Structure Foundation Exploration Report

PREPARED FOR

City of Conneaut

294 Main Street, Conneaut, OH 4403

ISSUED: February 2026

8150 Sterling Court| Mentor| OH | 44060 | 440-951-900 | www.ctconsultants.com

February 27, 2026 CT Project No. 232245

City of Conneaut
294 Main Street
Conneaut, Ohio 44030

Re: Structure Foundation Exploration Report
Proposed ATB Old Main Street Bridge Replacement
Conneaut, Ohio

Dear City of Conneaut Representative:

Following is the report of our Geotechnical Subsurface Exploration performed by CT Consultants, Inc. (CT) for the referenced project. This study was performed in accordance with Proposal No. P220609, dated June 9, 2023, and was authorized with a Subconsultant Services Agreement, dated February 16, 2024.

This report contains the results of our studies, our engineering interpretation of the results with respect to the project characteristics, and our recommendations for design and construction of pavements and bridge foundations.

A draft report was submitted to city of Conneaut and ODOT in June 2024, for review and comment. Comments were received and are incorporated herein. Additional comments from ODOT regarding a previous submittal provided January 7, 2026 are also addressed in this "FINAL" report. Should you have any questions regarding this report or require additional information, please contact our office.

Respectfully,

CT Consultants, Inc.

Imad El Hajjar, PE
Project Manager

Curtis E. Roupe, P.E.
AVP/ Group Leader

H:\2023\232245\PHASE\02 Geotechnical Engineering Services\Reports and Other Deliverables\Report Rev_4\232245 Geotech Report ATB Old Main Bridge PID 119470.docx

STRUCTURE FOUNDATION EXPLORATION REPORT

PROPOSED ATB OLD MAIN STREET BRIDGE REPLACEMENT
CONNEAUT, OHIO

FOR

CITY OF CONNEAUT
294 MAIN STREET
CONNEAUT, OHIO 44030

SUBMITTED

FEBRUARY 27, 2026
CT PROJECT NO. 232245

CT CONSULTANTS, INC.
8150 STERLING COURT
MENTOR, OHIO 44060
(440) 951-9000

CT consultants
engineers • architects • planners

TABLE OF CONTENTS

Page No.

PLATES

FIGURES

APPENDICES

1.0 INTRODUCTION

This report has been prepared for the proposed bridge replacement of the existing Old Main Street Bridge over the West Branch Conneaut Creek in Conneaut, Ohio. The project site is shown on the Site Location Map (Plate 1.0).

This study was performed in accordance with Proposal No. P220609, dated June 9, 2023, and was authorized with a Subconsultant Services Agreement, dated February 16, 2024.

1.1 Purpose and Scope of Exploration

The purpose of this exploration was to evaluate the subsurface conditions at the site. To accomplish this, CT performed four (4) test borings, field and laboratory soil testing and review of available geologic and soils data for the project area. The information provided in this data report will be incorporated into the final geotechnical exploration report which would be prepared in subsequent stages on the project.

This report summarizes our understanding of the proposed construction, describes the investigative and testing procedures utilized to evaluate the subsurface conditions at the site, and presents our findings from the field and laboratory testing. This report also presents our evaluations and conclusions in accordance with ODOT GDM Section 600 “Subgrade” (January 2024)and provides our design and construction recommendations for pavements.

This report includes:

- A description of the existing surface materials, subsurface soils, and groundwater conditions encountered in the borings.
- Design recommendations for bridge foundations, associated shaft, walls, and pavements.
- Recommendations concerning soil and groundwater-related construction procedures such as subgrade preparation, earthwork, pavement and foundation construction, and related field testing.

Pertinent Geotechnical Engineering Design Checklists are attached in Appendix D.

1.2 Proposed Construction

The project includes the proposed bridge replacement of the existing Old Main Street Bridge over the West Branch Conneaut Creek in Conneaut, Ohio.

The proposed bridge will be a two-span, 160-foot-long composite prestressed box beam bridge supported on new abutments and 1 pier. The proposed abutments will be located out of the stream flow of Conneaut Creek and portions of the existing abutment will remain in front of the new abutments as scour countermeasures. It will also consist of a 10-foot-wide shared use path for pedestrian traffic across the bridge.

It is our understanding that the west abutment will be supported on drilled shafts whereas the new pier and east abutment will be supported on shallow foundations.

Information regarding traffic loads was not provided at the time of this report. New pavements are anticipated to consist of flexible (asphalt) and/or rigid (concrete) sections for roadways.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 General Geology and Hydrogeology

Physiographic Region

The project site at 132 Old Main St, Conneaut, OH, is situated within the Glaciated Allegheny Plateau, a sub-region of the larger Appalachian Plateau. This physiographic province is characterized by a landscape of rolling hills, dissected plateaus, and broad river valleys, which were heavily influenced by Pleistocene glaciation. The glaciation left behind a varied topography with significant deposits of glacial till and outwash. The area's topography and geomorphology are critical factors in bridge construction, particularly for structures with in-water piers, where understanding soil and bedrock conditions is essential for foundation design and stability.

Quaternary Deposits

The Quaternary deposits at the site primarily consist of glacial till, outwash, and lacustrine sediments. Glacial till, an unsorted mixture of clay, silt, sand, gravel, and boulders, is prevalent and typically exhibits low permeability and varying degrees of consolidation. Outwash deposits, composed of stratified sands and gravels, are found in areas influenced by glacial meltwater and are more permeable, often forming the primary aquifers. Lacustrine sediments, including fine-grained silts and clays, were deposited in glacial lakes and are typically found in low-lying areas.

NRCS Soil Survey

The USDA Natural Resource Conservation Service (NRCS) Web Soil Survey for the project area identifies the predominant upper-profile soil as Otego silt loam. These soils are derived from alluvium formed on floodplains and are considered moderately well-drained. The Otego silt loam has good drainage characteristics, making it suitable for various types of construction. However, its alluvial nature means it can be susceptible to changes in moisture content and may exhibit variable bearing capacity.

Aquifers

Aquifers in the Conneaut area are found within both the unconsolidated glacial deposits and the underlying bedrock formations. Unconfined aquifers in the outwash sands and gravels provide significant groundwater storage and transmission capacity, recharged by precipitation and surface water infiltration. Confined aquifers within the glacial till and lacustrine sediments may also be present, with groundwater flow controlled by the permeability and continuity of these deposits.

Bedrock

The bedrock underlying the site is primarily composed of Devonian and Mississippian-age sedimentary formations, including sandstones, shales, and siltstones. These formations were deposited in ancient marine and fluvial environments, resulting in varied lithologies with different degrees of consolidation and fracturing. The depth to bedrock in this area can vary significantly due to the glacial and post-glacial topography, but it is typically encountered at relatively shallow depths beneath the Quaternary deposits.

Based on ODNR mapping, no mining or probable karst is indicated for the project site.

2.2 Site Reconnaissance

CT performed site reconnaissance on October 11, 2023. The eastern part of the site consist of predominantly commercial and residential properties while the western side is undeveloped and consist of mature woods. There is a creek below the bridge.

In the immediate area of the bridge, the pavement along Old Main Street was observed to generally be in poor condition and heavily distressed. The pavements were observed to have transverse, longitudinal, and fatigue cracks that were generally not sealed. Alligator cracks were observed in multiple areas along the roadway area. Several Potholes were also observed along the edge of the bridge with smaller potholes along the entirety of the bridge. At the ends of the bridge there appears to be longitudinal cracks as well as depressions in the concrete The foundation structures of the bridge appear to be slightly weathered, but overall in good condition.

The existing bridge deck appeared to be surfaced with asphalt with the ends being concrete. The bridge is approximately 180 feet long and 20 feet wide. There is also a concrete sidewalk on the north side of the bridge along the entirety of the bridge. A metal medium between the concrete ends and the asphalt of the bridge appears to be slightly rusted as well.

Within the site area, the agricultural area surrounding the bridge slopes down to the creek on both sides of the bridge. The creek below the bridge was approximately 15 to 20 feet below the road surface. A railroad track also runs north-south just west of the bridge.

Overhead utility lines were observed but were just north of the bridge.

3.0 EXPLORATION

3.1 Historic Borings

Based on our research, historic boring information was not available for the alignment of Old Main Street within the vicinity of either project location.

3.2 Project Exploration Program

This exploration included four test borings, designated as Borings B-001-0-23 through B-004-0-23, performed from the period of October 17 through 19, 2023, by CT Consultants. The borings have been identified in accordance with ODOT protocol, but the "-0-23" portion of the nomenclature is generally omitted for discussion in this report. The borings were located in the field by CT in accordance with a proposed boring location plan. The approximate locations of the borings are shown on the Test Boring Location Plans (Plates 2.0).

Based on boring location dimensions from existing site features obtained by CT, Latitude, Longitude, and ground surface elevation were estimated using Google Earth. This data is shown on the Logs of Test Borings.

Borings B-001 and B-003 were performed as ODOT Type E1 borings and were extended to a depth of 6 feet below existing pavements as an ODOT type A Boring. Boring B-002 was performed as an ODOT Type E1 boring to a depth of 6 feet below the creek bottom for scour analysis. Boring B-004 was performed as an ODOT Type A roadway boring for subgrade evaluation.

Experience indicates that the actual subsoil conditions at a site could vary from those generalized on the basis of test borings made at specific locations. Therefore, it is essential that a geotechnical engineer be retained to provide soil engineering services during the site preparation and pavement construction phases of the proposed project. This is to observe compliance with the design concepts, specifications, and recommendations, and to allow design changes in the event subsurface conditions differ from those anticipated prior to the start of construction.

3.3 Boring Methods

The test borings performed during this exploration were drilled using a truck-mounted rig with hollow-stem augers. All borings were continuously sampled for 6 feet using 18-inch split-spoon sample drives. Borings B-001 and B-003 were then sampled every 2½ feet until auger refusal. Boring B-2 was conducted from the bridge through a corehole in the bridge deck, with only one sample recovered from the creek bed before auger refusal. The samples were sealed in jars and transported to our laboratory for classification and testing.

Split-spoon soil samples were obtained by the Standard Penetration Test Method (ASTM D 1586). The Standard Penetration Test (SPT) consists of driving a 2-inch outside diameter split-spoon sampler into the soil with a 140-pound weight falling freely through a distance of 30 inches. The sampler was driven in three successive 6-inch increments, with the number of blows per increment being recorded. The number of blows per increment was recorded at each depth interval, and these data are presented under the "SPT" column on the Logs of Test Borings attached to this report. The sum of the number of blows required to advance the sampler the second and third 6-inch increments is termed the Standard Penetration Resistance, or Nm-value, and is typically reported in blows per foot (bpf). The Nm-values were corrected to an equivalent rod energy ratio of 60 percent, N60. The hammer/rod energy ratio for the CME 75 Truck 844 mounted drilling rig was 72.9 percent and was last calibrated on February 20, 2023. The N60-values are presented on the attached Logs of Test Borings.

One (1) Shelby tube sample, designated ST on the Log of Test Boring, was obtained in Boring B-003 from 13 to 15 feet below existing grades. The Shelby tube sample was obtained by hydraulically advancing a 3-inch diameter, thin-walled sampler approximately 24 inches beyond the hollow-stem auger into relatively undisturbed soil in accordance with ASTM D 1587. The Shelby tube was then extracted from the subsoils, and the ends were capped and sealed. The sample was transported to our laboratory where it was extruded, classified, and tested.

Upon encountering auger refusal in Borings B-001, B-2 and B-003, two 5-foot rock core runs were completed using an NQ2 diamond-bit core barrel and coring techniques in general accordance with ASTM D 2113. Recovery of the core is expressed as the percentage ratio of the recovered rock length to the total length of the core run. The Rock Quality Designation (RQD) is the percentage ratio of the summed length of rock pieces 4 inches in length and greater to the total length of the run or rock unit thickness. The RQD is expressed for each bedrock unit to provide clarity of the overall quality of rock within the unit descriptions. The rock core samples are designated as “NQ2-1” and “NQ2-2” on the Log of Test Boring attached to this report.

Soil conditions encountered in the test borings are presented in the Logs of Test Borings, along with information related to sample data, SPT results, water conditions observed in the borings, and laboratory test data. In conjunction with published data and typical correlations, the N_{60}-values can be evaluated as a measure of soil compactness/consistency as well as shear strength.

Field and laboratory data were incorporated into gINT™ software for presentation purposes. It should be noted that these logs have been prepared on the basis of laboratory classification and testing as well as field logs of the encountered soils.

3.4 Laboratory Testing Program

All samples were visually or manually classified in accordance with the ODOT Soil Classification System. All samples of the subsoils were also tested in our laboratory for moisture content (ASTM D 2216). Dry density determinations and unconfined compressive strength tests by the constant rate of strain method (ASTM D 2166) were performed on selected samples, including the Shelby tube sample. Unconfined compressive strength estimates were obtained for the remaining intact cohesive samples using a calibrated hand penetrometer. `

Laboratory testing was performed in accordance with ODOT GDM SECTION 600 “Subgrade” criteria, including mechanical soil classification consisting of an Atterberg limits test (ASTM D 4318) and a particle size analysis (ASTM D 6913 and D 7928) for two samples from each boring within 6 feet of the proposed subgrade. These test results are presented on the Logs of Test Borings and Grain Size Distribution sheets.

These test results are presented on the Logs of Test Borings.

4.0 FINDINGS

4.1 General Site Conditions

The project site is predominantly located along the Old Main Street bridge in Conneaut, Ohio. Roadway Grades in the project area ranged from Elevs. 591± to 595± in Borings B-001, B-003, and B-004 and 573± in Boring B-002.

All the borings were performed in existing pavements. The encountered surface materials consisted of asphalt ranging in thickness from approximately 1 to 8½ inches in Borings B-002, B-003, and B-004. Aggregate base material was present below the asphalt in Borings B-003 and B-004 and at the ground surface in Boring B-001 ranging in thickness from 4 to 15 ½ inches. The asphalt was underlain by concrete (bridge deck) in Boring B-002 that was approximately 31 inches thick.

The encountered pavement materials are summarized in the following table.

Table 4.1. Encountered Pavement and Subgrade Materials

Boring Number	Thickness (inches)		
	Asphalt	Aggregate Base	Concrete
B-001	-	4	-
B-002	1	-	31
B-003	8½	15½	-
B-004	4	8	-

In Borings B-001, B-003, and B-004 underlying the surface material, granular and cohesive fill material were encountered to depths of 5½ , 8½, and 4½ feet, respectively.

The granular fill material consisted of gravel and stone fragments with silt and sand (A-2-4) with varying amounts of clay. It should be noted that coal fragments were encountered in Boring B-4 within the granular fill layer. SPT N_{60}-values of 1 to 36 blows per foot (bpf), indicating very loose to dense compactness. Moisture contents ranged from 7 to 26 percent.

Isolated layers of Cohesive fill material were only encountered in Borings B-001 and B-004 with thickness on the order of 3 feet. The cohesive fill material consisted of sandy silt (A-4a) with varying amounts of coal fragments, gravel and clay. SPT N_{60}-values of 7 and 10 bpf were reported for the two samples recovered from the cohesive fill material.

4.2 General Soil and Rock Conditions

Based on the results of our field and laboratory tests, the subsoils encountered underlying the surface materials and existing fill materials can generally be characterized predominantly stiff to hard native cohesive soils underlain by native granular soil.

Stratum I consisted of predominantly stiff to hard cohesive soils encountered along the surface material in Borings B-001, B-003, and B-004 to depths of 8 feet, 16 feet and 7½ feet below existing grades, respectively. The Stratum I native cohesive soils consisted of silt and clay (A-6a) and sandy silt (A-4a) mixed with varying amounts of clay, gravel, and sand. SPT N_{60}-values ranged from 7 to 11 blows per foot (bpf). Unconfined compressive strength of 2,500 to 9,000 pounds per square foot (psf) (maximum reading obtainable using a hand penetrometer). Moisture contents ranged from 16 to 26 percent.

Stratum II consisted of predominantly medium dense native granular soils encountered underlying stratum I in Boring B-001 and B-003 to depths of 15½ feet and 18½ feet, respectively. The Stratum II granular soils consisted of fine sand (A-3), as well as coarse and fine sand (A-3a) mixed with varying amounts of sand, gravel, silt, and clay. A layer of decomposed bedrock sampled and classified as gravel and stone fragments (A-1-a) was encountered in Boring B-001. SPT N_{60}-values generally ranged from 16 to 17 blows per foot (bpf). Moisture contents ranged from 12 to 13 percent.

Shale bedrock was encountered underlying the native cohesive soils starting at approximately 15½ feet (Elev. 1038±), 23 feet (Elev. 570±) and 18½ feet (Elev. 573±) in Borings B-001, B-002 and B-003, respectively. Weathered rock that was able to be penetrated with the augers was encountered to depths of 24 (Elev. 569±) and 24½ feet (Elev. 567±) in Borings B-002 and B-003, respectively. This represents approximately 1 and 6 feet in Borings B-002 and B-003, respectively, of bedrock that was weathered and decomposed such that it was augerable. Within the weathered rock, the SPT generally resulted in split-spoon refusal (SSR, 50 or more blows for 6 inches or less penetration).

The depths of encountered weathered rock and auger refusal on more intact rock are summarized in the following table.

Table 4.2.A Summary of Encountered Rock Depths					
Boring Number	Ground Surface Elevation (feet)	Top of Weathered Rock Depth (feet)	Top of Weathered Rock Elevation (feet)	Top of Corable Rock Depth (feet)	Top of Corable Rock Elevation (feet)
B-001	592.7	15.7	577	15.7	577
B-002	570.1	0	570	24	569.2
B-003	591.7	18.7	573	24.5	567.2

Upon encountering auger refusal, the bedrock was cored for 10 feet in each boring, using 5-foot intervals. The recovered rock consisted of slightly to moderately weathered shale. Data for the cored bedrock is summarized in the following table.

Table 4.2.B Summary of Cored Rock						
Boring No.	Rock Core Run No.	Depth (feet)	Recovery (%)	RQD (%)	Slake Durability Index, I_{d2} (percent)	Comp. Strength (psi)
B-001	NQ2-8	15.7-20.7	80	0	52.5	2730, 2860
	NQ2-9	20.7-25.7	88	0	No Test	3350
B-002	NQ2-2	0.9-5.9	100	0	54.1	2280, 5630
	NQ2-3	5.9-10.9	90	0	No Test	5170
B-003	NQ2-12	24.5-29.5	95	0	70.1	7042, 2581
	NQ2-23	29.5-34.5	100	0	No Test	4994, 7473

Based on RQD values that were generally 0 percent, the rock mass quality in the cored bedrock profile can be generally characterized as very poor to poor. Based on compressive strength test results, the cored bedrock can be described as moderately strong to strong.

Additional descriptions of the stratigraphy encountered in the borings are presented on the Logs of Test Borings.

4.3 Groundwater Conditions

Groundwater was encountered during drilling in Borings B-001 and B-003 at depths of 8 feet and 20.3 feet, but not observed upon completion of drilling operations in any of the land borings. For Boring B-002, the top of water in the creek elevations was approximately 572 at the time of boring. It should be noted that the boreholes were drilled and backfilled within the same day, and stabilized water levels may not have occurred over this limited time period.

Apart from streamflow influences in the creek, it is our opinion that the "normal" groundwater level can generally be expected at depths corresponding to the bottom of the creek, on the order of 18 to 20 feet below roadway grades (Elev. 572±), or deeper (within bedrock). It should be noted that groundwater elevations can also fluctuate with seasonal and climatic influences, as well as streamflow conditions in the creek. Perched groundwater may be encountered within the pavement subbase, existing fill materials, or existing granular embankment materials that are underlain by relatively impermeable cohesive soils. Perched groundwater may also be encountered at the soil/bedrock interface. Therefore, the groundwater conditions may vary at different times of the year from those encountered during this exploration.

4.4 Scour Considerations

Scour considerations for the encountered subsurface conditions should be made as part of the vertical and lateral load evaluations for the drilled shafts and rock sockets. Pier scour in erodible rock was analyzed using the methodology outlined in HEC-18, Section 7.13. The calculated stream power did not exceed the critical stream power, therefore no significant pier scour would be expected at this location. Long term abrasion of the rock is evident and was observed to be approximately 0.71 ft over approximately 39 years, or 0.0182 ft/year. At this rate, it is expected that an additional 1.4 ft of abrasion will occur over the next 75 years. Abutment scour was analyzed using clear water methodology as outlined in HEC-18, Section 3.4 since there is no indication of movement of the bed material in the flow upstream of the crossing. Utilizing this methodology, negative scour is predicted, indicating that the abutments will not be subjected to scour. This information is based on a report prepared for the project by CT Consultants, Inc. titled ATB-MR-365-0.02 Bridge Scour Calculations, dated December 2025. Please refer to this document for the detailed calculations.

5.0 ANALYSES AND RECOMMENDATIONS

The following analyses and recommendations are based on our understanding of the proposed construction and upon the data obtained during our exploration. If the project information or location as outlined is incorrect or should change significantly, a review of these recommendations should be made by CT. Structural calculations are provided in Appendix B for reference.

5.1 West Abutment Foundation – Drilled Shafts

The Old Main Street Bridge replacement is designed to feature a two-span 160-foot composite prestressed box beam bridge supported on new abutments and one (1) pier. Due to the proximity of bedrock to the proposed foundation pier caps and the nearby train tracks, the west abutment of the new bridge is planned to be supported by a deep foundation system, specifically drilled shafts socketed into bedrock. The design of the bridge foundations will adhere to LRFD methods. The maximum total factored vertical loads for abutments are specified to be 307.8 kips.

The anticipated top of shaft elevations (i.e., bottom of abutment) are projected to be approximately 583.8 for the western abutment. For the abutments, it is planned to utilize 42-inches diameter shaft above bedrock and 36-inches diameter shaft in the socket. The diameter of bedrock sockets for drilled shafts are generally 6 inches less than the diameter of the shaft above the bedrock elevation. Regardless of shaft diameter, reinforcing steel cages should be based on the bedrock socket diameter.

5.1.1 Vertical Load Evaluations

The minimum prescribed rock socket length is 1.5 times the socket diameter. However, rock sockets should be increased to 5 feet in accordance with BDM 305.4.4.4, as the top of the rock was encountered within 10 feet of the bottom of the shaft cap. The proposed abutments are located out of the stream flow of the Conneaut Creek and the existing abutment stones will be removed and placed in front of the new abutment as scour countermeasures. Therefore, controlling scour elevation is not required for this site.

For detailed recommendations regarding rock socket lengths based on vertical resistance evaluations, please refer to the accompanying table.

Item	Boring B-001
	West Abutment
Recommended Minimum Rock Socket Length(1)	5 feet
Top of Rock Elevation (feet)	577
Bottom of Rock Socket Minimum Elevation (feet)	572

(1) Based on rock socket diameter of 36 inches as well as rock considerations discussed above.

Based on the rock conditions encountered in Boring B-001 for the Western Abutment, an unfactored unit tip resistance (q_p) of 1,073 kips per square foot (ksf) was calculated. Per LRFD guidance, this value was determined using an average of the compressive strength results at and within approximately 2 times the socket diameter below the end-bearing elevation. Based on the design methodologies utilized to evaluate unfactored unit tip resistance and AASHTO LRFD Table 10.5.5.2.4-1, a resistance factor of 0.50 should be utilized for design for tip resistance. **As such, the factored unit tip resistance was calculated to be 535 ksf.** Using the planned 36-inch diameter socket, this design value would provide sufficient resistance for the indicated factored vertical load.

It should be noted that the values for factored unit tip resistance listed above are based on bearing in competent rock that does not contain adverse jointing, open solution cavities, or joints that are filled with weathered material that would affect the bearing resistance of the rock, within a distance equal to two socket diameters below the tip of the drilled shaft rock socket. If such conditions are observed during socket installation at or in close proximity above the end-bearing elevation, it may be prudent to extend the sockets deeper.

The factored unit tip resistance evaluations presented above were based on rock conditions. We recommend the structural engineer also consider any limiting conditions associated with the stress limitations of the concrete.

It should be noted that the provided factored unit bearing resistance reflects end-bearing conditions only. Typically, design based on end-bearing alone is considered when sound bedrock underlies highly weathered rock. Conversely, design based on side shear resistance alone is considered when the drilled shaft cannot be adequately cleaned, or where large movement of the shaft would be required to mobilize the end bearing. For this project, significant movement is not expected to be required to mobilize the end bearing (for shafts installed beyond the less competent upper bedrock profile, into rock resulting at least in SSR), and it is assumed that due diligence will be exercised to install the shafts in a cleaned drill hole.

Drilled shafts should be constructed in accordance with ODOT Construction and Material Specifications (CMS) Item 524. It is also recommended that the center-to-center spacing between adjacent shafts be no less than 2 shaft diameters.

Due to the expected presence of groundwater at the soil/rock interface, as well as the encountered fill materials, it is likely that temporary steel casing will be required to support the walls of the shaft and to control groundwater seepage. If significant seepage is encountered and cannot be suitably pumped to dewater the drilled shaft, concrete will require placement by tremie methods. As the steel casing is withdrawn during concreting, sufficient concrete should be maintained above the bottom of the casing to counteract any hydrostatic head. Care must be taken during concreting and removal of any temporary liner so as to avoid the possibility of soil intrusions. The contractor should submit procedures for installation prior to the start of work.

Although cobbles or boulders were not noted in the borings performed for this exploration, they may be encountered at this site. Therefore, provisions should be made by the contractor to remove any obstructions, including debris, cobbles or boulders, if they are encountered during the drilling operations.

Drilled shafts should be clean and free of all loose material prior to the placement of concrete. A CT representative should verify that shafts are bearing on competent materials and that installation procedures meet specifications.

Based on ODOT guidelines, foundation plans should contain the following typical notes:

The maximum factored load to be supported by each drilled shaft is 308 kips at the abutments. This load is resisted entirely by tip resistance. At the West Abutment the factored tip resistance is 3,782 kips.

5.1.2 Lateral Load Evaluations

For lateral load-deflection evaluations using software, such as LPILE, recommended design parameters are summarized in the following tables based on the conditions encountered in the borings. Design values are provided based on Borings B-001 so evaluations can be made for the West Abutment.

Table 5.1.2.A. Subsurface Conditions and Recommended Lateral Load-Deflection Parameters – Boring B-001 [West Abutment]

Approx. Depth (feet)	Approx. Elevation (feet)	Generalized Layer Description	Approx. Total Unit Weight[1] (pcf)	Approx. Internal Angle of Friction (deg)	Average Undrained Shear Strength, S_u (psf)	Strain at 50% Maximum Stress, ε_{50}	Rock Mass Modulus, E_m (psi)	Rock Uniaxial Compressive Strength (psi)	k_{rm}
0.3 to 3	592.4 to 589.7	Medium stiff cohesive soils – Fill	118	-	875	0.007	–	–	–
3 to 5.5	589.7 to 587.4	Very loose to loose granular soil – Potential Fill	110	29	-	-	–	–	–
5.5 to 8	587.4 to 584.7	Stiff to very stiff cohesive soil – Native Soils	120	-	2,630	0.005	-	-	-
8 to 11.3	584.7 to 581.4	Medium dense granular soil – Native Soils	122	34	-	-	-	-	-
11.3 to 15.6	581.4 to 577.0	Gravel and Stone Fragments, Augerable Shale as Bedrock	140	43	–	–	-	-	-
15.6 to 25.7	577.0 to 567	Weak to slightly strong, weathered Shale as Bedrock RQD = 0%	150	-	-	-	1,400,000	2730 - 3350	0.000050

[1]Effective unit weight should be used below a depth of 20 feet (reduce by unit weight of water – 62.4 pcf).

A p–y analysis was performed in accordance with GDM Section 1501.7 using the parameters shown above. The vertical wall element was modeled from the proposed top of wall elevation to the estimated tip elevation, and fixity was achieved within the anticipated rock socket depth. The resulting head deflection of the vertical wall element was within the serviceability limit of 2 inches, satisfying the requirements of GDM Section 1501.6.

5.1.3 Scour Considerations

Scour considerations for the encountered subsurface conditions should be made as part of the vertical and lateral load evaluations for the drilled shafts and rock sockets. The proposed abutments are located out of the stream flow of the Conneaut Creek and the existing abutment stones will be removed and placed in front of the new abutment as scour countermeasures. For additional scour discussion, please refer to Section 4.4.

5.2 Pier Foundation - Shallow Foundations on Weathered Bedrock

For the pier support, it is planned that the bridge spread foundations be extended to bear on moderately weathered shale bedrock. Footing excavation should extend through highly weathered/fractured rock (particularly that which was augerable in the borings).

Based on the conditions encountered in Boring B-002, zero percent RQD and high recovery of 90 to 100 percent was determined for the rock extending to approximately Elev. 559.2. Furthermore, uniaxial compressive (UCS) strength of in the order of 2280 to 5630 psi was determined on the rock core samples collected at Elev. 569 to 565.

We understand that the bridge foundations will be designed using LRFD specifications. The following loads were provided. It was indicated that a foundation width of 9 feet was planned.

- Strength Limit State Maximum load: 10.5 ksf;
- Service Limit State Maximum load: 8.1 ksf

At the strength limit state, we recommend a nominal bearing resistance (q_n) of 95.3 ksf for foundations bearing on intact shale bedrock. At the strength limit state, the resistance factor (ϕ_b) is 0.45. Therefore, the factored bearing resistance (q_r) is 42.9 ksf. From a conventional allowable stress design comparison, this is roughly akin to calculating an ultimate bearing capacity and applying a factor of safety. This strength limit state bearing resistance is adequate based on the provided maximum strength limit state bearing pressure of 10.5 ksf.

Since the structure will be bearing on weathered bedrock, no adjustments to the bearing pressure are required at this structure location. The calculated unfactored bearing pressure of 95.3 ksf (0.30 ksi) is significantly lower than the estimated rock mass modulus of 57,120 ksf (605 ksi) , which satisfies the criterion outlined in GDM Section 1303.2.1—that bearing stress should be less than 50 times the rock mass modulus to assume negligible settlement. Supporting calculations for the rock mass modulus are provided in Appendix A.

Given that the foundation is resting on competent bedrock and the settlement is considered negligible, the service limit state bearing resistance is deemed adequate. We, therefore, anticipate that the **service limit state bearing resistance is adequate based on the provided maximum service limit state bearing pressure of 8.1 ksf.**

5.3 East Abutment Foundation - Shallow Foundations on Rock

For east abutment support, it is planned that the bridge spread foundations be extended to bear on augerable severely weathered shale bedrock. The augerable severely weathered bedrock is assumed to behave like cohesionless granular material. The bearing capacity, settlement, and overall stability of east abutment is computed based on this assumption.

We understand that the bridge foundations will be designed using LRFD specifications. The following loads were provided. It was indicated that a foundation width of 9 feet was planned.

- Strength Limit State Maximum load: 12.7 ksf;
- Service Limit State Maximum load: 8.9 ksf

At the strength limit state, we recommend a nominal bearing resistance (q_n) of 31.4 ksf for foundations bearing on augerable severely weathered bedrock. At the strength limit state, the resistance factor (ϕ_b) is 0.55. Therefore, the factored bearing resistance (q_r) is 17.3 ksf. From a conventional allowable stress design comparison, this is roughly akin to calculating an ultimate bearing capacity and applying a factor of safety. **This strength** limit state bearing resistance is adequate based on the provided maximum strength limit state bearing pressure of 12.7 ksf.

Since the structure will be bearing on weathered bedrock, no adjustments to the bearing pressure are required at this structure location. The calculated unfactored bearing pressure of 17.3 ksf is significantly lower than the estimated rock mass modulus of 7,920 ksf, which satisfies the criterion outlined in GDM Section 1303.2.1—that bearing stress should be less than 50 times the rock mass modulus to assume negligible settlement. Supporting calculations for the rock mass modulus are provided in Appendix A.

Given that the foundation is resting on competent bedrock and the settlement is considered negligible, the service limit state bearing resistance is deemed adequate. We, therefore, anticipate that the service limit state bearing resistance is adequate based on the provided maximum service limit state bearing pressure of 8.9 ksf.

5.3.1 East Abutment – Overall Stability

East abutment is checked against for potential overturning and sliding as per LRFD Section 10.6.3.5 and 10.6.3.4. In order to perform these, we assumed the east abutment as a semi-gravity cantilever wall.

Overturning stability was evaluated by comparing the calculated eccentricity of the wall geometry to the maximum eccentricity with the resultant force. It was assumed that the backfill will consist of cohesive soils with a minimum effective internal angle of friction (ϕ') of 30 degrees behind the stabilized earth section of fill. As such, a coefficient of active earth pressure, K_a, of 0.33 was used for the overturning analysis at the abutment sections. Based on the analysis the abutment were determined to be adequate with regard to eccentricity, as presented in attached calculations in Appendix A.

The LRFD factored sliding resistance (R_R) is determined by ϕR_n, where R_n is the nominal sliding resistance on the base of the wall, and ϕ is the resistance factor. For semi gravity cantilever walls, $\phi = 1.0$.

The abutment is anticipated to bear augerable weathered rock. We assumed the augerable weathered rock as cohesionless granular soils having an internal angel of friction of 30 degrees with no cohesion. The factored sliding resistance provided by the foundation base is 2,454 kips. Calculations are attached in Appendix A.

5.3.2 East Abutment Wingwall Foundation - Shallow Foundations on Soils

For the east abutment support, it is planned that the bridge spread foundations be extended to bear on the existing native soils or the underlying weathered bedrock.

Based on the conditions encountered in the borings, the soils at the anticipated foundation bearing elevation are expected to consist of:

- Stratum I – very stiff to hard native cohesive soils, or
- Stratum II - medium dense native granular soils, or
- Bedrock - Highly Weathered / Decomposed Shale

The native cohesive soils are considered generally suitable for support of the proposed abutment and will govern the bearing capacity design. However, with any installation within a creek area, there may be areas of encountered sediment at bearing elevations, which would require over-excavation. The bearing soils should be confirmed as being native cohesive soils with an unconfined compressive strength of at least 3,000 pounds per square foot (hand penetrometer reading of 1.5 or greater).

We understand that the abutment foundations will be designed using LRFD specifications. At the strength limit state, we recommend a nominal bearing resistance (q_n) of 8.5 ksf (undrained) and 25.4 ksf (drained) for the abutment base bearing on the native cohesive soils. As such, undrained conditions govern the design. At the strength limit state, the resistance factor (ϕ_b) is 0.55. Therefore, the factored bearing resistance (q_r) is 4.7 ksf (undrained) and 13.9 ksf (drained). From a conventional allowable stress design comparison, this is roughly akin to calculating an ultimate bearing capacity and applying a factor of safety.

Settlement of the abutment was calculated by conventional consolidation theory utilizing recompression indices for the over-consolidated soils, based on empirical relations using moisture content. Based on a bearing pressure of 4.7 ksf (undrained) and 13.9 ksf (drained), using the service limit state bearing resistance indicated above, total settlement was calculated to be on the order of ½ to ¾ inches.

Although not anticipated to be prevalent, if unsuitable bearing soils are encountered during culvert installation, over-excavation should extend through these materials to suitable bearing soils. The base of the over-excavation should be widened 6 inches for every foot of depth extending beyond the edge of the culvert. The over-excavated areas should be backfilled with lean concrete having a minimum compressive strength of 1,500 pounds per square inch (psi) or other flowable controlled-density fill having a minimum compressive strength of 300 psi. If foundations will be placed at the base of the over-excavation or the lean concrete fill option will be utilized, widening the footing over-excavation will not be required. If the controlled-density fill or aggregate fill option is utilized, the footing over-excavation shall be widened as discussed above.

5.3.3 Lateral Earth Pressures

Based on the conditions encountered in the borings performed for this investigation, the soils at the east (forward) abutment that will support the wingwall above the rock are predominately consist of either native cohesive or granular soils.

For wingwalls that are restrained at the top of the wall, lateral earth pressures should be assumed for “at-rest” conditions. It is anticipated that excavated on-site granular soils will comprise the majority of the backfill behind the existing abutment walls. For the encountered subsurface soils, an at-rest lateral earth pressure coefficient (K_o) of 0.48 should be used along with a total soil unit weight of 122 pounds per cubic foot (pcf) in determining the lateral pressure acting on the walls.

For the encountered subsurface soils, an active lateral earth pressure coefficient (K_a) of 0.32, and passive lateral earth pressure coefficient (K_p) of 3.12 should be used along with a total soil unit weight of 122 pcf in determining the lateral pressures acting on the walls.

Although unlikely, lateral loading due to hydrostatic pressures below the design groundwater depth should be included in design of below-grade walls. Depending on the design methodology, total lateral pressures would be the resultant of the hydrostatic pressures in combination with submerged (or “effective”) unit weights of the soil. An effective unit weight of 57 pcf should be used for lateral earth pressure design below the design groundwater depth.

It should be noted that the above K-parameters may be used for general design of subsurface structures, retaining walls, and possible excavation support systems associated with the project. However, certain types of braced excavations may account for method-specific earth pressure distributions, for which the above parameters should be reviewed and utilized in the proper context of the design method/system.

Lateral load due to hydrostatic pressures below the design groundwater depth should be included in design of below-grade walls. Additionally, the earth pressures indicated above are based on a level backfill condition behind the culvert wall. If there are areas beyond the horizontal roadway portion of the backfill area that include sloping backfill behind the top of the wall, surcharge loading or equivalent higher earth pressure coefficients should be evaluated, based on backfill material, backfill slope, and proximity to the wall. In general, 50 percent of the vertical surcharge load may be assumed for lateral loading in the design of the wall.

Backfill for the abutment should be placed concurrently on both sides to avoid unbalanced forces that could cause sliding. If this method of backfilling is not possible and one side will be backfilled prior than the other, sliding can be evaluated as presented below.

We recommend that passive pressure be considered negligible at the toe of the wall due to the potential for erosion and/or freeze-thaw behavior that would significantly reduce reliance on passive earth pressure. As such, the LRFD nominal sliding resistance (R_R) is determined by $\phi_T R_T$, where RT is the nominal sliding resistance on the base of the footing.

For cohesive soils, nominal sliding resistance R_T is the lesser of the following:

- The cohesion (c) of the clay, for which we recommend c be taken as 3,000 psf, or
- Although not anticipated to be the case, where footings are supported on at least 6 inches of compacted granular material, one-half the normal stress on the interface between the footing and soil.

For sliding resistance on clays, the resistance factor ϕ_T should be taken as 0.85.

5.4 Subgrades and Pavements

An evaluation of the subgrade soils was completed in general accordance with ODOT Geotechnical Design Manual Section 600. As part of this evaluation, the ODOT "Subgrade Analysis" worksheet (V14.76, 1102/0611/2422) was completed and is attached to this report.

Final pavement grades are assumed to approximate existing grades. Based on the existing pavement cross-sections encountered in the borings, the proposed subgrade is presumed to be 12 inches below the existing top of pavement grades (represented as a 1 feet cut in the ODOT "Subgrade Analysis" worksheet).

Based on the GDM, soils classified as ODOT A-4b, A-2-5, A-5, A-7-5, A-8a, A-8b, or rock have been designated as being problematic with respect to pavement subgrade support. None of these soil types were encountered at planned subgrade elevations in the borings performed for this exploration. However, unsuitable Uncontrolled Fill (UCF) consisting of coal fragments was encountered in B-004 at 1.5 to 3 feet below existing ground surface, having a thickness of 24 inches. It, therefore, is recommended to remove and replace it with granular engineered fill.

The type and thickness of subgrade modification is determined by the GDM criteria based on the average, low SPT N_{60}-value (N60L) of the subgrade soils in a particular portion of the project area, hand penetrometer value, soil type, and moisture content. Based on these criteria, subgrade modification is anticipated.

Where undercut and replacement is utilized, all fill should consist of ODOT Item 304 Aggregate Base or Item 703.16C, Granular Material Type B or Type C. It is recommended that geotextile fabric (referenced in ODOT Item 204, and specified as ODOT Item 712.09, Type D) be utilized on the subgrade at the bottom of the undercut zone. Although not anticipated to be required based on the conditions encountered in the borings and the proposed sections and grades, if particularly unstable subgrades are encountered during construction, or undercuts exceed approximately 18 inches, a geogrid could be used to reduce the total undercut and replacement of the unsuitable soils by 6 inches.

