulomb	0	33
7-Sandy Silt	⬜	122	Mohr-Coulomb	210	26
8-Silt	🟩	122	Mohr-Coulomb	220	26
9-Coarse and Fine Sand	🟨	128	Mohr-Coulomb	0	35
10-Gravel with Sand	🟥	135	Mohr-Coulomb	0	38
MSE Wall	▤	120	Mohr-Coulomb	5000	0

ROCSCIENCE

SLIDEINTERPRET 9.040

Project	HUR-CR 11-2.96 - BASELINE ROAD/BROADWAY ST		
Group	Effective - Translational	*Scenario*	Master Scenario
Drawn By	BPA	*Company*	NEAS
Date	1/14/2026, 4:18:09 PM	*File Name*	RA_RW_011526_pile_E-B.slmd

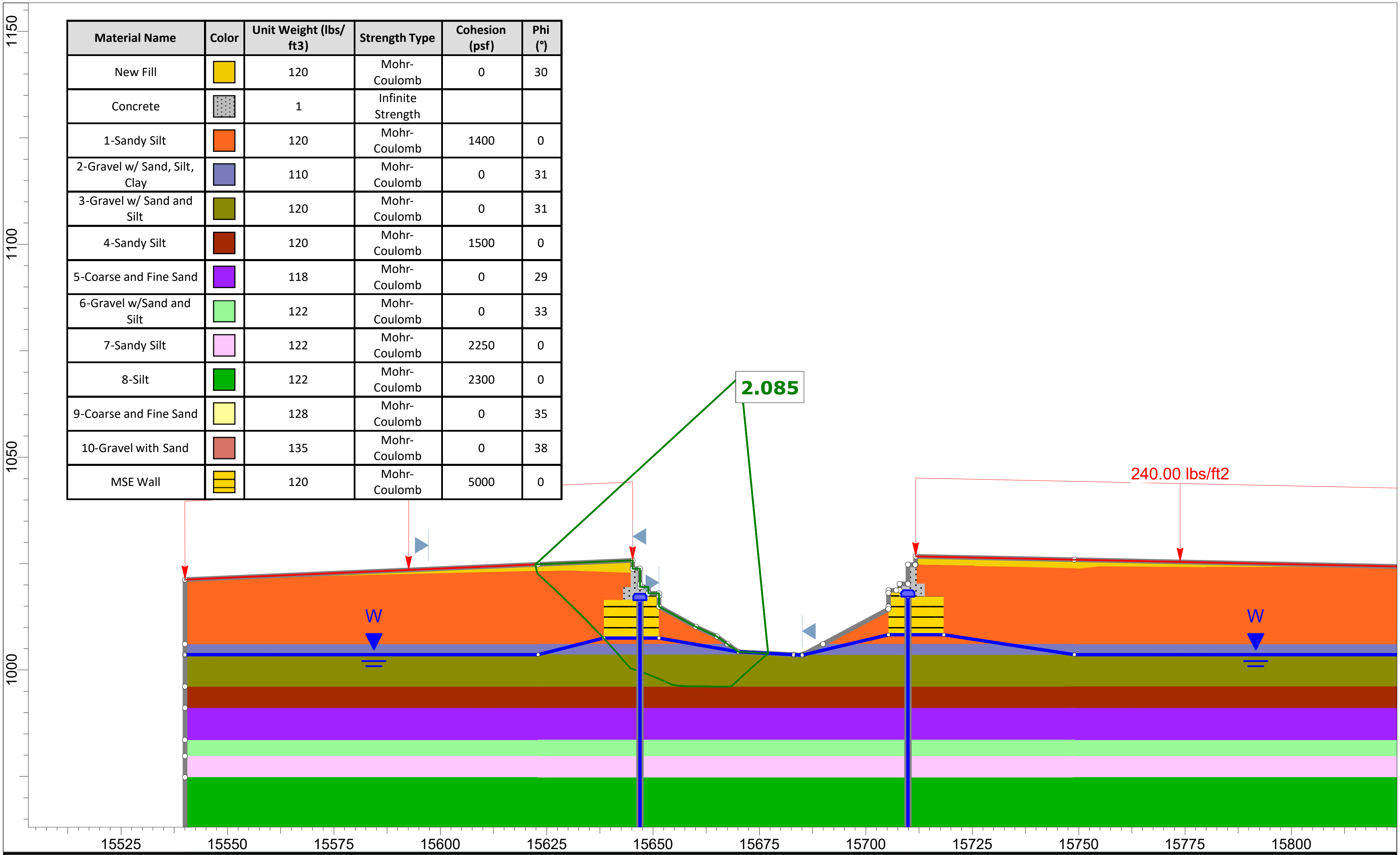
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Fill	🟨	120	Mohr-Coulomb	0	30
Concrete	▨	1	Infinite Strength		
1-Sandy Silt	🟧	120	Mohr-Coulomb	1400	0
2-Gravel w/ Sand, Silt, Clay	🟦	110	Mohr-Coulomb	0	31
3-Gravel w/ Sand and Silt	🟩	120	Mohr-Coulomb	0	31
4-Sandy Silt	🟫	120	Mohr-Coulomb	1500	0
5-Coarse and Fine Sand	🟪	118	Mohr-Coulomb	0	29
6-Gravel w/Sand and Silt	🟩	122	Mohr-Coulomb	0	33
7-Sandy Silt	🟪	122	Mohr-Coulomb	2250	0
8-Silt	🟩	122	Mohr-Coulomb	2300	0
9-Coarse and Fine Sand	🟨	128	Mohr-Coulomb	0	35
10-Gravel with Sand	🟥	135	Mohr-Coulomb	0	38
MSE Wall	▤	120	Mohr-Coulomb	5000	0