Due to the relatively small area for pavement replacement, sulfate content testing was not performed to evaluate potential concerns with global chemical stabilization. It was anticipated that undercutting and replacement with new granular engineered fill would be more economical for this project.

5.4.1 Flexible (Asphalt) and Rigid Pavement Design

In Boring B-004 underlying UCF and loose A-2-4 zones to a depth of 4½ feet below the existing ground surface. These soils may govern the overall subgrade support conditions. As such, we recommend that the selected replacement pavement section granular engineered fill. It should also be noted that the subgrades should be compacted to at least 100 percent of the maximum dry density as determined by ASTM D 698 (Standard Proctor).

All pavement design and paving operations should conform to ODOT specifications. The pavement and subgrade preparation procedures outlined in this report should result in reasonably workable and satisfactory pavement. It should be recognized, however, that all pavements need repairs or overlays over time as a result of progressive yielding under repeated loading for a prolonged period.

It is recommended that placement of aggregate base, and placement of asphalt be performed within as short a time period as possible. Exposure of the aggregate base to rain, snow, or freezing conditions may lead to deterioration of the subgrade and/or base materials due to excessive moisture conditions and to difficulties in achieving the required compaction.

For short projects where the pavement replacement is less than 300 feet, ODOT encourages to use Pavement Design Manual Appendix C. Based on the GDM Subgrade analysis, a design CBR value of 8 percent was determined for the project. It should be noted that the CBR determination by the subgrade analysis spreadsheet is based on the average Group Index of all the evaluated samples, which was 8

It should also be noted that the design CBR value is based on subgrades compacted to at least 100 percent of the maximum dry density as determined by ASTM D 698 (Standard Proctor).

All pavement design and paving operations should conform to ODOT specifications. The pavement and subgrade preparation procedures outlined in this report should result in reasonably workable and satisfactory pavement. It should be recognized, however, that all pavements need repairs or overlays over time as a result of progressive yielding under repeated loading for a prolonged period.

5.5 Construction

5.5.1 Sedimentation and Erosion Control

In planning the implementation of earthwork operations, special consideration should be given to provide measures to prevent or reduce soil erosion and the subsequent sedimentation into nearby waterways. These measures may include some or all of the following:

1. Scheduling of earthwork operations such that erodible areas are kept as small as possible and are exposed for the shortest possible time.
2. Using special grading practices, along with diversion or interceptor structures, to reduce the amount of run-off water from an erodible area.
3. Providing vegetative buffer zones, filter berms, or sedimentation basins to trap sediment from surface run-off water.

A specific and detailed soil erosion and sedimentation control program and permits may be required by local, state, or federal regulatory agencies.

5.5.2 Site Preparation

Prior to proceeding with construction operations, site preparation activities should include the removal of any structures or substructures which are not appropriated for spillway protection, as well as topsoil, root systems, and vegetation from all proposed structure areas.

Replacement pavement subgrade preparation recommendations are provided in Subgrade and Pavement Section.

5.5.3 Excavations and Slopes

The sides of temporary excavations for subsurface drainage pipe, utility installations, and other construction should be adequately sloped to provide stable sides and safe working conditions. Otherwise, the excavation must be properly braced against lateral movements. For the relatively shallow depth of excavation activity anticipated for this project, laid-back slopes are likely to be most feasible and economical. In any case, applicable Occupational Safety and Health Administration (OSHA) safety standards must be followed.

Based on the test borings, it is likely that excavations will encounter a range of soil conditions that include the following OSHA designations:

- OSHA Type A soils (cohesive soils with unconfined compressive strengths of 3,000 pounds per square foot (psf) or greater),
- OSHA Type B soils (cohesive soils with unconfined compressive strengths greater than 1,000 psf but less than 3,000 psf), and
- OSHA Type C soils (cohesive soils with unconfined compressive strengths of 1,000 psf or less, granular soils, weathered bedrock, and existing fill materials).

For temporary excavations in Type A, B, C soils, side slopes should be constructed no steeper than ¾ horizontal to 1 vertical (¾H:1V), 1H:1V, and 1½H:1V, respectively. For situations where an excavation encounters a lower strength soil underlying a higher strength soil, the slope of the entire excavation is governed by the lower strength soil. In all cases, flatter slopes may be required if lower strength soils or adverse seepage conditions are encountered during construction.

For permanent excavation slopes, we recommend that grades be no steeper than 3H:1V without a more extensive geotechnical evaluation of the proposed construction plans and site conditions.

Temporary shoring calculations are included in Appendix C.

5.5.4 Rock Excavation

For bridge foundation installation, augerable weathered/fractured rock should be excavated. Additionally, the encountered rock in Boring B-002 and B-003 indicates rock excavation beyond the depth of auger refusal will likely be required in some areas to encounter proper foundation bearing material.

As stated in Section 5.2, RQD values of zero percent and high recovery of 90 to 100 percent were determined for the rock extending to approximate Elev. 567 in Boring B-002 and B-003. As such, footings should be extended to the more suitable material that was encountered below this elevation in Boring B-003.

Based on test data from the rock cores, our evaluations indicate that the weathered/ fractured (augerable) shale bedrock and cored highly fractured to fractured bedrock may be rippable using conventional excavation equipment such as a backhoe or track excavator, with some assistance from pneumatic chippers, jackhammers, or hydraulic wedging equipment.

5.5.5 Construction Dewatering and Groundwater Control

Groundwater conditions encountered in the borings were summarized in Section 4.3. Apart from streamflow influences in the creek, it is our opinion that the "normal" groundwater level can generally be expected at depths corresponding to the bottom of the creek, on the order of 8 to 20 feet (Elevs. 584± to 571±). It should be noted that groundwater elevations can also fluctuate with seasonal and climatic influences, as well as streamflow conditions in the creek. Perched groundwater may be encountered within the pavement subbase, existing fill materials, or existing granular embankment materials that are underlain by relatively impermeable cohesive soils. Perched groundwater may also be encountered at the soil/bedrock interface.

It is our experience that adequate control of groundwater seepage or surface water run-off into shallow excavations should be achievable by minor dewatering systems, such as pumping from prepared sumps. As mentioned in Section 5.1, it is likely that temporary steel casing will be required to support the walls of the drilled shafts and to control groundwater seepage. In the event excessive seepage is encountered during construction, CT should be notified to evaluate whether other dewatering methods are required.

5.5.6 Fill

Material for engineered fill or backfill required to achieve design grades should meet ODOT Item 203 "Embankment Fill" placement and compaction requirements.

The upper profile on-site soils consist predominantly of cohesive soils. For these cohesive soils, a sheepsfoot roller should provide the most effective soil compaction. For new granular engineered fill or dense-graded aggregate pavement base materials, a vibratory smooth-drum roller would be required to provide effective compaction.

6.0 QUALIFICATION OF RECOMMENDATIONS

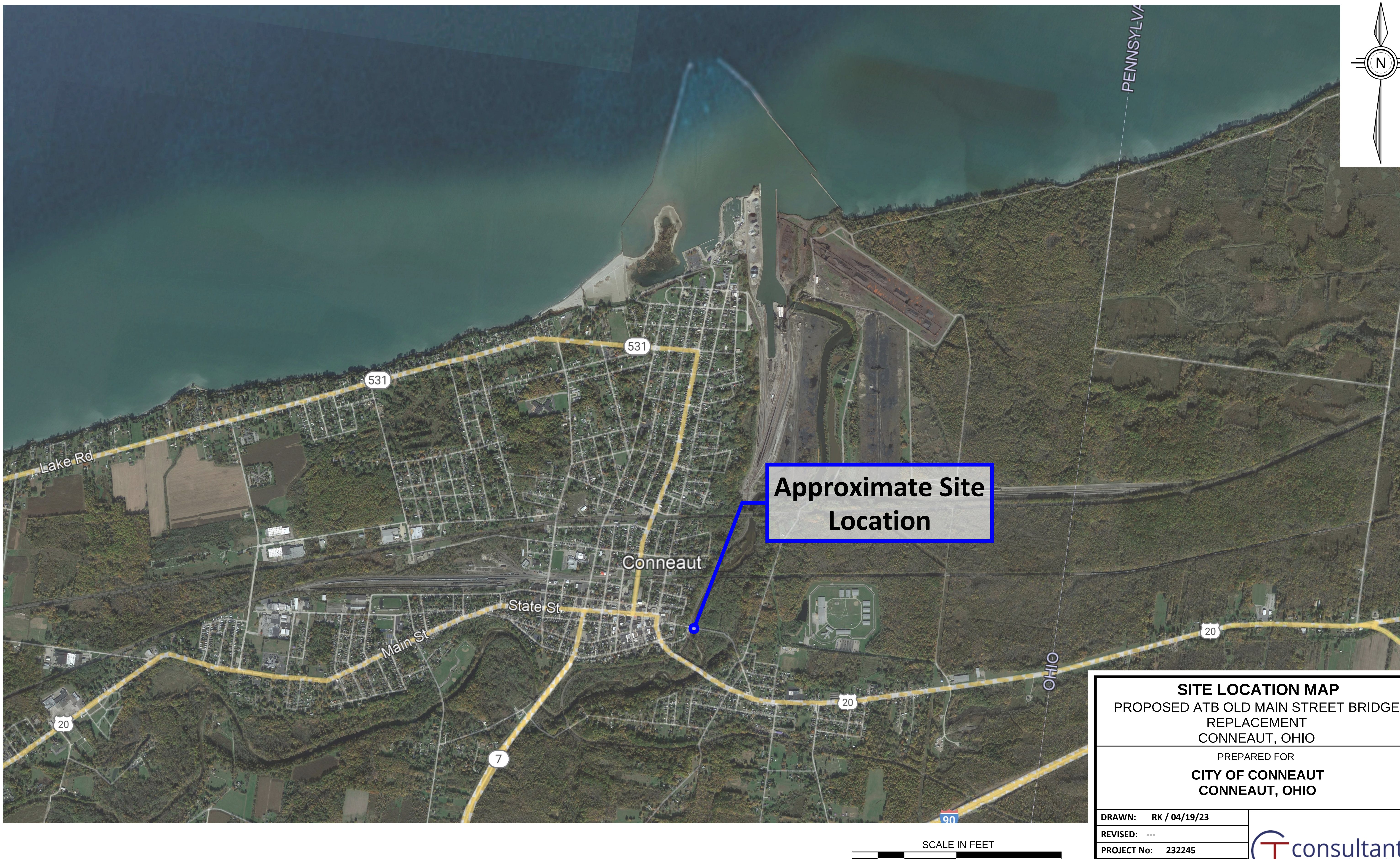
Our evaluation of design and construction conditions for the proposed bridge replacement and pavement reconstruction has been based on our understanding of the site and project information and the data obtained during our field exploration. The general subsurface conditions were based on interpretation of the data obtained at specific boring locations. Regardless of the thoroughness of a subsurface exploration, there is the possibility that conditions between borings will differ from those at the boring locations, that conditions are not as anticipated by the designers, or that the construction process has altered the soil conditions. This potential is increased for previously developed sites. Therefore, experienced geotechnical engineers should observe earthwork and foundation construction to confirm that the conditions anticipated in design are noted. Otherwise, CT assumes no responsibility for construction compliance with the design concepts, specifications, or recommendations.

The design recommendations in this report have been developed on the basis of the previously described project characteristics and subsurface conditions. If project criteria or locations change, a qualified geotechnical engineer should be permitted to determine whether the recommendations must be modified. The findings of such a review will be presented in a supplemental report.

The nature and extent of variations between the borings may not become evident until the course of construction. If such variations are encountered, it will be necessary to reevaluate the recommendations of this report after on-site observations of the conditions.

Our professional services have been performed, our findings derived, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied. CT is not responsible for the conclusions, opinions, or recommendations of others based on this data.

Plates



SITE LOCATION MAP
PROPOSED ATB OLD MAIN STREET BRIDGE REPLACEMENT
CONNEAUT, OHIO

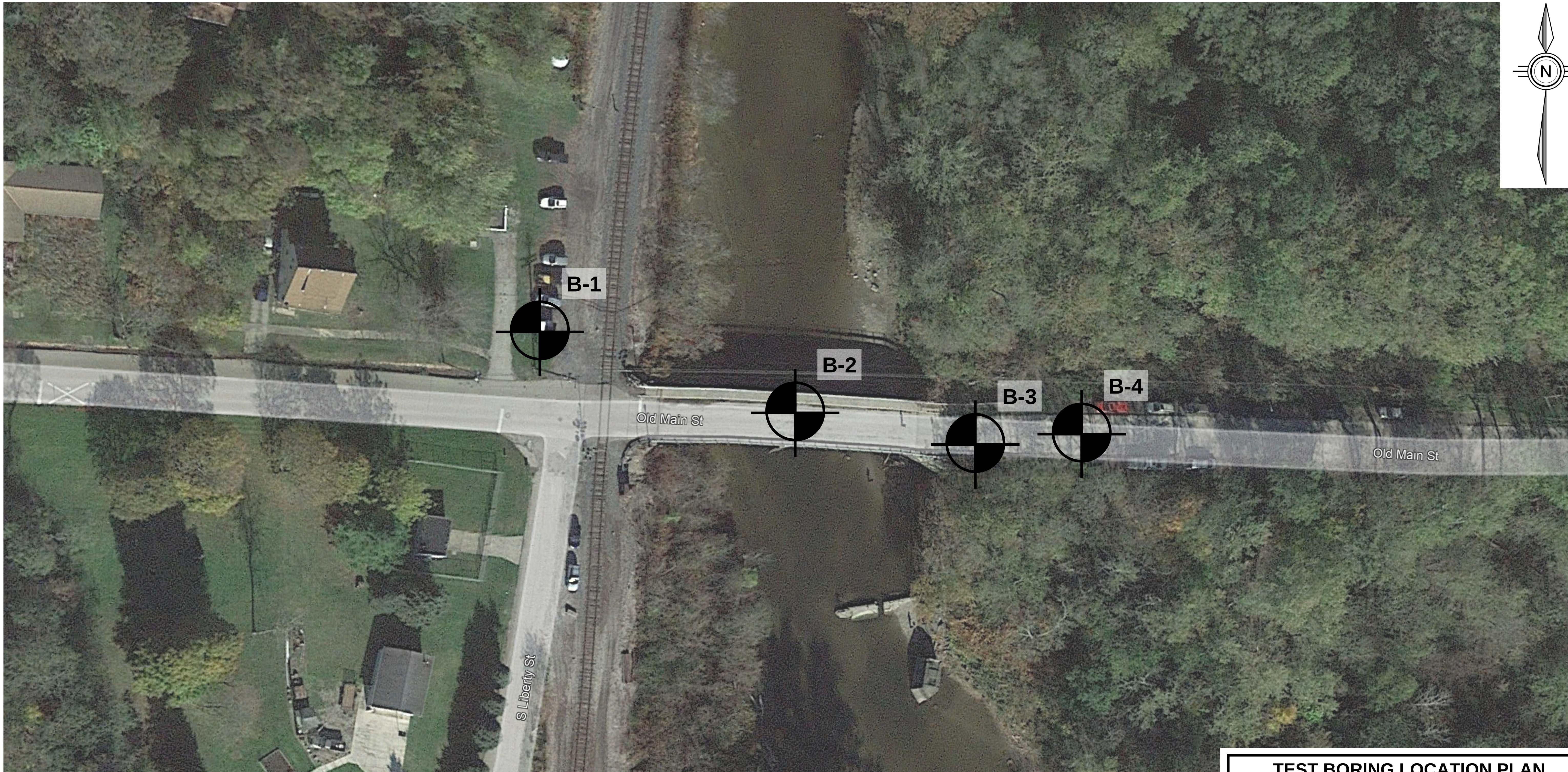
PREPARED FOR
CITY OF CONNEAUT
CONNEAUT, OHIO

DRAWN: RK / 04/19/23
REVISED: ---
PROJECT No: 232245

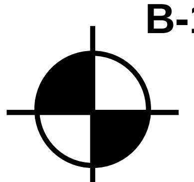
CT consultants engineers • architects • planners

PLATE 1.0

BASE PLAN "SITE AERIAL PLAN" DATED 11/6/2021 OBTAINED FROM GOOGLE EARTH.



LEGEND:



APPROXIMATE TEST BORING LOCATION

SCALE IN FEET

0' 75' 150'

BASE PLAN "SITE AERIAL PLAN" DATED 11/6/2021 OBTAINED FROM GOOGLE EARTH.

TEST BORING LOCATION PLAN

PROPOSED ATB OLD MAIN STREET BRIDGE REPLACEMENT

CONNEAUT, OHIO

PREPARED FOR

CITY OF CONNEAUT
CONNEAUT, OHIO

DRAWN:	RK / 04/19/23
REVISED:	---
PROJECT No:	232245

PLATE 2.0

CT consultants
engineers • architects • planners

Figures

Logs of Test Borings
Legend Key
Grain Size Distribution
Rock Core Photo Logs
Point Load Test Results
Slake Durability Test Results

PROJECT: OLD MAIN STREET BRIDGE	DRILLING FIRM / OPERATOR: CT CONSULTANTS / CW	DRILL RIG: CME 75 TRUCK 844	STATION / OFFSET: 10+32, 68' LT.
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: TTL / KKC	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. (OLD MAIN ST.)
PID: 119471 SFN: 0461254	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 2/20/23	ELEVATION: 592.7 (NAVD88) EOB: 25.7 ft.
START: 10/19/23 END: 10/19/23	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 72.9	LAT / LONG: 41.943784, -80.551162

EXPLORATION ID B-001-0-23

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 592.7	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
AGGREGATE BASE - 4 INCHES	592.4																	
MEDIUM STIFF, BROWN/BLACK, **SANDY SILT**, SOME CLAY, LITTLE COAL FRAGMENTS, LITTLE GRAVEL, WET FILL	589.7		4 3 3	7	100	SS-1	-	16	15	14	35	20	NP	NP	NP	26	A-4a (4)	
VERY LOOSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT**, TRACE CLAY, DAMP FILL @4.8': LOOSE, DARK BROWN	587.4		0 0 1	1	100	SS-2	-	38	14	15	29	4	NP	NP	NP	14	A-2-4 (0)	
			1 1 2	4	100	SS-3A	-	-	-	-	-	-	-	-	-	-	A-2-4 (V)	
STIFF, BLUE/GRAY/BROWN, **SILT AND CLAY**, LITTLE GRAVEL, LITTLE SAND, MOIST	586.7					SS-3B	1.25	-	-	-	-	-	-	-	-	20	A-6a (V)	
VERY STIFF TO HARD, BLUE/GRAY/BROWN, **SILT AND CLAY**, LITTLE SAND, DAMP Qu - 28.9 PSI	584.7	W 584.7	2 3 5	10	100	SS-4	4.00	0	3	7	22	68	32	21	11	19	A-6a (8)	
MEDIUM DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE CLAY, LITTLE SHALE FRAGMENTS, TRACE SILT, MOIST	581.4		4 7 7	17	100	SS-5	-	-	-	-	-	-	-	-	-	13	A-3a (V)	
DENSE TO VERY DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS**, MOIST TO DAMP **SHALE**, GRAY, SEVERELY WEATHERED, WEAK, HIGHLY FRACTURED. [INFERRED FROM DRILLING]	577.0	TR	10 27 50/1"	-	100	SS-6	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
			50/3"	-	100	SS-7	-	-	-	-	-	-	-	-	-	11	A-1-a (V)	
SHALE, GRAY, HIGHLY WEATHERED, WEAK TO SLIGHTLY STRONG, JOINTED, HIGHLY FRACTURED, OPEN TO NARROW; RQD 0%, REC 72%. @15.8': SDI, Id2 = 52.5 % @17.1': Qu - 2,730 PSI @18.3': Qu - 2,860 PSI	572.0		0		80	NQ2-8											CORE	
SHALE, GRAY, HIGHLY TO SLIGHTLY WEATHERED, WEAK TO SLIGHTLY STRONG, JOINTED, HIGHLY FRACTURED, OPEN TO NARROW; RQD 0%, REC 88%. @21.3': Qu - 3,350 PSI	567.0	EOB	0		88	NQ2-9											CORE	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 7 CF CEMENT-BENTONITE GROUT

PROJECT: OLD MAIN STREET BRIDGE TYPE: BRIDGE PID: 119471 SFN: 0461254 START: 10/17/23 END: 10/17/23	DRILLING FIRM / OPERATOR: CT CONSULTANTS / CW SAMPLING FIRM / LOGGER: TTL / KKC DRILLING METHOD: 3.25" HSA / NQ2 SAMPLING METHOD: SPT / NQ2	DRILL RIG: CME 75 TRUCK 844 HAMMER: CME AUTOMATIC CALIBRATION DATE: 2/20/23 ENERGY RATIO (%): 72.9	STATION / OFFSET: 11+71, 6' RT. ALIGNMENT: CL CONST. (OLD MAIN ST.) ELEVATION: 570.1 (NAVD88) EOB: 10.9 ft. LAT / LONG: 41.943617, -80.550660	EXPLORATION ID B-002-0-23 PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 570.1	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
SHALE, GRAY, SEVERELY WEATHERED, WEAK, HIGHLY FRACTURED.	569.2	TR 570.1 – 0.9	50/5"	-	100	SS-1	-	-	-	-	-	-	-	-	-	9	Rock (V)	
SHALE, GRAY, MODERATELY TO HIGHLY WEATHERED, SLIGHTLY TO MODERATELY STRONG, JOINTED, HIGHLY FRACTURED, NARROW; RQD 0%, REC 100%. @0.9': SDI, Id2 = 54.1 % @2.6': Qu - 2,280 PSI @5.5': Qu - 5,630 PSI	564.2	0.9 – 5.9	0		100	NQ2-2											CORE	
SHALE, GRAY, MODERATELY TO HIGHLY WEATHERED, SLIGHTLY TO MODERATELY STRONG, JOINTED, HIGHLY FRACTURED, NARROW; RQD 0%, REC 90%. @6.2': Qu - 5,170 PSI	559.2	5.9 – 10.9 EOB	0		90	NQ2-3											CORE	

NOTES: THE BORING IS DRILLED THROUGH THE BRIDGE DECK. BRIDGE DECK ELEVATION WAS 593.2 FEET.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 1 BAG ASPHALT PATCH; PUMPED 3 CF CEMENT-BENTONITE GROUT

PROJECT: OLD MAIN STREET BRIDGE	DRILLING FIRM / OPERATOR: CT CONSULTANTS / CW	DRILL RIG: CME 75 TRUCK 844	STATION / OFFSET: 12+67, 9' LT.
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: TTL / KKC	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. (OLD MAIN ST.)
PID: 119471 SFN: 0461254	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 2/20/23	ELEVATION: 591.7 (NAVD88) EOB: 34.5 ft.
START: 10/18/23 END: 10/18/23	SAMPLING METHOD: SPT / ST / NQ2	ENERGY RATIO (%): 72.9	LAT / LONG: 41.943638, -80.550312

EXPLORATION ID B-003-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 591.7	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
ASPHALT - 8.5 INCHES	591.0																	
AGGREGATE BASE - 15.5 INCHES	589.7	1																
DENSE, GRAY/BROWN, **CRUSHED STONE WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP FILL	588.7	2	11 15 15	36	44	SS-1	-	50	23	9	16	2	NP	NP	NP	8	A-1-b (0)	
LOOSE TO MEDIUM DENSE, BROWN, **CRUSHED STONE WITH SAND AND SILT**, TRACE CLAY, DAMP FILL		3	4 5 3	10	100	SS-2	-	-	-	-	-	-	-	-	-	7	A-2-4 (V)	
@4.5': GRAY/BROWN		5	4 4 5	11	100	SS-3	-	28	21	16	31	4	NP	NP	NP	10	A-2-4 (0)	
	583.4	6–7	4 4 4	10	100	SS-4	-	-	-	-	-	-	-	-	-	12	A-2-4 (V)	
HARD, BROWN, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, TRACE IRON OXIDE STAIN SEAM, MOIST	580.7	8–10	2 3 3	7	100	SS-5	4.50	-	-	-	-	-	-	-	-	16	A-4a (V)	
STIFF TO VERY STIFF, BROWN, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, TRACE IRON OXIDE STAIN SEAM, DAMP		11–12	4 5 4	11	100	SS-6	-	-	-	-	-	-	-	-	-	26	A-4a (V)	
@13': GRAY/BROWN, LITTLE GRAVEL	575.7	13–15			100	ST-7	2.75	10	15	15	39	21	25	18	7	18	A-4a (5)	
MEDIUM DENSE, BROWN, **FINE SAND**, LITTLE GRAVEL, TRACE SILT, TRACE CLAY, MOIST	573.0	16–18	2 5 8	16	100	SS-8	-	-	-	-	-	-	-	-	-	12	A-3 (V)	
MEDIUM DENSE, GRAY, **SEVERELY WEATHERED SHALE**, MOIST [INFERRED FROM DRILLING] @19.3': BLUE/GRAY	571.4	TR 19–20, W 571.4	5 5 13	22	100	SS-9	-	-	-	-	-	-	-	-	-	13	Rock (V)	
BLUE/GRAY, **WEATHERED SHALE SHALE**, GRAY, SEVERELY WEATHERED, WEAK, HIGHLY WEATHERED.		21	50/5"	-	100	SS-10	-	-	-	-	-	-	-	-	-	9	Rock (V)	
	567.2	22–24	50/5"	-	100	SS-11	-	-	-	-	-	-	-	-	-	6	Rock (V)	
SHALE, GRAY, MODERATELY WEATHERED, SLIGHTLY TO MODERATELY STRONG, JOINTED, HIGHLY FRACTURED, NARROW; RQD 0%, REC 95%. @25.4': Qu - 7,042 PSI @25.8': SDI, Id2 = 70.1 % @28.1': Qu - 2,581 PSI	562.2	25–29	0		95	NQ2-12											CORE	

PID: 119471	SFN: 0461254	PROJECT: OLD MAIN STREET BRIDGE	STATION / OFFSET: 12+67, 9' LT.	START: 10/18/23	END: 10/18/23	PG 2 OF 2	B-003-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 561.7	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
SHALE, GRAY, MODERATELY TO HIGHLY WEATHERED, SLIGHTLY TO MODERATELY STRONG, JOINTED, HIGHLY FRACTURED, NARROW; RQD 0%, REC 100%. *(continued)* @30.3': Qu - 4,994 PSI @32.1': Qu - 7,473 PSI	557.2	31 32 33 34 EOB	0		100	NQ2-13											CORE	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.25 BAG ASPHALT PATCH; PUMPED 10 CF CEMENT-BENTONITE GROUT

PROJECT: OLD MAIN STREET BRIDGE	DRILLING FIRM / OPERATOR: CT CONSULTANTS / CW	DRILL RIG: CME 75 TRUCK 844	STATION / OFFSET: 13+21, 3' RT.	EXPLORATION ID B-004-0-23
TYPE: SUBGRADE	SAMPLING FIRM / LOGGER: TTL / KKC	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. (OLD MAIN ST.)	
PID: 119471 SFN: 0461254	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 2/20/23	ELEVATION: 590.2 (NAVD88) EOB: 7.5 ft.	PAGE 1 OF 1
START: 10/18/23 END: 10/18/23	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 72.9	LAT / LONG: 41.943589, -80.550067	

MATERIAL DESCRIPTION AND NOTES	ELEV. 590.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
ASPHALT - 4 INCHES	589.9																	
AGGREGATE BASE - 8 INCHES	589.2																	
MEDIUM DENSE, BLACK/GRAY, **COAL FRAGMENTS**, TRACE SILT, TRACE CLAY, MOIST FILL	587.5		3 5 5	12	100	SS-1	-	-	-	-	-	-	-	-	-	15	UCF (V)	
LOOSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT**, TRACE CLAY, DAMP FILL	585.7		3 2 2	5	100	SS-2	-	28	29	15	25	3	NP	NP	NP	7	A-2-4 (0)	
STIFF, BROWN, **SANDY SILT**, SOME GRAVEL, TRACE CLAY, DAMP FILL			5 5 3	10	100	SS-3	-	27	22	13	33	5	NP	NP	NP	10	A-4a (1)	
	583.4		7 6 3	11	100	SS-4A	-	-	-	-	-	-	-	-	-	-	A-4a (V)	
STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, MOIST	582.7	EOB				SS-4B	1.50	-	-	-	-	-	-	-	-	22	A-6a (V)	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.25 BAG ASPHALT PATCH; AUGER CUTTINGS MIXED WITH 0.5 BAG BENTONITE CHIPS

KEY TO SYMBOLS

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT OLD MAIN STREET BRIDGE
PID 119471
OGE NUMBER N/A
PROJECT TYPE STRUCTURE FOUNDATION

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

- A-1-B: Ohio DOT: A-1-b, gravel and/or stone fragments with sand
- A-2-4: Ohio DOT: A-2-4, gravel and/or stone fragments with sand and silt
- F S — A-3: Ohio DOT: A-3, fine sand
- A-3A: Ohio DOT: A-3a, coarse and fine sand
- A-4A: Ohio DOT: A-4a, sandy silt
- A-6A: Ohio DOT: A-6a, silt and clay
- C — COAL: Ohio DOT: Coal or coal blossom
- PAVEMENT OR BASE: Ohio DOT: Pavement or Aggregate base
- SHALE: Ohio DOT: Shale
- WEATHERED SHALE: Ohio DOT: Highly or Severely Weathered Shale

SAMPLER SYMBOLS

- Thin Walled Undisturbed Sample

WELL CONSTRUCTION SYMBOLS

- Bentonite: Bottom of hole
- Soil Cuttings Backfill mixed with Bentonite Pellets or Chips
- Asphalt or Concrete Pavement Patch

ABBREVIATIONS

LL	- LIQUID LIMIT (%)
PI	- PLASTIC INDEX (%)
W	- MOISTURE CONTENT (%)
DD	- DRY DENSITY (PCF)
NP	- NON PLASTIC
-200	- PERCENT PASSING NO. 200 SIEVE
PP	- POCKET PENETROMETER (TSF)
TV	- TORVANE
PID	- PHOTOIONIZATION DETECTOR
UC	- UNCONFINED COMPRESSION
ppm	- PARTS PER MILLION

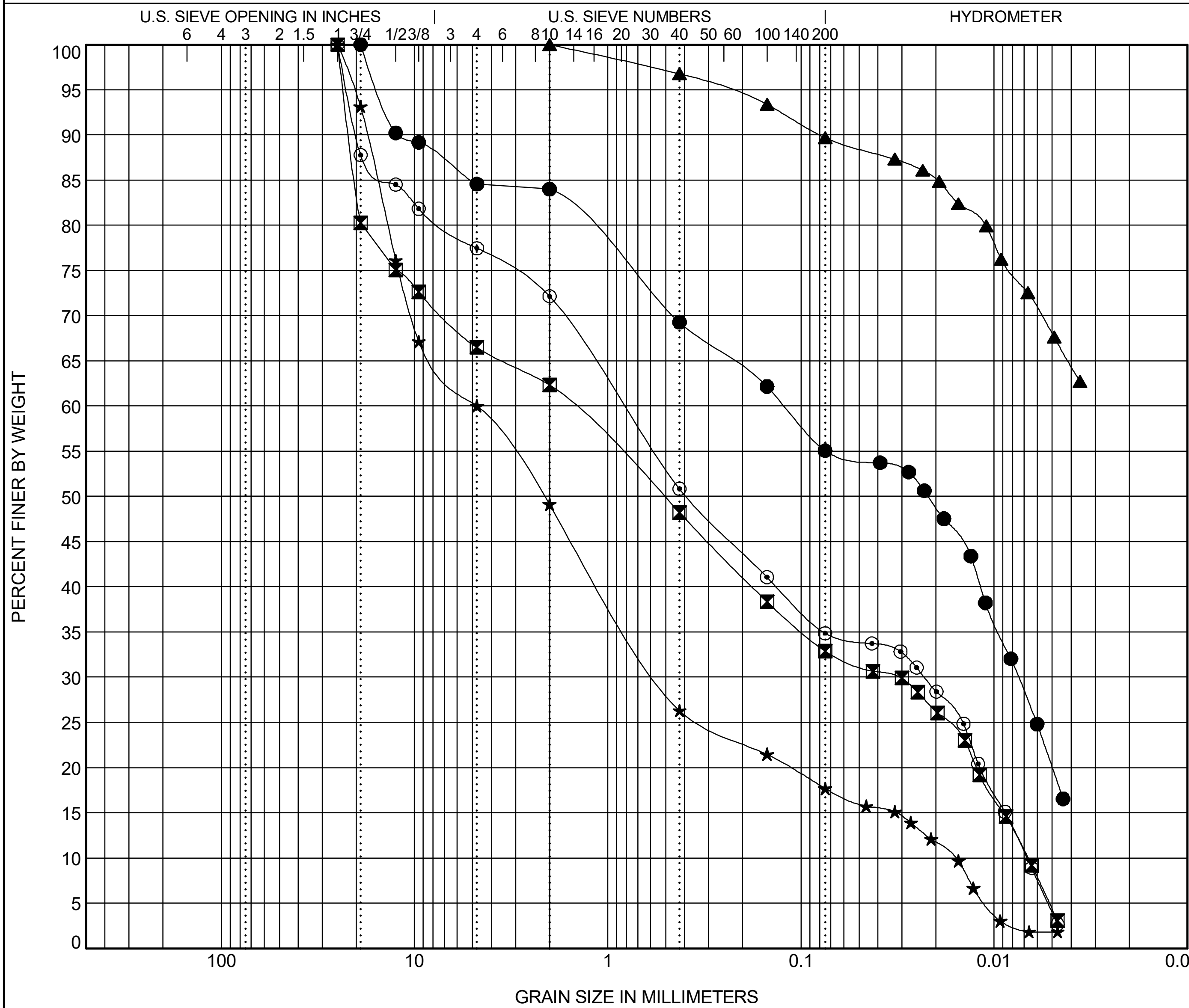
- ▽ Water Level at Time Drilling, or as Shown
- ▼ Water Level at End of Drilling, or as Shown
- ▼ Water Level After 24 Hours, or as Shown

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT OLD MAIN STREET BRIDGE **PID** 119471

OGE NUMBER N/A **PROJECT TYPE** STRUCTURE FOUNDATION



Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
● B-001-0-23	1.5	**A-4a ~ SANDY SILT with GRAVEL(ML)**	**NP**	**NP**	**NP**
⊠ B-001-0-23	3.0	**A-2-4 ~ SILTY SAND with GRAVEL(SM)**	**NP**	**NP**	**NP**
▲ B-001-0-23	6.0	**A-6a ~ LEAN CLAY(CL)**	**32**	**21**	**11**
★ B-003-0-23	1.5	**A-1-b ~ SILTY SAND with GRAVEL(SM)**	**NP**	**NP**	**NP**
⊙ B-003-0-23	4.5	**A-2-4 ~ SILTY SAND with GRAVEL(SM)**	**NP**	**NP**	**NP**

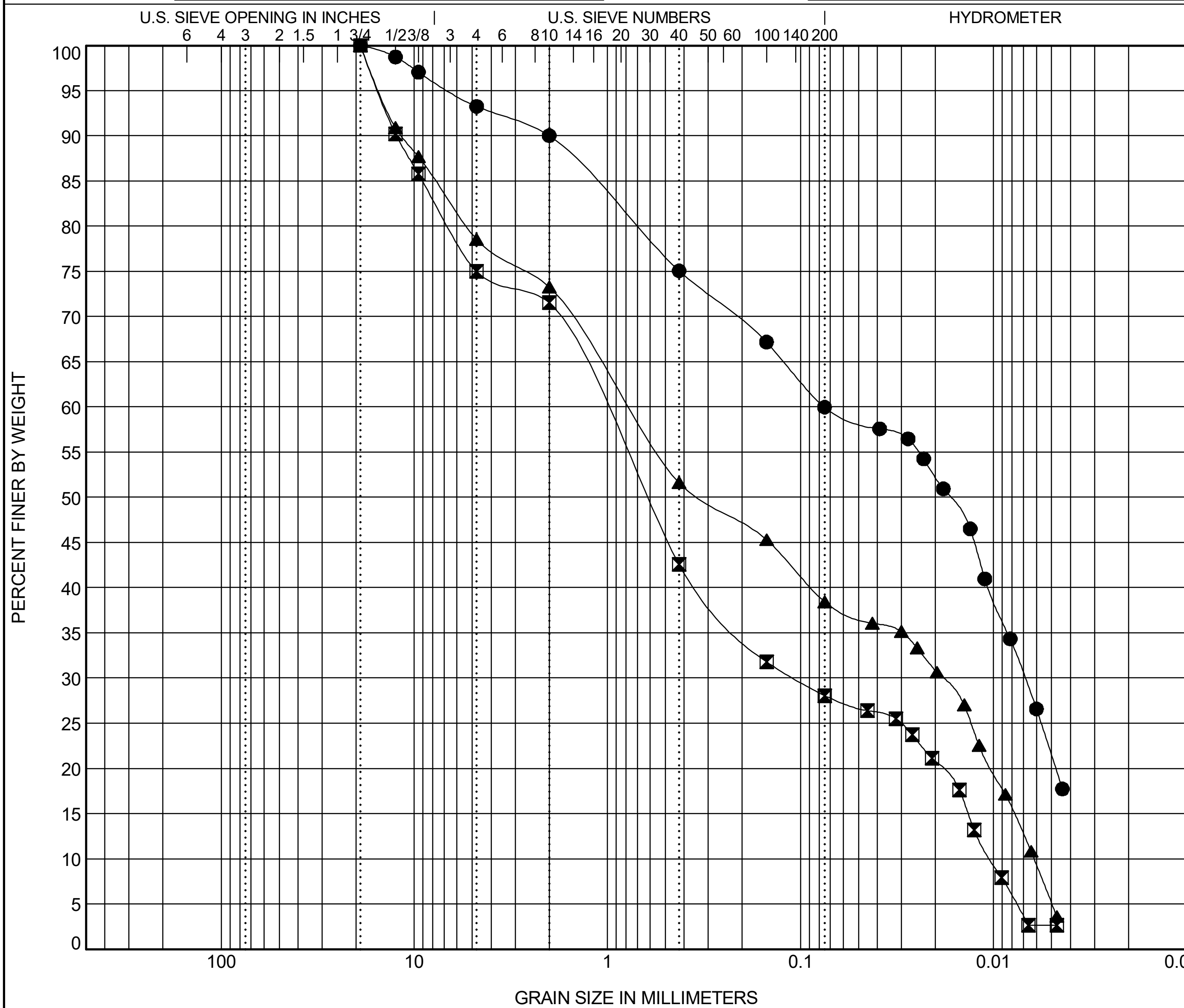
Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
● B-001-0-23	1.5	**11.789**	**0.022**	**0.008**		**16**	**15**	**14**	**35**	**20**		
⊠ B-001-0-23	3.0	**21.749**	**0.516**	**0.032**	**0.007**	**38**	**14**	**15**	**29**	**4**	**0.10**	**231.27**
▲ B-001-0-23	6.0	**0.079**				**0**	**3**	**7**	**22**	**68**		
★ B-003-0-23	1.5	**17.585**	**2.144**	**0.546**	**0.016**	**50**	**23**	**9**	**16**	**2**	**3.93**	**296.43**
⊙ B-003-0-23	4.5	**19.973**	**0.388**	**0.023**	**0.007**	**28**	**21**	**16**	**31**	**4**	**0.09**	**121.94**

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT OLD MAIN STREET BRIDGE **PID** 119471

OGE NUMBER N/A **PROJECT TYPE** STRUCTURE FOUNDATION



COBBLES	GRAVEL	SAND coarse	SAND fine	SILT	CLAY

Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
● B-003-0-23	13.0	**A-4a ~ SANDY SILTY CLAY(CL-ML)**	25	18	7
⧗ B-004-0-23	3.0	**A-2-4 ~ SILTY SAND with GRAVEL(SM)**	NP	NP	NP
▲ B-004-0-23	4.5	**A-4a ~ SILTY SAND with GRAVEL(SM)**	NP	NP	NP

Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
● B-003-0-23	13.0	1.995	0.017	0.007		10	15	15	39	21		
⧗ B-004-0-23	3.0	12.335	0.632	0.108	0.01	28	29	15	25	3	1.03	104.34
▲ B-004-0-23	4.5	11.575	0.325	0.019	0.006	27	22	13	33	5	0.07	125.25

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING

Office of Geotechnical Engineering

B-001-0-23

Core Date: October 19, 2023					Ground Surface Elevation: 592.7'			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-8	15.7'	20.7'	577.0'	572.0'	48/60	80%	0/60	0%
NQ2-9	20.7'	25.7'	572.0'	567.0'	53/60	88%	0/60	0%
Old Main Street Bridge, PID 119471								

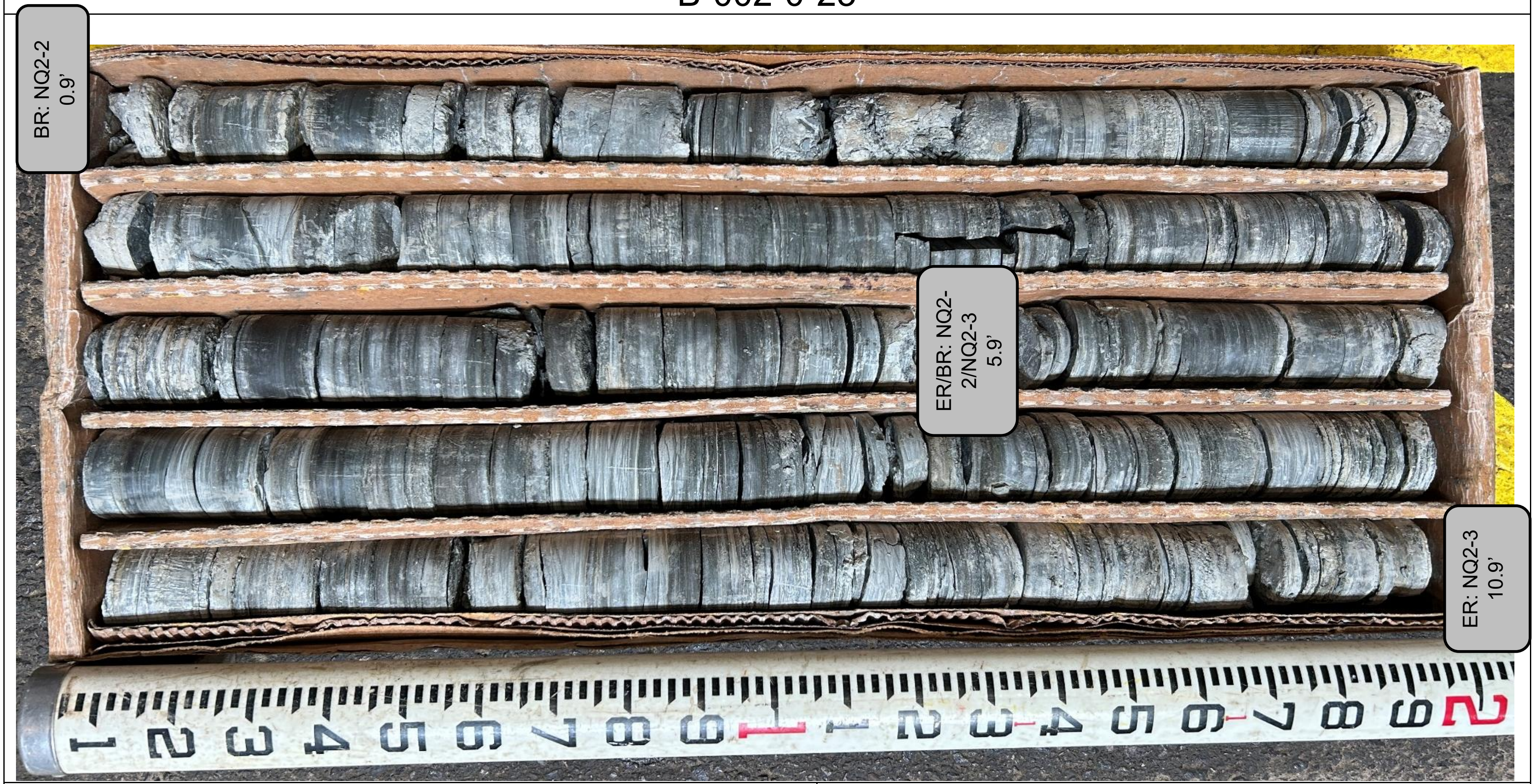
Prepared by: CT

CT Project No.: 232245

Ohio Department of Transportation
Division of Engineering

Office of Geotechnical Engineering

B-002-0-23



Core Date: October 17, 2023					Ground Surface Elevation: 570.1'			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-2	0.9'	5.9'	569.2'	564.2'	60/60	100%	0/60	0%
NQ2-3	5.9'	10.9'	564.2'	559.2'	54/60	90%	0/60	0%
Old Main Street Bridge, PID 119471								

Prepared by:

CT Project No.: 232245

Ohio Department of Transportation
Division of Engineering

Office of Geotechnical Engineering

B-003-0-23

Core Date: October 18, 2023					Ground Surface Elevation: 591.7'			
Run #:	Depth		Elevation		Recovery		RQD	
NQ2-12	24.5'	29.5'	567.2'	562.2'	57/60	95%	0/60	0%
NQ2-13	29.5'	34.5'	562.2'	557.2'	60/60	100%	0/60	0%
Old Main Street Bridge, PID 119471								

Prepared by:

CT Project No.: 232245

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING

Office of Geotechnical Engineering

PROJECT: ATB Old Main Street Bridge | DISTRICT No.: | PID No. 119471 | Tech: KKC

Axial Point Load Strength Calc*: $Is = P / (D_e^2)$ $D_e^2 = 4A/\pi$ A = (WD') Strength = Is * K K= 23

Boring #	Sample Elevation (ft)	Material Type	W (in)	W (mm)	D (in)	D (mm)	L/D	Failure Load (kN)	Penetration (in)	Penetration (mm)	Is_{50} (MPa)	Is_{50} (psi)	Strength S, (MPa)	Strength S (psi)
B-1	575.6 – 575.5	Shale	1.98	50.3	1.30	33.0	0.7	1.50	0.176	4.5	0.76	**110.9**	18.9	2730
	574.4 – 574.3	Shale	1.98	50.3	1.26	32.0	0.6	1.5	0.184	4.7	0.79	**114.7**	19.7	2860
	571.4 – 571.3	Shale	1.98	50.3	1.40	35.6	0.7	2.00	0.175	4.4	0.95	**138.3**	23.1	3350
											Average Strength (Sc)			2980
B-2	567.5 – 567.4	Shale	1.98	50.3	1.40	35.6	0.7	1.50	0.053	1.3	0.66	**96.4**	15.7	2280
	564.6 – 564.5	Shale	1.98	50.3	1.35	34.3	0.7	3.50	0.076	1.9	1.62	**234.9**	38.8	5630
	563.9 – 563.8	Shale	1.98	50.3	1.30	33.0	0.7	3.00	0.110	2.8	1.46	**212.2**	35.6	5170
											Average Strength (Sc)			4360
B-3	566.3 – 566.2	Shale	1.98	50.3	1.28	32.5	0.6	4.00	0.115	2.9	1.98	**287.7**	48.6	7042
	563.6 – 563.5	Shale	1.98	50.3	1.25	31.8	0.6	1.50	0.058	1.5	0.73	**106.0**	17.8	2581
	561.4 – 561.3	Shale	1.98	50.3	1.34	34.0	0.7	3.00	0.108	2.7	1.42	**206.6**	34.4	4994
	559.6 – 559.5	Shale	1.98	50.3	1.42	36.1	0.7	4.50	0.185	4.7	2.13	**309.3**	51.5	7473
											Average Strength (Sc)			5523

Comments:

SLAKE DURABILITY TEST

ASTM D 4644

Office of Geotechnical Engineering

Lab No.	
Report Date:	12/4/2025
Tech:	KC

County		Route		Section	
Boring Number	B-001-0-23	District		PID	119471
Station	10+32	Offset	68	Offset Direction	LT
Latitude	41.943784	Longitude	-80.551162	Ground Elev. (ft)	592.7
Sample Number	NQ2-8	Top Depth (ft)	15.8	Bottom Depth (ft)	16.5

Geologic Unit	
Description	SHALE

NATURAL MOISTURE DETERMINATION

Pan ID	Sample Weight (g)	Tare Weight (g)		IN: 4/16/2024	OUT: 4/17/2024	Moisture Content (%)
2	576.8	1260.3	Time	Not Applicable	Not Applicable	1.39
			Mass	1837.1	1829.1	

Start Time (mil):	End Time (mil):		**First Cycle (I_{d1})**					
0	10		Drum ID	Tare Weight (g)		IN: 4/17/2024	OUT: 4/17/2024	Final Dry Mass (g)
Start Temp. (°C):	End Temp. (°C):	Avg. Temp. (°C):	A	1260.3	Time	Not Applicable	Not Applicable	
20	21	20.5			Mass	2177.3	1718.8	458.5

Start Time (mil):	End Time (mil):		**Second Cycle (I_{d2})**					
			Drum ID	Tare Weight (g)		IN: 4/17/2024	OUT: 4/17/2024	Final Dry Mass (g)
Start Temp. (°C):	End Temp. (°C):	Avg. Temp. (°C):	A	1260.3	Time	Not Applicable	Not Applicable	
20	21	20.5			Mass	1857.5	1558.9	298.6

After First Cycle

After Second Cycle

Slake Durability Index
I_{d2} ={(W_F - C)/(B-C)}*100

I_{d2}
52.5
Retained Material
Type: I
Reference below

WF = Drum mass + oven dried specimen after second cycle
B = Drum mass + specimen prior to test
C = Drum mass

From ASTM D4644

T1 — Retained pieces remain virtually unchanged

T2 — Retained pieces remain virtually unchanged

T3 — Retained material is exclusively small pieces

SLAKE DURABILITY TEST

ASTM D 4644

Office of Geotechnical Engineering

Lab No.	
Report Date:	12/4/2025
Tech:	KC

County		Route		Section	
Boring Number	B-002-0-23	District		PID	119471
Station	11+71	Offset	6	Offset Direction	RT
Latitude	41.943617	Longitude	-80.55066	Ground Elev. (ft)	570.1
Sample Number	NQ2-2	Top Depth (ft)	0.9	Bottom Depth (ft)	1.7

Geologic Unit	
Description	SHALE

NATURAL MOISTURE DETERMINATION

Pan ID	Sample Weight (g)	Tare Weight (g)		IN: 4/16/2024	OUT: 4/17/2024	Moisture Content (%)
2	619.1	1259.3	Time	Not Applicable	Not Applicable	1.29
			Mass	1878.4	1870.4	

Start Time (mil):	End Time (mil):		**First Cycle (I_{d1})**					
0	10		Drum ID	Tare Weight (g)		IN: 4/17/2024	OUT: 4/17/2024	Final Dry Mass (g)
Start Temp. (^{o}C):	End Temp. (^{o}C):	Avg. Temp. (^{o}C):	B	1259.3	Time	Not Applicable	Not Applicable	
20	21	20.5			Mass	2279.9	1769.6	510.3

Start Time (mil):	End Time (mil):		**Second Cycle (I_{d2})**					
			Drum ID	Tare Weight (g)		IN: 4/17/2024	OUT: 4/17/2024	Final Dry Mass (g)
Start Temp. (^{o}C):	End Temp. (^{o}C):	Avg. Temp. (^{o}C):	B	1259.3	Time	Not Applicable	Not Applicable	
20	21	20.5			Mass	1921.1	1590.2	330.9

After First Cycle

After Second Cycle

Slake Durability Index $I_{d2} = \{(W_F - C)/(B-C)\}*100$
I_{d2}
54.1
Retained Material
Type: I
Reference below

WF = Drum mass + oven dried specimen after second cycle

B = Drum mass + specimen prior to test

C = Drum mass

From ASTM D4644

T1	Retained pieces remain virtually unchanged	T2	Retained pieces remain virtually unchanged	T3	Retained material is exclusively small pieces

SLAKE DURABILITY TEST

ASTM D 4644

Office of Geotechnical Engineering

Lab No.	
Report Date:	12/4/2025
Tech:	KC

County		Route		Section	
Boring Number	B-003-0-23	District		PID	119471
Station	12+67	Offset	9	Offset Direction	LT
Latitude	41.943638	Longitude	-80.550312	Ground Elev. (ft)	591.7
Sample Number	NQ2-12	Top Depth (ft)	25.8	Bottom Depth (ft)	26.5

Geologic Unit	
Description	SHALE

NATURAL MOISTURE DETERMINATION

Pan ID	Sample Weight (g)	Tare Weight (g)		IN: 4/16/2024	OUT: 4/17/2024	Moisture Content (%)
2	625.1	1260.3	Time	Not Applicable	Not Applicable	1.28
			Mass	1885.4	1877.4	

First Cycle (I_{d1})

Start Time (mil):	End Time (mil):		Drum ID	Tare Weight (g)		IN: 4/17/2024	OUT: 4/17/2024	Final Dry Mass (g)
0	10							
Start Temp. (°C):	End Temp. (°C):	Avg. Temp. (°C):	A	1260.3	Time	Not Applicable	Not Applicable	
20	21	20.5			Mass	2258.7	1759.5	499.2

Second Cycle (I_{d2})

Start Time (mil):	End Time (mil):		Drum ID	Tare Weight (g)		IN: 4/17/2024	OUT: 4/17/2024	Final Dry Mass (g)
Start Temp. (°C):	End Temp. (°C):	Avg. Temp. (°C):	A	1260.3	Time	Not Applicable	Not Applicable	
20	21	20.5			Mass	2125.1	1692.7	432.4

After First Cycle

After Second Cycle

Slake Durability Index

I_{d2} ={(W_F - C)/(B-C)}*100

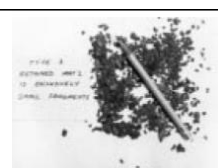
I_{d2}
70.1
Retained Material
Type: I
Reference below

WF = Drum mass + oven dried specimen after second cycle

B = Drum mass + specimen prior to test

C = Drum mass

From ASTM D4644



T1	Retained pieces remain virtually unchanged	T2	Retained pieces remain virtually unchanged	T3	Retained material is exclusively small pieces

Appendix A

Engineering Calculations
(Including ODOT Subgrade Analysis Spreadsheets)

Project No.:	**232245**
Project:	**ATB-Old Main Street Bridge**
Calcs by:	**MSI**
Date:	**2/11/2025**
Revision:	**2**
Date:	**msi, 12/3/2025**
Chekced:	**ihj, 12/4/2025**
Calcs:	**Drilled Shaft Rock Sockets - Vertical Resistance**
Location:	**ATB-Old Main Street**
Substructure:	**Rear (West) Abutment**
Boring(s):	B-001-0-23
Ground Surface Elevation (ft):	593.71
Bottom of Abutment Elev (ft):	583.8
Top of Rock Elevation (ft):	577
Length of Shaft in Soil (ft):	6.8
Shaft in Soil Diameter (in):	42
Shaft in Rock Diameter (in):	36
Shaft in Rock Diameter (ft):	3
End-Bearing at 1.5 x B	
Length of Socket (ft):	4.5
May increase the shaft diameter in soil to 3.5 ft and reduce the shaft diameter in rock to 3 ft for lateral resistance.	
Shaft in Rock Diameter (ft):	3
In this case, 1.5 x B	
Length of Socket (ft):	4.5
BDM 305.4.4.4, minimum 5' socket if rock within 10 ft of ground surface or bottom of shaft cap.	
As noted above, shaft in soil (ft):	6.8
Therefore, governing Length of Socket (ft):	5
End-Bearing Elev. (ft):	572

Calcs:	**Drilled Shaft Rock Sockets - Vertical Resistance**
Location:	**ATB-Old Main Street**
Substructure:	**Rear (West) Abutment**
Look at rock core Qu at bearing to	
2B below bearing:	
2B below foundation/shaft bearing Elev.:	566
Qu (psi):	2730
	2860
	3350
Use Average Qu (psi):	2980
Average Qu (ksf):	429
End-Bearing Resistance (AASHTO LRFD 10.8.3.5.4c-1)	
qp=2.5qu	
(Unfactored) qp (ksf):	**1073**
Resistance Factor (AASHTO LRFD Table 10.5.5.2.4-1)	
f=	0.5
Factored Bearing Resistance (ksf)=	536
Say, Factored Bearing Resistance (ksf)=	**535**
For 3 ft diameter socket,	
Available Resistance (kips)=	3782
Based on provided loading	
Indicated Total Factored Load (kips)=	307.8
Suitable Vertical Resistance?	**YES**
For 3 ft diameter socket,	
Available Resistance (kips)=	3782

Project No.:	**232245**
Project:	**ATB-Old Main ST. Bridge**
Calcs by:	**msi**
Date:	**2/12/2025**
Review:	**2**
Revision:	**2**
Revision by:	**msi, 12/3/2025**
Checked by:	**ihj, 12/4/2025**
Calcs:	**Drilled Shaft Rock Sockets - Lateral Resistance**
Location:	**Old Main St., Ashtabula, OH**
Substructure:	**Rear (West) Abutment**
Boring(s):	B-001-0-23
GSE (ft):	592.7
Long-Term GWT (ft):	586.7 — Based on PL>MC for sample SS 3B
Bottom of Shaft Elev. (ft):	572 — 5 feet below TR

Soil

Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**Avg. N60**	**HP (tsf)**	**Qu (tsf)**
Layer 1 - Fill	**Med. Stiff, A-4a**	0	3	592.7	589.7	7	-	-
	Depth below bottom of Pier Cap:			-20.7	-17.7			
Total Unit Wt (pcf):	118	GDM Table 400-4		**Use**	**118**	**pcf**		
Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1								
N60, Su (ksf)=	0.875							
HP, Su (ksf)=	-							
Qu, Su (ksf)=	-							
Say, Su (ksf)=	**0.875**							
Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual								
Su = 1-2 ksf, epsilon 50 =	**0.01**							

Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**N60**	**HP (tsf)**	**Qu (tsf)**
Layer 2 - Fill	**Very loose to loose, A-2-4**	3	5.3	589.7	587.4	2.5	-	-
	Depth below bottom of Pier Cap:			-17.7	-15.4			
Total Unit Wt (pcf):	110	GDM Table 400-4		**Use**	**110**	**pcf**		
Internal Angle of Friction Determination (GDM 404.2):								
N160 (bpf)=CN*N60	AASHTO LRFD 10.4.6.2.4							
CN=0.77log(40/sigma-v'), with CN<2.0								
CN at	4.15	ft						
sigma-v' (ksf):	0.48							
CN=	1.5	<2 so use:	1.5					
N160 (bpf)=	4							
AASHTO LRFD Table 10.4.6.2.4-1								
N160	Mid-Range Phi (deg)							
4	29.5							
		use	29.5	deg				
GDM Table 400-3 phi Adjustment								
A-3a	-0.5							
Phi (deg) =	**29**	< ODOT Maximum 46 deg, ok						
k Evaluation From LPILE 2018 Technical Manual								
Parameters:	Loose sand and silt							
Range of k-value (pci) =	3.5 to 10.4							
Very loose to loose range of N60	k (pci)							
0	3.5							
10	10.4							
Interpolate for 2.5 bpf for this layer:	5.2							
Say k (pci) =	**5**	**Sand (Reese)**						

Calcs:	**Drilled Shaft Rock Sockets - Lateral Resistance**							
Location:	**Old Main St., Ashtabula, OH**							
Substructure:	**Rear (West) Abutment**							
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**N60**	**HP (tsf)**	**Qu (tsf)**
Layer 3	**Stiff to V. Stiff, A-6a**	5.3	8	587.4	584.7	10	2.63	-
			Depth below bottom of Pier Cap:	-15.4	-12.7			
Total Unit Wt (pcf):	120	GDM Table 400-4		**Use**	**120**	**pcf**		
Su = N60 x 125 (N60<= 52 bpf) per GDM 404.1				Based on Unit Wt for native A-4a in B-002.				
N60, Su (ksf)=	1.25							
HP, Su (ksf)=	2.63							
Say, Su (ksf)=	**2.63**							
Evaluation of Strain at half stress (epsilon 50) from LPILE 2018 Technical Manual								
Su = 2-4 ksf, epsilon 50 =	**0.008**							
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**N60**	**HP (tsf)**	**Qu (tsf)**
Layer 4 = Layer 1 in Lpile	**Medium dense, A-3a**	8	11.3	584.7	581.4	17	-	-
			Depth below bottom of Pier Cap:	-12.7	-9.4			
Total Unit Wt (pcf):	122	GDM Table 400-4		**Use**	**122**	**pcf**		
Internal Angle of Friction Determination (GDM 404.2):								
N160 (bpf)=CN*N60	AASHTO LRFD 10.4.6.2.4							
CN=0.77log(40/sigma-v'), with CN<2.0								
CN at	9.65	ft						
sigma-v' (ksf):	1.03							
CN=	1.2	<2 so use:	1.2					
N160 (bpf)=	21							
AASHTO LRFD Table 10.4.6.2.4-1								
N160	Mid-Range Phi (deg)							
10	32.5							
30	37.5							
N160	Phi (deg)							
21	35.2	use	34.5	deg				
GDM Table 400-3 phi Adjustment								
A-3a	-0.5							
Phi (deg) =	**34**	< ODOT Maximum 46 deg, ok						
k Evaluation From LPILE 2019 Technical Manual								
Parameters:	Loose sand and silt							
Internal Angle of Friction (deg)=	34	deg.						
k-value (pci) based on Figure 3.34 =	75	approx. for fine sand below the water table.						
Say k (pci) =	**75**	**L-Pile (2019), Figure 3.34**						
Layer	**Soil Type**	**Top Depth (ft)**	**Bottom Depth (ft)**	**Top Elev. (ft)**	**Bottom Elev. (ft)**	**N60**	**HP (tsf)**	**Qu (tsf)**
Layer 5 = Layer 2 in Lpile	**Very dense, A-1-a**	11.3	15.7	581.4	577	>100	-	-
			Depth below bottom of Pier Cap:	-3.8	-0.5			
Total Unit Wt (pcf):	140	GDM Table 400-4		**Use**	**140**	**pcf**		
Internal Angle of Friction Determination (GDM 404.2):								
N160 (bpf)=CN*N60	AASHTO LRFD 10.4.6.2.4							
CN=0.77log(40/sigma-v'), with CN<2.0								
CN at	13.5	ft						
sigma-v' (ksf):	0.17							
CN=	1.8	<2 so use:	1.8					
N160 (bpf)=	#VALUE!							
AASHTO LRFD Table 10.4.6.2.4-1								
N160	Mid-Range Phi (deg)							
>50	43							
N160	Phi (deg)							
>50	43	use	43	deg.	*Conservatively.*			
GDM Table 400-3 phi Adjustment		*Phi-prime cannot be more than 45 deg (ODOT GDM 2025, Section 400). Therefore, using 43 (max) from LRFD Table 10.4.6.2.4-1*						
A-1-a	2.5	*Not Applied.*						
Phi (deg) =	**43**	< ODOT Maximum 46 deg, ok						
k Evaluation From LPILE 2018 Technical Manual								
Parameters:	Very dense, A-1-a							
Internal Angle of Friction (deg)=	43	deg.						
k-value (pci) based on Figure 3.34 =	190	approx. for fine sand below the water table.						
Say k (pci) =	**190**	**L-Pile (2019), Figure 3.34**						
Bedrock = Layer 3 in Lpile								

Calcs:	Drilled Shaft Rock Sockets - Lateral Resistance									
Location:	Old Main St., Ashtabula, OH									
Substructure:	Rear (West) Abutment									
Layer	Soil Type	Top Depth (ft)	Bottom Depth (ft)	Top Elev. (ft)	Bottom Elev. (ft)	RQD (%)	Rec (%)	Avg. Qu (psi)	Total Unit Wt (pcf)	
Layer 6	Shale - Highly Weathered	15.7	25.7	577	567	0	100	2980	150	at 17-21 ft
	Weak to Slightly Strong									
			Depth below bottom of Pier Cap:	-5	5					
Total Unit Wt (pcf):	150 - 160	GDM Table 400-5		Use	150	pcf				
Qu (psi)=	2,730									
From GDM Table 400-6										
Qu (psi)	E (psi)									
2,250	200,000									
3,600	320,000									
Interpolation for Qu (psi) = 2730, E(psi):	242,667									
From GDM Table 400-6, say E (psi) =	242,667									
If Strain at 242667 psi is 1%, then strain at half max stress (krm) is calculated by:										
Half max stress = Qu/2 =	1,365	psi								
krm = 1% x (1365 psi / 242667 psi) =	0.0056	%								
krm (decimal format) =	0.000056									

```
================================================================================

                        LPile for Version 2022-12.012

                          License ID : 5279320353
                     License Type : (Office Cloud License)

                 Analysis of Individual Piles and Drilled Shafts
                Subjected to Lateral Loading Using the p-y Method
                         © 1985-2024 by Ensoft, Inc.
                             All Rights Reserved

                This software is licensed for exclusive use by:
                                Verdantas Inc.


================================================================================


This model was prepared by:
ihajjar




--------------------------------------------------------------------------------
                           Files Used for Analysis
--------------------------------------------------------------------------------

Path to file locations:
\\verdantas.com\CT\Projects\2023\232245\PHASE\02 Geotechnical Engineering
Services\Project Data\Calculations\L-Pile Analysis\Rear (West) Abutment Drilled
Shaft\

Name of input data file:
42_to_36-inch Dia.lp12d

Name of output report file:
42_to_36-inch Dia.lp12o

Name of plot output file:
42_to_36-inch Dia.lp12p

Name of runtime message file:
42_to_36-inch Dia.lp12r



--------------------------------------------------------------------------------
                          Date and Time of Analysis
```

Date: January 7, 2026   Time: 15:34:02

Problem Title

Project Name: ATB Old Main Street Bridtge

Job Number: 232245

Client: City of Conneaut

Engineer: msi

Description: Lateral Shaft Resistance - Rear Abutment

Program Options and Settings

Computational Options:
 - Conventional Analysis
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)

Analysis Control Options:

```
 - Maximum number of iterations allowed                 =          500
 - Deflection tolerance for convergence                 =   1.0000E-05 in
 - Maximum allowable deflection                         =     100.0000 in
 - Number of pile increments                            =          100
```

Loading Type and Number of Cycles of Loading:
 - Static loading specified

- Analysis uses p-y modification factors for p-y curves
- Analysis uses layering correction (Method of Georgiadis)
- Analysis includes loading by multiple distributed lateral loads acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 2
Total length of pile = 11.800 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	42.0000
2	6.800	42.0000
3	6.800	36.0000
4	11.800	36.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

```
   Section 1 is an elastic pile
   Cross-sectional Shape                                 =   Circular Pile
   Length of section                                     =     6.800000 ft
   Width of top of section                               =    42.000000 in
   Width of bottom of section                            =    42.000000 in
   Top Area                                              =        1018. sq. in
   Bottom Area                                           =        1018. sq. in
   Moment of Inertia at Top                              =       82448. in^4
   Moment of Inertia at Bottom                           =       82448. in^4
   Elastic Modulus                                       =     3604997. psi

Pile Section No. 2:

   Section 2 is an elastic pile
   Cross-sectional Shape                                 =   Circular Pile
   Length of section                                     =     5.000000 ft
   Width of top of section                               =    36.000000 in
   Width of bottom of section                            =    36.000000 in
   Top Area                                              =        1018. sq. in
   Bottom Area                                           =        1018. sq. in
   Moment of Inertia at Top                              =       82448. in^4
   Moment of Inertia at Bottom                           =       82448. in^4
   Elastic Modulus                                       =     3604997. psi



--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------

The soil profile is modelled using 3 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer             =       0.0000 ft
   Distance from top of pile to bottom of layer          =     2.400000 ft
   Effective unit weight at top of layer                 =    59.600000 pcf
   Effective unit weight at bottom of layer              =    59.600000 pcf
   Friction angle at top of layer                        =    34.000000 deg.
   Friction angle at bottom of layer                     =    34.000000 deg.
   Subgrade k at top of layer                            =    75.000000 pci
   Subgrade k at bottom of layer                         =    75.000000 pci


Layer 2 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer             =     2.400000 ft
   Distance from top of pile to bottom of layer          =     6.800000 ft
   Effective unit weight at top of layer                 =    77.600000 pcf
   Effective unit weight at bottom of layer              =    77.600000 pcf
```

Friction angle at top of layer = 43.000000 deg.
Friction angle at bottom of layer = 43.000000 deg.
Subgrade k at top of layer = 190.000000 pci
Subgrade k at bottom of layer = 190.000000 pci

Layer 3 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer = 6.800000 ft
Distance from top of pile to bottom of layer = 11.800000 ft
Effective unit weight at top of layer = 87.600000 pcf
Effective unit weight at bottom of layer = 87.600000 pcf
Uniaxial compressive strength at top of layer = 2730. psi
Uniaxial compressive strength at bottom of layer = 3350. psi
Initial modulus of rock at top of layer = 1400000. psi
Initial modulus of rock at bottom of layer = 1400000. psi
RQD of rock at top of layer = 10.000000 %
RQD of rock at bottom of layer = 10.000000 %
k rm of rock at top of layer = 0.0000500
k rm of rock at bottom of layer = 0.0000500

(Depth of the lowest soil layer extends 0.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Angle of Friction deg.	Uniaxial qu psi	RQD %	E50 or krm	kpy pci	Rock Mass Modulus psi
1	Sand	0.00	59.6000	34.0000	--	--	--	75.0000	--
	(Reese, et al.)	2.4000	59.6000	34.0000	--	--	--	75.0000	--
2	Sand	2.4000	77.6000	43.0000	--	--	--	190.0000	--
	(Reese, et al.)	6.8000	77.6000	43.0000	--	--	--	190.0000	--
3	Weak	6.8000	87.6000	--	2730.	10.0000	5.00E-05	--	1400000.
	Rock	11.8000	87.6000	--	3350.	10.0000	5.00E-05	--	1400000.

Modification Factors for p-y Curves

Distribution of p-y modifiers with depth defined using 2 points

Point No.	Depth X ft	p-mult	y-mult
1	0.000	1.0000	1.0000
2	11.800	1.0000	1.0000

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	2	V = 63620. lbs	S = 0.0000 in/in	307800.	No	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Pile Section No. 2:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	6695.
2	2.4000	1.8613	Yes	No	6695.	102095.
3	6.8000	6.8000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Pile-head Rotation (Loading Type 2)

Shear force at pile head = 63620.0 lbs
Rotation of pile head = 0.000E+00 radians
Axial load at pile head = 307800.0 lbs

(Zero slope for this load indicates fixed-head conditions)

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.01352	-2883754.	63620.	0.00	1037.	2.97E+11	0.00	0.00	0.00
0.1180	0.01351	-2793665.	63619.	-1.35E-05	1014.	2.97E+11	-1.435	150.3792	0.00
0.2360	0.01348	-2703573.	63616.	-2.66E-05	991.0111	2.97E+11	-2.864	300.7584	0.00
0.3540	0.01344	-2613482.	63611.	-3.93E-05	968.0642	2.97E+11	-4.280	451.1376	0.00
0.4720	0.01337	-2523393.	63604.	-5.15E-05	945.1181	2.97E+11	-5.680	601.5168	0.00
0.5900	0.01329	-2433311.	63595.	-6.33E-05	922.1736	2.97E+11	-7.057	751.8960	0.00
0.7080	0.01319	-2343237.	63584.	-7.47E-05	899.2313	2.97E+11	-8.406	902.2752	0.00
0.8260	0.01308	-2253176.	63571.	-8.57E-05	876.2922	2.97E+11	-9.722	1053.	0.00
0.9440	0.01295	-2163130.	63556.	-9.62E-05	853.3568	2.97E+11	-11.001	1203.	0.00
1.0620	0.01281	-2073101.	63540.	-1.06E-04	830.4259	2.97E+11	-12.239	1353.	0.00
1.1800	0.01265	-1983092.	63522.	-1.16E-04	807.5001	2.97E+11	-13.432	1504.	0.00
1.2980	0.01248	-1893106.	63502.	-1.25E-04	784.5801	2.97E+11	-14.576	1654.	0.00
1.4160	0.01229	-1803146.	63480.	-1.34E-04	761.6666	2.97E+11	-15.667	1805.	0.00
1.5340	0.01210	-1713213.	63458.	-1.42E-04	738.7601	2.97E+11	-16.702	1955.	0.00
1.6520	0.01189	-1623310.	63433.	-1.50E-04	715.8613	2.97E+11	-17.679	2105.	0.00
1.7700	0.01167	-1533439.	63408.	-1.58E-04	692.9706	2.97E+11	-18.594	2256.	0.00
1.8880	0.01144	-1443602.	63381.	-1.65E-04	670.0886	2.97E+11	-19.445	2406.	0.00

2.0060	0.01121	-1353801.	63353.	-1.72E-04	647.2158	2.97E+11	-20.230	2556.	0.00
2.1240	0.01096	-1264038.	63323.	-1.78E-04	624.3526	2.97E+11	-20.947	2707.	0.00
2.2420	0.01070	-1174314.	63293.	-1.84E-04	601.4994	2.97E+11	-21.594	2857.	0.00
2.3600	0.01044	-1084631.	63262.	-1.89E-04	578.6567	2.97E+11	-22.170	3008.	0.00
2.4780	0.01017	-994991.	63206.	-1.94E-04	555.8246	2.97E+11	-57.438	8000.	0.00
2.5960	0.00989	-905463.	63124.	-1.98E-04	533.0214	2.97E+11	-58.528	8381.	0.00
2.7140	0.00960	-816051.	63040.	-2.03E-04	510.2476	2.97E+11	-59.431	8762.	0.00
2.8320	0.00931	-726757.	62956.	-2.06E-04	487.5037	2.97E+11	-60.145	9143.	0.00
2.9500	0.00902	-637581.	62870.	-2.09E-04	464.7902	2.97E+11	-60.670	9524.	0.00
3.0680	0.00872	-548526.	62784.	-2.12E-04	442.1073	2.97E+11	-61.007	9905.	0.00
3.1860	0.00842	-459592.	62697.	-2.15E-04	419.4552	2.97E+11	-61.156	10286.	0.00
3.3040	0.00811	-370779.	62611.	-2.17E-04	396.8342	2.97E+11	-61.119	10667.	0.00
3.4220	0.00781	-282089.	62524.	-2.18E-04	374.2442	2.97E+11	-60.898	11048.	0.00
3.5400	0.00750	-193520.	62439.	-2.19E-04	351.6851	2.97E+11	-60.496	11429.	0.00
3.6580	0.00718	-105072.	62353.	-2.20E-04	329.1568	2.97E+11	-59.916	11810.	0.00
3.7760	0.00687	-16743.	62269.	-2.20E-04	306.6590	2.97E+11	-59.162	12191.	0.00
3.8940	0.00656	71466.	62186.	-2.20E-04	320.5973	2.97E+11	-58.240	12572.	0.00
4.0120	0.00625	159559.	62104.	-2.20E-04	343.0350	2.97E+11	-57.154	12953.	0.00
4.1300	0.00594	247537.	62024.	-2.19E-04	365.4435	2.97E+11	-55.911	13334.	0.00
4.2480	0.00563	335402.	61946.	-2.17E-04	387.8233	2.97E+11	-54.516	13715.	0.00
4.3660	0.00532	423157.	61870.	-2.16E-04	410.1751	2.97E+11	-52.978	14096.	0.00
4.4840	0.00502	510805.	61796.	-2.13E-04	432.4996	2.97E+11	-51.304	14477.	0.00
4.6020	0.00472	598349.	61725.	-2.11E-04	454.7976	2.97E+11	-49.503	14857.	0.00
4.7200	0.00442	685793.	61656.	-2.08E-04	477.0701	2.97E+11	-47.584	15238.	0.00
4.8380	0.00413	773140.	61590.	-2.04E-04	499.3178	2.97E+11	-45.556	15619.	0.00

4.9560	0.00384	860394.	61527.	-2.00E-04	521.5419	2.97E+11	-43.430	16000.	0.00
5.0740	0.00356	947559.	61467.	-1.96E-04	543.7434	2.97E+11	-41.218	16381.	0.00
5.1920	0.00329	1034639.	61410.	-1.91E-04	565.9233	2.97E+11	-38.930	16762.	0.00
5.3100	0.00302	1121639.	61357.	-1.86E-04	588.0828	2.97E+11	-36.579	17143.	0.00
5.4280	0.00276	1208564.	61307.	-1.81E-04	610.2230	2.97E+11	-34.177	17524.	0.00
5.5460	0.00251	1295417.	61260.	-1.75E-04	632.3451	2.97E+11	-31.739	17905.	0.00
5.6640	0.00227	1382204.	61217.	-1.68E-04	654.4503	2.97E+11	-29.279	18286.	0.00
5.7820	0.00203	1468930.	61177.	-1.61E-04	676.5399	2.97E+11	-26.811	18667.	0.00
5.9000	0.00181	1555599.	61141.	-1.54E-04	698.6149	2.97E+11	-24.350	19048.	0.00
6.0180	0.00160	1642215.	61108.	-1.47E-04	720.6767	2.97E+11	-21.913	19429.	0.00
6.1360	0.00140	1728785.	61079.	-1.39E-04	742.7265	2.97E+11	-19.517	19810.	0.00
6.2540	0.00120	1815312.	61053.	-1.30E-04	764.7654	2.97E+11	-17.178	20191.	0.00
6.3720	0.00103	1901800.	61030.	-1.21E-04	786.7945	2.97E+11	-14.914	20572.	0.00
6.4900	8.61E-04	1988255.	61011.	-1.12E-04	808.8150	2.97E+11	-12.745	20953.	0.00
6.6080	7.09E-04	2074679.	60994.	-1.02E-04	830.8280	2.97E+11	-10.688	21334.	0.00
6.7260	5.72E-04	2161079.	60980.	-9.22E-05	852.8344	2.97E+11	-8.765	21715.	0.00
6.8440	4.48E-04	2247456.	37490.	-8.17E-05	793.0579	2.97E+11	-33169.	1.05E+08	0.00
6.9620	3.40E-04	2267322.	-9222.	-7.10E-05	797.3950	2.97E+11	-32808.	1.37E+08	0.00
7.0800	2.47E-04	2221401.	-55123.	-6.03E-05	787.3695	2.97E+11	-32024.	1.83E+08	0.00
7.1980	1.69E-04	2111266.	-97561.	-4.99E-05	763.3249	2.97E+11	-27917.	2.33E+08	0.00
7.3160	1.06E-04	1945150.	-130222.	-4.03E-05	727.0587	2.97E+11	-18214.	2.44E+08	0.00
7.4340	5.53E-05	1742511.	-150150.	-3.15E-05	682.8185	2.97E+11	-9931.	2.54E+08	0.00
7.5520	1.66E-05	1519954.	-159378.	-2.37E-05	634.2301	2.97E+11	-3103.	2.64E+08	0.00
7.6700	-1.19E-05	1291173.	-159944.	-1.70E-05	584.2826	2.97E+11	2303.	2.75E+08	0.00
7.7880	-3.16E-05	1067007.	-153801.	-1.14E-05	535.3428	2.97E+11	6373.	2.85E+08	0.00

7.9060	-4.42E-05	855617.	-142754.	-6.84E-06	489.1923	2.97E+11	9230.	2.96E+08	0.00
8.0240	-5.10E-05	662732.	-128414.	-3.22E-06	447.0818	2.97E+11	11024.	3.06E+08	0.00
8.1420	-5.33E-05	491950.	-112171.	-4.71E-07	409.7967	2.97E+11	11919.	3.16E+08	0.00
8.2600	-5.23E-05	345064.	-95179.	1.52E-06	377.7286	2.97E+11	12081.	3.27E+08	0.00
8.3780	-4.90E-05	222401.	-78360.	2.87E-06	350.9488	2.97E+11	11675.	3.37E+08	0.00
8.4960	-4.42E-05	123145.	-62412.	3.70E-06	329.2794	2.97E+11	10851.	3.48E+08	0.00
8.6140	-3.85E-05	45647.	-47829.	4.10E-06	312.3600	2.97E+11	9747.	3.58E+08	0.00
8.7320	-3.26E-05	-12309.	-34924.	4.18E-06	305.0816	2.97E+11	8479.	3.68E+08	0.00
8.8500	-2.67E-05	-53263.	-23861.	4.02E-06	314.0227	2.97E+11	7147.	3.79E+08	0.00
8.9680	-2.12E-05	-79887.	-14676.	3.71E-06	319.8353	2.97E+11	5826.	3.89E+08	0.00
9.0860	-1.62E-05	-94829.	-7311.	3.29E-06	323.0975	2.97E+11	4577.	4.00E+08	0.00
9.2040	-1.19E-05	-100594.	-1634.	2.82E-06	324.3561	2.97E+11	3440.	4.10E+08	0.00
9.3220	-8.22E-06	-99461.	2529.	2.35E-06	324.1086	2.97E+11	2441.	4.20E+08	0.00
9.4400	-5.23E-06	-93433.	5385.	1.89E-06	322.7927	2.97E+11	1592.	4.31E+08	0.00
9.5580	-2.88E-06	-84213.	7146.	1.46E-06	320.7797	2.97E+11	895.9323	4.41E+08	0.00
9.6760	-1.09E-06	-73196.	8026.	1.09E-06	318.3745	2.97E+11	346.2539	4.52E+08	0.00
9.7940	2.10E-07	-61484.	8222.	7.69E-07	315.8176	2.97E+11	-68.588	4.62E+08	0.00
9.9120	1.09E-06	-49910.	7916.	5.03E-07	313.2908	2.97E+11	-364.077	4.72E+08	0.00
10.0300	1.64E-06	-39066.	7264.	2.91E-07	310.9234	2.97E+11	-557.699	4.83E+08	0.00
10.1480	1.92E-06	-29341.	6396.	1.28E-07	308.8000	2.97E+11	-667.518	4.93E+08	0.00
10.2660	2.00E-06	-20953.	5420.	8.62E-09	306.9688	2.97E+11	-711.064	5.04E+08	0.00
10.3840	1.94E-06	-13991.	4418.	-7.46E-08	305.4489	2.97E+11	-704.518	5.14E+08	0.00
10.5020	1.79E-06	-8442.	3450.	-1.28E-07	304.2374	2.97E+11	-662.159	5.24E+08	0.00
10.6200	1.58E-06	-4220.	2559.	-1.58E-07	303.3157	2.97E+11	-596.048	5.35E+08	0.00
10.7380	1.34E-06	-1193.	1772.	-1.71E-07	302.6549	2.97E+11	-515.894	5.45E+08	0.00

```
   10.8560   1.09E-06   798.7360      1103.  -1.72E-07   302.5688   2.97E+11
-429.083   5.56E+08       0.00
   10.9740   8.53E-07      1931.   557.9819  -1.66E-07   302.8159   2.97E+11
-340.808   5.66E+08       0.00
   11.0920   6.25E-07      2379.   136.6475  -1.55E-07   302.9138   2.97E+11
-254.297   5.76E+08       0.00
   11.2100   4.13E-07      2318.   -164.526  -1.44E-07   302.9004   2.97E+11
-171.090   5.87E+08       0.00
   11.3280   2.17E-07      1913.   -350.345  -1.34E-07   302.8121   2.97E+11
-91.366   5.97E+08       0.00
   11.4460   3.33E-08      1326.   -425.155  -1.26E-07   302.6838   2.97E+11
-14.298   6.08E+08       0.00
   11.5640  -1.41E-07   709.3443   -391.694  -1.21E-07   302.5493   2.97E+11
61.5585   6.18E+08       0.00
   11.6820  -3.11E-07   216.4717   -250.511  -1.19E-07   302.4417   2.97E+11
137.8522   6.28E+08       0.00
   11.8000  -4.79E-07       0.00       0.00  -1.19E-07   302.3944   2.97E+11
215.9774   3.19E+08       0.00

* The above values of total stress are combined axial and bending stresses.


Output Summary for Load Case No. 1:

Pile-head deflection              =     0.01352035 inches
Computed slope at pile head       =       0.000000 radians
Maximum bending moment            =      -2883754. inch-lbs
Maximum shear force               =       -159944. lbs
Depth of maximum bending moment   =       0.000000 feet below pile head
Depth of maximum shear force      =     7.67000000 feet below pile head
Number of iterations              =              6
Number of zero deflection points =              3


--------------------------------------------------------------------------------
            Summary of Pile-head Responses for Conventional Analyses
--------------------------------------------------------------------------------

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians


Load Load                Load                  Axial    Pile-head  Pile-head  Max
Shear Max Moment
Case Type   Pile-head    Type     Pile-head   Loading  Deflection  Rotation    in
```

```
Pile    in Pile
 No.  1      Load 1       2         Load 2       lbs       inches     radians      lbs
     in-lbs
---- ----- ---------- ---------- ---------- ---------- ---------- ----------
---------- ----------
  1  V, lb     63620.   S, rad         0.00    307800.    0.01352       0.00
-159944.  -2883754.
```

Maximum pile-head deflection = 0.0135203475 inches
Maximum pile-head rotation   = -0.0000000000 radians = -0.000000 deg.