ROCSCIENCE

SLIDEINTERPRET 9.040

Project	HUR-CR 11-2.96 - BASELINE ROAD/BROADWAY ST		
Group	Total - Circular	*Scenario*	Master Scenario
Drawn By	BPA	*Company*	NEAS
Date	1/14/2026, 4:18:09 PM	*File Name*	RA_RW_011526_pile_T-C.slmd



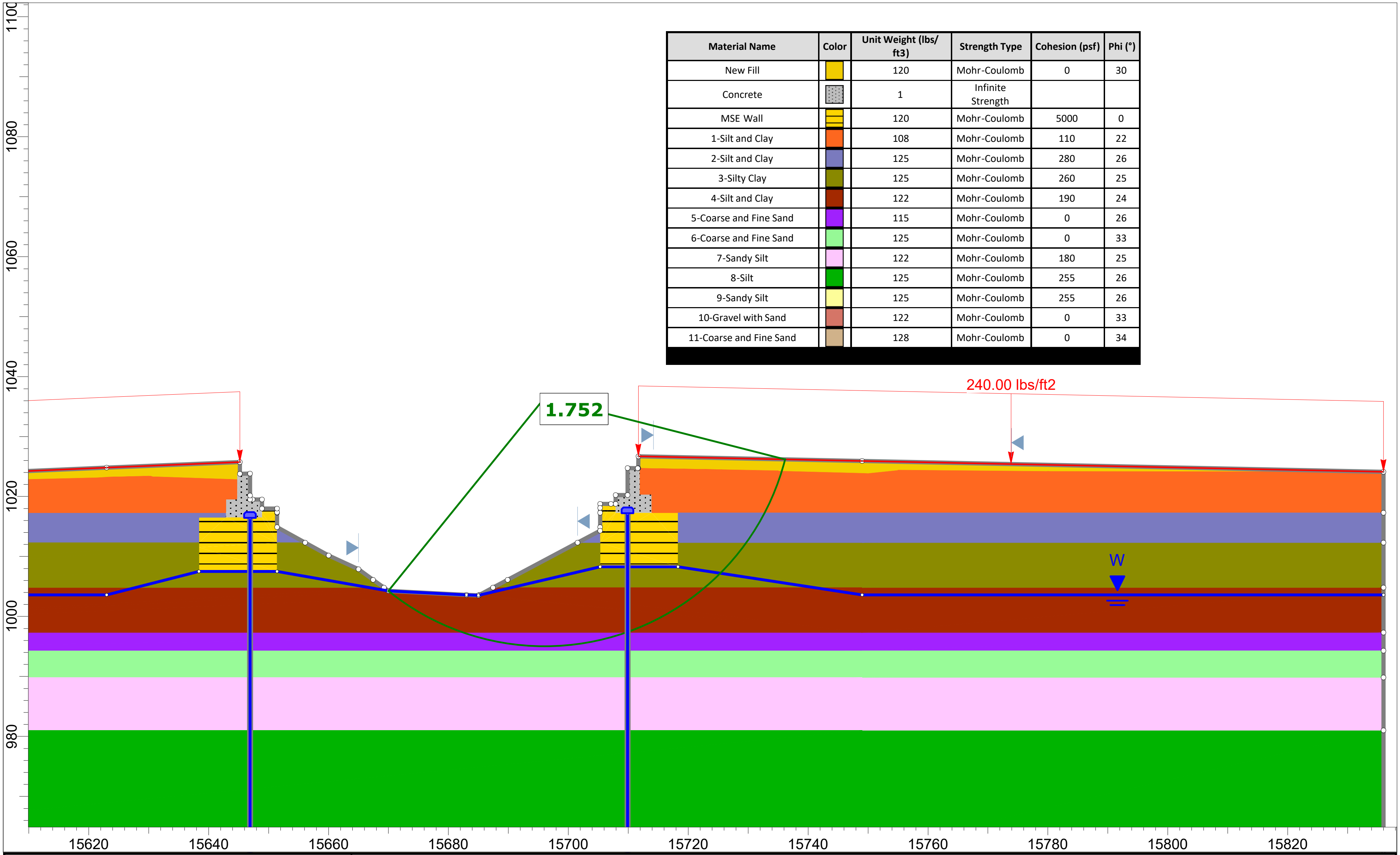
Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Fill	🟨	120	Mohr-Coulomb	0	30
Concrete	▨	1	Infinite Strength		
1-Sandy Silt	🟧	120	Mohr-Coulomb	1400	0
2-Gravel w/ Sand, Silt, Clay	🟦	110	Mohr-Coulomb	0	31
3-Gravel w/ Sand and Silt	🟫	120	Mohr-Coulomb	0	31
4-Sandy Silt	🟥	120	Mohr-Coulomb	1500	0
5-Coarse and Fine Sand	🟪	118	Mohr-Coulomb	0	29
6-Gravel w/Sand and Silt	🟩	122	Mohr-Coulomb	0	33
7-Sandy Silt	🟪	122	Mohr-Coulomb	2250	0
8-Silt	🟩	122	Mohr-Coulomb	2300	0
9-Coarse and Fine Sand	🟨	128	Mohr-Coulomb	0	35
10-Gravel with Sand	🟥	135	Mohr-Coulomb	0	38
MSE Wall	▤	120	Mohr-Coulomb	5000	0

ROCSCIENCE

SLIDEINTERPRET 9.040

Project	HUR-CR 11-2.96 - BASELINE ROAD/BROADWAY ST		
Group	Total - Translational	*Scenario*	Master Scenario
Drawn By	BPA	*Company*	NEAS
Date	1/14/2026, 4:18:09 PM	*File Name*	RA_RW_011526_pile_T-B.slmd