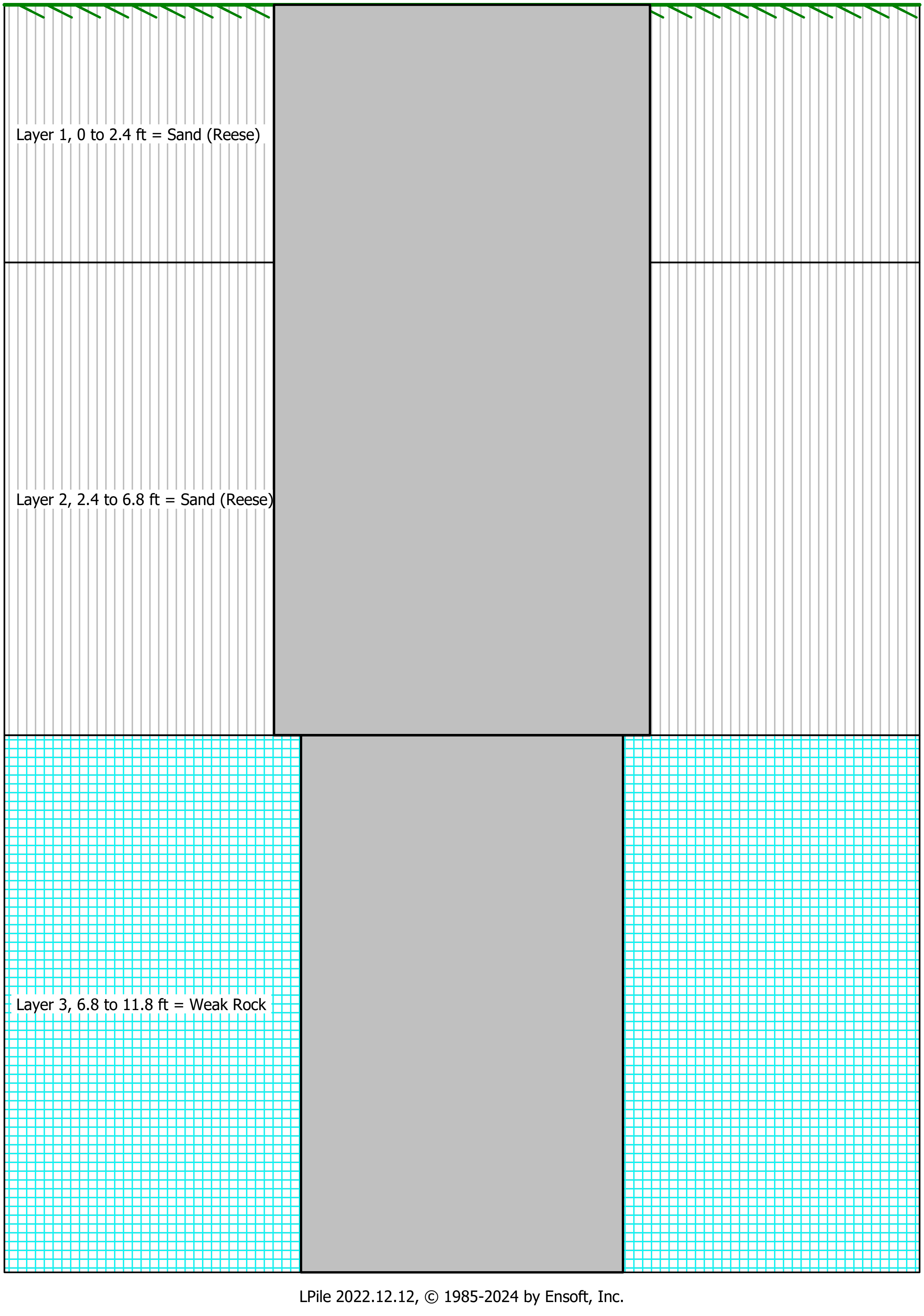
```
--------------------------------------------------------------------------------
                           Summary of Warning Messages
--------------------------------------------------------------------------------
```

The following warning was reported 301 times

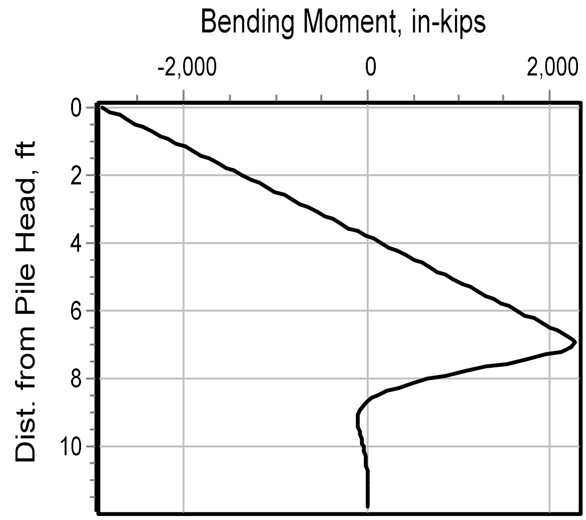
**** Warning ****

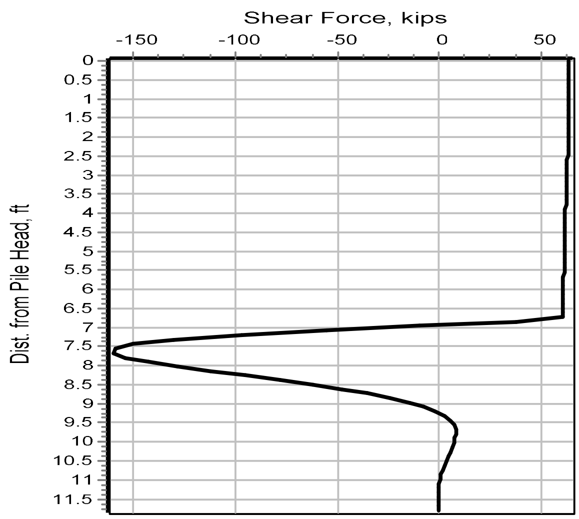
An unreasonable input value for unconfined compressive strength has been
specified for a soil defined using the weak rock criteria. The input value
is greater than 500 psi. Please check your input data for correctness.

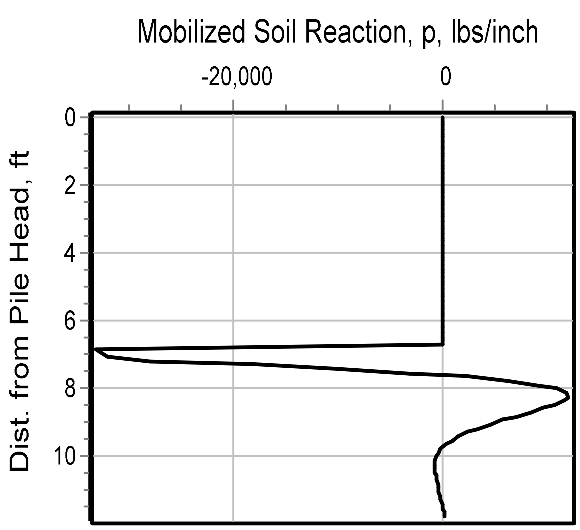
The analysis ended normally.

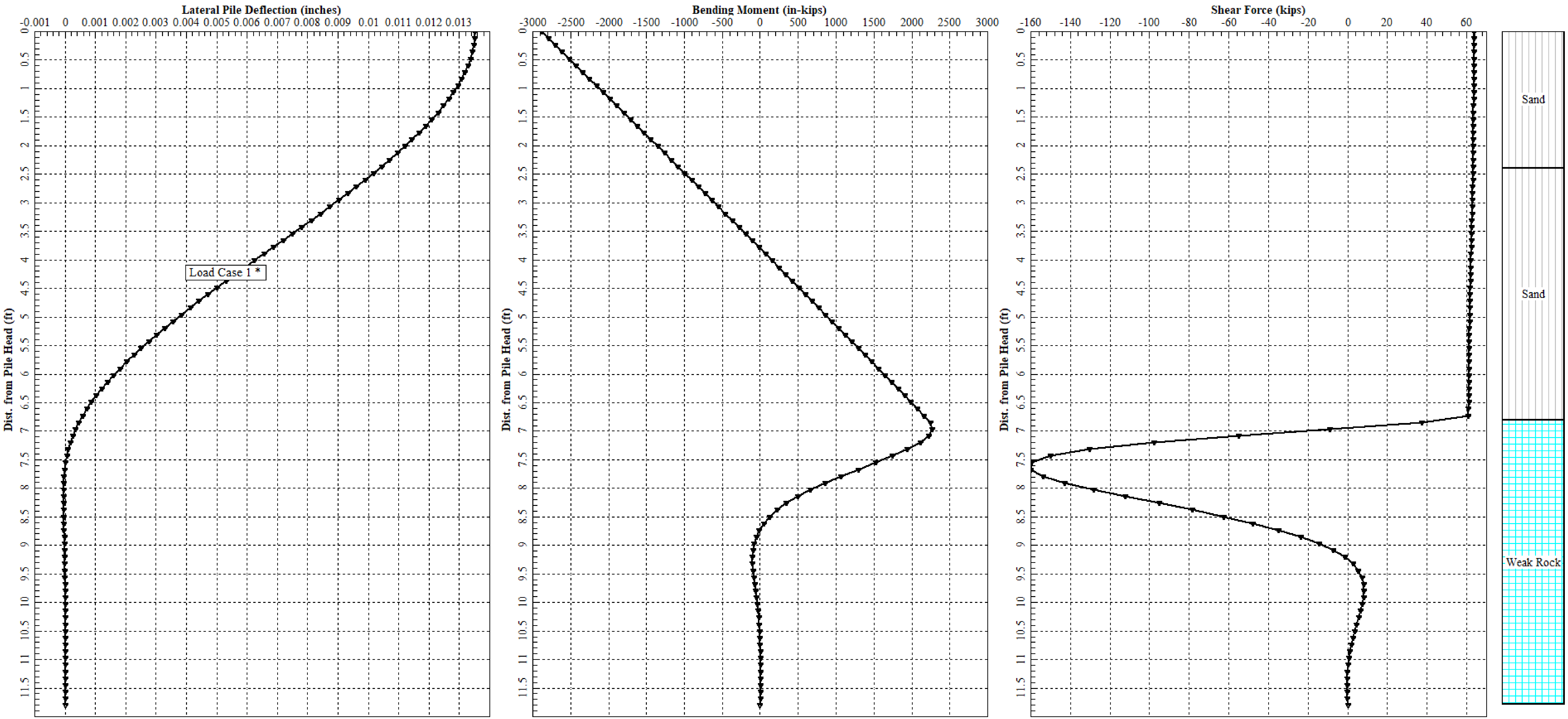

Layer 1, 0 to 2.4 ft = Sand (Reese)
Layer 2, 2.4 to 6.8 ft = Sand (Reese)
Layer 3, 6.8 to 11.8 ft = Weak Rock


Deflection, inches
0
0.005
0.01
Dist. from Pile Head, ft


Bending Moment, in-kips
-2,000
0
2,000
Dist. from Pile Head, ft


Shear Force, kips
-150
-100
-50
0
50
Dist. from Pile Head, ft


Mobilized Soil Reaction, p, lbs/inch
-20,000
0
Dist. from Pile Head, ft



Lateral Pile Deflection (inches)
-0.001 0 0.001 0.002 0.003 0.004 0.005 0.006 0.007 0.008 0.009 0.01 0.011 0.012 0.013
Load Case 1 *
Dist. from Pile Head (ft)
0 0.5 1 1.5 2 2.5 3 3.5 4 4.5 5 5.5 6 6.5 7 7.5 8 8.5 9 9.5 10 10.5 11 11.5
Bending Moment (in-kips)
-3000 -2500 -2000 -1500 -1000 -500 0 500 1000 1500 2000 2500 3000
Shear Force (kips)
-160 -140 -120 -100 -80 -60 -40 -20 0 20 40 60
Sand
Sand
Weak Rock

Project Name: ATB Old Main Street Bridge Replacement
Project Number: 232245
Calculated by: Bikal Sah
Reviewed By: Imad El. Hajjar / MS Iqbal 12/17/2025 Rev2

Scour Determination

Upper Elevation Limit for Analysis = 588.09 feet, based on 100-year floodplain
Lower Elevation Limit for Analysis = 563.56 feet, based on 6 feet below bottom of river

Table 2. Scour Parameters for Cored Rock

Boring Number	Rock Core Run No.	Depth (feet)	Approximate Elevation (feet)	Unconfined Compressive Strength, Q_u (psi)	Slake Durability Index, S_{DI} (percent)	Rock Quality Designation, RQD (percent)	Unit Weight (pcf)	Rock Mass Rating, RMR (Superseded by GSI)	Geologic Strength Index, GSI	Erodibility Index, K	Critical Shear Stress, τ_c (Pa) as per GDM 1302.1.2		Critical Shear Stress, τ_c (psf) (Unit Conversion)
Forward Abutment													
B-001-0-23	NQ2-8	15.7 - 20.7	577 - 572	2730	52.5	0	150	37	25	1.13	$\tau_c = \rho*\{(1000*K^{0.75})/(7.853*\rho)\}^{2/3}$ where K is Erodibility Index and ρ = 1000 kg/m^3	269.0	5.62
B-001-0-23	NQ2-8	15.7 - 20.7	577 - 572	2860	Not Tested	0	150	37	25	1.18		275.3	5.75
B-001-0-23	NQ2-9	20.7 - 25.7	572 - 567	3350	Not Tested	0	150	37	25	1.39		298.0	6.22
Pier													
B-002-0-23	NQ2-2	0.9 - 5.9	569.2 - 564.2	2280	54.1	0	150	37	35	0.47	$\tau_c = \rho*\{(1000*K^{0.75})/(7.853*\rho)\}^{2/3}$ where K is Erodibility Index and ρ = 1000 kg/m^3	173.8	3.63
B-002-0-23	NQ2-2	0.9 - 5.9	569.2 - 564.2	5630	Not Tested	0	150	39	35	1.16		273.1	5.70
B-002-0-23	NQ2-3	5.9 - 10.9	564.2 - 559.2	5170	Not Tested	0	150	39	35	1.07		261.7	5.47
Rear Abutment													
B-003-0-23	NQ2-12	24.5 - 29.5	567.2 - 562.2	7042	70.1	0	150	39	28	1.46	$\tau_c = \rho*\{(1000*K^{0.75})/(7.853*\rho)\}^{2/3}$ where K is Erodibility Index and ρ = 1000 kg/m^3	305.5	6.38
B-003-0-23	NQ2-12	24.5 - 29.5	567.2 - 562.2	2581	Not Tested	0	150	37	28	0.53		184.9	3.86
B-003-0-23	NQ2-13	29.5 - 34.5	562.2 - 557.2	4994	Not Tested	0	150	39	28	1.03		257.2	5.37
B-003-0-23	NQ2-13	29.5 - 34.5	562.2 - 557.2	7473	Not Tested	0	150	39	28	1.55		314.7	6.57

[1] Q_u is average of two tested specimens for NQ2.

[2] For actual depths and elevations of the tested samples, please refer to Point Load and Slake Durability Test results.

Project Name:	ATB Old Main Street Bridge Replacement
Project Number:	232245
Calculated by:	Bikal Sah
Reviewed By:	MS Iqbal - 12/17/2025 Rev2

RMR Calculations

Note: RMR has been superseded by GSI, but is included herein for comparison

B-001-0-23 (NQ2-8)

Parameter	Value	Note
1	2	Qu =393 ksf
2	3	RQD = 0%
3	20	Highly Fractured - Most <2"
4	12	Shale, Narrow apperture
5	0	Below groundwater, gray coloring
RMR =	**37**	

B-001-0-23 (NQ2-8)

Parameter	Value	Note
1	2	Qu =412 ksf
2	3	RQD = 0%
3	20	Highly Fractured - Most <2"
4	12	Shale, Narrow apperture
5	0	Below groundwater, gray coloring
RMR =	**37**	

B-001-0-23 (NQ2-9)

Parameter	Value	Note
1	2	Qu =482 ksf
2	3	RQD = 0%
3	20	Highly Fractured - Most <2"
4	12	Shale, Narrow apperture
5	0	Below groundwater, gray coloring
RMR =	**37**	

B-002-0-23 (NQ2-2)

Parameter	Value	Note
1	2	Qu =328 ksf
2	3	RQD = 0%
3	20	Highly Fractured - Most <2"
4	12	Shale, Narrow to Tight apperture
5	0	Below groundwater, gray coloring
RMR =	**37**	

B-002-0-23 (NQ2-2)

Parameter	Value	Note
1	4	Qu =811 ksf
2	3	RQD = 0%
3	20	Highly Fractured - Most <2"
4	12	Shale, Narrow apperture
5	0	Below groundwater, gray coloring
RMR =	**39**	

Project Name:	ATB Old Main Street Bridge Replacement
Project Number:	232245
Calculated by:	Bikal Sah
Reviewed By:	MS Iqbal - 12/17/2025 Rev2

RMR Calculations

Note: RMR has been superseded by GSI, but is included herein for comparison

B-002-0-23 (NQ2-3)

Parameter	Value	Note
1	4	Qu =744 ksf
2	3	RQD = 0%
3	20	Highly Fractured - Most <2"
4	12	Shale, Narrow apperture
5	0	Below groundwater, gray coloring
RMR =	**39**	

B-003-0-23 (NQ2-12)

Parameter	Value	Note
1	4	Qu =1014 ksf
2	3	RQD = 0%
3	20	Highly Fractured - Most <2"
4	12	Weathered rough Shale, Narrow to Tight apperture
5	0	Below groundwater, gray coloring
RMR =	**39**	

B-003-0-23 (NQ2-12)

Parameter	Value	Note
1	2	Qu =372 ksf
2	3	RQD = 0%
3	20	Mostly Fractured - >2" pieces
4	12	Shale, Narrow apperture
5	0	Below groundwater, gray coloring
RMR =	**37**	

B-003-0-23 (NQ2-13)

Parameter	Value	Note
1	4	Qu =719 ksf
2	3	RQD = 0%
3	20	Highly Fractured - Most <2"
4	12	Shale, Narrow apperture
5	0	Below groundwater, gray coloring
RMR =	**39**	

B-003-0-23 (NQ2-13)

Parameter	Value	Note
1	4	Qu =1076 ksf
2	3	RQD = 0%
3	20	Highly Fractured - Most <2"
4	12	Weathered rough Shale, Narrow to Tight apperture
5	0	Below groundwater, gray coloring
RMR =	**39**	

Project Name:	ATB Old Main Street Bridge Replacement
Project Number:	232245
Calculated by:	MS Iqbal
Reviewed By:	MS Iqbal - 12/12/2025 Rev2

Geological Strength Index (GSI)

as per AASHTO LRFD Figure 10.4.6.4-1 (Hoek and Marinos, 2000)

B-001-0-23 (NQ2-8)

Parameter	Description	Upper Value	Lower Value	Note
Structure	Disintegrated	30	20	Rock mass was significant disintegrated (Highly Fractured Shale).
Surface Conditions	Poor	30	20	Surface quality is fair to Poor. However, Poor is considered conservatively.
GSI (Average)		**25**		

B-001-0-23 (NQ2-8)

Parameter	**Value**	**Note**		
Parameter	**Description**	**Upper Value**	**Lower Value**	**Note**
Structure	Disintegrated	30	20	Rock mass was significant disintegrated (Highly Fractured Shale).
Surface Conditions	Poor	30	20	Surface quality is fair to Poor. However, Poor is considered conservatively.
GSI (Average)		**25**		

B-001-0-23 (NQ2-9)

Parameter	Description	Upper Value	Lower Value	Note
Structure	Disintegrated	30	20	Rock mass was significant disintegrated (Highly Fractured Shale).
Surface Conditions	Poor	30	20	Surface quality is fair to Poor. However, Poor is considered conservatively.
GSI (Average)		**25**		

B-002-0-23 (NQ2-2)

Parameter	Description	Upper Value	Lower Value	Note
Structure	Blocky / Disturbed / Seamy to Disintegrated	45	25	Rock mass was significant disintegrated.
Surface Conditions	Fair to Poor	45	25	Surface quality is fair to Poor.
GSI (Average)		**35**		

B-002-0-23 (NQ2-2)

Parameter	Description	Upper Value	Lower Value	Note
Structure	Blocky / Disturbed / Seamy to Disintegrated	45	25	Rock mass was significant disintegrated.
Surface Conditions	Fair to Poor	45	25	Surface quality is fair to Poor.
GSI (Average)		**35**		

B-002-0-23 (NQ2-3)

Parameter	Description	Upper Value	Lower Value	Note
Structure	Blocky / Disturbed / Seamy to Disintegrated	45	25	Rock mass was significant disintegrated.
Surface Conditions	Fair to Poor	45	25	Surface quality is fair to Poor.
GSI (Average)		**35**		

Project Name: ATB Old Main Street Bridge Replacement
Project Number: 232245
Calculated by: MS Iqbal
Reviewed By: MS Iqbal - 12/12/2025 Rev2

Geological Strength Index (GSI)

as per AASHTO LRFD Figure 10.4.6.4-1 (Hoek and Marinos, 2000)

B-003-0-23 (NQ2-12)

Parameter	**Value**	**Note**		
Parameter	**Description**	**Upper Value**	**Lower Value**	**Note**
Structure	Blocky / Disturbed / Seamy to Disintegrated	35	20	Rock mass was significant disintegrated (schistosity).
Surface Conditions	Fair to Poor	35	20	Surface quality is fair to Poor.
GSI (Average)		**28**		

B-003-0-23 (NQ2-12)

Parameter	**Description**	**Upper Value**	**Lower Value**	**Note**
Structure	Blocky / Disturbed / Seamy to Disintegrated	35	20	Rock mass was significant disintegrated (schistosity).
Surface Conditions	Fair to Poor	35	20	Surface quality is fair to Poor.
GSI (Average)		**28**		

B-003-0-23 (NQ2-13)

Parameter	**Description**	**Upper Value**	**Lower Value**	**Note**
Structure	Blocky / Disturbed / Seamy to Disintegrated	35	20	Rock mass was significant disintegrated (schistosity).
Surface Conditions	Fair to Poor	35	20	Surface quality is fair to Poor.
GSI (Average)		**28**		

B-003-0-23 (NQ2-13)

Parameter	**Description**	**Upper Value**	**Lower Value**	**Note**
Structure	Blocky / Disturbed / Seamy to Disintegrated	35	20	Rock mass was significant disintegrated (schistosity).
Surface Conditions	Fair to Poor	35	20	Surface quality is fair to Poor.
GSI (Average)		**28**		

Project Name:	ATB Old Main Street Bridge Replacement
Project Number:	232245
Calculated by:	Bikal Sah
Reviewed By:	msi, 12/17/2025

Erodability Index (K) Calculations

B-001-0-23 (NQ2-8)

Parameter	Value	Note
J_s (0.4 min)	0.4	Indecipherable due to wire line coring method
J_n (5 max)	5	Highly Fractured - Most <2"
J_r (1 min)	3	Mostly Undulating Joint-Rough surface
J_a (5 max)	2	Slightly altered joint walls, sandy particles seen in joint
RQD	0	
K_b (0.10 min)	0.1	
K_d	1.5	
Q_u (MPa)	18.8	
M_s (MPa)	18.8	
K	**1**	

B-001-0-23 (NQ2-8)

Parameter	Value	Note
J_s (0.4 min)	0.4	Indecipherable due to wire line coring method
J_n (5 max)	5	Highly Fractured - Most <2"
J_r (1 min)	3	Mostly Undulating Joint-Rough surface
J_a (5 max)	2	**Slightly altered joint walls, sandy particles seen in joint**
RQD	0	
K_b (0.10 min)	0.1	
K_d	1.5	
Q_u (MPa)	19.7	
M_s (MPa)	19.7	
K	**1.18**	

B-001-0-23 (NQ2-9)

Parameter	Value	Note
J_s (0.4 min)	0.4	Indecipherable due to wire line coring method
J_n (5 max)	5	Highly Fractured - Most <2"
J_r (1 min)	3	Mostly Undulating Joint-Rough surface
J_a (5 max)	2	**Slightly altered joint walls, sandy particles seen in joint**
RQD	0	
K_b (0.10 min)	0.1	
K_d	1.5	
Q_u (MPa)	23.1	
M_s (MPa)	23.1	
K	**1.39**	

Project Name:	ATB Old Main Street Bridge Replacement
Project Number:	232245
Calculated by:	Bikal Sah
Reviewed By:	msi, 12/17/2025

Erodability Index (K) Calculations

B-002-0-23 (NQ2-2)

Parameter	Value	Note
J_s (0.4 min)	0.4	Indecipherable due to wire line coring method
J_n (5 max)	5	Highly Fractured - Most <2"
J_r (1 min)	1.5	Undulating, Slickensided
J_a (5 max)	2	**Slightly altered joint walls, sandy particles seen in joint**
RQD	0	
K_b (0.10 min)	0.1	
K_d	0.75	
Q_u (MPa)	15.7	
M_s (MPa)	15.7	
K	**0.47**	

B-002-0-23 (NQ2-2)

Parameter	Value	Note
J_s (0.4 min)	0.4	Indecipherable due to wire line coring method
J_n (5 max)	5	Highly Fractured - Most <2"
J_r (1 min)	1.5	Undulating, Slickensided
J_a (5 max)	2	Slightly altered joint walls, sandy particles seen in joint
RQD	0	
K_b (0.10 min)	0.1	As RQD is zero, min. K_b is considered
K_d	0.75	
Q_u (MPa)	38.8	
M_s (MPa)	38.8	
K	**1.16**	

B-002-0-23 (NQ2-3)

Parameter	Value	Note
J_s (0.4 min)	0.4	Indecipherable due to wire line coring method
J_n (5 max)	5	Highly Fractured - Most <2"
J_r (1 min)	1.5	Undulating, Slickensided
J_a (5 max)	2	Slightly altered joint walls, sandy particles seen in joint
RQD	0	
K_b (0.10 min)	0.1	As RQD is zero, min. K_b is considered
K_d	0.75	
Q_u (MPa)	35.6	
M_s (MPa)	35.6	
K	**1.07**	

Project Name:	ATB Old Main Street Bridge Replacement
Project Number:	232245
Calculated by:	Bikal Sah
Reviewed By:	msi, 12/17/2025

Erodability Index (K) Calculations

B-003-0-23 (NQ2-12)

Parameter	Value	Note
J_s (0.4 min)	0.4	Indecipherable due to wire line coring method
J_n (5 max)	5	Highly Fractured - Most <2"
J_r (1 min)	1.5	Undulating, Slickensided
J_a (5 max)	2	Slightly altered joint walls, sandy particles seen in joint
RQD	0	
K_b (0.10 min)	0.1	As RQD is zero, min. K_b is considered
K_d	0.75	
Q_u (MPa)	48.6	
M_s (MPa)	48.6	
K	**1.46**	

B-003-0-23 (NQ2-12)

Parameter	Value	Note
J_s (0.4 min)	0.4	Indecipherable due to wire line coring method
J_n (5 max)	5	Mostly Fractured - >2" pieces
J_r (1 min)	1.5	Undulating, Slickensided
J_a (5 max)	2	Slightly altered joint walls, sandy particles seen in joint
RQD	0	
K_b (0.10 min)	0.1	As RQD is zero, min. K_b is considered
K_d	0.75	
Q_u (MPa)	17.8	
M_s (MPa)	17.8	
K	**0.53**	

B-003-0-23 (NQ2-13)

Parameter	Value	Note
J_s (0.4 min)	0.4	Indecipherable due to wire line coring method
J_n (5 max)	5	Highly Fractured - Most <2"
J_r (1 min)	1.5	Undulating, Slickensided
J_a (5 max)	2	Slightly altered joint walls, sandy particles seen in joint
RQD	0	
K_b (0.10 min)	0.1	As RQD is zero, min. K_b is considered
K_d	0.75	
Q_u (MPa)	34.4	
M_s (MPa)	34.4	
K	**1.03**	

B-003-0-23 (NQ2-13)

Parameter	Value	Note
J_s (0.4 min)	0.4	Indecipherable due to wire line coring method
J_n (5 max)	5	Highly Fractured - Most <2"
J_r (1 min)	1.5	Undulating, Slickensided
J_a (5 max)	2	Slightly altered joint walls, sandy particles seen in joint
RQD	0	
K_b (0.10 min)	0.1	As RQD is zero, min. K_b is considered
K_d	0.75	
Q_u (MPa)	51.5	
M_s (MPa)	51.5	
K	**1.55**	

By:	msi	Date:	12/16/2025	Checked:	IJH	Date:	12/16/2025

GENERAL FOUNDATION INFORMATION:

Central Pier:

Bottom Elevation	565	ft	Approximate bearing elevation
W or B	9	ft	Width of Pier
L	35	ft	Length of Pier
$\gamma_{concrete}$	0.15	Kcf	Unit weight of concrete

ECCENTRICITY, e in the Direction of Width of Pier (B), Z-Direction GDM 1303.1.2

$V_{strength\ Load}$	2,095.71	kips	Maximum applied factored load at strength limit state for soil pressure of 10.5 ksf.
M_{Pier}	3534.32	kip-ft/ft	Sum of moment in Z-direction at strength limit state for soil pressure of 10.5 ksf.

$e_B = \Sigma M / \Sigma V$

$\boxed{e_B = 1.69}$

ECCENTRICITY, e in the Direction of Length of Pier (L), X-Direction GDM 1303.1.2

$V_{strength\ Load}$	2095.71	kips	Maximum applied factored load at strength limit state for soil pressure of 10.5 ksf.
M_{Pier}	915.36	kip-ft/ft	Sum of moment in X-direction at strength limit state for soil pressure of 10.5 ksf.

$e_L = \Sigma M / \Sigma V$

$\boxed{e_L = 0.44}$

LIMITING ECCENTRICITY, e_{Limit}

in the Direction of Width of Pier (B), Z-Direction AASHTO LRFD 10.6.3.3

$e_{Limit} = 0.45 * B$

$\boxed{e_{Limit} = 4.05}$

in the Direction of Length of Pier (L), X-Direction AASHTO LRFD 10.6.3.3

$e_{Limit} = 0.45 * L$

$\boxed{e_{Limit} = 15.75}$

EFFECTIVE FOOTING DIMENSIONS AASHTO LRFD 10.6.1.3

$B' = B - 2e_B$

B' =	5.63	ft.
B' =	6.00	ft. approx.

$L' = L - 2e_L$

L' =	34.13	ft.
L' =	34.00	ft. approx.

Date: September 17, 2025

Project Name:	Old Main Street Bridge
Project No.	232245
Calculated by	MSI
Checked by	IHJ, 10/23/2025
Method	LRFD Shallow Foundation on Rock
Structure	Central Pier
Boring ID	B-002-0-23
Severely weathered weak and highly fractured gray shale is exposed at elevation of 570 feet.	
STRENGTH LIMIT STATE DESIGN	
As per Geotechnical Design Manual (GDM) Section 1303.3.3 • Bedrock slope of 2H:1V or less • Rock Mass Rating (RMR) ≤ 70 • q_u ≤7,500 psi then calculate drained shear strength properties (c' and ϕ') in accordance with Bieniawski (1989). The Bieniawski (1989) drained shear strength equations are as follows: c' = 0.104 x RMR (ksf) ϕ' = RMR/2 + 5° (deg)	
RMR = 39 q_u = 5,630 psi Unit Weight, γ = 150 pcf	RMR [4+3+20+12+0 = 39] is computed based on the recovered rock cores at Elevation 565 in boring B-002-0-23. Point load test at Elevation 565 feet resulted a UCS of 5,630 psi.
Drained Cohesion c' = 0.104 x 39 = 4.056 ksf = **4,056** psf	
Drained Friction Angle ϕ' = 39/2 + 5 = 24.5° ≈ **25**°	
Footing Dimensions Width, B = 9 feet, Effective Width, B' = 6 feet Length, L = 35 feet, Effective Length, L' = 34 feet Footing Depth, D_f = 3/12 feet	
Based on AASHTO 10.6.3.1.2a Nominal bearing resistance of spread footing on cohesionless soils	

Date: September 17, 2025

$q_n = cN_{cm} + \gamma_q D_f N_{qm} C_{wq} + 0.5\gamma_f B N_{\gamma m} C_{w\gamma}$ (10.6.3.1.2a-1) in which: $N_{cm} = N_c s_c i_c$ (10.6.3.1.2a-2) $N_{qm} = N_q s_q d_q i_q$ (10.6.3.1.2a-3) $N_\gamma m = N_\gamma s_\gamma i_\gamma$ (10.6.3.1.2a-4)	
N_c = 20.7	AASHTO LRFD Table 10.6.3.1.2a-1
N_q = 10.7	AASHTO LRFD Table 10.6.3.1.2a-1
N_γ = 10.9	AASHTO LRFD Table 10.6.3.1.2a-1
i_c = 1	No inclination
i_q = 1	No inclination
i_γ = 1	No inclination
s_c = 1.0912	1+ (B'/L')(N_q/N_c) AASHTO Table 10.6.3.1.2a-3
s_γ = 0.9294	1 - 0.4 x (B'/L') AASHTO Table 10.6.3.1.2a-3
s_q = 1.08229	1+((B'/L')tan ϕ) AASHTO Table 10.6.3.1.2a-3
d_q = 1	$d_q = 1 + 2\tan\phi_f(1-\sin\phi_f)^2 \arctan\left(\frac{D_f}{B}\right)$ (10.6.3.1.2a-10)
C_{wq} = 0.5	D_w = 0 AASHTO Table 10.6.3.1.2a-2
$C_{w\gamma}$ = 0.5	D_w = 0 AASHTO Table 10.6.3.1.2a-2
N_{cm} =20.7 x 1.0912 x1 = 22.5878	AASHTO LRFD 10.6.3.1.2a-2
N_{qm} = 10.7 x 1.08229 x 1 x 1 = 11.5805	AASHTO LRFD 10.6.3.1.2a-3
$N_{\gamma m}$ = 10.9 x 0.9294 x 1 = 10.1305	AASHTO LRFD 10.6.3.1.2a-4
Nomila Bearing Resistance, q_n	AASHTO LRFD 10.6.3.1.2a-1
q_n = (4056 x 22.5878) + (150 x 0.25 x 11.5805 x 0.5) + (0.5 x 150 x 9 x 10.1305 x 0.5) q_n = **95,252.29** psf = 95.3 ksf	
For Piers, footing on rock Reduction Factor, $\varnothing_b$ = 0.45	AASHTO LRFD Table 10.5.5.2.2-1

Bearing Capacity Factored, q_r q_r = 95,252.29 x 0.45 = 42, 863.53 psf ≈ **42.86** ksf
SERVICE LIMIT STATE DESIGN
The calculated unfactored bearing pressure of 0.30 ksi is significantly lower than the estimated rock mass modulus of 605 ksi, which satisfies the criterion outlined in GDM Section 1303.2.1—that bearing stress should be less than 50 times the rock mass modulus to assume negligible settlement. Therefore, no adjustments to the bearing pressure are required at this structure location. Supporting calculations for the rock mass modulus are provided in the attached documentation.

Project Name: ATB Old Main St.	Calculations by: Muhammad Shahid Iqbal
	Checked by: IHJ, 10/23/2025

GDM Section 1303.2.1

Settlement of foundations bearing on bedrock may be assumed to be negligible if the maximum Service Limit State bearing stress is less than fifty (50) times the rock mass modulus, E_m.