RETAINING WALL 2

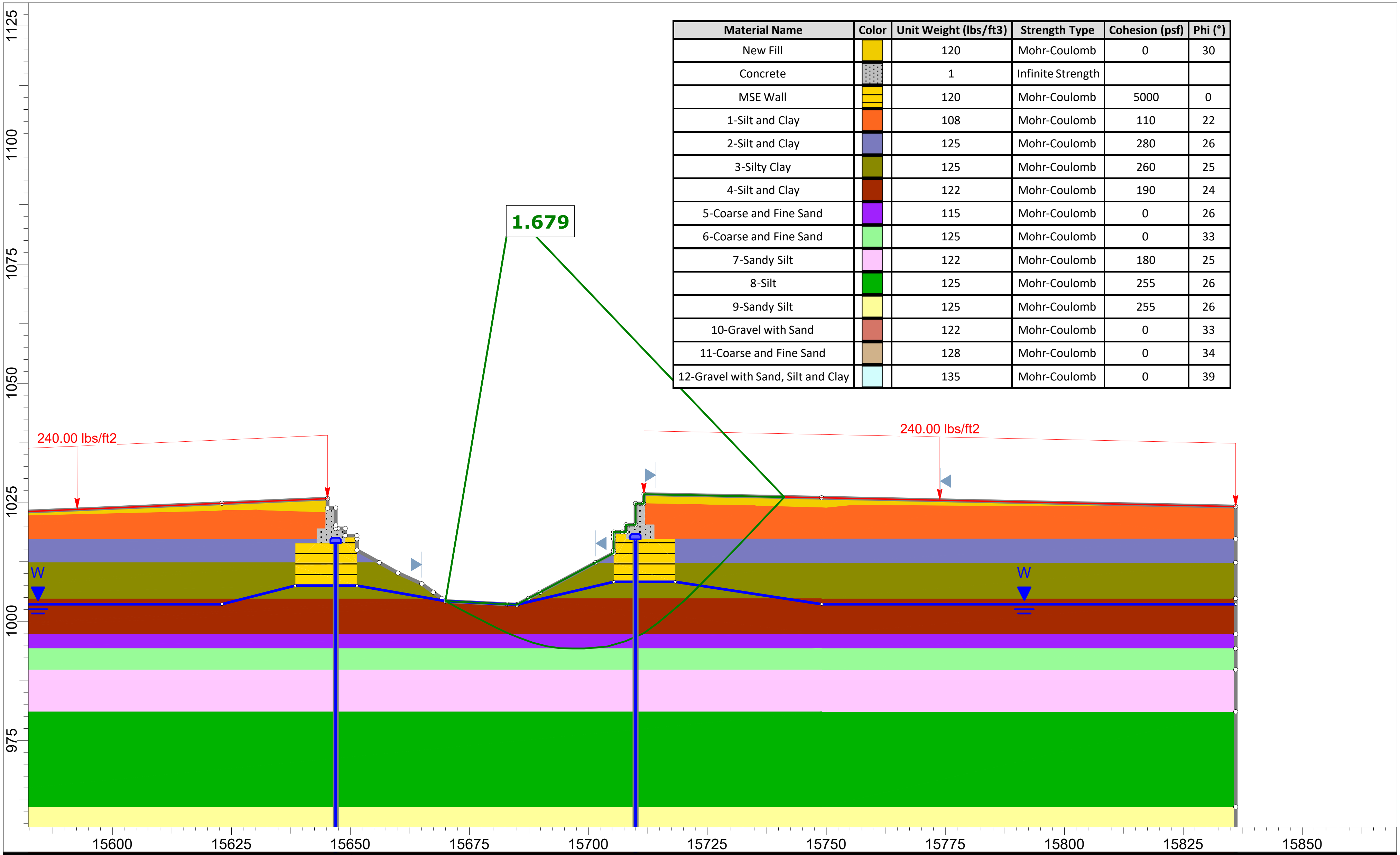


Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Fill	🟨	120	Mohr-Coulomb	0	30
Concrete	▨	1	Infinite Strength		
MSE Wall	☰	120	Mohr-Coulomb	5000	0
1-Silt and Clay	🟧	108	Mohr-Coulomb	110	22
2-Silt and Clay	🟦	125	Mohr-Coulomb	280	26
3-Silty Clay	🟫	125	Mohr-Coulomb	260	25
4-Silt and Clay	🟥	122	Mohr-Coulomb	190	24
5-Coarse and Fine Sand	🟪	115	Mohr-Coulomb	0	26
6-Coarse and Fine Sand	🟩	125	Mohr-Coulomb	0	33
7-Sandy Silt	🟪	122	Mohr-Coulomb	180	25
8-Silt	🟩	125	Mohr-Coulomb	255	26
9-Sandy Silt	🟨	125	Mohr-Coulomb	255	26
10-Gravel with Sand	🟥	122	Mohr-Coulomb	0	33
11-Coarse and Fine Sand	🟫	128	Mohr-Coulomb	0	34

ROCSCIENCE

SLIDEINTERPRET 9.041

Project	HUR-CR 11-2.96 - BASELINE ROAD/BROADWAY ST		
Group	Effective - Circular	*Scenario*	Master Scenario
Drawn By	BPA	*Company*	NEAS
Date	1/14/2026, 4:18:09 PM	*File Name*	FA_RW_011526_pile_E-C.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Fill	🟨	120	Mohr-Coulomb	0	30
Concrete	▨	1	Infinite Strength		
MSE Wall	▤	120	Mohr-Coulomb	5000	0
1-Silt and Clay	🟧	108	Mohr-Coulomb	110	22
2-Silt and Clay	🟦	125	Mohr-Coulomb	280	26
3-Silty Clay	🟫	125	Mohr-Coulomb	260	25
4-Silt and Clay	🟥	122	Mohr-Coulomb	190	24
5-Coarse and Fine Sand	🟪	115	Mohr-Coulomb	0	26
6-Coarse and Fine Sand	🟩	125	Mohr-Coulomb	0	33
7-Sandy Silt	■	122	Mohr-Coulomb	180	25
8-Silt	🟩	125	Mohr-Coulomb	255	26
9-Sandy Silt	🟨	125	Mohr-Coulomb	255	26
10-Gravel with Sand	🟥	122	Mohr-Coulomb	0	33
11-Coarse and Fine Sand	🟫	128	Mohr-Coulomb	0	34
12-Gravel with Sand, Silt and Clay	■	135	Mohr-Coulomb	0	39