STEPS		REFERENCES
Boring No.	B-002-0-23	
Structure	Central Pier	
Rock Core Elev. (feet)	569.2 to 564.2 (NQ2-2)	Boring log
RQD	0	-do-
Recovery	100	-do-
Strength of Rock Core, q_u (psi)	5,630	Point load test at Elevation 565 feet resulted a UCS of 5,630 psi.
Strength of Rock Core, q_u (MPa)	38.82	Unit conversion
Rock Mass Rating (RMR)	39	Bieniawski (1989) RMR [4+3+20+12+0 = 39] is computed based on the recovered rock cores at Elevation 565 in boring B-002-0-23.
GSI	35	Based on Scour Analysis Table
Rock Mass Modulus, E_m (GPa) $E_m\left(GPa\right)=\sqrt{\frac{q_u}{100}}10^{\frac{GSI-10}{40}}$ for $q_u \le 100$ MPa $E_m\left(GPa\right)=10^{\frac{GSI-10}{40}}$ for $q_u > 100$ MPa	0.0197 x 4.2170 = 0.0831	Hoek and Brown 1997; Hoek et al. 2002; AASHTO LRFD Table 10.4.6.5.1
Approx. E_m (ksi)	12.1	After unit conversion
50 x E_m based on SPT (ksi)	605	GDM Section 1303.2.1
The unfactored LRFD bearing pressure (ksf)	95.3	Calculations are attached
The unfactored LRFD bearing pressure (ksi)	0.30	Unit conversion

The bearing pressure is less than 50 times E_m of the severely weathered rock. Therefore, the settlement is negligible.

By:	msi	Date:	12/12/2025 Rev2	Checked:	IJH	Date:	10/23/2025

GENERAL FOUNDATION INFORMATION:

Forward Abutment:

Bottom Elevation	572	ft	Approximate bearing elevation	
B	9	ft	Width	B is the governing direction for this structure
L	48	ft	Length	
$\gamma_{concrete}$	0.15	Kcf	Unit weight	

ECCENTRICITY, e, in Governing Direction

GDM 1303.1.2

NOTE: At strength 1b. Provided by Structural Engineer.

$e_B = \Sigma M / \Sigma V$

ΣM_O = 136.22

ΣM_R = 308.65 — Distance from point O the resultant intersect with the base.

ΣV = 72.30

x = 2.38 $x = (\Sigma M_O - \Sigma M_R) / \Sigma V$

$\boxed{e_B = 2.12}$ $e = B/2 - x$ — Wall Eccentricity

LIMITING ECCENTRICITY, e_{Limit}

In Governing Direction — AASHTO LRFD 10.6.3.3

$e_{Limit} = 0.45 * B$

$\boxed{e_{Limit} = 4.05}$

EFFECTIVE FOOTING DIMENSIONS — AASHTO LRFD 10.6.1.3

$B' = B - 2e_B$

B' =	4.77	ft.
B' =	5.00	ft. approx.

$L' = L - 2e_L$ — NOTE: Not applicable

By: msi Date: 10/14/2025 Checked: IJH Date: 10/24/2025

GENERAL FOUNDATION INFORMATION:

Forward Abutment: Width W = 5.3' , Length L = 48' Width = 5.3' is effective width
Bearing at approxiamtely 572 feet.

GENERAL SOIL INFORMATION:

Anticipated Bearing Conditions:

Predominantly severely weathered rock.
The weathered rock is assumed to behave like cohesionless granular material.
The estimated UCS is 19.5 psi based on SPT data.

Based on Soil Strength Evaluation Spreadsheet,

USE c' =	0	ksf - cohesionless soil
USE ϕ'=	30	Degrees

Groundwater

Model groundwater in creek above foundation bearing elevation.

STRENGTH LIMIT STATE:

$q_R = \varphi_b * q_n$ (AASTHO LRFD 10.6.3.1.1-1)

q_R = factored resistance at strength limit state (ksf)
φ_b = resistance factor (Article 10.5.5.2.2)
q_n = nominal bearing resistance (ksf)

$q_n = cN_{cm} + \gamma_q D_f N_{qm} C_{wq} + 0.5\gamma_f B N_{gm} C_{wg}$ (AASTHO LRFD 10.6.3.1.2a-1)
$N_{cm} = N_c s_c i_c$ (AASTHO LRFD 10.6.3.1.2a-2)
$N_{qm} = N_q s_q d_q i_q$ (AASTHO LRFD 10.6.3.1.2a-3)
$N_{gm} = N_g s_g i_g$ (AASTHO LRFD 10.6.3.1.2a-4)

c = cohesion, undrained shear strength (ksf)
N_c = cohesion term (Table 10.6.3.1.2a-1)
N_q = surcharge term (Table 10.6.3.1.2a-1)
N_g = unit weight term (Table 10.6.3.1.2a-1)
g = total (moist) unit weight (kcf)
D_f = footing embedment depth (ft)
B = footing width (ft)
C_{wq} , C_{wg} = groundwater correction factors (Table 10.6.3.1.2a-2)
s_c , s_g , s_q = shape correction factors (Table 10.6.3.1.2a-3)
d_q = shear resistance thought cohesionless material correction factor (Table 10.6.3.1.2a-4)
i_c , i_g , i_q = inclination correction factors

By:	msi	Date:	10/14/2025	Checked:	IJH	Date:	10/24/2025

Setup	$c =$	0	ksf	assumed zero in cohesionless soil	
	$\phi_f =$	30	degrees		
	$N_c =$	30.1	*unitless*		
	$N_q =$	18.4	*unitless*	for soil with a $\phi_f = 30$ Degrees	
	$N_\gamma =$	22.4	*unitless*		
	$\gamma =$	0.125	kcf	(assumed in upper 1.5 feet above bearing)	
	$D_f =$	0.3	ft	Minimum embedment of the footing into rock	
	$B =$	5.3	ft	Width Stage 2 plans	
	$L =$	48	ft	Length Stage 2 plans	
	$D_w =$	0	ft	highest anticipated groundwater depth	
	$C_{wq} =$	0.5	*unitless*	where $D_w = 0.0$	$1.5B + D_f =$ 8.2
	$C_{w\gamma} =$	0.5	*unitless*	(above D_f)	
	$s_c =$	1.07	*unitless*	$s_c = 1 + (B/L)*(Nq/Nc)$	$s_c = 1 + (B/(5L))$
	$s_\gamma =$	0.96	*unitless*	for $\phi_f > 0$ $s_g = 1 - 0.4 * (B/L)$	for $\phi_f = 0$ $s_g = 1$
	$s_q =$	1.05	*unitless*	$s_q = 1+((B/L)*\tan\phi_f)$	$s_q = 1$
	$d_q =$	1.0	*unitless*	taken as 1 since cohesionless soil on top of weathered rock	
	i_c , i_γ , i_q =	1.0	*unitless*	Assumed loaded without inclination	

calculation	$N_{cm} = N_c s_c i_c$	= 30.1 * 1.067 * 1 =	32.117
	$N_{qm} = N_q s_q d_q i_q$	= 18.4 * 0.956 * 1 * 1	17.59
	$N_{gm} = N_g s_g i_g$	= 22.4 * 1.05 * 1 =	23.52

$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$

= (0*32.117) + (0.125*0.25*17.59*0.5) + (0.5*5.3*23.52*0.5) =

= (0) + (0.275) + (31.164) =

$cN_{cm} =$	0
$\gamma D_f N_{qm} C_{wq} =$	0.275
$0.5\gamma B N_{\gamma m} C_{w\gamma} =$	31.164

$q_n =$ 31.44 ksf

$\varphi_b =$ 0.55 AASHTO LRFD Table 11.5.7-1 - SLS Res. Factor for Perm. Retain. Walls

$q_R = \varphi_b * q_n$ = 0.55 * 31.439 = 17.29 ksf

Factored resistance at the strength limit state for the ABUTMENT footing bearing in the WEATHERED BEDROCK is equal to 17.3 ksf

By: msi Date: 10/14/2025 Checked: IJH Date: 10/24/2025

SERVICE LIMIT STATE:

Based on a factored bearing pressure of 17.3 ksf.
Referring to GDM Section 1303.2.1, the settlement of foundations bearing on bedrock may be assumed to be negligible if the maximum Service Limit State bearing stress is less than fifty (50) times the rock mass modulus, Em.

The unfactored bearing pressure is 0.12 ksi which is significantly lower than estimated 50^*E_m of weathered rock which is 55 ksi. Therefore, no adjustments to the bearing pressure are required at this structure location.
Supporting calculations for the rock mass modulus are provided in the attached documentation.

.

By: MSI Date: 10/20/2025 Checked: IHJ Date: 10/23/2025

GENERAL FOUNDATION INFORMATION:

Forward Abutment: Width W = 5.3' , Length L = 48' w = 5.3 is effective width
Bearing at approximately 572 feet.

GENERAL SOIL INFORMATION:

Anticipated Bearing Conditions:

Predominantly severely weathered rock.
The weathered rock is assumed to behave like cohesion less granular material.
The estimated UCS is 19.5 psi based on SPT data.

USE c' =	0	ksf	cohesionless soil
USE ϕ'=	30	Degrees	backfill material

Groundwater:

Model groundwater in creek above foundation bearing elevation.

FAILURE BY OVERTURNING:

Assumptions:

The rear abutment is assumed as semi-gravity cantilever wall

SETUP:

W	5.3	feet	*Width of fwd.abutment*
L	48	feet	*Length of fwd. abutment*
z	17.3	feet	*height of fwd. abutment*
b	25	feet	*length of the backfill material (assuming same as length of approach slab)*
$\gamma_{concrete}$	0.15	ksf	*Unit weight of the concrete*
P	12.16	ksf	*Maximum strength load for Fwd. Abutment*
V	5253	kips	*Estimated based on P (ksi) for Fwd. Abutment Wall with total area of 48' x 9'*
ϕ_f	30	degrees	*internal angle of friction of the soil (backfill)*
c	0	ksf	*cohesion of the soil (backfill)*
γ	0.125	kcf	*Unit weight of the soil (backfill)*
K_a	0.33	unitless	*Coefficient of active earth pressure (backfill)*

$$\mathbf{K_a = \tan^2 (45^0 - \phi_f /2)}$$

RESISTING MOMENTS:

A_{wall}	254.4	Area (W x L) of Wall (ft^2)	
$A_{wall\ base}$	75	Area (b x 3) of Wall Base (ft^2)	Based on the Fwd. Abutment wall sections
$A_{backfill}$	1200	Area (L x b) of Backfill (ft^2)	
P_{wall}	38.16	Weight / unit length of Wall (kip/ft)	
$P_{wall\ base}$	11.25	Weight / unit length of Wall Base (kip/ft)	
$P_{backfill}$	150	Weight / unit length of Backfill (kip/ft)	
X_{wall}	2.65	Moment Arm Wall (ft)	*Assumed half of the abutment width*
X_{wall} base	12.5	Moment Arm Wall Base (ft)	*Assumed half of the approach slab length*
$X_{backfill}$	12.5	Moment Arm Backfill (ft)	*Assumed half of the approach slab length*

Resisting Moment, M_r

M_{wall}	101.124
$M_{wall\ base}$	140.63
$M_{backfill}$	1875

M_{RT} 2,116.75 Total Resisting Moment (kip)

Overturning Moment, M_O

$P_a = 0.5 * K_a * \gamma * z^2$

$P_a =$ 6.24 kips/ft

$X_{backfill}$ 5.77 Moment Arm, typically taken at one-thrid of height of wall (ft)

$M_o = P_a * X_{backfill}$

$M_o =$ 35.96 kip

FACTOR OF SAFETY (FS):

$FS = M_r / M_O$

FS = 59

By: MSI Date: 10/20/2025 Checked: IHJ Date: 10/23/2025

GENERAL FOUNDATION INFORMATION:

Forward Abutment: Width W = 5.3' , Length L = 48' Width = 5.3' is effective width
Bearing at approximately 572 feet.

GENERAL SOIL INFORMATION:

Anticipated Bearing Conditions:

Predominantly severely weathered rock.
The weathered rock is assumed to behave like cohesion less granular material.
The estimated UCS is 19.5 psi based on SPT data.

USE c' =	0	ksf - cohesionless soil
USE ϕ'=	30	Degrees

Groundwater:

Model groundwater in creek above foundation bearing elevation.

FAILURE BY SLIDING: AASHTO LRFD 10.6.3.4

$R_R = \phi R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$ (kips) AASHTO LRFD 10.6.3.4-1

ϕ resistance factor (dim)
R_n nominal sliding resistance against failure by sliding (kips)
ϕ_τ resistance factor for shear resistance between soil and foundation specified in Table 10.5.5.2.2-1
R_τ nominal sliding resistance between soil and foundation (kips)
ϕ_{ep} resistance factor for passive resistance specified in Table 10.5.5.2.2-1
R_{ep} nominal passive resistance of soil available throughout the design life of the structure (kips)

for footings on the cohesionless soils

$R_\tau = CV \tan \phi_f$ AASHTO LRFD 10.6.3.4-2

C 1.0 for concrete cast against soil
0.8 for precast concrete footing
V total vertical force (kips)
ϕ_f internal friction angle of drained soil (degrees)

Since the forward abutment wall (assumed semi-gravity cantilever wall) rests on the severely weathered rock, assumed as cohesionless granular material, AASHTO LRFD 10.6.3.4-2 section is applicable.

SETUP:

C	1	unitless		
P	12.16	ksf	*Maximum strength load for Fwd. Abutment*	
V	5253	kips	*Estimated based on P (ksi) for Fwd. Abutment Wall with total area of 48' x 9'*	
ϕ_f	30	degrees	*Estimated based on overburden pressure and SPT data*	
ϕ_τ	0.8	unitless	*concrete on sand*	AASHTO LRFD Table 10.5.5.2.2.1
ϕ_{ep}	0.5	unitless		AASHTO LRFD Table 10.5.5.2.2.1
ϕ	1	unitless		AASHTO LRFD Table 11.5.7-1
γ	0.125	kcf	*Unit weight of the backfill material*	
K_p	3.00	unitless	*Coefficient of passive earth pressure (backfill)*	$\mathbf{K_p = \tan^2 (45^0 + \phi_f /2)}$
z	17.3	feet	*height of the fwd. abutment based on the stage 2 plans*	
c	0	ksf	*cohesion of the soil (backfill)*	

Nominal Sliding Resistance between Soil and Foundation (kips):

$R_\tau =$ 3,032.89

Nominal Passive Resistance of Soil (kips):

$R_{ep} = 0.5 * \gamma * z^2 * K_p$ AASHTO LRFD Figure 3.11.5.4-1

$R_{ep} =$ 56.12

Nominal Sliding Resistance against Failure by Sliding (kips):

$R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$

$R_n =$ 2,426.31 + 28.06

$R_n =$ 2,454.37

Factored Resistance against Sliding (kips): AASHTO LRFD 10.6.3.4

$R_R = \phi R_n$

$\boxed{R_R = 2,454.37}$

PID: 119860
Date: October 14, 2025

Project Name: ATB Old Main St.	Calculations by: Muhammad Shahid Iqbal
	Checked by: IHJ, 10/23/2025

GDM Section 1303.2.1

Settlement of foundations bearing on bedrock may be assumed to be negligible if the maximum Service Limit State bearing stress is less than fifty (50) times the rock mass modulus, E_m.

STEPS		REFERENCES
Boring No.	B-003-0-23	
N_{85} at Elev. 1156-feet (bpf)	22 + [(50/5”) x 12”] + [(50/5”) x 12”] = 262 blow per foot	BDM Section 404.3
N_{90} (bpf)	(72.5/90) x 262 = 212	-do-
q_u from SPT Data(ksf)	0.092 x N_{90} = 19.5	-do-
q_u from SPT Data(psi)	136	-do-
q_u from SPT Data (MPa)	0.93	General unit conversion
Geological Stregnth Index (GSI)	25	Assumed minimum value possible for severely weathered rock
Rock Mass Modulus, E_m (GPa) $E_m(GPa)=\sqrt{\frac{q_u}{100}}10^{\frac{GSI-10}{40}}$ for $q_u \le 100$ MPa $E_m(GPa)=10^{\frac{GSI-10}{40}}$ for $q_u > 100$ MPa	0.0031 x 2.3714 = 0.0072	Hoek and Brown 1997; Hoek et al. 2002; AASHTO LRFD Table 10.4.6.5.1
Approx. E_m (ksi), based on SPT Data	1.1	After unit conversion
50 x E_m based on SPT (ksi)	55	GDM Section 1303.2.1
The unfactored LRFD bearing pressure (ksf)	17.3	Calculations are attached
The unfactored LRFD bearing pressure (ksi)	0.12	Unit conversion

The bearing pressure is less than 50 times E_m of the severely weathered rock. Therefore, the settlement is negligible.

By:	msi	Date:	10/22/2025	Checked:	IJH	Date:	10/23/2025

GENERAL FOUNDATION INFORMATION:

Wingwall at Rear Abutment:

Bottom Elevation	580.1	ft	Approximate bearing elevation	
B	7	ft	Width	B is the governing direction for this structure
L	12.5	ft	Length	
$\gamma_{concrete}$	0.15	Kcf	Unit weight	

ECCENTRICITY, e, in Governing Direction GDM 1303.1.2

$e_B = \Sigma M / \Sigma V$ NOTE: At strength 1b. Provided by Structural Engineer.

ΣM_O 34.93

ΣM_R 55.58

ΣV 13.39

x 1.54 $x = (\Sigma M_O - \Sigma M_R)/ \Sigma V$ Distance from point O the resultant intersect with the base.

$e_B =$ 1.96 $e = B/2 - x$ Wall Eccentricity

LIMITING ECCENTRICITY, e_{Limit}

In Governing Direction AASHTO LRFD 10.6.3.3

$e_{Limit} = 0.45 * B$

$e_{Limit} =$ 3.15

EFFECTIVE FOOTING DIMENSIONS AASHTO LRFD 10.6.1.3

$B' = B - 2e_B$

B' = 3.08 ft.

B' = 3.00 ft. approx.

$L' = L - 2e_L$ NOTE: Not applicable

By:	msi	Date:	10/20/2025	Checked:	IJH	Date:	10/23/2025

GENERAL FOUNDATION INFORMATION:

Wingwall at Forward Abutment: Width W = 3' , Length L = 12.5'
Bearing at approxiamtely 580 feet.

GENERAL SOIL INFORMATION:

Anticipated Bearing Conditions:

Predominantly Very Stiff to Hard Soil underlain by 1' zone very dense sand and then weathered bedrock.

Based on Soil Strength Evaluation Spreadsheet,

USE c' =	0	ksf - cohesionless soil
USE ϕ'=	30	Degrees

Groundwater

Model groundwater in creek above foundation bearing elevation.

STRENGTH LIMIT STATE:

$q_R = \varphi_b * q_n$ (AASTHO LRFD 10.6.3.1.1-1)

q_R = factored resistance at strength limit state (ksf)
φ_b = resistance factor (Article 10.5.5.2.2)
q_n = nominal bearing resistance (ksf)

$q_n = cN_{cm} + \gamma_q D_f N_{qm} C_{wq} + 0.5\gamma_f B N_{gm} C_{wg}$ (AASTHO LRFD 10.6.3.1.2a-1)
$N_{cm} = N_c s_c i_c$ (AASTHO LRFD 10.6.3.1.2a-2)
$N_{qm} = N_q s_q d_q i_q$ (AASTHO LRFD 10.6.3.1.2a-3)
$N_{gm} = N_g s_g i_g$ (AASTHO LRFD 10.6.3.1.2a-4)

c = cohesion, undrained shear strength (ksf)
N_c = cohesion term (Table 10.6.3.1.2a-1)
N_q = surcharge term (Table 10.6.3.1.2a-1)
N_g = unit weight term (Table 10.6.3.1.2a-1)
g = total (moist) unit weight (kcf)
D_f = footing embedment depth (ft)
B = footing width (ft)
C_{wq} , C_{wg} = groundwater correction factors (Table 10.6.3.1.2a-2)
s_c , s_g , s_q = shape correction factors (Table 10.6.3.1.2a-3)
d_q = shear resistance thought cohesionless material correction factor (Table 10.6.3.1.2a-4)
i_c , i_g , i_q = inclination correction factors

By: msi Date: 10/20/2025 Checked: IJH Date: 10/23/2025

Setup	$c =$	0	ksf	assumed zero in cohesionless soil
	$\phi_f =$	30	degrees	
	$N_c =$	30.1	*unitless*	
	$N_q =$	18.4	*unitless*	for soil with a $\phi_f = 30$ Degrees
	$N_\gamma =$	22.4	*unitless*	
	$\gamma =$	0.125	kcf	(assumed in upper 1.5 feet above bearing)
	$D_f =$	6.5	ft	Minimum embedment of the footing into rock
	$B =$	3	ft	Width Stage 2 plans
	$L =$	12.5	ft	Length Stage 2 plans
	$D_w =$	0	ft	highest anticipated groundwater depth
	$C_{wq} =$	0.5	*unitless*	where $D_w = 0.0$ $1.5B + D_f =$ 11
	$C_{w\gamma} =$	0.5	*unitless*	(above D_f)
	$s_c =$	1.15	*unitless*	$s_c = 1 + (B/L)*(Nq/Nc)$ $s_c = 1 + (B/(5L))$
	$s_\gamma =$	0.90	*unitless*	for $\phi_f > 0$ $s_g = 1 - 0.4 * (B/L)$ for $\phi_f = 0$ $s_g = 1$
	$s_q =$	1.11	*unitless*	$s_q = 1+((B/L)*\tan\phi_f)$ $s_q = 1$
	$d_q =$	1.0	*unitless*	taken as 1 since cohesionless soil on top of weathered rock
	i_c , i_γ , $i_q =$	1.0	*unitless*	Assumed loaded without inclination

calculation

$N_{cm} = N_c s_c i_c$ = 30.1 * 1.147 * 1 = 34.525

$N_{qm} = N_q s_q d_q i_q$ = 18.4 * 0.904 * 1 * 1 = 16.634

$N_{gm} = N_g s_g i_g$ = 22.4 * 1.108 * 1 = 24.819

$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$

= (0*34.525) + (0.125*6.5*16.634*0.5) + (0.5*3*24.819*0.5) =

= (0) + (6.758) + (18.614) =

$cN_{cm} =$	0
$\gamma D_f N_{qm} C_{wq} =$	6.758
$0.5\gamma B N_{\gamma m} C_{w\gamma} =$	18.614

$q_n =$ 25.37 ksf

$\varphi_b =$ 0.55 AASHTO LRFD Table 11.5.7-1 - SLS Res. Factor for Perm. Retain. Walls

$q_R = \varphi_b * q_n$ = 0.55 * 25.372 = 13.95 ksf

Factored resistance at the strength limit state for the wingwall footing bearing in the very stiff to hard cohesive soils is equal to 14 ksf

By: msi Date: 10/20/2025 Checked: IJH Date: 10/23/2025

SERVICE LIMIT STATE:

At this structure location, the maximum service limit state bearing pressure is 2.6 ksf, and the maximum strength limit state bearing pressure is 4.3 ksf, both of which are significantly lower than the factored bearing resistance of 14 ksf.
Settlement analysis performed under the Service Limit State, using the applied bearing pressure of 4.3 ksf, yielded total settlements in the range of 0.50 to 0.68 inches, which is below the maximum allowable settlement of 1 inch typically considered acceptable per GDM Section 1303.2.
This confirms that the foundation design satisfies both serviceability and strength requirements, with the Service Limit State loads being well within the bounds of the factored bearing resistance.

Furthermore, we consider the settlement to be insignificant at the wingwall footing, especially since the wingwall is proposed to be structurally connected to the abutment and the abutment is bearing on rock.

No need to reduce BC (see attached *Settlement Calculation*)

.

By:	IJH	Date:	2/26/2025	Checked:	IJH	Date:	10/23/2025
				Revision:	msi	Date:	10/7/2025

GENERAL FOUNDATION INFORMATION:

Wingwall at Forward Abutment: Width W = 3' , Length L = 12.5'
Bearing at approxiamtely 580 feet.

GENERAL SOIL INFORMATION:

Anticipated Bearing Conditions:

Predominantly Very Stiff to Hard Cohesive Soils underlain by 1' zone very dense sand, and then weathered bedrock.

Based on Soil Strength Evaluation Spreadsheet,

USE c =	1.5	ksf for these soils

Groundwater

Model groundwater in creek above foundation bearing elevation.

STRENGTH LIMIT STATE:

$q_R = \varphi_b * q_n$ (AASTHO LRFD 10.6.3.1.1-1)

- q_R = factored resistance at strength limit state (ksf)
- φ_b = resistance factor (Article 10.5.5.2.2)
- q_n = nominal bearing resistance (ksf)

$q_n = cN_{cm} + gD_fN_{qm}C_{wq} + 0.5gBN_{gm}C_{wg}$ (AASTHO LRFD 10.6.3.1.2a-1)

$N_{cm} = N_cs_ci_c$ (AASTHO LRFD 10.6.3.1.2a-2)

$N_{qm} = N_qs_qd_qi_q$ (AASTHO LRFD 10.6.3.1.2a-3)

$N_{gm} = N_gs_gi_g$ (AASTHO LRFD 10.6.3.1.2a-4)

- c = cohesion, undrained shear strength (ksf)
- N_c = cohesion term (Table 10.6.3.1.2a-1)
- N_q = surcharge term (Table 10.6.3.1.2a-1)
- N_g = unit weight term (Table 10.6.3.1.2a-1)
- g = total (moist) unit weight (kcf)
- D_f = footing embedment depth (ft)
- B = footing width (ft)
- C_{wq} , C_{wg} = groundwater correction factors (Table 10.6.3.1.2a-2)
- s_c , s_g , s_q = shape correction factors (Table 10.6.3.1.2a-3)
- d_q = shear resistance thought cohesionless material correction factor (Table 10.6.3.1.2a-4)
- i_c , i_g , i_q = inclination correction factors

By: IJH Date: 2/26/2025 Checked: IJH Date: 10/7/2025

Setup	c =	1.5	ksf	
	f_f =	0	degrees	assumed zero in cohesive soil
	N_c =	5.14	*unitless*	
	N_q =	1.0	*unitless*	for soil with a f_f = 0 Degrees
	N_g =	0.0	*unitless*	
	g =	0.125	kcf	(assumed in upper 1.5 feet above bearing)
	D_f =	6.5	ft	(Depth below creek bottom)
	B =	3	ft	Width
	L =	12.5	ft	Length (measured on Google Earth)
	D_w =	0	ft	highest anticipated groundwater depth
	C_{wq} =	0.5	*unitless*	where D_w = 0.0 $1.5B + D_f$ = 11
	C_{wg} =	0.5	*unitless*	(above $D_{f)}$
	s_c =	1.048	*unitless*	$s_c = 1 + (B/(5L))$ $s_c = 1 + (B/(5L))(Nq/Nc)$
	s_g =	1.0	*unitless*	for $\phi_f = 0$ $s_g = 1$ for $\phi_f > 0$ $s_g = 1 - 0.4(B/L)$
	s_q =	1.0	*unitless*	$s_q = 1$ $s_q = 1 + ((B/L)\tan(f_f))$
	d_q =	1.0	*unitless*	taken as 1 since cohesive soil D_f / B = 2.166667
	i_c , i_g , i_q =	1.0	*unitless*	Assumed loaded without inclination

calculation

$N_{cm} = N_c s_c i_c$ = 5.14 * 1.048 * 1 = 5.387

$N_{qm} = N_q s_q d_q i_q$ = 1 * 1 * 1 * 1 = 1

$N_{gm} = N_g s_g i_g$ = 0 * 1 * 1 = 0

$q_n = cN_{cm} + gD_f N_{qm} C_{wq} + 0.5gBN_{gm}C_{wg}$

= (1.5*5.387) + (0.125*6.5*1*0.5) + (0.5*3*0*0.5) =

= (8.081) + (0.406) + (0) =

cN_{cm} = 8.081

$gD_f N_{qm} C_{wq}$ = 0.406

$0.5gBN_{gm}C_{wg}$ = 0

q_n = 8.49 ksf

φ_b = 0.55 AASHTO LRFD Table 11.5.7-1 - Strength Limit State Res. Factor for Perm. Retaining Walls

$q_R = \varphi_b * q_n$ = 0.55 * 8.487 = 4.67 ksf

Factored resistance at the strength limit state for the wingwall footing bearing in the very stiff to hard cohesive soils is equal to 4.7 ksf

By: IJH Date: 2/26/2025 Checked: IJH Date: 10/7/2025

SERVICE LIMIT STATE:

At this structure location, the maximum service limit state bearing pressure is 2.6 ksf, and the maximum strength limit state bearing pressure is 4.3 ksf, both of which are lower than the factored bearing resistance of 4.7 ksf.
Settlement analysis performed under the Service Limit State, using the applied bearing pressure of 4.3 ksf, yielded total settlements in the range of 0.50 to 0.68 inches, which is below the maximum allowable settlement of 1 inch typically considered acceptable per GDM Section 1303.2.
This confirms that the foundation design satisfies both serviceability and strength requirements, with the Service Limit State loads being well within the bounds of the factored bearing resistance.

Furthermore, we consider the settlement to be insignificant at the wingwall footing, especially since the wingwall is proposed to be structurally connected to the abutment and the abutment is bearing on rock.

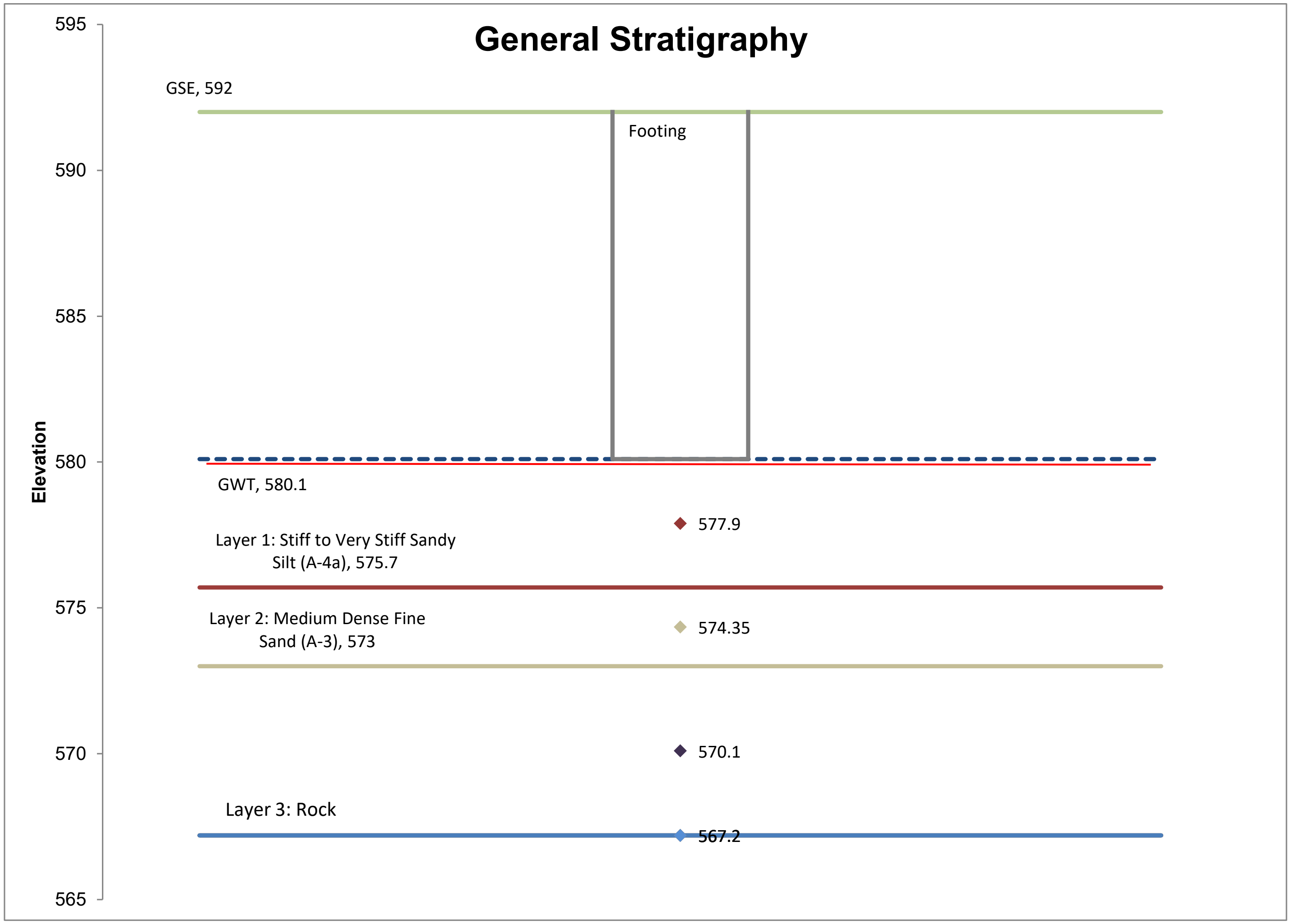
No need to reduce BC (see attached *Settlement Calculation*)

.

Project Name:	ATB Old Main Bridge	Boring Number	B-003-0-21	Rev. 2	by:	msi
Project Number:	232245	Analysis Type	Boussinesq Continuous	Date: 10/8/2025	Checked by:	IHJ, 10/23/25
Calculated by:	IJH 2/26/25					

Wingwall - Indicated 4.9 ksf bearing pressure.

Layers and Soil Type	H (feet)	C_r	e_o	σ_v (psf)	z (feet)	b (feet)	(z-Df) / b	I_z	Δ p@ 4700 psf	(check) sigma v+ΔP	Δ H (inches)
Layer 1: Stiff to Very Stiff Sandy Silt (A-4a)	4.4	0.022	0.59	1696	2.2	7	0.3	0.9	4230	5926	0.40
Layer 2: Medium Dense Fine Sand (A-3)	2.7	0.012	0.45	1936	5.75	7	0.8	0.6	2820	4756	0.10
Layer 3: Rock	5.8	0.009333	0.43	2223	10	7	1.4	0.4	1880	4103	0.12
Layer 3: Rock	0	0	0.30	2419	12.9	7	1.8	0.3	1410	3829	0.00
Layer 3: Rock	0	0	0.30	2419	12.9	7	1.8	0.3	1410	3829	0.00
Layer 3: Rock	0	0	0.30	2419	12.9	7	1.8	0.3	1410	3829	0.00



Total Δ H (in.)	0.62
+15%	0.72
-15%	0.53

$pc = s_u/(0.11+0.0037(PI))$

s_u (psf)= 1,500

Max PI = 7

pc (psf)= 11,038

All $\sigma_v + \Delta$ P < pc, so all Cr

OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES
Geotechnical Design Manual Section 600

Ashtabula, Old Main Street
119471

ATB-OLD MAIN STREET BRIDGE

CT/VERDANTAS

Prepared By: MSI
Date prepared: Thursday, October 23, 2025

Shahid Iqbal
8150 STERLING COURT
MENTOR, OH 44060

MUHAMMAD.IQBAL@VERDANTAS.COM

NO. OF BORINGS: 1

NO. OF DCPS:

Add DCP Test Data Worksheets

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	b-004-0-23	US-20	13+21	3	RIGHT	CME AUTOMATIC	73	590.2	589.2	1.0 C

Ohio Department of Transportation

Subgrade Analysis

V. 14.81 7/21/2025

#	Boring	Sample	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)
			From	To	From	To	N_{60}	N_{60L}		LL	PL	PI	% Silt	% Clay	P200	M_C	M_{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
1	b	SS-1	1.5	3.0	0.5	2.0	12									15	0	UCF			UCF	N_{60} & Mc	24''	12''	**REMOVE AND REPLACE 24-INCH UCF**
	004-0	SS-2	3.0	4.5	2.0	3.5	5						25	3	28	7	10	A-2-4	0			N_{60}			**204 Geotextile**
	23	SS-3	4.5	6.0	3.5	5.0	10						33	5	38	10	10	A-4a	5						
		SS-4	6.0	7.5	5.0	6.5	11	5								22	14	A-6a	10						

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.81 7/21/2025

PID: 119471

County-Route-Section: Ashtabula, Old Main Street
No. of Borings: 1

Geotechnical Consultant: CT/VERDANTAS
Prepared By: MSI
Date prepared: 10/23/2025

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

Excavate and Replace Stabilization Options	
Global Geotextile Average(N60L): Average(HP):	21" 0"
Global Geogrid Average(N60L): Average(HP):	15" 0"

Design CBR	8

% Samples within 3 feet of subgrade			
$N_{60} \leq 5$	25%	$HP \leq 0.5$	0%
$N_{60} < 12$	25%	$0.5 < HP \leq 1$	0%
$12 \leq N_{60} < 15$	25%	$1 < HP \leq 2$	0%
$N_{60} \geq 20$	0%	$HP > 2$	0%
M+	25%		
Rock	0%		
Unsuitable Soil	50%		

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

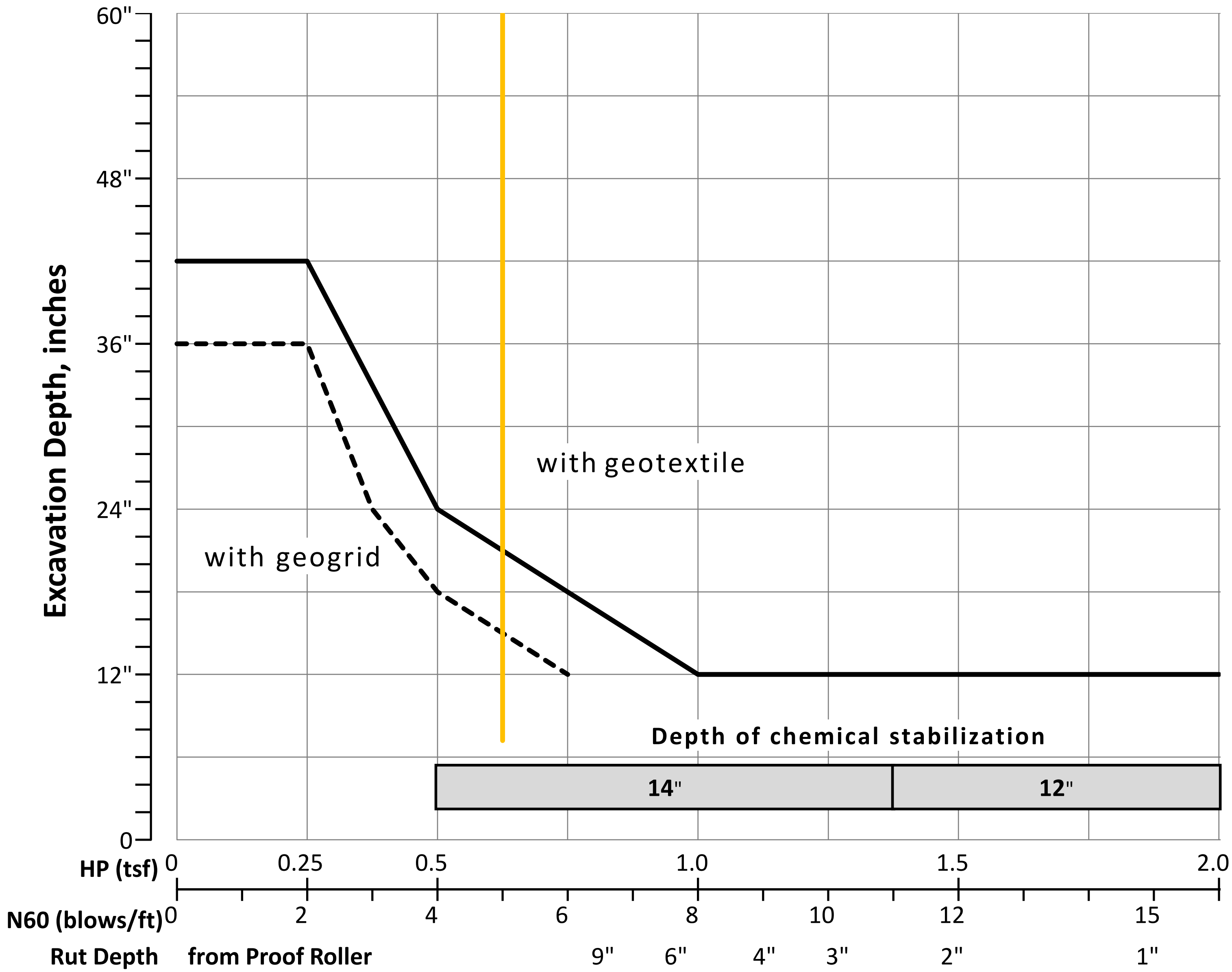
% Proposed Subgrade Surface	
Unstable & Unsuitable	150%
Unstable	100%
Unsuitable (Soil & Rock)	50%

	N_{60}	N_{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M_C	M_{OPT}	GI
Average	10	5	NP	0	0	0	29	4	33	14	9	5
Maximum	12	5	NP	0	0	0	33	5	38	22	14	10
Minimum	5	5	NP	0	0	0	25	3	28	7	0	0

Classification Counts by Sample

ODOT Class	UCF	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals
Count	1	0	0	0	1	0	0	0	0	0	1	0	0	1	0	0	0	0	0	4
Percent	25%	0%	0%	0%	25%	0%	0%	0%	0%	0%	25%	0%	0%	25%	0%	0%	0%	0%	0%	100%
% Rock|Granular|Cohesive	25%	0%	50%									25%								100%
Surface Class Count	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
Surface Class Percent	50%	0%	0%	0%	50%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	100%

Fig. 600-1 – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
NP	0.50	☐ HP
5.00	6.00	☐ N60L

Average HP
Average N_{60L}

Appendix B

Relevant Structural Calculations

ISOLATED FOOTING DESIGN

Code: AASHTO LRFD 9
Units: US
Pier View: Upstation.