ROCSCIENCE

SLIDEINTERPRET 9.041

Project HUR-CR 11-2.96 - BASELINE ROAD/BROADWAY ST

Group Effective - Translational

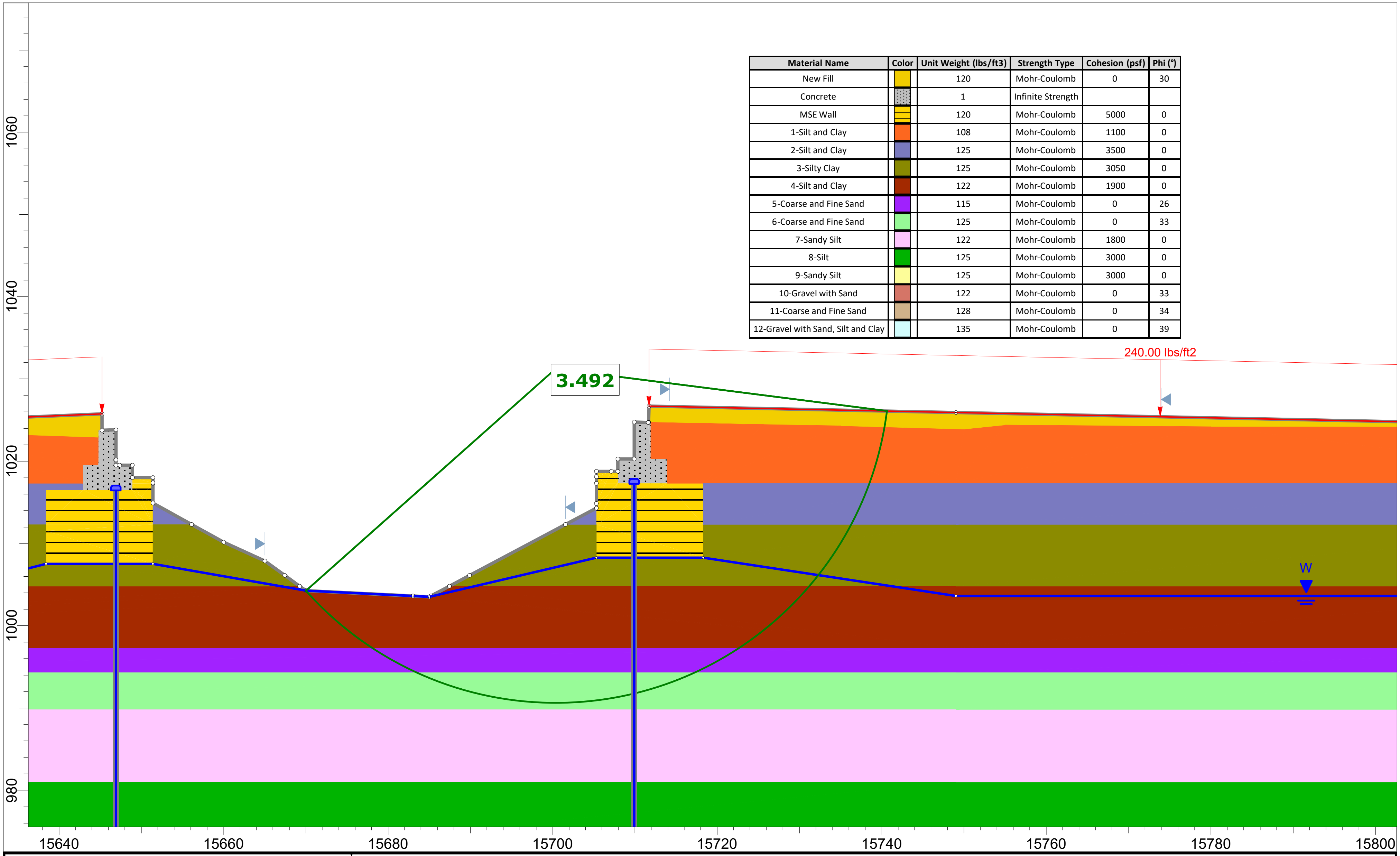
Scenario Master Scenario

Drawn By BPA

Company NEAS

Date 1/14/2026, 4:18:09 PM

File Name FA_RW_011526_pile_E-T.slmd

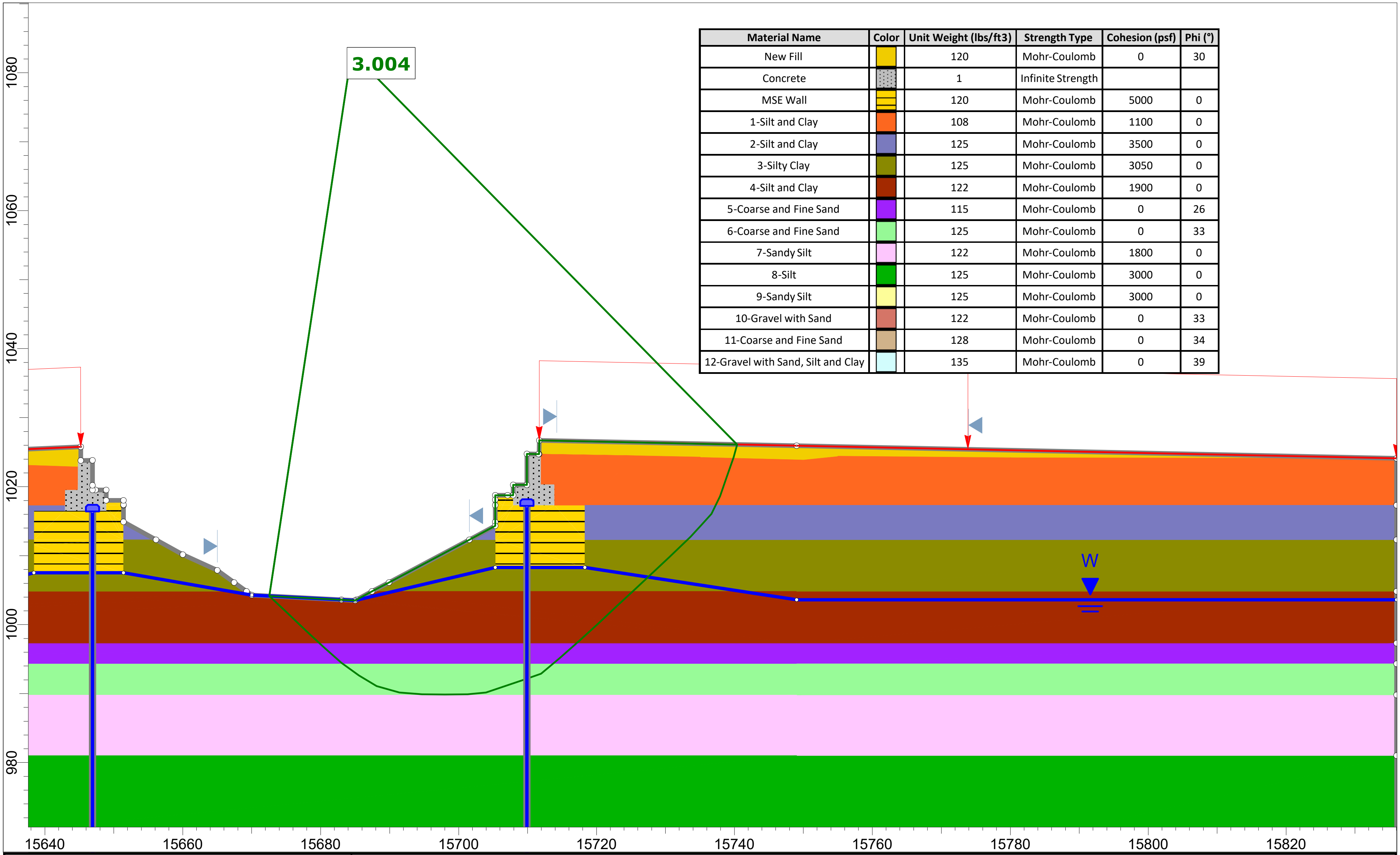


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Fill	🟨	120	Mohr-Coulomb	0	30
Concrete	▨	1	Infinite Strength		
MSE Wall	▤	120	Mohr-Coulomb	5000	0
1-Silt and Clay	🟧	108	Mohr-Coulomb	1100	0
2-Silt and Clay	🟦	125	Mohr-Coulomb	3500	0
3-Silty Clay	🟫	125	Mohr-Coulomb	3050	0
4-Silt and Clay	🟥	122	Mohr-Coulomb	1900	0
5-Coarse and Fine Sand	🟪	115	Mohr-Coulomb	0	26
6-Coarse and Fine Sand	🟩	125	Mohr-Coulomb	0	33
7-Sandy Silt	■	122	Mohr-Coulomb	1800	0
8-Silt	🟩	125	Mohr-Coulomb	3000	0
9-Sandy Silt	🟨	125	Mohr-Coulomb	3000	0
10-Gravel with Sand	■	122	Mohr-Coulomb	0	33
11-Coarse and Fine Sand	■	128	Mohr-Coulomb	0	34
12-Gravel with Sand, Silt and Clay	■	135	Mohr-Coulomb	0	39

ROCSCIENCE

Project	HUR-CR 11-2.96 - BASELINE ROAD/BROADWAY ST		
Group	Total - Circular	*Scenario*	Master Scenario
Drawn By	BPA	*Company*	NEAS
Date	1/14/2026, 4:18:09 PM	*File Name*	FA_RW_011526_pile_T-C.slmd

SLIDEINTERPRET 9.041



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Fill	🟨	120	Mohr-Coulomb	0	30
Concrete	▨	1	Infinite Strength		
MSE Wall	▤	120	Mohr-Coulomb	5000	0
1-Silt and Clay	🟧	108	Mohr-Coulomb	1100	0
2-Silt and Clay	🟪	125	Mohr-Coulomb	3500	0
3-Silty Clay	🟫	125	Mohr-Coulomb	3050	0
4-Silt and Clay	🟥	122	Mohr-Coulomb	1900	0
5-Coarse and Fine Sand	🟪	115	Mohr-Coulomb	0	26
6-Coarse and Fine Sand	🟩	125	Mohr-Coulomb	0	33
7-Sandy Silt	🟪	122	Mohr-Coulomb	1800	0
8-Silt	🟩	125	Mohr-Coulomb	3000	0
9-Sandy Silt	🟨	125	Mohr-Coulomb	3000	0
10-Gravel with Sand	🟥	122	Mohr-Coulomb	0	33
11-Coarse and Fine Sand	🟫	128	Mohr-Coulomb	0	34
12-Gravel with Sand, Silt and Clay	🟦	135	Mohr-Coulomb	0	39

ROCSCIENCE

SLIDEINTERPRET 9.041

Project	HUR-CR 11-2.96 - BASELINE ROAD/BROADWAY ST
Group	Total - Translational
Scenario	Master Scenario
Drawn By	BPA
Company	NEAS
Date	1/14/2026, 4:18:09 PM
File Name	FA_RW_011526_pile_T-T.slmd