GEOMETRY

Name : Spread 9 ft x 35 ft
Shape : Rectangular, Type : Spread
Bf(X) = 35.00 ft, Hf(Z) = 9.00 ft, Thickness(Y) = 48.00 in
Ag = 315.00 ft^2, Ix = 2126.25 ft^4, Iz = 32156.26 ft^4
Footing eccentric: Start at X = -17.50 ft from centerline of column.
Footing eccentric: Start at Z = 4.50 ft from centerline of column.
Columns located on the footing:
Column No. 1 at x = 0.00 ft, Round D = 396.00 in

DESIGN PARAMETERS

f'c = 4000.00 psi	fy = 60000.00 psi
phi tens = 0.90	
phi comp = 0.75	phi shear = 0.90
Tens above = 0.005	Comp below = 0.002
Ec = 4266.2 ksi	Es = 29000.0 ksi
Crack check as per current LRFD	
Crack control Exposure = 1.00	
Concrete Type : Normal Weight.	

Reinforcement Schedule

Dir	Quantity	Size	Bar dist. in	As total in^2	Spacing in	Hook
X	11	US#8[M25]	3.50	8.69	10.10	None
X	11	US#8[M25]	44.50	8.69	10.10	None
Z	36	US#8[M25]	4.50	28.44	11.80	None
Z	36	US#8[M25]	43.50	28.44	11.80	None

Max Soil Pressures, Service

Corner	X ft	Z ft	comb	Ovs	P kips	Mxx kft	Mzz kft	Soil press. ksf
1	-17.50	-4.50	695	1.000	-1649.52	-679.98	2652.19	8.12
			376	1.000	-1460.22	592.39	-1098.62	2.78
2	17.50	-4.50	687	1.000	-1649.52	-679.98	-1567.11	7.53
			389	1.000	-1568.39	801.61	2365.52	2.00
3	17.50	4.50	704	1.000	-1649.52	679.98	-1553.11	7.52
			527	1.000	-1568.39	-801.61	1898.03	2.25
4	-17.50	4.50	725	1.000	-1649.52	679.98	2666.19	8.13
			514	1.000	-1460.22	-592.39	-1561.81	2.53

Max Soil Pressures, Factored

Corner	X ft	Z ft	comb	Ovs	P kips	Mxx kft	Mzz kft	Soil press. ksf
1	-17.500	-4.500	21	---	-2095.71	-915.36	3534.32	10.51
			94	---	-1452.12	549.21	-2639.90	2.01
2	17.500	-4.500	13	---	-2095.71	-915.36	-2145.51	9.76
			745	---	-1168.17	944.43	960.29	1.19
3	17.500	4.500	30	---	-2095.71	915.36	-2126.67	9.75
			77	---	-1641.42	-915.36	3403.51	1.42
4	-17.500	4.500	51	---	-2095.71	915.36	3553.16	10.52
			750	---	-1168.17	-944.43	-287.57	1.55

Footing Design : Notes

Only max. positive pressure is considered for design.
Load effects (P, Mxx and Mzz) are reported at bottom of the footing.
Soil pressure are calculated based on the load effects at bottom of the footing.
Comb 0 -- For cases when there is no moment in a section, Zero moment might be reported for combination number 0.

Max. Soil Pressure Used in Design:

Factored soil pressure	10.52 ksf	Comb 51
Service soil pressure	8.13 ksf	Comb 725

Flexure

Dir	Loc ft	d in	Mmax kft	Comb	CL	Asb_req in^2	Asb_prv in^2	Asb_eff in^2	Ast_req in^2	Ast_prv in^2	Ast_eff in^2
X	-16.31	44.50	63.5	51	T	3.24	8.69	3.55	3.24	8.69	3.55
X	16.31	44.50	63.5	51	T	3.24	8.69	3.55	3.24	8.69	3.55
Z	-1.73	43.50	1340.6	51	T	16.33	28.44	17.59	16.33	28.44	17.59
Z	1.73	43.50	1340.6	51	T	16.33	28.44	17.59	16.33	28.44	17.59

Flexure Note

CL: Section classification as per LRFD 2006 interims for provided reinforcement.
C = Compression controlled, I = In-Transition, T = Tension controlled.
Required reinforcement is based on phi for tension controlled sections..
Comb 0 -- For cases when there is no moment in a section, Zero moment might be reported for combination number 0.

Crack control and fatigue check

Cracking/Fatigue

Dir	Loc ft	d in	Service Mmax kft	Service Comb	Service fs ksi	Service Srq in	Service Spr in	Fatigue fs ksi	Fatigue FTH ksi
X	-16.31	44.50	47.9	725	9.05	66.7	10.1	0.00	0.00
X	16.31	44.50	47.9	725	9.05	66.7	10.1	0.00	0.00
Z	-1.73	43.50	1010.7	725	22.03	24.5	11.8	0.00	0.00
Z	1.73	43.50	1010.7	725	22.03	24.5	11.8	0.00	0.00

Cracking/Fatigue Note

Comb 0 -- For cases when there is no moment in a section, Zero moment might be reported for combination number 0.

One Way Shear (Beta-Theta Method)

Col	Dir	Dist ft	Comb	dv in	Vu kips	Mu kft	theta deg	beta	phi*Vc kips
1	X	-19.96	Outside of Footing	---	---	---	---	---	---
	X	19.96	Outside of Footing	---	---	---	---	---	---
	Z	-5.31	Outside of Footing	---	---	---	---	---	---
	Z	5.31	Outside of Footing	---	---	---	---	---	---

One Way Shear Note

Comb 0 -- For cases when there is no moment in a section, Zero moment might be reported for combination number 0.

Two Way Shear

#	Bo ft	Ao ft^2	Comb	Avg. dv in	Vu kips	phi*Vc kips
Columns						
1	86.61	256.25	51	43.35	618.2	6192.6

Two Way Shear Note

TWO WAY SHEAR IN FOOTING IS NOT DESIGNED AND STIRRUPS ARE NOT CONSIDERED.
Comb 0 -- For cases when there is no moment in a section, Zero moment might be reported for combination number 0.

JOB NO. *232245* JOB NAME Conneaut Old Main Street
SUBJECT *Design FA with AASHTO LRFD* PREPARED BY *MPS* DATE *16-Oct-25*
Footing Elevation 572.0 CHECKED BY *JDB* DATE *16-Oct-25*
H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Cast-In-Place Concrete Cantilever Abutment on Spread Footing, LRFD

This Spreadhseet is adjusted for Rock

General

Design a cast-in-place (CIP) concrete Abutment supported on a spread footing conforming to the LRFD Bridge Design Specifications and the ODOT Bridge Manual.

Establish Project Requirements

See attached sketch for CIP concrete wall parameters

Summary of Calculation Below

Bearing

Service I Case 4 =	8.916	ksf
Eccentricity, e	1.881	ft
Strength 1b (MAX) Case 4	12.700	ksf
Eccentricity, e	2.115	ft
CDR, Sliding	1.149	
CDR, OT	1.256	

Design Parameters

Project Parameters

Soil Properties

Soil Data

Backfill soil Design Paramers

ϕ_f	*31.00*	°	Angle of internal friction.
γ_f	*125.00*	pcf	Unit weight of soil.
β	*0.00*	°	Angle of backslope.
c_f	*0.00*	ksf	Cohesion
δ_{Wall}	*0.00*	°	Friction angle between the soil and the wall.

Foundation Soil Design Parameters

ϕ_{fd}	*31.00*	°	Angle of internal friction.
γ_{fd}	*125.00*	pcf	Unit weight of soil.
β	*0.00*	°	Angle of backslope.
t	*0.00*	ft	pavement thickness
c_{fd}	*0.00*	ksf	Cohesion
δ_{Ftg}	*0.00*	°	Friction angle between the soil and the footing.

Concrete Data

γ_C	*150.0*	pcf	Unit weight of concrete.
f_Y	*60.0*	ksi	Steel yield.
f'_c	*4.0*	ksi	Concrete strength.

JOB NO.	*232245*	**JOB NAME**	**Conneaut Old Main Street**		
SUBJECT	*Design FA with AASHTO LRFD*	**PREPARED BY**	*MPS*	**DATE**	*16-Oct-25*
	Footing Elevation 572.0	**CHECKED BY**	*JDB*	**DATE**	*16-Oct-25*

H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Live Load Surcharge Parameters

$L_{traffic}$	*0.0*	ft	Distance from wall backface to edge of traffic
H/2	0.00	ft	Distance from wall backface where live loadsurcharge is considered in the wall design
H_{eq}	*2.000*	ft	Surcharge.

Resistance Factors

ϕ_b	*0.45*	Bearing resistance (Shallow Foundations) LRFD [Table 10.5.5.2.2-1]
ϕ_s	*1.00*	Sliding resistance LRFD [Table 11.5.7-1]
ϕ_t	*1.00*	Sliding resistance (shear resistance between soil and foundation) LRFD [Table 11.5.7-1]
ϕ_{ep}	*0.50*	Sliding resistance (passive resistance) LRFD [Table10.5.5.2.2-1]

Wall Geometry

He	*14.96*	ft	Exposed wall height, ft
Df	*6.80*	ft	Footing cover, ft
H=He+Df =	*21.76*	ft	Design wall height
Tt	*3.00*	ft	Section thickness.
b_1	*0.00*	in/ft	Front wall batter (b_1H:12V)
b_2	*0.00*	in/ft	Back wall batter (b_2H:12V)
bwh	*0.00*	in/ft	Back wall height

Preliminary Wall Dimensioning

H	*21.760*	ft	Design Wall height.
B	*9.000*	ft	Footing length.
A	*2.500*	ft	Toe length.
D	*3.00*	ft	Footing thickness.

Shear Key Dimensioning

Dkey	*0.00*	ft	Depth of shear key from bottom of footing, ft
Dw	*0.00*	ft	Width of shear key, ft
XK	*0.00*	ft	Distance from toe to shear key, ft

Other Wall Dimensioning

h'=H-D=	18.76		Stem + backwall height
T1 = b1h'/12=	0.00		Stem front batter width
T2=b2h'/12=	0.00		Stem back batter width
Tb = T1+Tt+T2=	3.00	ft	Stem thickness at bottom of wall
C=B-A-Tb=	3.50	ft	Heel projection
Φ = atan (12/b_2) =	90.00	deg	Angle of back face of wall to horizontal
b =	12.00	in	Concrete strip width for design
y1 = Df =	6.80	ft	Bottom of footing depth
y2 = Df + Dkey =	6.80	ft	Bottom of shear key depth
	21.76	ft	Retained soil height (DO NOT neglect top 6 in wall reveal)
h_{cws} =	0.00	ft	shear key depth

JOB NO.	*232245*	**JOB NAME**	**Conneaut Old Main Street**		
SUBJECT	*Design FA with AASHTO LRFD*	**PREPARED BY**	*MPS*	**DATE**	*16-Oct-25*
	Footing Elevation 572.0	**CHECKED BY**	*JDB*	**DATE**	*16-Oct-25*

H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Compute Earth Pressure Coefficients

Compute Active Earth Pressure Coefficient

ϕ_f	31.00 °	0.54105 rad
β	0.00 °	0.00000 rad
δ_{Wall}	0.00 °	0.00000 rad
Φ	90.00 °	1.57080 rad

$$K_A = \frac{\text{Sin}^2(\phi f+\Phi)}{\Gamma'\text{Sin}(\Phi)^2\text{Sin}(\Phi\text{-'dWall})} \qquad \mathbf{0.320}$$

$$\Gamma = (1+((\text{Sin}(\phi_f+\text{dWall})\text{Sin}(\phi_f-\beta))/(\text{Sin}(\Phi\text{-'dWall})\text{Sin}(\Phi+\beta)))^2 \qquad \mathbf{2.295}$$

Compute Passive Earth Pressure Coefficient

$$K_P = \text{Tan}^2(45^o + \phi f/2) \qquad \mathbf{3.124}$$

Compute Unfactored Loads

Active Earth Force Resultant (kip/ft), FT

$$F_T = (K_A \gamma_f h^2)/2$$

9.473 kip/ft Active soil force / foot width of wall.

Live Load Surcharge Load (kip/ft), Fsur

$$F_{SUR} = K_A \gamma_f h_{eq} h$$

1.741 kip/ft

Vertical Loads (kip/ft), V

$\mathbf{R_{DW}} = V_1 =$	2.47	kip/ft	Future Wearing Surface (DW)
$\mathbf{R_{DC}} = V_2 =$	1.56	kip/ft	Approach Slab (DC)
$\mathbf{R_{DC}} = V_3 =$	14.340	kip/ft	Superstructure (DC)
$\mathbf{R_{DC}} = V_4 =$	0.510	kip/ft	Backwall (DC)
$\mathbf{R_{DC}} = V_5 = DB'\gamma_C =$	4.050	kip/ft	Footing (DC)
$\mathbf{R_{DC}} = V_6 = Tbh'\gamma_C =$	8.442	kip/ft	Stem (DC)
$\mathbf{R_{EV}} = V_7 = Ch'\gamma_f =$	8.21	kip/ft	Soil backfill Vertical Earth Load (EV)
$\mathbf{R_{LL}} = V_8 =$	12.22	kip/ft	Live Load (LL) without impact

JOB NO. *232245* **JOB NAME** Conneaut Old Main Street

SUBJECT *Design FA with AASHTO LRFD* **PREPARED BY** *MPS* **DATE** *16-Oct-25*

Footing Elevation 572.0 **CHECKED BY** *JDB* **DATE** *16-Oct-25*

H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Moments produced from vertical loads about Point 'O' (kip-ft/ft), Mvi (Moment Arm (ft))

d_{v1} = A+1 = 3.50 ft
d_{v2} = A + Tb - 0.5 = 5.00 ft
d_{v3} = A + 1 = 3.50 ft
d_{v4} = A + Tb - 0.75 = 4.75 ft
d_{v5} = b/2 = 4.50 ft
d_{v6} = A + Tb/2 = 4.00 ft
d_{v7} = 7.25 ft
d_{v8} = A+1 = 3.50 ft

Horizontal Loads (kip/ft), Hi

$\mathbf{R_{LS} = H_1} = F_{SUR}Cos(90deg\text{-}\Theta+\delta_{Wall})$ = 1.74 kip/ft Live load surcharge (LS)
$\mathbf{R_{EH} = H_2} = F_T Cos(90deg\text{-}\Theta+\delta_{Wall})$ = 9.47 kip/ft Active earth force(horizontal component) (EH)

Moments produced from horizontal loads about about Point 'O' (kip-ft/ft), MH

d_{h1} = h/2 = 10.88 ft
d_{h2} = h/3 = 7.25 ft

Use the following construction stagesfor analysis

CASE 1 wall DL
CASE 2 wall DL + Earth Pressure
CASE 3 Abut DL + Earth Pressure + LLsurcharge
CASE 4 Abut DL + Earth Pressure + Llsurcharge +DL superst + LL

Service 1 Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	SER 1 max/min	SER 1 PV k/ft	SER 1 PH k/ft	SER 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	2.47		3.50	1.00	2.47				8.65	8.65
V_2	DCas	1.56		5.00	1.00	1.56		7.80	7.80	7.80	7.80
V_3	DC_{super}	14.34		3.50	1.00	14.34					50.19
V_4	DC_{bw}	0.51		4.75	1.00	0.51		2.42	2.42	2.42	2.42
V_5	DCftg	4.05		4.50	1.00	4.05		18.23	18.23	18.23	18.23
V_6	DCstem	8.44		4.00	1.00	8.44		33.77	33.77	33.77	33.77
V_7	EV_{bf}	8.21		7.25	1.00	8.21			59.50	59.50	59.50
V_8	LL	12.22		3.50	1.00	12.22					42.77
H_1	LS		1.74	-10.88	1.00		1.74			-18.95	-18.95
H_2	EH		9.47	-7.25	1.00		9.47		-68.71	-68.71	-68.71
∑Pv		51.80						14.562	22.77	25.24	51.80
∑Ph			11.21						9.473	11.214	11.214
M res.								62.22	121.72	130.36	223.32
M over.								0.00	-68.71	-87.66	-87.66
∑ M								62.22	53.01	42.71	135.67

JOB NO. *232245* JOB NAME Conneaut Old Main Street
SUBJECT *Design FA with AASHTO LRFD* PREPARED BY *MPS* DATE *16-Oct-25*
Footing Elevation 572.0 CHECKED BY *JDB* DATE *16-Oct-25*
H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Service 1 Load Combination (Continued)

Bearing Resistance at Base of the Wall

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

		CASE 1	CASE 2	CASE 3	CASE 4
$x = (\sum MR - \sum MO)/\sum V =$	ft	4.27	2.33	1.69	2.62
$e = B/2 - x =$	ft	0.23	2.17	2.81	**1.88**
If e is less than zero, use zero					
Compute the ultimate bearing stress for Rock (11.3.6.2)					
Is resultant in middle 1/3 of Base: B/3 - 2absE > 0		Inside	Outside	Outside	Outside
If Inside					
$\sigma_{vmax} = \sum V/B(1+6e/B) =$	ksf/ft	1.86	6.19	8.05	12.97
$\sigma_{vmin} = \sum V/B(1-6e/B) =$	ksf/ft	1.37	-1.13	-2.45	-1.46
If Outside					
$\sigma_{vmax} = 2\sum V/(3(B/2)-e) =$	ksf/ft	2.19	4.02	4.72	**8.92**
$\sigma_{vmin} = 0$	ksf/ft	0.00	0.00	0.00	0.00

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR=\phi_b q_n =$ 17.30 ksf Rock Bearing per Geotech

Capacity:Demand Ratio (CDR)

	CASE 1	CASE 2	CASE 3	CASE 4
$CDR_{Bearing1} = qR/\sigma v =$	7.88	2.79	2.15	1.33
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Factor of Safety Overturning (not per LRFD)

	CASE 1	CASE 2	CASE 3	CASE 4
FS_O = ABSOLUTE $\sum MR/\sum MO =$	#DIV/0!	1.77	1.49	2.55
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

	CASE 1	CASE 2	CASE 3	CASE 4
$e_{max} = 0.45B =$	4.05	4.05	4.05	4.05
$CDR_{Eccentricity1} = emax/e =$	17.80	1.86	1.44	2.15
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

(Input values are shown underlined and in italics.)

Sliding Resistance at Base of the Wall

Factored Sliding Force Ru = ∑Ph =	0.00	9.47	11.21	11.21
Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep}$ =				
$\emptyset\tau R\tau = \sum V \tan(\emptyset_{fd})$ =	8.75	13.68	15.17	31.12

where
$\emptyset\tau$ = 1.00

Compute passive resistance throughout the design life of the wall

$r_{ep1} = kp\, \gamma_{fd}\, y1$ = Nominal passive pressure at y1 = 2.66 kip/ft

$r_{ep2} = kp\, \gamma_{fd}\, y2$ = Nominal passive pressure at y2 = 2.66 kip/ft

$\phi epR_{ep} = (r_{ep1} + r_{ep2})(y_2 - y_1)/2$ = 0.00 kip/ft

ϕ_{ep} = 0.50

Compute nominal resistance against failure by sliding, Rn

$Rn = \emptyset\tau R_\tau + \emptyset_{ep} R_{ep}$ =	8.75	13.68	15.17	31.12

Compute factored resistance against failure by sliding, R_R

$R_R = \emptyset sRn$ =	8.75	13.68	15.17	31.12

$\emptyset s$ = 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.44	1.35	2.78
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Strength 1a (MIN) Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	STR 1 max/min	STR 1 PV k/ft	STR 1 PH k/ft	STR 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	2.47		3.50	0.65	1.61				5.62	5.62
V_2	DCas	1.56		5.00	0.90	1.40		7.02	7.02	7.02	7.02
V_3	DC_{super}	14.34		3.50	0.90	12.91					45.17
V_4	DC_{bw}	0.51		4.75	0.90	0.46		2.18	2.18	2.18	2.18
V_5	DCftg	4.05		4.50	0.90	3.65		16.40	16.40	16.40	16.40
V_6	DCstem	8.44		4.00	0.90	7.60		30.39	30.39	30.39	30.39
V_7	EV_{bf}	8.21		7.25	1.00	8.21			59.50	59.50	59.50
V_8	LL	12.22		3.50	1.75	21.39					74.85
H_1	LS		1.74	-10.88	1.75		3.05			-33.16	-33.16
H_2	EH		9.47	-7.25	0.90		8.53		-68.71	-61.84	-61.84
∑Pv		51.80						13.106	21.31	22.92	57.21
∑Ph			11.21						8.526	11.573	11.573
M res.								55.99	115.50	121.12	241.14
M over.								0.00	-68.71	-94.99	-94.99
∑ M								55.99	46.79	26.12	146.14

(Input values are shown underlined and in italics.)

Strength 1a (MIN) Load Combination

Bearing Resistance at Base of the Wall

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

		CASE 1	CASE 2	CASE 3	CASE 4
$x = (\sum MR - \sum MO)/\sum V =$	ft	4.27	2.20	1.14	2.55
$e = B/2 - x =$	ft	0.23	2.30	3.36	1.95
If e is less than zero, use zero					
Compute the ultimate bearing stress for Rock (11.3.6.2)					
Is resultant in middle 1/3 of Base: B/3 - 2absE > 0		Inside	Outside	Outside	Outside
If Inside					
$\sigma_{vmax} = \sum V/B(1+6e/B) =$	ksf/ft	1.68	6.01	8.25	14.60
$\sigma_{vmin} = \sum V/B(1-6e/B) =$	ksf/ft	1.24	-1.27	-3.16	-1.89
If Outside					
$\sigma_{vmax} = 2\sum V/(3(B/2)-e) =$	ksf/ft	1.97	3.81	4.52	9.90
$\sigma_{vmin} = 0$	ksf/ft	0.00	0.00	0.00	0.00

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR = \phi_b q_n =$ 17.30 ksf Rock Bearing per Geotech

Capacity:Demand Ratio (CDR)

	CASE 1	CASE 2	CASE 3	CASE 4
$CDR_{Bearing1} = qR/\sigma v =$	8.76	2.88	2.10	1.18
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**
Factor of Safety Overturning (not per LRFD)				
FS_O = ABSOLUTE $\sum MR/\sum MO$ =	#DIV/0!	1.68	1.28	2.54
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**
Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)				
Maximum eccentricity				
$e_{max} = 0.45B =$	4.05	4.05	4.05	4.05
$CDR_{Eccentricity1} = emax/e =$	17.80	1.76	1.21	2.08
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

JOB NO. *232245* JOB NAME Conneaut Old Main Street
SUBJECT *Design FA with AASHTO LRFD* PREPARED BY *MPS* DATE *16-Oct-25*
Footing Elevation 572.0 CHECKED BY *JDB* DATE *16-Oct-25*
H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Sliding Resistance at Base of the Wall

Factored Sliding Force Ru = ∑Ph =	0.00	8.53	11.57	11.57

Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep}$ =

$\emptyset\tau R\tau = \sum V \tan(\emptyset_{fd})$ =	7.87	12.81	13.77	34.38

where $\emptyset\tau$ = 1
$\emptyset\tau$ = 1.00

Compute passive resistance throughout the design life of the wall

$r_{ep1} = kp\,\gamma_{fd}\,y1$ = Nominal passive pressure at y1 = 2.66 kip/ft
$r_{ep2} = kp\,\gamma_{fd}\,y2$ = Nominal passive pressure at y2 = 2.66 kip/ft

$\phi ep R_{ep} = (r_{ep1} + r_{ep2})(y_2 - y_1)/2$ = 0.00 kip/ft
ϕ_{ep} = 0.50
Compute nominal resistance against failure by sliding, Rn

$Rn = \emptyset\tau R_\tau + \emptyset_{ep} R_{ep}$ =	7.87	12.81	13.77	34.38

Compute factored resistance against failure by sliding, R_R

$R_R = \emptyset s Rn$ =	7.87	12.81	13.77	34.38

$\emptyset s$ = 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.50	1.19	2.97
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Strength 1b (MAX) Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	STR 1 max/min	STR 1 PV k/ft	STR 1 PH k/ft	STR 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	2.47		3.50	1.50	3.71				12.97	12.97
V_2	DCas	1.56		5.00	1.25	1.95		9.75	9.75	9.75	9.75
V_3	DC_{super}	14.34		3.50	1.25	17.93					62.74
V_4	DC_{bw}	0.51		4.75	1.25	0.64		3.03	3.03	3.03	3.03
V_5	DCftg	4.05		4.50	1.25	5.06		22.78	22.78	22.78	22.78
V_6	DCstem	8.44		4.00	1.25	10.55		42.21	42.21	42.21	42.21
V_7	EV_{bf}	8.21		7.25	1.35	11.08			80.33	80.33	80.33
V_8	LL	12.22		3.50	1.75	21.39					74.85
H_1	LS		1.74	-10.88	1.75		3.05			-33.16	-33.16
H_2	EH		9.47	-7.25	1.50		14.21		-68.71	-103.06	-103.06
∑Pv		51.80						18.203	29.28	32.99	72.30
∑Ph			11.21						14.209	17.257	17.257
M res.								77.77	158.10	171.07	308.65
M over.								0.00	-68.71	-136.22	-136.22
∑ M								77.77	89.39	34.85	172.43

JOB NO. *232245* JOB NAME Conneaut Old Main Street
SUBJECT *Design FA with AASHTO LRFD* PREPARED BY *MPS* DATE *16-Oct-25*
Footing Elevation 572.0 CHECKED BY *JDB* DATE *16-Oct-25*
H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Strength 1b (MAX) Load Combination

Bearing Resistance at Base of the Wall

		CASE 1	CASE 2	CASE 3	CASE 4
Compute resultant location (distance from Point 'O' Figure E14-1.4-3)					
$x = (\sum MR - \sum MO)/\sum V =$	ft	4.27	3.05	1.06	$\boxed{2.39}$
$e = B/2 - x =$	ft	0.23	1.45	3.44	2.11
If e is less than zero, use zero					
Compute the ultimate bearing stress for Rock (11.3.6.2)					
Is resultant in middle 1/3 of Base: B/3 - 2absE > 0		Inside	Inside	Outside	Outside
If Inside					
$\sigma_{vmax} = \sum V/B(1+6e/B) =$	ksf/ft	2.33	6.39	12.08	19.36
$\sigma_{vmin} = \sum V/B(1-6e/B) =$	ksf/ft	1.72	0.11	-4.75	-3.29
If Outside					
$\sigma_{vmax} = 2\sum V/(3(B/2)-e) =$	ksf/ft	2.74	4.86	6.56	$\boxed{12.70}$
$\sigma_{vmin} = 0$	ksf/ft	0.00	0.00	0.00	0.00

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR = \phi_b q_n =$ 17.30 ksf Rock Bearing per Geotech

Capacity:Demand Ratio (CDR)

	CASE 1	CASE 2	CASE 3	CASE 4
$CDR_{Bearing1} = qR/\sigma v =$	7.43	2.71	2.64	1.36
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Factor of Safety Overturning (not per LRFD)

	CASE 1	CASE 2	CASE 3	CASE 4
FS_O = ABSOLUTE $\sum MR/\sum MO =$	#DIV/0!	2.30	1.26	2.27
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

	CASE 1	CASE 2	CASE 3	CASE 4
$e_{max} = 0.45B =$	4.05	4.05	4.05	4.05
$CDR_{Eccentricity1} = emax/e =$	17.80	2.80	1.18	1.91
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Sliding Resistance at Base of the Wall

	CASE 1	CASE 2	CASE 3	CASE 4
Factored Sliding Force $Ru = \sum Ph =$	0.00	14.21	17.26	17.26
Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep} =$				
$\emptyset\tau R\tau = \sum V \tan(\emptyset_{fd}) =$	10.94	17.59	19.82	43.44

where

$\emptyset\tau =$ 1.00

Compute passive resistance throughout the design life of the wall

(Input values are shown underlined and in italics.)

$r_{ep1} = kp\ \gamma_{fd}\ y1 =$ Nominal passive pressure at y1 = 2.66 kip/ft

$r_{ep2} = kp\ \gamma_{fd}\ y2 =$ Nominal passive pressure at y2 = 2.66 kip/ft

$\phi epR_{ep} = (r_{ep1} + r_{ep2})(y_2\text{-}y_1)/2 =$ 0.00 kip/ft

$\phi_{ep} =$ 0.50

Compute nominal resistance against failure by sliding, Rn

$Rn = \emptyset\tau R_\tau + \emptyset_{ep} R_{ep} =$	10.94	17.59	19.82	43.44

Compute factored resistance against failure by sliding, R_R

$R_R = \emptyset sRn =$	10.94	17.59	19.82	43.44
$\emptyset s =$ 1.00				

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.24	**1.15**	**2.52**
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

may be removed if rock excavation prohibits

Strength 3 a (MIN) Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	STR 1 max/min	STR 1 PV k/ft	STR 1 PH k/ft	STR 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	2.47		3.50	0.65	1.61				5.62	5.62
V_2	DCas	1.56		5.00	0.90	1.40		7.02	7.02	7.02	7.02
V_3	DC_{super}	14.34		3.50	0.90	12.91					45.17
V_4	DC_{bw}	0.51		4.75	0.90	0.46		2.18	2.18	2.18	2.18
V_5	DCftg	4.05		4.50	0.90	3.65		16.40	16.40	16.40	16.40
V_6	DCstem	8.44		4.00	0.90	7.60		30.39	30.39	30.39	30.39
V_7	EV_{bf}	8.21		7.25	1.00	8.21			59.50	59.50	59.50
V_8	LL	12.22		3.50	0.00	0.00					0.00
H_1	LS		1.74	-10.88	0.00		0.00			0.00	0.00
H_2	EH		9.47	-7.25	0.90		8.53		-68.71	-61.84	-61.84
∑Pv		51.80						13.106	21.31	22.92	35.82
∑Ph			11.21						8.526	8.526	8.526
M res.								55.99	115.50	121.12	166.29
M over.								0.00	-68.71	-61.84	-61.84
∑ M								55.99	46.79	59.28	104.45

JOB NO. *232245* JOB NAME Conneaut Old Main Street
SUBJECT *Design FA with AASHTO LRFD* PREPARED BY *MPS* DATE *16-Oct-25*
Footing Elevation 572.0 CHECKED BY *JDB* DATE *16-Oct-25*
H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Strength 3a (MIN) Load Combination

Bearing Resistance at Base of the Wall

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

		CASE 1	CASE 2	CASE 3	CASE 3
$x = (\sum MR - \sum MO)/\sum V =$	ft	4.27	2.20	2.59	2.92
$e = B/2 - x =$	ft	0.23	2.30	1.91	1.58

If e is less than zero, use zero

Compute the ultimate bearing stress for Rock (11.3.6.2)

Is resultant in middle 1/3 of Base: B/3 - 2absE > 0		Inside	Outside	Outside	Outside
If Inside					
$\sigma_{vmax} = \sum V/B(1+6e/B) =$	ksf/ft	1.68	6.01	5.80	8.19
$\sigma_{vmin} = \sum V/B(1-6e/B) =$	ksf/ft	1.24	-1.27	-0.70	-0.22
If Outside					
$\sigma_{vmax} = 2\sum V/(3(B/2)-e) =$	ksf/ft	1.97	3.81	3.96	6.01
$\sigma_{vmin} = 0$	ksf/ft	0.00	0.00	0.00	0.00

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR = \phi_b q_n =$ 17.30 ksf

Capacity:Demand Ratio (CDR)

$CDR_{Bearing1} = qR/\sigma v =$	8.76	2.88	2.99	2.11
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Factor of Safety Overturning (not per LRFD)

FS_O = ABSOLUTE $\sum MR/\sum MO =$	#DIV/0!	1.68	1.96	2.69
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

$e_{max} = 0.45B =$	4.05	4.05	4.05	4.05
$CDR_{Eccentricity1} = emax/e =$	17.80	1.76	2.12	2.56
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Sliding Resistance at Base of the Wall

Factored Sliding Force $Ru = \sum Ph =$	0.00	8.53	8.53	8.53
Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep} =$				
$\emptyset\tau R\tau = \sum V \tan(\emptyset_{fd}) =$	7.87	12.81	13.77	21.53

JOB NO. *232245* JOB NAME Conneaut Old Main Street
SUBJECT *Design FA with AASHTO LRFD* PREPARED BY *MPS* DATE *16-Oct-25*
Footing Elevation 572.0 CHECKED BY *JDB* DATE *16-Oct-25*
H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

where
Øτ = 1.00

Compute passive resistance throughout the design life of the wall

$r_{ep1} = kp\ \gamma_{fd}\ y1 =$ Nominal passive pressure at y1 = 2.66 kip/ft
$r_{ep2} = kp\ \gamma_{fd}\ y2 =$ Nominal passive pressure at y2 = 2.66 kip/ft

$\phi epR_{ep} = (r_{ep1} + r_{ep2})(y_2-y_1)/2 =$ 0.00 kip/ft
$\phi_{ep} =$ 0.50

Compute nominal resistance against failure by sliding, Rn

$Rn = Ø\tau R_\tau + Ø_{ep}R_{ep} =$	7.87	12.81	13.77	21.53

Compute factored resistance against failure by sliding, R_R

$R_R = ØsRn =$	7.87	12.81	13.77	21.53

Øs = 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.50	1.62	2.52
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Strength 3b (MAX) Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	STR 1 max/min	STR 1 PV k/ft	STR 1 PH k/ft	STR 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	2.47		3.50	1.50	3.71				12.97	12.97
V_2	DCas	1.56		5.00	1.25	1.95		9.75	9.75	9.75	9.75
V_3	DC_{super}	14.34		3.50	1.25	17.93					62.74
V_4	DC_{bw}	0.51		4.75	1.25	0.64		3.03	3.03	3.03	3.03
V_5	DCftg	4.05		4.50	1.25	5.06		22.78	22.78	22.78	22.78
V_6	DCstem	8.44		4.00	1.25	10.55		42.21	42.21	42.21	42.21
V_7	EV_{bf}	8.21		7.25	1.35	11.08			80.33	80.33	80.33
V_8	LL	12.22		3.50	0.00	0.00					0.00
H_1	LS		1.74	-10.88	0.00		0.00			0.00	0.00
H_2	EH		9.47	-7.25	1.50		14.21		-68.71	-103.06	-103.06
∑Pv		51.80						18.203	29.28	32.99	50.91
∑Ph			11.21						14.209	14.209	14.209
M res.								77.77	158.10	171.07	233.81
M over.								0.00	-68.71	-103.06	-103.06
∑ M								77.77	89.39	68.00	130.74

Strength 3b (MAX) Load Combination
Bearing Resistance at Base of the Wall
Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

CASE 1	CASE 2	CASE 3	CASE 3

$x = (\sum MR - \sum MO)/\sum V =$	ft	4.27	3.05	2.06	2.57
$e = B/2 - x =$	ft	0.23	1.45	2.44	1.93

If e is less than zero, use zero

Compute the ultimate bearing stress for Rock (11.3.6.2)

Is resultant in middle 1/3 of Base: B/3 - 2absE > 0		Inside	Inside	Outside	Outside
If Inside					
$\sigma_{vmax} = \sum V/B(1+6e/B) =$	ksf/ft	2.33	6.39	9.62	12.94
$\sigma_{vmin} = \sum V/B(1-6e/B) =$	ksf/ft	1.72	0.11	-2.29	-1.63
If Outside					
$\sigma_{vmax} = 2\sum V/(3(B/2)-e) =$	ksf/ft	2.74	4.86	5.96	8.80
$\sigma_{vmin} = 0$	ksf/ft	0.00	0.00	0.00	0.00

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR=\phi_b q_n =$ 17.30 ksf Rock Bearing per Geotech

Capacity:Demand Ratio (CDR)

$CDR_{Bearing1} = qR/\sigma v =$	6.31	2.71	1.80	1.34
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Factor of Safety Overturning (not per LRFD)

FS_O = ABSOLUTE $\sum MR/\sum MO =$	#DIV/0!	2.30	1.66	2.27
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

$e_{max} = 0.45B =$	4.05	4.05	4.05	4.05
$CDR_{Eccentricity1} = emax/e =$	17.80	2.80	1.66	2.10
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Sliding Resistance at Base of the Wall

Factored Sliding Force Ru = $\sum Ph =$	0.00	14.21	14.21	14.21

Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep} =$

$\emptyset\tau R\tau = \sum V \tan(\emptyset_{fd}) =$	10.94	17.59	19.82	30.59

where
$\emptyset\tau$ = 1.00

(Input values are shown underlined and in italics.)

Compute passive resistance throughout the design life of the wall

$r_{ep1} = kp\ \gamma_{fd}\ y1 =$ Nominal passive pressure at y1 = 2.66 kip/ft

$r_{ep2} = kp\ \gamma_{fd}\ y2 =$ Nominal passive pressure at y2 = 2.66 kip/ft

$\phi epR_{ep} = (r_{ep1} + r_{ep2})(y_2\text{-}y_1)/2 =$ 0.00 kip/ft

$\phi_{ep} =$ 0.50

Compute nominal resistance against failure by sliding, Rn

$Rn = \emptyset\tau R_\tau + \emptyset_{ep}R_{ep} =$	10.94	17.59	19.82	30.59

Compute factored resistance against failure by sliding, R_R

$R_R = \emptyset sRn =$	10.94	17.59	19.82	30.59

$\emptyset s =$ 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.24	1.39	2.15
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Strength 5 a (MIN) Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	STR 1 max/min	STR 1 PV k/ft	STR 1 PH k/ft	STR 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	2.47		3.50	0.65	1.61				5.62	5.62
V_2	DCas	1.56		5.00	0.90	1.40		7.02	7.02	7.02	7.02
V_3	DC_{super}	14.34		3.50	0.90	12.91					45.17
V_4	DC_{bw}	0.51		4.75	0.90	0.46		2.18	2.18	2.18	2.18
V_5	DCftg	4.05		4.50	0.90	3.65		16.40	16.40	16.40	16.40
V_6	DCstem	8.44		4.00	0.90	7.60		30.39	30.39	30.39	30.39
V_7	EV_{bf}	8.21		7.25	1.00	8.21			59.50	59.50	59.50
V_8	LL	12.22		3.50	1.35	16.50					57.74
H_1	LS		1.74	-10.88	1.35		2.35			-25.58	-25.58
H_2	EH		9.47	-7.25	0.90		8.53		-68.71	-61.84	-61.84
∑Pv		51.80						13.106	21.31	22.92	52.32
∑Ph			11.21						8.526	10.876	10.876
M res.								55.99	115.50	121.12	224.03
M over.								0.00	-68.71	-87.42	-87.42
∑ M								55.99	46.79	33.70	136.61

(Input values are shown underlined and in italics.)

Strength 5a (MIN) Load Combination

Bearing Resistance at Base of the Wall

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

		CASE 1	CASE 2	CASE 3	CASE 3
$x = (\sum MR - \sum MO)/\sum V =$	ft	4.27	2.20	1.47	2.61
$e = B/2 - x =$	ft	0.23	2.30	3.03	1.89
If e is less than zero, use zero					
Compute the ultimate bearing stress for Rock (11.3.6.2)					
Is resultant in middle 1/3 of Base: B/3 - 2absE > 0		Inside	Outside	Outside	Outside
If Inside					
$\sigma_{vmax} = \sum V/B(1+6e/B) =$	ksf/ft	1.68	6.01	7.69	13.13
$\sigma_{vmin} = \sum V/B(1-6e/B) =$	ksf/ft	1.24	-1.27	-2.60	-1.51
If Outside					
$\sigma_{vmax} = 2\sum V/(3(B/2)-e) =$	ksf/ft	1.97	3.81	4.38	9.01
$\sigma_{vmin} = 0$	ksf/ft	0.00	0.00	0.00	0.00

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR = \phi_b q_n =$ 17.30 ksf Rock Bearing per Geotech

Capacity:Demand Ratio (CDR)

	CASE 1	CASE 2	CASE 3	CASE 3
$CDR_{Bearing1} = qR/\sigma v =$	8.76	2.88	2.25	1.32
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**
Factor of Safety Overturning (not per LRFD)				
FS_O = ABSOLUTE $\sum MR/\sum MO$ =	#DIV/0!	1.68	1.39	2.56
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

$e_{max} = 0.45B =$	4.05	4.05	4.05	4.05

(Input values are shown underlined and in italics.)

$CDR_{Eccentricity1}$ = emax/e =	17.80	1.76	1.34	2.14
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Sliding Resistance at Base of the Wall

Factored Sliding Force Ru = $\sum Ph$ =	0.00	8.53	10.88	10.88
Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep}$ =				
$\emptyset\tau R\tau = \sum V \tan(\emptyset_{fd})$ =	7.87	12.81	13.77	31.44

where
$\emptyset\tau$ = 1.00

Compute passive resistance throughout the design life of the wall

$r_{ep1} = kp\ \gamma_{fd}\ y1$ = Nominal passive pressure at y1 = 2.66 kip/ft
$r_{ep2} = kp\ \gamma_{fd}\ y2$ = Nominal passive pressure at y2 = 2.66 kip/ft

$\phi ep R_{ep} = (r_{ep1} + r_{ep2})(y_2 - y_1)/2$ = 0.00 kip/ft
ϕ_{ep} = 0.50
Compute nominal resistance against failure by sliding, Rn

$Rn = \emptyset\tau R_\tau + \emptyset_{ep} R_{ep}$ =	7.87	12.81	13.77	31.44

Compute factored resistance against failure by sliding, R_R

$R_R = \emptyset s Rn$ =	7.87	12.81	13.77	31.44

$\emptyset s$ = 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.50	1.27	2.89
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

JOB NO. *232245* JOB NAME Conneaut Old Main Street
SUBJECT *Design FA with AASHTO LRFD* PREPARED BY *MPS* DATE *16-Oct-25*
Footing Elevation 572.0 CHECKED BY *JDB* DATE *16-Oct-25*
H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Strength 5b (MAX) Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	STR 1 max/min	STR 1 PV k/ft	STR 1 PH k/ft	STR 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	2.47		3.50	1.50	3.71				12.97	12.97
V_2	DCas	1.56		5.00	1.25	1.95		9.75	9.75	9.75	9.75
V_3	DC_{super}	14.34		3.50	1.25	17.93					62.74
V_4	DC_{bw}	0.51		4.75	1.25	0.64		3.03	3.03	3.03	3.03
V_5	DCftg	4.05		4.50	1.25	5.06		22.78	22.78	22.78	22.78
V_6	DCstem	8.44		4.00	1.25	10.55		42.21	42.21	42.21	42.21
V_7	EV_{bf}	8.21		7.25	1.35	11.08			80.33	80.33	80.33
V_8	LL	12.22		3.50	1.35	16.50					57.74
H_1	LS		1.74	-10.88	1.35		2.35			-25.58	-25.58
H_2	EH		9.47	-7.25	1.50		14.21		-68.71	-103.06	-103.06
∑Pv		51.80						18.203	29.28	32.99	67.41
∑Ph			11.21						14.209	16.560	16.560
M res.								77.77	158.10	171.07	291.54
M over.								0.00	-68.71	-128.64	-128.64
∑ M								77.77	89.39	42.43	162.90

Strength 5b (MAX) Load Combination
Bearing Resistance at Base of the Wall
Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

		CASE 1	CASE 2	CASE 3	CASE 3
x = (∑MR -∑MO)/∑V =	ft	4.27	3.05	1.29	2.42
e = B/2 -x =	ft	0.23	1.45	3.21	2.08
If e is less than zero, use zero					
Compute the ultimate bearing stress for Rock (11.3.6.2)					
Is resultant in middle 1/3 of Base: B/3 - 2absE > 0		Inside	Inside	Outside	Outside
If Inside					
σ_{vmax} = ∑V/B(1+6e/B) =	ksf/ft	2.33	6.39	11.52	17.89
σ_{vmin} = ∑V/B(1-6e/B) =	ksf/ft	1.72	0.11	-4.19	-2.91
If Outside					
σ_{vmax} = 2∑V/(3(B/2)-e) =	ksf/ft	2.74	4.86	6.41	11.81
σ_{vmin} = 0	ksf/ft	0.00	0.00	0.00	0.00

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR=\phi_b q_n$ = 17.30 ksf Rock Bearing per Geotech

Capacity:Demand Ratio (CDR)

	CASE 1	CASE 2	CASE 3	CASE 3
$CDR_{Bearing1}$ = qR/σv =	6.31	2.71	1.50	0.97
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**NG**

Factor of Safety Overturning (not per LRFD)

(Input values are shown underlined and in italics.)

FS_O = ABSOLUTE ∑MR/∑MO =	#DIV/0!	2.30	1.33	2.27
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

$e_{max} = 0.45B$ =	4.05	4.05	4.05	4.05
$CDR_{Eccentricity1}$ = emax/e =	17.80	2.80	1.26	1.94
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Sliding Resistance at Base of the Wall

Factored Sliding Force Ru = ∑Ph =	0.00	14.21	16.56	16.56

Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep}$ =

$\emptyset\tau R\tau$ = ∑V Tan ($\emptyset_{fd}$) =	10.94	17.59	19.82	40.50

where
$\emptyset\tau$ = 1.00

Compute passive resistance throughout the design life of the wall

$r_{ep1} = kp\, \gamma_{fd}\, y1$ = Nominal passive pressure at y1 = 2.66 kip/ft

$r_{ep2} = kp\, \gamma_{fd}\, y2$ = Nominal passive pressure at y2 = 2.66 kip/ft

$\phi epR_{ep} = (r_{ep1} + r_{ep2})(y_2 - y_1)/2$ = 0.00 kip/ft

ϕ_{ep} = 0.50

Compute nominal resistance against failure by sliding, Rn

$Rn = \emptyset\tau R_\tau + \emptyset_{ep} R_{ep}$ =	10.94	17.59	19.82	40.50

Compute factored resistance against failure by sliding, R_R

$R_R = \emptyset sRn$ =	10.94	17.59	19.82	40.50

$\emptyset s$ = 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.24	1.20	2.45
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

JOB NO.	*232245*	JOB NAME	Conneaut Old Main Street		
SUBJECT	*Design FA with AASHTO LRFD*	PREPARED BY	*MPS*	DATE	*16-Oct-25*
	Footing Elevation 580.1	CHECKED BY	*JDB*	DATE	*16-Oct-25*

H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Cast-In-Place Concrete Cantilever Wall on Spread Footing, LRFD

This Spreadhseet is adjusted for Soil

General

Design a cast-in-place (CIP) concrete wall supported on a spread footing conforming to the LRFD Bridge Design Specifications and the ODOT Bridge Manual.

Establish Project Requirements

See attached sketch for CIP concrete wall parameters

Summary of Calculation Below

Bearing

Service I Case 4 =	2.631 ksf
Eccentricity, e	1.536 ft
Strength 1b (MAX) Case 4	4.341 ksf
Eccentricity, e	1.957 ft
CDR, Sliding	1.143
CDR, OT	1.578

Design Parameters

Project Parameters

Design Life = 75 years — Wall design life (min) LRFD [11.5.1]

Soil Properties

Soil Data

Backfill soil Design Paramers

ϕ_f	*31.00*	°	Angle of internal friction.
γ_f	*125.00*	pcf	Unit weight of soil.
β	*0.00*	°	Angle of backslope.
c_f	*0.00*	ksf	Cohesion
δ_{Wall}	*0.00*	°	Friction angle between the soil and the wall.

Foundation Soil Design Parameters

ϕ_{fd}	*31.00*	°	Angle of internal friction.
γ_{fd}	*125.00*	pcf	Unit weight of soil.
β	*0.00*	°	Angle of backslope.
t	*0.00*	ft	pavement thickness
c_{fd}	*0.00*	ksf	Cohesion
δ_{Ftg}	*0.00*	°	Friction angle between the soil and the footing.

Concrete Data

γ_C	*150.0*	pcf	Unit weight of concrete.
f_Y	*60.0*	ksi	Steel yield.
f'_c	*4.0*	ksi	Concrete strength.

JOB NO.	*232245*	JOB NAME	Conneaut Old Main Street		
SUBJECT	*Design FA with AASHTO LRFD*	PREPARED BY	*MPS*	DATE	*16-Oct-25*
	Footing Elevation 580.1	CHECKED BY	*JDB*	DATE	*16-Oct-25*

H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Live Load Surcharge Parameters

$L_{traffic}$	*0.0*	ft	Distance from wall backface to edge of traffic
H/2	0.00	ft	Distance from wall backface where live loadsurcharge is considered in the wall design
H_{eq}	*2.000*	ft	Surcharge.

Resistance Factors

ϕ_b	*0.45*		Bearing resistance (Shallow Foundations) LRFD [Table 10.5.5.2.2-1]
ϕ_s	*1.00*		Sliding resistance LRFD [Table 11.5.7-1]
ϕ_t	*1.00*		Sliding resistance (shear resistance between soil and foundation) LRFD [Table 11.5.7-1]
ϕ_{ep}	*0.50*		Sliding resistance (passive resistance) LRFD [Table10.5.5.2.2-1]

Wall Geometry

He	*8.56*	ft	Exposed wall height, ft
Df	*5.10*	ft	Footing cover, ft
H=He+Df =	*13.66*	ft	Design wall height
Tt	*2.00*	ft	Section thickness.
b_1	*0.00*	in/ft	Front wall batter (b_1H:12V)
b_2	*0.00*	in/ft	Back wall batter (b_2H:12V)
bwh	*0.00*	in/ft	Back wall height

Preliminary Wall Dimensioning

H	*13.660*	ft	Design Wall height.	Do not Design for 0.67 height
B	*7.000*	ft	Footing length.	
A	*1.750*	ft	Toe length.	
D	*2.00*	ft	Footing thickness.	

Shear Key Dimensioning

Dkey	*0.00*	ft	Depth of shear key from bottom of footing, ft
Dw	*0.00*	ft	Width of shear key, ft
XK	*0.00*	ft	Distance from toe to shear key, ft

Other Wall Dimensioning

h'=H-D=	11.66		Stem + backwall height
T1 = b1h'/12=	0.00		Stem front batter width
T2=b2h'/12=	0.00		Stem back batter width
Tb = T1+Tt+T2=	2.00	ft	Stem thickness at bottom of wall
C=B-A-Tb=	3.25	ft	Heel projection
Φ = atan (12/b_2) =	90.00	deg	Angle of back face of wall to horizontal
b =	12.00	in	Concrete strip width for design
y1 = Df =	5.10	ft	Bottom of footing depth
y2 = Df + Dkey =	5.10	ft	Bottom of shear key depth
	13.16	ft	Retained soil height (neglect top 6 in wall reveal)
h_{cws} =	0.00	ft	shear key depth

JOB NO.	*232245*	JOB NAME	Conneaut Old Main Street		
SUBJECT	*Design FA with AASHTO LRFD*	PREPARED BY	*MPS*	DATE	*16-Oct-25*
	Footing Elevation 580.1	CHECKED BY	*JDB*	DATE	*16-Oct-25*

H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Compute Earth Pressure Coefficients

Compute Active Earth Pressure Coefficient

ϕ_f	31.00 °	0.54105 rad
β	0.00 °	0.00000 rad
δ_{Wall}	0.00 °	0.00000 rad
Φ	90.00 °	1.57080 rad

$$K_A = \frac{\mathrm{Sin}^2(\phi f+\Phi)}{\Gamma'\mathrm{Sin}(\Phi)^2\mathrm{Sin}(\Phi-'\mathrm{dWall})} \qquad \mathbf{0.320}$$

$$\Gamma = (1+((\mathrm{Sin}(\phi_f+\mathrm{dWall})\mathrm{Sin}(\phi_f-\beta))/(\mathrm{Sin}(\Phi-'\mathrm{dWall})\mathrm{Sin}(\Phi+\beta)))^2 \qquad \mathbf{2.295}$$

Compute Passive Earth Pressure Coefficient

K_P = $\mathrm{Tan}^2(45^\circ + \phi f/2)$ **3.124**

Compute Unfactored Loads

Active Earth Force Resultant (kip/ft), FT

$F_T = (K_A\, \gamma_f\, h^2)/2$

3.465 kip/ft Active soil force / foot width of wall.

Live Load Surcharge Load (kip/ft), Fsur

$F_{SUR} = K_A\, \gamma_f\, h_{eq} h$

1.053 kip/ft

Vertical Loads (kip/ft), V

$\mathbf{R_{DW}} = V_1 =$	0.00	kip/ft	Future Wearing Surface (DW)
$\mathbf{R_{DC}} = V_2 =$	0.00	kip/ft	Approach Slab (DC)
$\mathbf{R_{DC}} = V_3 =$	0.000	kip/ft	Superstructure (DC)
$\mathbf{R_{DC}} = V_4 =$	0.000	kip/ft	Backwall (DC)
$\mathbf{R_{DC}} = V_5 = DB'\gamma_C =$	2.100	kip/ft	Footing (DC)
$\mathbf{R_{DC}} = V_6 = Tbh'\gamma_C =$	3.498	kip/ft	Stem (DC)
$\mathbf{R_{EV}} = V_7 = Ch'\gamma_f =$	4.74	kip/ft	Soil backfill Vertical Earth Load (EV)
$\mathbf{R_{LL}} = V_8 =$	0.00	kip/ft	Live Load (LL) without impact

JOB NO. *232245* JOB NAME Conneaut Old Main Street

SUBJECT *Design FA with AASHTO LRFD* PREPARED BY *MPS* DATE *16-Oct-25*

Footing Elevation 580.1 CHECKED BY *JDB* DATE *16-Oct-25*

H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Moments produced from vertical loads about Point 'O' (kip-ft/ft), Mvi (Moment Arm (ft))

d_{v1} = A+1 = 2.75 ft

d_{v2} = A + Tb - 0.5 = 3.25 ft

d_{v3} =A + 1 = 2.75 ft

d_{v4}= A + Tb - 0.75 = 3.00 ft

d_{v5} =b/2 = 3.50 ft

d_{v6} = A + Tb/2 = 2.75 ft

d_{v7} = 5.38 ft

d_{v8} = A+1 = 2.75 ft

Horizontal Loads (kip/ft), Hi

$\mathbf{R_{LS} = H_1}$ = $F_{SUR}Cos(90deg-\Theta+\delta_{Wall})$ = 1.05 kip/ft Live load surcharge (LS)

$\mathbf{R_{EH} = H_2}$ = $F_TCos(90deg-\Theta+\delta_{Wall})$ = 3.46 kip/ft Active earth force(horizontal component) (EH)

Moments produced from horizontal loads about about Point 'O' (kip-ft/ft), MH

d_{h1} =h/2 = 6.58 ft

d_{h2} = h/3 = 4.39 ft

Use the following construction stagesfor analysis

CASE 1 wall DL

CASE 2 wall DL + Earth Pressure

CASE 3 Abut DL + Earth Pressure + LLsurcharge

CASE 4 Abut DL + Earth Pressure + Llsurcharge +DL superst + LL

Service 1 Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	SER 1 max/min	SER 1 PV k/ft	SER 1 PH k/ft	SER 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	0.00		2.75	1.00	0.00				0.00	0.00
V_2	DCas	0.00		3.25	1.00	0.00		0.00	0.00	0.00	0.00
V_3	DC_{super}	0.00		2.75	1.00	0.00					0.00
V_4	DC_{bw}	0.00		3.00	1.00	0.00		0.00	0.00	0.00	0.00
V_5	DCftg	2.10		3.50	1.00	2.10		7.35	7.35	7.35	7.35
V_6	DCstem	3.50		2.75	1.00	3.50		9.62	9.62	9.62	9.62
V_7	EV_{bf}	4.74		5.38	1.00	4.74			25.46	25.46	25.46
V_8	LL	0.00		2.75	1.00	0.00					0.00
H_1	LS		1.05	-6.58	1.00		1.05			-6.93	-6.93
H_2	EH		3.46	-4.39	1.00		3.46		-15.20	-15.20	-15.20
∑Pv		10.33						5.598	10.33	10.33	10.33
∑Ph			4.52						3.465	4.518	4.518
M res.								16.97	42.43	42.43	42.43
M over.								0.00	-15.20	-22.13	-22.13
∑ M								16.97	27.23	20.30	20.30

JOB NO. *232245* JOB NAME Conneaut Old Main Street
SUBJECT *Design FA with AASHTO LRFD* PREPARED BY *MPS* DATE *16-Oct-25*
Footing Elevation 580.1 CHECKED BY *JDB* DATE *16-Oct-25*
H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Service 1 Load Combination (Continued)

Bearing Resistance at Base of the Wall

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

		CASE 1	CASE 2	CASE 3	CASE 4
$x = (\sum MR - \sum MO)/\sum V =$	ft	3.03	2.63	1.96	1.96
$e = B/2 - x =$	ft	0.47	0.87	1.54	1.54

If e is less than zero, use zero

Compute the ultimate bearing stress for Soil (11.6.3.2)

		CASE 1	CASE 2	CASE 3	CASE 4
$\sigma_{vmax} = \sum V/(B-2e) =$	ksf/ft	0.92	1.96	2.63	2.63

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR=\phi_b q_n =$ 4.70 ksf Soil Bearing per geotech

Capacity:Demand Ratio (CDR)

	CASE 1	CASE 2	CASE 3	CASE 4
$CDR_{Bearing1} = qR/\sigma v =$	5.09	2.40	1.79	1.79
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Factor of Safety Overturning (not per LRFD)(Need 2 here for AASHTO Standard)

	CASE 1	CASE 2	CASE 3	CASE 4
FS_O = ABSOLUTE $\sum MR/\sum MO =$	#DIV/0!	2.79	1.92	1.92
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

	CASE 1	CASE 2	CASE 3	CASE 4
$e_{max} = 0.45B =$	3.15	3.15	3.15	3.15
$CDR_{Eccentricity1} = emax/e =$	6.72	3.64	2.05	2.05
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

(Input values are shown underlined and in italics.)

Sliding Resistance at Base of the Wall

Factored Sliding Force Ru = ∑Ph =	0.00	3.46	4.52	4.52
Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep}$ =				
$\emptyset\tau R\tau = \sum V \tan(\emptyset_{fd})$ =	3.36	6.21	6.21	6.21

where
$\emptyset\tau$ = 1.00

Compute passive resistance throughout the design life of the wall

$r_{ep1} = kp\ \gamma_{fd}\ y1$ = Nominal passive pressure at y1 = 1.99 kip/ft

$r_{ep2} = kp\ \gamma_{fd}\ y2$ = Nominal passive pressure at y2 = 1.99 kip/ft

$\phi epR_{ep} = (r_{ep1} + r_{ep2})(y_2 - y_1)/2$ = 0.00 kip/ft

ϕ_{ep} = 0.50

Compute nominal resistance against failure by sliding, Rn

$Rn = \emptyset\tau R_\tau + \emptyset_{ep} R_{ep}$ =	3.36	6.21	6.21	6.21

Compute factored resistance against failure by sliding, R_R

$R_R = \emptyset sRn$ =	3.36	6.21	6.21	6.21

$\emptyset s$ = 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.79	1.37	1.37
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Strength 1a (MIN) Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	STR 1 max/min	STR 1 PV k/ft	STR 1 PH k/ft	STR 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	0.00		2.75	0.65	0.00				0.00	0.00
V_2	DCas	0.00		3.25	0.90	0.00		0.00	0.00	0.00	0.00
V_3	DC_{super}	0.00		2.75	0.90	0.00					0.00
V_4	DC_{bw}	0.00		3.00	0.90	0.00		0.00	0.00	0.00	0.00
V_5	DCftg	2.10		3.50	0.90	1.89		6.62	6.62	6.62	6.62
V_6	DCstem	3.50		2.75	0.90	3.15		8.66	8.66	8.66	8.66
V_7	EV_{bf}	4.74		5.38	1.00	4.74			25.46	25.46	25.46
V_8	LL	0.00		2.75	1.75	0.00					0.00
H_1	LS		1.05	-6.58	1.75		1.84			-12.13	-12.13
H_2	EH		3.46	-4.39	0.90		3.12		-15.20	-13.68	-13.68
∑Pv		10.33						5.038	9.78	9.78	9.78
∑Ph			4.52						3.118	4.961	4.961
M res.								15.27	40.73	40.73	40.73
M over.								0.00	-15.20	-25.81	-25.81
∑ M								15.27	25.53	14.93	14.93

JOB NO. *232245* JOB NAME Conneaut Old Main Street
SUBJECT *Design FA with AASHTO LRFD* PREPARED BY *MPS* DATE *16-Oct-25*
Footing Elevation 580.1 CHECKED BY *JDB* DATE *16-Oct-25*
H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Strength 1a (MIN) Load Combination

Bearing Resistance at Base of the Wall

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

		CASE 1	CASE 2	CASE 3	CASE 4
$x = (\sum MR - \sum MO)/\sum V =$	ft	3.03	2.61	1.53	1.53
$e = B/2 - x =$	ft	0.47	0.89	1.97	1.97

If e is less than zero, use zero

Compute the ultimate bearing stress for Soil (11.6.3.2)

$\sigma_{vmax} = \sum V/(B-2e) =$	ksf/ft	0.83	1.87	3.20	3.20

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR=\phi_b q_n =$ 4.70 ksf Soil Bearing per geotech

Capacity:Demand Ratio (CDR)

$CDR_{Bearing1} = qR/\sigma v =$	5.66	2.51	1.47	1.47
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Factor of Safety Overturning (not per LRFD)

FS_O = ABSOLUTE $\sum MR/\sum MO =$	#DIV/0!	2.68	1.58	1.58
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

$e_{max} = 0.45B =$	3.15	3.15	3.15	3.15
$CDR_{Eccentricity1} = emax/e =$	6.72	3.55	1.60	1.60
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

(Input values are shown underlined and in italics.)

Sliding Resistance at Base of the Wall

Factored Sliding Force Ru = ∑Ph =	0.00	3.12	4.96	4.96

Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep}$ =

$\emptyset\tau R\tau = \sum V \tan(\emptyset_{fd})$ =	3.03	5.87	5.87	5.87

where $\emptyset\tau$ = 1
$\emptyset\tau$ = 1.00

Compute passive resistance throughout the design life of the wall

$r_{ep1} = kp\ \gamma_{fd}\ y1$ = Nominal passive pressure at y1 = 1.99 kip/ft
$r_{ep2} = kp\ \gamma_{fd}\ y2$ = Nominal passive pressure at y2 = 1.99 kip/ft

$\phi epR_{ep} = (r_{ep1} + r_{ep2})(y_2\text{-}y_1)/2$ = 0.00 kip/ft
ϕ_{ep} = 0.50

Compute nominal resistance against failure by sliding, Rn

$Rn = \emptyset\tau R_\tau + \emptyset_{ep} R_{ep}$ =	3.03	5.87	5.87	5.87

Compute factored resistance against failure by sliding, R_R

$R_R = \emptyset sRn$ =	3.03	5.87	5.87	5.87

$\emptyset s$ = 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.88	1.18	1.18
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Strength 1b (MAX) Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	STR 1 max/min	STR 1 PV k/ft	STR 1 PH k/ft	STR 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	0.00		2.75	1.50	0.00				0.00	0.00
V_2	DCas	0.00		3.25	1.25	0.00		0.00	0.00	0.00	0.00
V_3	DC_{super}	0.00		2.75	1.25	0.00					0.00
V_4	DC_{bw}	0.00		3.00	1.25	0.00		0.00	0.00	0.00	0.00
V_5	DCftg	2.10		3.50	1.25	2.63		9.19	9.19	9.19	9.19
V_6	DCstem	3.50		2.75	1.25	4.37		12.02	12.02	12.02	12.02
V_7	EV_{bf}	4.74		5.38	1.35	6.39			34.37	34.37	34.37
V_8	LL	0.00		2.75	1.75	0.00					0.00
H_1	LS		1.05	-6.58	1.75		1.84			-12.13	-12.13
H_2	EH		3.46	-4.39	1.50		5.20		-15.20	-22.80	-22.80
∑Pv		10.33						6.998	13.39	13.39	13.39
∑Ph			4.52						5.197	7.040	7.040
M res.								21.21	55.58	55.58	55.58
M over.								0.00	-15.20	-34.93	-34.93
∑ M								21.21	40.38	20.66	20.66

JOB NO. *232245* JOB NAME Conneaut Old Main Street
SUBJECT *Design FA with AASHTO LRFD* PREPARED BY *MPS* DATE *16-Oct-25*
Footing Elevation 580.1 CHECKED BY *JDB* DATE *16-Oct-25*
H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Strength 1b (MAX) Load Combination

Bearing Resistance at Base of the Wall

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

		CASE 1	CASE 2	CASE 3	CASE 4
$x = (\sum MR - \sum MO)/\sum V =$	ft	3.03	3.02	1.54	1.54
$e = B/2 - x =$	ft	0.47	0.48	1.96	$\boxed{1.96}$

If e is less than zero, use zero

Compute the ultimate bearing stress for Soil (11.6.3.2)

		CASE 1	CASE 2	CASE 3	CASE 4
$\sigma_{vmax} = \sum V/(B-2e) =$	ksf/ft	1.15	2.22	4.34	$\boxed{4.34}$

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR=\phi_b q_n =$ 4.70 ksf Soil Bearing per geotech

Capacity:Demand Ratio (CDR)

	CASE 1	CASE 2	CASE 3	CASE 4
$CDR_{Bearing1} = qR/\sigma v =$	4.07	2.12	1.08	1.08
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Factor of Safety Overturning (not per LRFD)

	CASE 1	CASE 2	CASE 3	CASE 4
FS_O = ABSOLUTE $\sum MR/\sum MO =$	#DIV/0!	3.66	1.59	1.59
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

	CASE 1	CASE 2	CASE 3	CASE 4
$e_{max} = 0.45B =$	3.15	3.15	3.15	3.15
$CDR_{Eccentricity1} = emax/e =$	6.72	6.50	1.61	1.61
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Sliding Resistance at Base of the Wall

	CASE 1	CASE 2	CASE 3	CASE 4
Factored Sliding Force Ru = $\sum Ph$ =	0.00	5.20	7.04	7.04

Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep} =$

	CASE 1	CASE 2	CASE 3	CASE 4
$\emptyset\tau R\tau = \sum V \tan(\emptyset_{fd}) =$	4.20	8.05	8.05	8.05

where
$\emptyset\tau$ = 1.00

Compute passive resistance throughout the design life of the wall

JOB NO. *232245* JOB NAME Conneaut Old Main Street
SUBJECT *Design FA with AASHTO LRFD* PREPARED BY *MPS* DATE *16-Oct-25*
Footing Elevation 580.1 CHECKED BY *JDB* DATE *16-Oct-25*
H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

$r_{ep1} = kp\ \gamma_{fd}\ y1 =$ Nominal passive pressure at y1 = 1.99 kip/ft

$r_{ep2} = kp\ \gamma_{fd}\ y2 =$ Nominal passive pressure at y2 = 1.99 kip/ft

$\phi epR_{ep} = (r_{ep1} + r_{ep2})(y_2\text{-}y_1)/2 =$ 0.00 kip/ft

$\phi_{ep} =$ 0.50

Compute nominal resistance against failure by sliding, Rn

$Rn = \emptyset\tau R_\tau + \emptyset_{ep}R_{ep} =$ 4.20 8.05 8.05 8.05

Compute factored resistance against failure by sliding, R_R

$R_R = \emptyset sRn =$ 4.20 8.05 8.05 8.05

$\emptyset s =$ 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru = NA 1.55 **1.14** **1.14**

Is the CDR greater than 1.0 ? **OK** **OK** **OK** **OK**

may be removed if rock excavation prohibits

Strength 3 a (MIN) Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	STR 1 max/min	STR 1 PV k/ft	STR 1 PH k/ft	STR 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	0.00		2.75	0.65	0.00				0.00	0.00
V_2	DCas	0.00		3.25	0.90	0.00		0.00	0.00	0.00	0.00
V_3	DC_{super}	0.00		2.75	0.90	0.00					0.00
V_4	DC_{bw}	0.00		3.00	0.90	0.00		0.00	0.00	0.00	0.00
V_5	DCftg	2.10		3.50	0.90	1.89		6.62	6.62	6.62	6.62
V_6	DCstem	3.50		2.75	0.90	3.15		8.66	8.66	8.66	8.66
V_7	EV_{bf}	4.74		5.38	1.00	4.74			25.46	25.46	25.46
V_8	LL	0.00		2.75	0.00	0.00					0.00
H_1	LS		1.05	-6.58	0.00		0.00			0.00	0.00
H_2	EH		3.46	-4.39	0.90		3.12		-15.20	-13.68	-13.68
∑Pv		10.33						5.038	9.78	9.78	9.78
∑Ph			4.52						3.118	3.118	3.118
M res.								15.27	40.73	40.73	40.73
M over.								0.00	-15.20	-13.68	-13.68
∑ M								15.27	25.53	27.05	27.05

(Input values are shown underlined and in italics.)

Strength 3a (MIN) Load Combination

Bearing Resistance at Base of the Wall

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

		CASE 1	CASE 2	CASE 3	CASE 4
$x = (\sum MR - \sum MO)/\sum V =$	ft	3.03	2.61	2.77	2.77
$e = B/2 - x =$	ft	0.47	0.89	0.73	0.73

If e is less than zero, use zero

Compute the ultimate bearing stress for Soil (11.6.3.2)

		CASE 1	CASE 2	CASE 3	CASE 4
$\sigma_{vmax} = \sum V/(B-2e) =$	ksf/ft	0.83	1.87	1.77	1.77

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR=\phi_b q_n =$ 4.70 ksf Soil Bearing per geotech

Capacity:Demand Ratio (CDR)

	CASE 1	CASE 2	CASE 3	CASE 4
$CDR_{Bearing1} = qR/\sigma v =$	5.66	2.51	2.66	2.66
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Factor of Safety Overturning (not per LRFD)

	CASE 1	CASE 2	CASE 3	CASE 4
FS_O = ABSOLUTE $\sum MR/\sum MO =$	#DIV/0!	2.68	2.98	2.98
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

	CASE 1	CASE 2	CASE 3	CASE 4
$e_{max} = 0.45B =$	3.15	3.15	3.15	3.15
$CDR_{Eccentricity1} = emax/e =$	6.72	3.55	4.30	4.30
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Sliding Resistance at Base of the Wall

	CASE 1	CASE 2	CASE 3	CASE 4
Factored Sliding Force $Ru = \sum Ph =$	0.00	3.12	3.12	3.12

Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep} =$

	CASE 1	CASE 2	CASE 3	CASE 4
$\emptyset\tau R\tau = \sum V \tan(\emptyset_{fd}) =$	3.03	5.87	5.87	5.87

(Input values are shown underlined and in italics.)

where
$\varnothing\tau$ = 1.00

Compute passive resistance throughout the design life of the wall

$r_{ep1} = kp\ \gamma_{fd}\ y1$ = Nominal passive pressure at y1 = 1.99 kip/ft

$r_{ep2} = kp\ \gamma_{fd}\ y2$ = Nominal passive pressure at y2 = 1.99 kip/ft

$\phi epR_{ep} = (r_{ep1} + r_{ep2})(y_2 - y_1)/2$ = 0.00 kip/ft
ϕ_{ep} = 0.50
Compute nominal resistance against failure by sliding, Rn

$Rn = \varnothing\tau R_\tau + \varnothing_{ep}R_{ep}$ =	3.03	5.87	5.87	5.87

Compute factored resistance against failure by sliding, R_R

$R_R = \varnothing sRn$ =	3.03	5.87	5.87	5.87

$\varnothing s$ = 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.88	1.88	1.88
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Strength 3b (MAX) Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	STR 1 max/min	STR 1 PV k/ft	STR 1 PH k/ft	STR 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	0.00		2.75	1.50	0.00				0.00	0.00
V_2	DCas	0.00		3.25	1.25	0.00		0.00	0.00	0.00	0.00
V_3	DC_{super}	0.00		2.75	1.25	0.00					0.00
V_4	DC_{bw}	0.00		3.00	1.25	0.00		0.00	0.00	0.00	0.00
V_5	DCftg	2.10		3.50	1.25	2.63		9.19	9.19	9.19	9.19
V_6	DCstem	3.50		2.75	1.25	4.37		12.02	12.02	12.02	12.02
V_7	EV_{bf}	4.74		5.38	1.35	6.39			34.37	34.37	34.37
V_8	LL	0.00		2.75	0.00	0.00					0.00
H_1	LS		1.05	-6.58	0.00		0.00			0.00	0.00
H_2	EH		3.46	-4.39	1.50		5.20		-15.20	-22.80	-22.80
∑Pv		10.33						6.998	13.39	13.39	13.39
∑Ph			4.52						5.197	5.197	5.197
M res.								21.21	55.58	55.58	55.58
M over.								0.00	-15.20	-22.80	-22.80
∑ M								21.21	40.38	32.79	32.79

Strength 3b (MAX) Load Combination
Bearing Resistance at Base of the Wall CASE 1 CASE 2 CASE 3 CASE 3
Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

(Input values are shown underlined and in italics.)

$x = (\sum MR - \sum MO)/\sum V =$	ft	3.03	3.02	2.45	2.45
$e = B/2 - x =$	ft	0.47	0.48	1.05	1.05

If e is less than zero, use zero
Compute the ultimate bearing stress for Soil (11.6.3.2)

$\sigma_{vmax} = \sum V/(B-2e) =$	ksf/ft	1.15	2.22	2.74	2.74

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR=\phi_b q_n =$ 4.70 ksf Soil Bearing per geotech

Capacity:Demand Ratio (CDR)

$CDR_{Bearing1} = qR/\sigma v =$	4.07	2.12	1.72	1.72
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Factor of Safety Overturning (not per LRFD)

FS_O = ABSOLUTE $\sum MR/\sum MO =$	#DIV/0!	3.66	2.44	2.44
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

$e_{max} = 0.45B =$	3.15	3.15	3.15	3.15
$CDR_{Eccentricity1} = emax/e =$	6.72	6.50	2.99	2.99
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Sliding Resistance at Base of the Wall

Factored Sliding Force Ru = $\sum Ph =$	0.00	5.20	5.20	5.20

Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep} =$

$\emptyset\tau R\tau = \sum V \tan(\emptyset_{fd}) =$	4.20	8.05	8.05	8.05

where
$\emptyset\tau$ = 1.00

Compute passive resistance throughout the design life of the wall

$r_{ep1} = kp\ \gamma_{fd}\ y1 =$ Nominal passive pressure at y1 = 1.99 kip/ft

$r_{ep2} = kp\ \gamma_{fd}\ y2 =$ Nominal passive pressure at y2 = 1.99 kip/ft

$\phi epR_{ep} = (r_{ep1} + r_{ep2})(y_2-y_1)/2 =$ 0.00 kip/ft

$\phi_{ep} =$ 0.50

Compute nominal resistance against failure by sliding, Rn

$Rn = \emptyset\tau R_\tau + \emptyset_{ep}R_{ep} =$	4.20	8.05	8.05	8.05

Compute factored resistance against failure by sliding, R_R

$R_R = \emptyset sRn =$	4.20	8.05	8.05	8.05

$\emptyset s =$ 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.55	1.55	1.55
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Strength 5 a (MIN) Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	STR 1 max/min	STR 1 PV k/ft	STR 1 PH k/ft	STR 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	0.00		2.75	0.65	0.00				0.00	0.00
V_2	DCas	0.00		3.25	0.90	0.00		0.00	0.00	0.00	0.00
V_3	DC_{super}	0.00		2.75	0.90	0.00					0.00
V_4	DC_{bw}	0.00		3.00	0.90	0.00		0.00	0.00	0.00	0.00
V_5	DCftg	2.10		3.50	0.90	1.89		6.62	6.62	6.62	6.62
V_6	DCstem	3.50		2.75	0.90	3.15		8.66	8.66	8.66	8.66
V_7	EV_{bf}	4.74		5.38	1.00	4.74			25.46	25.46	25.46
V_8	LL	0.00		2.75	1.35	0.00					0.00
H_1	LS		1.05	-6.58	1.35		1.42			-9.35	-9.35
H_2	EH		3.46	-4.39	0.90		3.12		-15.20	-13.68	-13.68
∑Pv		10.33						5.038	9.78	9.78	9.78
∑Ph			4.52						3.118	4.540	4.540
M res.								15.27	40.73	40.73	40.73
M over.								0.00	-15.20	-23.03	-23.03
∑ M								15.27	25.53	17.70	17.70

JOB NO.	*232245*	JOB NAME	Conneaut Old Main Street		
SUBJECT	*Design FA with AASHTO LRFD*	PREPARED BY	*MPS*	DATE	*16-Oct-25*
	Footing Elevation 580.1	CHECKED BY	*JDB*	DATE	*16-Oct-25*

H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Strength 5a (MIN) Load Combination

Bearing Resistance at Base of the Wall

		CASE 1	CASE 2	CASE 3	CASE 4
Compute resultant location (distance from Point 'O' Figure E14-1.4-3)					
$x = (\sum MR - \sum MO)/\sum V =$	ft	3.03	2.61	1.81	1.81
$e = B/2 - x =$	ft	0.47	0.89	1.69	1.69
If e is less than zero, use zero					
Compute the ultimate bearing stress for Soil (11.6.3.2)					
$\sigma_{vmax} = \sum V/(B-2e) =$	ksf/ft	0.83	1.87	2.70	2.70

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR=\phi_b q_n =$ 4.70 ksf Soil Bearing per geotech

Capacity:Demand Ratio (CDR)

	CASE 1	CASE 2	CASE 3	CASE 4
$CDR_{Bearing1} = qR/\sigma v =$	5.66	2.51	1.74	1.74
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Factor of Safety Overturning (not per LRFD)

	CASE 1	CASE 2	CASE 3	CASE 4
FS_O = ABSOLUTE $\sum MR/\sum MO =$	#DIV/0!	2.68	1.77	1.77
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

	CASE 1	CASE 2	CASE 3	CASE 4
$e_{max} = 0.45B =$	3.15	3.15	3.15	3.15

(Input values are shown underlined and in italics.)

$CDR_{Eccentricity1}$ = emax/e =	6.72	3.55	1.86	1.86
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Sliding Resistance at Base of the Wall

Factored Sliding Force Ru = ∑Ph =	0.00	3.12	4.54	4.54

Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep}$ =

$\emptyset\tau R\tau = \sum V \text{ Tan } (\emptyset_{fd})$ =	3.03	5.87	5.87	5.87

where
$\emptyset\tau$ = 1.00

Compute passive resistance throughout the design life of the wall

$r_{ep1} = kp\ \gamma_{fd}\ y1$ = Nominal passive pressure at y1 = 1.99 kip/ft
$r_{ep2} = kp\ \gamma_{fd}\ y2$ = Nominal passive pressure at y2 = 1.99 kip/ft

$\phi epR_{ep} = (r_{ep1} + r_{ep2})(y_2 - y_1)/2$ = 0.00 kip/ft
ϕ_{ep} = 0.50
Compute nominal resistance against failure by sliding, Rn

$Rn = \emptyset\tau R_\tau + \emptyset_{ep} R_{ep}$ =	3.03	5.87	5.87	5.87

Compute factored resistance against failure by sliding, R_R

$R_R = \emptyset sRn$ =	3.03	5.87	5.87	5.87

$\emptyset s$ = 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.88	1.29	1.29
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

JOB NO.	*232245*	JOB NAME	Conneaut Old Main Street		
SUBJECT	*Design FA with AASHTO LRFD*	PREPARED BY	*MPS*	DATE	*16-Oct-25*
	Footing Elevation 580.1	CHECKED BY	*JDB*	DATE	*16-Oct-25*

H:\2023\232245\PHASE\01 Base Design Services\Bridge Design\Abutments\Forward Abutment

(Input values are shown underlined and in italics.)

Strength 5b (MAX) Load Combination

LOAD	LRFD Load Type	unfact. vert. loads k/ft	unfact. hor. loads k/ft	ARM	STR 1 max/min	STR 1 PV k/ft	STR 1 PH k/ft	STR 1 max Moments CASE 1	CASE 2	CASE 3	CASE 4
V_1	DW_{ws}	0.00		2.75	1.50	0.00				0.00	0.00
V_2	DCas	0.00		3.25	1.25	0.00		0.00	0.00	0.00	0.00
V_3	DC_{super}	0.00		2.75	1.25	0.00					0.00
V_4	DC_{bw}	0.00		3.00	1.25	0.00		0.00	0.00	0.00	0.00
V_5	DCftg	2.10		3.50	1.25	2.63		9.19	9.19	9.19	9.19
V_6	DCstem	3.50		2.75	1.25	4.37		12.02	12.02	12.02	12.02
V_7	EV_{bf}	4.74		5.38	1.35	6.39			34.37	34.37	34.37
V_8	LL	0.00		2.75	1.35	0.00					0.00
H_1	LS		1.05	-6.58	1.35		1.42			-9.35	-9.35
H_2	EH		3.46	-4.39	1.50		5.20		-15.20	-22.80	-22.80
∑Pv		10.33						6.998	13.39	13.39	13.39
∑Ph			4.52						5.197	6.619	6.619
M res.								21.21	55.58	55.58	55.58
M over.								0.00	-15.20	-32.15	-32.15
∑ M								21.21	40.38	23.43	23.43

Strength 5b (MAX) Load Combination

Bearing Resistance at Base of the Wall

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

		CASE 1	CASE 2	CASE 3	CASE 3
x = (∑MR -∑MO)/∑V =	ft	3.03	3.02	1.75	1.75
e = B/2 -x =	ft	0.47	0.48	1.75	1.75

If e is less than zero, use zero

Compute the ultimate bearing stress for Soil (11.6.3.2)

σ_{vmax} = ∑V/(B-2e) =	ksf/ft	1.15	2.22	3.83	3.83

Compute factored bearing resistance, qR, LRFD [Eq 10.6.3.1.1]

$qR=\phi_b q_n$ = 4.70 ksf Soil Bearing per geotech

Capacity:Demand Ratio (CDR)

$CDR_{Bearing1}$ = qR/σv =	4.07	2.12	1.23	1.23
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Factor of Safety Overturning (not per LRFD)

(Input values are shown underlined and in italics.)

FS_O = ABSOLUTE ∑MR/∑MO =	#DIV/0!	3.66	1.73	1.73
Is the FS greater than 1.0 ?	**#DIV/0!**	**OK**	**OK**	**OK**

Limiting Eccentricity at Base of the Wall (LRFD 11.6.3.3)

Maximum eccentricity

e_{max} = 0.45B =	3.15	3.15	3.15	3.15
$CDR_{Eccentricity1}$ = emax/e =	6.72	6.50	1.80	1.80
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Sliding Resistance at Base of the Wall

Factored Sliding Force Ru = ∑Ph =	0.00	5.20	6.62	6.62

Sliding Resistance, $R_R = \emptyset_s Rn = \emptyset_t Rt + \emptyset_{ep} R_{ep}$ =

$\emptyset\tau R\tau = \sum V \tan(\emptyset_{fd})$ =	4.20	8.05	8.05	8.05

where
$\emptyset\tau$ = 1.00

Compute passive resistance throughout the design life of the wall

$r_{ep1} = kp\, \gamma_{fd}\, y1$ = Nominal passive pressure at y1 = 1.99 kip/ft
$r_{ep2} = kp\, \gamma_{fd}\, y2$ = Nominal passive pressure at y2 = 1.99 kip/ft

$\phi epR_{ep} = (r_{ep1} + r_{ep2})(y_2 - y_1)/2$ = 0.00 kip/ft
ϕ_{ep} = 0.50

Compute nominal resistance against failure by sliding, Rn

$Rn = \emptyset\tau R_\tau + \emptyset_{ep} R_{ep}$ =	4.20	8.05	8.05	8.05

Compute factored resistance against failure by sliding, R_R

$R_R = \emptyset sRn$ =	4.20	8.05	8.05	8.05

$\emptyset s$ = 1.00

Capacity:Demand Ratio (CDR)

CRDSliding1 = RR/Ru =	NA	1.55	1.22	1.22
Is the CDR greater than 1.0 ?	**OK**	**OK**	**OK**	**OK**

Appendix C

Temporary Shoring Calculations

SOLDIER PILE & LAGGING DESIGN CALCULATIONS

TEMPORARY SHORING DESIGN

Project Parameters

- Replacement of the existing Main Street bridge over Conneaut Creek will require construction of new forward and rear abutments.

- The rear abutment is adjacent to an existing active rail line and will require excavation to construct. The railroad tracks are straight, and the excavation will be parallel to them.

- Due to the excavation's proximity to the tracks, shoring will be required.

- The temporary shoring system will consist of soldier piles and timber lagging. The soldier piles will be steel wide-flange member enbedded in concrete-filled, rock socket drilled shafts. The lagging will span between piles.

- The distance from centerline of railroad track to face of the temporary shoring will be 18'-0" . Therefore, per the figure below, in addition to active eath pressure and dead load surcharges, we also have to design for the Railroad Cooper E80 Live load.

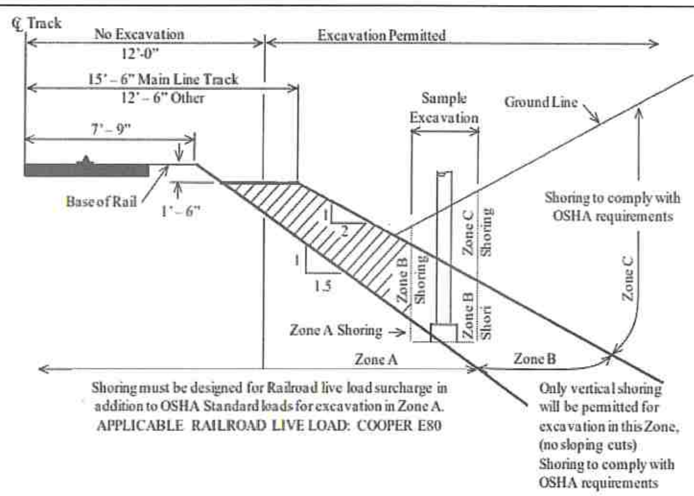


Figure 8-2. General Railroad Requirements (GTS section 6, p6)

Dead Load Surcharge Calculation

- Roadbed *(balast and track ties)*

The existing grade at the approach to the rear abutment is essentially flat. The top of rail is also approximately flush with the existing grade. These conditions will remain unchanged during construction of the rear abutment. Therefore, the roadbed dead load will already be accounted for by the retained soil active earth pressure.

- Track

Conservatively, assume the combined weight of rails, inside guard rails and fasteners is approximately 200 lb/linear ft. Since this load will be distributed to the underlying soils via the ties, which are typically 9'-0" long, the equivalent uniform load will be: 200 lb/ft / 9 ft = 22.2 psf

- Spoils

Per the soils report, the unit weight of the retained soils is approximately 125 lb/ft^3 . We will assume a 2'-0" allowance for spoil piled next to the excavation, which equates to : 2' x 125 lb/ft^3 = 250 psf

- For simplicity, we will address the above surcharge loads by increasing the theoretical excavation depth.
- From the geotechnical borings, rock was encountered at the following reference elevations (See CT Consultants Soils Report and Verdantas Sheet P16):

ground El. =	593.71	ft
btm of footing EL. =	583.90	ft
top of decomposed bedrock *(sand & gravel)* EL. =	581.40	ft
top of rock EL. =	577.00	ft

Δ EL. between top of decomposed bedrock *(sand & gravel)* & btm. of footing
= 583.90' - 581.40' = 2.50 ft

$\therefore$ soil wt. = 2.500' x 125 lb/ft^3 = 312.5 psf *> than 272.2 psf dead load surcharge req'd, ∴ Ok*

Note: *Based upon Boring B-001 stratigraphy, the Geotechnical Engineer determined the top of non-corable decomposed bedrock (sand and gravel) and corable weathered bedrock to be at elevations 581.4 and 577.0, respectively. Similarly, based upon the SPT/RQD values, and absent the need to meet the long-term serviceability requirements of the permanent replacement bridge structure, the Geotechnical engineer also determined that it was acceptable to start embedment of the temporary shoring in the non-corablede composed bedrock (sand and gravel) layer. Therefore, top of weathered rock elevation 581.40 is used as the basis for the temporary design calculations that follow.*

TEMPORARY SHORING DESIGN - Continued

(Determination of Surcharge Forces from Rail Line Adjacent to Bridge Abutment)

References:

1. Das , B. M, Principles of Foundation Engineering - 7th Edition , Cengage Learning, 2011, Pages 348 - 350

Reference Elevations *(re Verdantas Sheet P.06)*

ground El. = 593.71 ft

btm of footing EL. = 583.90 ft

top of decomposed (sand & gravel) bedrock EL. = 581.40 ft

Calculate Surcharge Strip Live Load Intensity 'q' Based on E80 Cooper Load

ht. from btm of tie to top of shoring , H_1 = 0.000 ft

tie length, L_{tie} = 9.000 ft — **See attached Figure 5.1 for calculation equations*

$L_d = L_{tie} + H_1$ = 9.000 ft

$q = 80{,}000/(5L_d)$ = 1,778 psf

Calculate Lateral Pressure 'σ' Due to the Surcharge *(@ depth $H_2 = D$)*

- Conservatively per the Note on Sheet 2, the ' D' dimension in Figure 5.1 is taken as the distance from existing grade to the top of the decomposed bedrock (sand and gravel) layer. Continuing with Reference 1, Figure 7.4, depth of pt. being evaluated with Boussinesq eqn , for $H = H_2 = D$

dist. from top of shoring to the dredge line, D = 12.31 ft, ∴ use 12.333 ft

horiz. dist. btwn track centerline to face of shoring , S = 18.000 ft

$a' = L_d$ = 9.000 ft — **See attached Figure 7.4 from Reference 1 for calculation equations*

$b' = S - L_{tie}/2$ = 13.500 ft

for $H = H_2 = D$ = 12.333 ft

$\theta_1 = \arctan(b'/H)$ =	0.831 rad =	47.586 deg	
$\theta_2 = \arctan[(a'+b')/H]$ =	1.069 rad =	61.271 deg	*(eqn 7-35)*
$\beta = \theta_2 - \theta_1$ =	0.239 rad =	13.685 deg	*(eqn 7-36)*
$\alpha = (\theta_1 + \theta_2)/2$ =	0.950 rad =	54.428 deg	
* $\sigma = 2q(\beta - \sin\beta\cos\alpha)]/\pi$ =	356.90 psf		*(eqn 7-33)*

**Note: α and β are in radians for this equation*

Calculate the Distance z' to the Resultant Shear Force *(for H =D, measured from bottom of excavation)*

$R = (a' + b')^2(90 - \theta_2)$ = 14,544.19 *(eqn 7-38)*

$Q = b'^2(90 - \theta_1)$ = 7,729.99 *(eqn 7-39)*

$z' = H - [H^2(\theta_2 - \theta_1) + (R - Q) - 57.30a'H]/[2H(\theta_2 - \theta_1)]$ = 4.822 ft *(eqn 7-37)*

Calculate Resultant Shear Force 'R_x' due to Surcharge *(for H = D)*

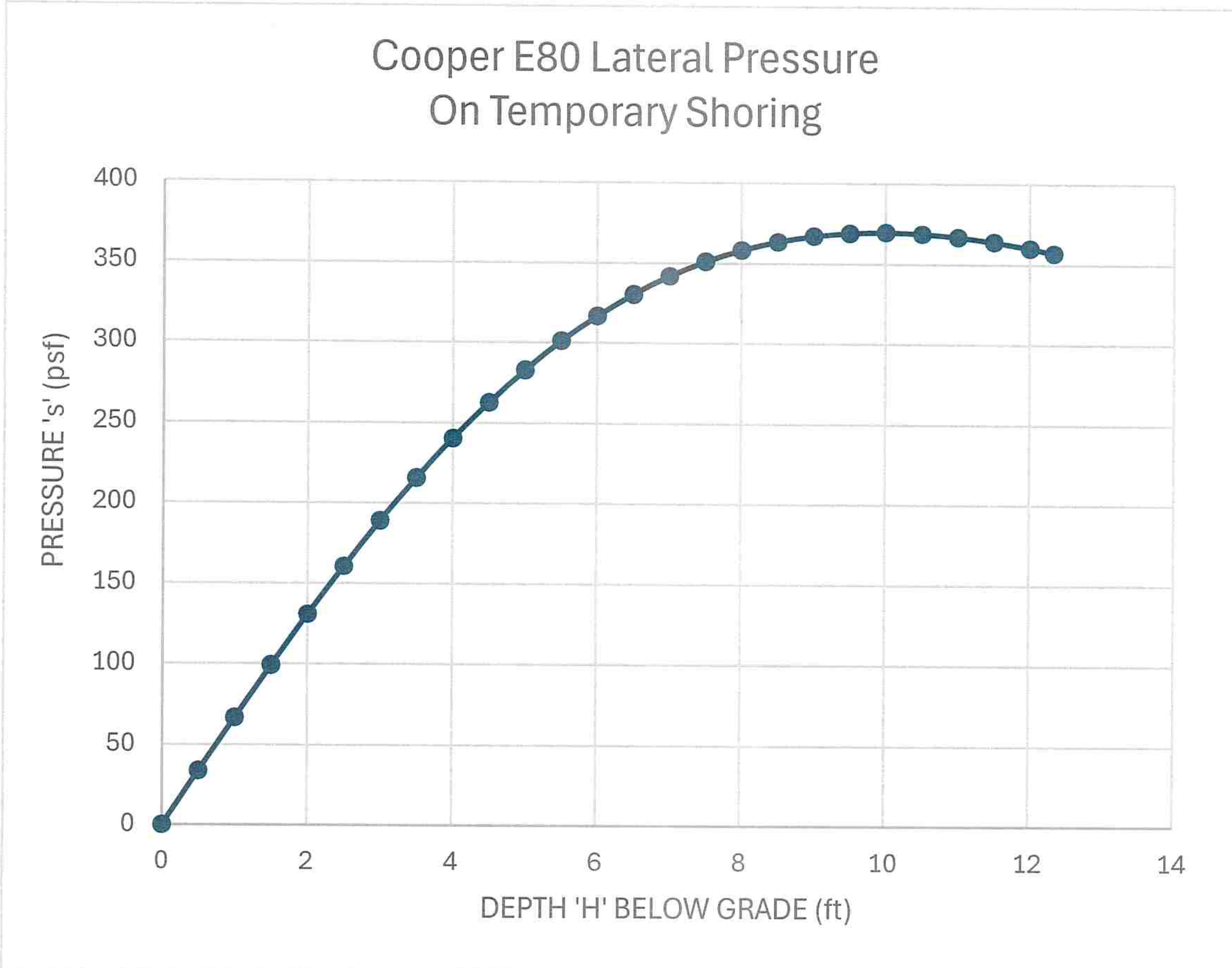
$P = q[H(\theta_2 - \theta_1)]/90$ = 3.334 kip/ft of wall *(eqn 7-34)*

Calculate Depth Resultant Moment 'M' *(@ H = D)*

$M = Pz'$ = 16.078 k-ft/ft of wall

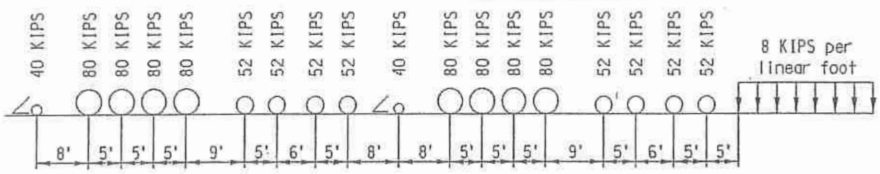
Calculate Lateral Pressures 'σ' Between Top of Wall and Top of Rock *(z = 0 to z =12.333 ft)*

z (ft)	θ_1 (rad)	θ_2 (rad)	β (rad)	α (rad)	σ (psf)
0.000	0.000	0.000	0.000	0.000	0.000
0.500	1.5338	1.5486	0.0148	1.5412	33.48
1.000	1.4969	1.5264	0.0295	1.5116	66.60
1.500	1.4601	1.5042	0.0441	1.4822	99.01
2.000	1.4237	1.4821	0.0584	1.4529	130.39
2.500	1.3877	1.4601	0.0725	1.4239	160.44
3.000	1.3521	1.4382	0.0861	1.3952	188.89
3.500	1.3171	1.4165	0.0994	1.3668	215.52
4.000	1.2827	1.3949	0.1121	1.3388	240.15
4.500	1.2490	1.3734	0.1244	1.3112	262.66
5.000	1.2161	1.3521	0.1360	1.2841	282.94
5.500	1.1839	1.3311	0.1471	1.2575	300.96
6.000	1.1526	1.3102	0.1576	1.2314	316.70
6.500	1.1221	1.2896	0.1675	1.2058	330.20
7.000	1.0924	1.2692	0.1767	1.1808	341.53
7.500	1.0637	1.2490	0.1853	1.1564	350.75
8.000	1.0358	1.2292	0.1933	1.1325	357.99
8.500	1.0089	1.2096	0.2007	1.1092	363.35
9.000	0.9828	1.1903	0.2075	1.0865	366.97
9.500	0.9576	1.1713	0.2137	1.0644	369.00
10.000	0.9332	1.1526	0.2193	1.0429	369.56
10.500	0.9098	1.1342	0.2244	1.0220	368.80
11.000	0.8871	1.1161	0.2290	1.0016	366.86
11.500	0.8652	1.0983	0.2331	0.9818	363.87
12.000	0.8442	1.0808	0.2367	0.9625	359.96
12.333	0.8305	1.0694	0.2388	0.9500	356.90


Cooper E80 Lateral Pressure
On Temporary Shoring
PRESSURE 's' (psf)
400
350
300
250
200
150
100
50
0
0
2
4
6
8
10
12
14
DEPTH 'H' BELOW GRADE (ft)

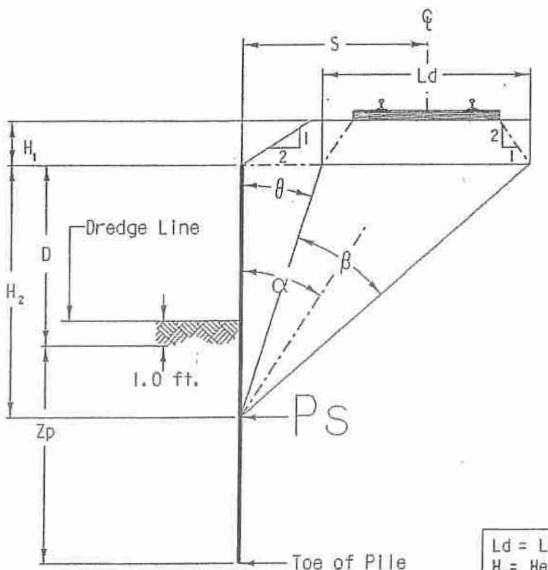
5. APPENDIX

5.1 LIVE LOAD PRESSURE DUE TO COOPER E80 LOADING



COOPER E80 LOAD

SCALE: (NOT TO SCALE)



Ps = Lateral Presssure due to Live Load

PS for Track Parallel to Shoring

$$Ps = \frac{2q}{\pi}\left[\beta - \sin\beta\cos\left(2 \times \left(\theta + \frac{\beta}{2}\right)\right)\right]$$

Calculated using the Boussinesq Strip Load Equation.
α and β are angles measured in radians.

PS for Track Perpendicular to Shoring

$$Ps = K_A\, q$$

where $K_A = \tan^2(45 - \frac{\phi}{2})$
ϕ is the angle of internal friction in degrees

q is the intensity of strip load due to E80 Railroad live load and shall be calculated as follows:

For $H_1 = 0$, $q = \dfrac{80{,}000 \text{ lb}}{(5 \text{ feet})(9 \text{ feet})} = 1{,}778$ psf

For $H_1 > 0$, $q = \dfrac{80{,}000 \text{ lb}}{(5 \text{ feet})(Ld)}$

Ld = Length of tie (9 feet) plus H_1.
H_1 = Height from the bottom of tie to the top of shoring.
H_2 = Depth of point being evaluated with the Boussinesq equation.
S = Distance perpendicular from centerline of track to the face of shoring.
D = Top of shoring to one foot below dredge line.
Zp = The minimum embedment depth.

7.6 Lateral Earth Pressure Due to Surcharge

In several instances, the theory of elasticity is used to determine the lateral earth pressure on unyielding retaining structures caused by various types of surcharge loading, such as *line loading* (Figure 7.14a) and *strip loading* (Figure 7.14b).

According to the theory of elasticity, the stress at any depth, z, on a retaining structure caused by a line load of intensity q/unit length (Figure 7.14a) may be given as

$$\sigma = \frac{2q}{\pi H}\frac{a^2 b}{(a^2 + b^2)^2} \tag{7.29}$$

where σ = horizontal stress at depth $z = bH$

(See Figure 7.14a for explanations of the terms a and b.)

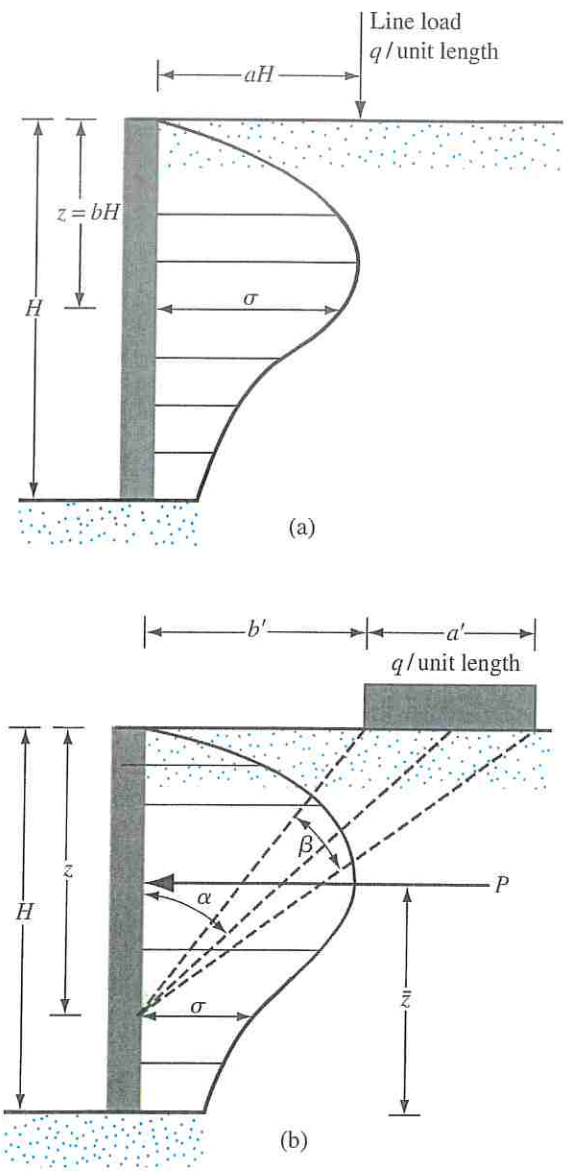


Figure 7.14 Lateral earth pressure caused by (a) line load and (b) strip load

However, because soil is not a perfectly elastic medium, some deviations from Eq. (7.29) may be expected. The modified forms of this equation generally accepted for use with soils are as follows:

$$\sigma = \frac{4a}{\pi H}\frac{a^2 b}{(a^2 + b^2)} \qquad \text{for } a > 0.4 \tag{7.30}$$

and

$$\sigma = \frac{q}{H}\frac{0.203b}{(0.16 + b^2)^2} \qquad \text{for } a \leq 0.4 \tag{7.31}$$

Figure 7.14b shows a strip load with an intensity of q/unit area located at a distance b' from a wall of height H. Based on the theory of elasticity, the horizontal stress, σ, at any depth z on a retaining structure is

$$\sigma = \frac{q}{\pi}(\beta - \sin\beta\cos 2\alpha) \tag{7.32}$$

(The angles α and β are defined in Figure 7.14b.)

However, in the case of soils, the right-hand side of Eq. (7.32) is doubled to account for the yielding soil continuum, or

$$\sigma = \frac{2q}{\pi}(\beta - \sin\beta\cos 2\alpha) \tag{7.33}$$

The total force per unit length (P) due to the *strip loading only* (Jarquio, 1981) may be expressed as

$$P = \frac{q}{90}[H(\theta_2 - \theta_1)] \tag{7.34}$$

where

$$\theta_1 = \tan^{-}\left(\frac{b'}{H}\right) \quad \text{(deg)} \tag{7.35}$$

$$\theta_2 = \tan^{-1}\left(\frac{a' + b'}{H}\right) \text{(deg)} \tag{7.36}$$

The location $\bar{z}$ (see Figure 7.14b) of the resultant force, P, can be given as

$$\bar{z} = H - \left[\frac{H^2(\theta_2 - \theta_1) + (R - Q) - 57.3a'H}{2H(\theta_2 - \theta_1)}\right] \tag{7.37}$$

where

$$R = (a' + b')^2(90 - \theta_2) \tag{7.38}$$

$$Q = b'^2(90 - \theta_1) \tag{7.39}$$

Example 7.8

Refer to Figure 7.14b. Here, $a' = 2$ m, $b' = 1$ m, $q = 40$ kN/m², and $H = 6$ m. Determine the total force on the wall (kN/m) caused by the strip loading only.

Solution From Eqs. (7.35) and (7.38),

$$\theta_1 = \tan^{-1}\left(\frac{1}{6}\right) = 9.46°$$

$$\theta_2 = \tan^{-1}\left(\frac{2+1}{6}\right) = 26.57°$$

From Eq. (7.34)

$$P = \frac{q}{90}[H(\theta_2 - \theta_1)] = \frac{40}{90}[6(26.57 - 9.46)] = \mathbf{45.63\ kN/m}$$

Example 7.9

Refer to Example 7.8. Determine the location of the resultant $\bar{z}$.

Solution

From Eqs. (7.38) and (7.39),

$$R = (a' + b')^2(90 - \theta_2) = (2+1)^2(90 - 26.57) = 570.87$$

$$Q = b'^2(90 - \theta_1) = (1)^2(90 - 9.46) = 80.54$$

From Eq. (7.37),

$$\bar{z} = H - \left[\frac{H^2(\theta_2 - \theta_1) + (R - Q) - 57.3a'H}{2H(\theta_2 - \theta_1)}\right]$$

$$= 6 - \left[\frac{(6)^2(26.57 - 9.46) + (570.87 - 80.54) - (57.3)(2)(6)}{(2)(6)(26.57 - 9.46)}\right] = \mathbf{3.96\ m}$$

7.7 Active Earth Pressure for Earthquake Conditions

Coulomb's active earth pressure theory (see Section 7.5) can be extended to take into account the forces caused by an earthquake. Figure 7.15 shows a condition of active pressure with a granular backfill ($c' = 0$). Note that the forces acting on the soil failure wedge in Figure 7.15

COOPER E80 SURCHARGE

SPREADSHEET VALIDATION

STRIP LOAD INTENSITY 'q'

L_Δ = TIE LENGTH + H_1 (FOR LEVEL GROUND, $H_1 = 0$)

= 9' + 0'

= 9'

$q = \frac{80{,}000}{5' L_\Delta} = \frac{80{,}000\ lb}{5'(9')} = 1{,}111.8$ PSF, USE 1,118 PSF

CALCULATE LATERAL PRESSURE DUE TO STRIP LOAD

S = 18' (DIST. FROM RAIL ℄ TO FACE OF SHORING)

$a = L_\Delta = 9'$

$b = S - a/2 = 18' - 9'/2 = 13.5'$

GROUND EL. = 593.71'

TOP WEATHERED ROCK EL. = 581.40'

} H = 593.71' - 581.40 = 12.31' ∴ USE 12'-4"

$\theta_1 = ATAN(b/H)$

$= ATAN(13.5'/12.333')$

$= 41.586° = 0.831$ RAD

$\theta_2 = ATAN[(a+b)/H]$

$= ATAN[(9'+13.5')/12.333']$

$= 61.271° = 1.069$ RAD

$\beta = \theta_2 - \theta_1 = 61.271° - 41.586° = 13.685° = 0.239$ RAD

$\alpha = (\theta_1 + \theta_2)/2 = (61.271° + 41.586°)/2 = 54.428° = 0.950$ RAD

PRESSURE 'σ' @ TOP OF ROCK

$\sigma = 2q(\beta - \sin\beta \cos 2\alpha)/\pi$

$= 2(1{,}118\ psf)[0.239\ RAD - \sin(0.239\ RAD)\cos(2 \times 0.950\ RAD)]/\pi$

$= 357.2$ PSF

COOPER E80 SURCHARGE - CONT'D

RESULTANT SHEAR FORCE

$P = q[H(\theta_2 - \theta_1)]/90$

$= 1.118 \text{ KSF} [12.333'(61.211° - 41.586°)]/90$

$= 3.334$ KIP / FT. OF WALL

MOMENT ARM $\bar{Z}'$ (DIST. FROM BTM EXCAVATION TO RESULTANT)

$R = (a+b)^2(90-\theta_2) = (9' + 13.5')^2 \cdot (90° - 61.211°)$

$= 14,544.19$

$Q = b^2(90-\theta_1) = (90° - 41.586°)(13.500')^2 = 7,729.99$

$$\bar{Z}' = H - \left\{ \frac{H^2(\theta_2 - \theta_1) - (R - Q) + 57.30\,aH}{2H(\theta_2 - \theta_1)} \right\}$$

$H^2(\theta_2 - \theta_1) = (12.333')^2(61.211° - 41.586°) = 2081.631$

$R - Q = 14,544.19 - 7,729.99 = 6,814.203$

$57.30(9')(12.333') = 6,360.3$

$2H(\theta_2 - \theta_1) = 2(12.333')(61.211° - 41.586°) = 337.562$

$$\therefore \bar{Z}' = 12.333' - \left\{ \frac{2,081.631 + 6,814.203 - 6,360.3}{337.562} \right\} = 4.822'$$

RESULTANT MOMENT

$M = P\bar{Z}' = 3.334$ KIP/FT OF WALL $(4.822')$

$= 16.077$ K-FT/FT OF WALL

TEMPORARY SHORING DESIGN - CONTINUED

(Determination of Active & Live Load Lateral Earth Pressures)

Reference Elevations *(re CT Consultants Sheet P.06)*

ground El. = 593.71 ft

top of decomposed bedrock *(sand & gravel)* EL. = 581.40 ft

Calculate Retained Soil Height 'H'

- As previously discussed, to account for the required dead loads from soil, track and road bed, the retained soil height used, is the distance between existing grade and top of decomposed bedrock (sand and gravel).

H = 12.31 ft, ∴ use 12.333 ft

Retained Soil Properties

- The soil borings indicate that the retained soil is granular, with some cohesion and a unit weight of 125 lb/ft^3. Clearly, it is a mixed soil. Averaging represenative values from several sources we can assume an approximate $\phi = 36^o$. Therefore, for calculation of active earth pressure use:

unit weight, γ = 125 lb/ft^3

active earth coeff., K_a = 0.296

Calculate Live Load Surcharge Forces

- Per AASHTO 3.11.6.4-2 use a surcharge equivalent to 2-ft of soil

surcharge soil ht., H_s = 2.000 ft

surcharge pressure., $p_s = K_a \gamma H_s$ = 74 psf *(uniform lateral)*

Calculate Resultant Forces From Live Load Surcharge (@H = D)

shear force, $R_x = P_s H$ = 0.913 kip/ft of wall

moment, $M_s = R_x H/2$ = 5.628 kip-ft/ft of wall

Calculate Active Earth Lateral Forces

active earth pressure., $p_a = K_a \gamma H$ = 456.33 psf *(linearly varing lateral, max.)*

shear force, $R_x = P_a H/2$ = 2.814 kip/ft of wall

moment, $M_a = R_x H/3$ = 11.569 k-ft/ft of wall

Lateral Pressure 'Ps' Summary Between Top of Wall and Top of Decomposed Rock (H = 0 to H = 25 ft)

	Live Load	Active
H (ft)	ps (psf)	Pa (psf)
0.000	74	0.00
0.500	74	18.50
1.000	74	37.00
1.500	74	55.50
2.000	74	74.00
2.500	74	92.50
3.000	74	111.00
3.500	74	129.50
4.000	74	148.00
4.500	74	166.50
5.000	74	185.00
5.500	74	203.50
6.000	74	222.00
6.500	74	240.50
7.000	74	259.00
7.500	74	277.50
8.000	74	296.00
8.500	74	314.50
9.000	74	333.00
9.500	74	351.50
10.000	74	370.00
10.500	74	388.50
11.000	74	407.00
11.500	74	425.50
12.000	74	444.00
12.333	74	456.33

Project	ATB OLD MAIN ST. BRIDGE
Project No.	232245
Sheet No.	14 of 27
Calculated By	J. BROCK
Date	02/19/2025
Checked By	MPS
Date	03/19/2025
Subject	TEMP SHORING

your trusted advisor consultants engineers architects planners

www.ctconsultants.com

LATERAL EARTH PRESSURE

SPREADSHEET VALIDATION

LATERAL PRESSURE FROM SURCHARGE

$H_s = 1'$ OF SOIL

$\gamma = 125\ lb/ft^3$

$K_a = 0.296$

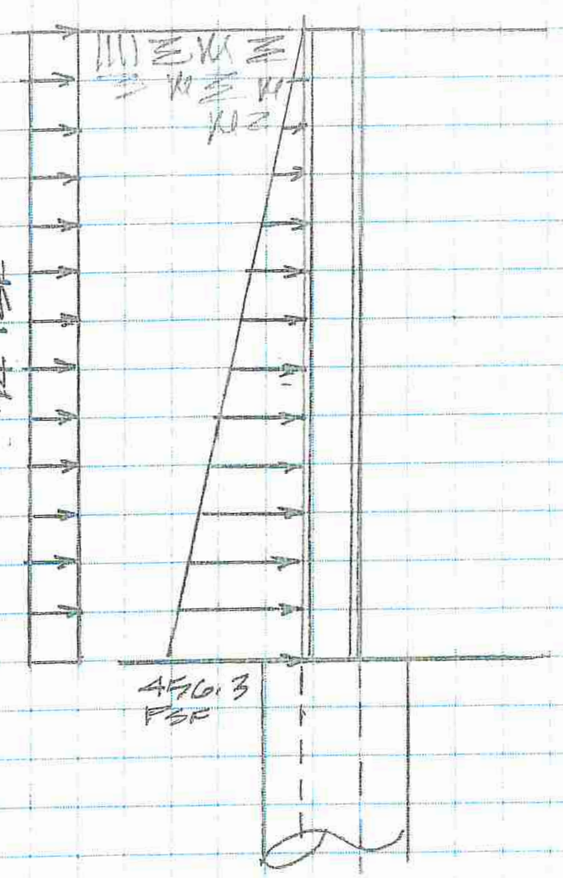
$\therefore P_s = 1'(125\ lb/ft^3)(0.296) = 14\ psf$

ACTIVE EARTH PRESSURE

$P_a = K_a \gamma H$

$= 0.296(125\ lb/ft^3)(12.333')$

$= 456.33\ psf$



EXIST GRADE EL. 593.71

DECOMPOSED BEDROCK (SAND & GRAVEL) EL. 581.4

LATERAL EARTH PRESSURE

RESULTANT FORCES @ TOP OF DECOMPOSED BEDROCK (SAND & GRAVEL)

LIVE LOAD

SHEAR = 74 psf (12.333')/1000 = 0.913^k/FT OF WALL

M.A. = 12.333'/2 = 6.167'

MOMENT = 0.913^k/FT (6.167') = 5.628 K-FT/FT OF WALL

ACTIVE EARTH PRESSURE

SHEAR = 456.33 psf (12.333')/2000 = 2.814 K/FT OF WALL

M.A. = 12.333'/3 = 4.111'

MOMENT = 2.814 K/FT OF WALL (4.111') = 11.569 K-FT/FT OF WALL

3.11.6.4—Live Load Surcharge (*LS*)

A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall. If the surcharge is for a highway, the intensity of the load shall be consistent with the provisions of Article 3.6.1.2. If the surcharge is for other than a highway, the Owner shall specify and/or approve appropriate surcharge loads.

The increase in horizontal pressure due to live load surcharge may be estimated as:

$$\Delta_p = k\gamma_s h_{eq} \quad (3.11.6.4\text{-}1)$$

where:

Δ_p = constant horizontal earth pressure due to live load surcharge (ksf)
γ_s = total unit weight of soil (kcf)
k = coefficient of lateral earth pressure
h_{eq} = equivalent height of soil for vehicular load (ft)

Equivalent heights of soil, h_{eq}, for highway loadings on abutments and retaining walls may be taken from Tables 3.11.6.4-1 and 3.11.6.4-2. Linear interpolation shall be used for intermediate wall heights.

The wall height shall be taken as the distance between the surface of the backfill and the bottom of the footing along the pressure surface being considered.

Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0

Table 3.11.6.4-2—Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic

Retaining Wall Height (ft)	h_{eq} (ft) Distance from wall backface to edge of traffic	
	0.0 ft	1.0 ft or Further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

The load factor for both vertical and horizontal components of live load surcharge shall be taken as specified in Table 3.4.1-1 for live load surcharge.

C3.11.6.4

The tabulated values for h_{eq} were determined by evaluating the horizontal force against an abutment or wall from the pressure distribution produced by the vehicular live load of Article 3.6.1.2. The pressure distributions were developed from elastic half-space solutions using the following assumptions:

- Vehicle loads are distributed through a two-layer system consisting of pavement and soil subgrade.
- Poisson's ratio for the pavement and subgrade materials are 0.2 and 0.4, respectively.
- Wheel loads were modeled as a finite number of point loads distributed across the tire area to produce an equivalent tire contact stress.
- The process for equating wall moments resulting from the elastic solution with the equivalent surcharge method used a wall height increment of 0.25 ft.

The value of the coefficient of lateral earth pressure k is taken as k_o, specified in Article 3.11.5.2, for walls that do not deflect or move, or k_a, specified in Articles 3.11.5.3, 3.11.5.6 and 3.11.5.7, for walls that deflect or move sufficiently to reach minimum active conditions.

The analyses used to develop Tables 3.11.6.4-1 and 3.11.6.4-2 are presented in Kim and Barker (1998).

The values for h_{eq} given in Tables 3.11.6.4-1 and 3.11.6.4-2 are generally greater than the traditional 2.0 ft of earth load historically used in the AASHTO specifications, but less than those prescribed in previous editions (i.e., before 1998) of this specification. The traditional value corresponds to a 20.0-kip single unit truck formerly known as an H10 truck, Peck et al. (1974). This partially explains the increase in h_{eq} in previous editions of this specification. Subsequent analyses, i.e., Kim and Barker (1998), show the importance of the direction of traffic, i.e., parallel for a wall and perpendicular for an abutment on the magnitude of h_{eq}. The magnitude of h_{eq} is greater for an abutment than for a wall due to the proximity and closer spacing of wheel loads to the back of an abutment compared to a wall.

The backface of the wall should be taken as the pressure surface being considered. Refer to Article C11.5.5 for application of surcharge pressures on retaining walls.

TEMPORARY SHORING DESIGN - CONTINUED

(Combination All Lateral Forces)

- Since these are construction conditions, all loads are service level loads. Therefore, the loads can be combined 'unfactored by simple addition.
- There are only two load cases that need to be considered:

***Case 1** = Cooper E80 Surcharge + Active Earth Pressure*

@ top of Decomposed Rock

Component	R_x kip/ft	Mom. Arm (ft)	M kip-ft/ft	
Cooper E80 Surchage	3.334	4.822	16.078	* Loads are per linear ft of wall
Active Earth Pressure	2.814	4.111	11.569	
	6.148		27.647	← controls

***Case 2** = Live Load Surcharge + Active Earth Pressure*

@ top of Decomposed Rock

Component	R_x kip/ft	Mom. Arm (ft)	M kip-ft/ft	
AASHTO Live Load Surcharge	0.913	6.167	5.628	* Loads are per linear ft of wall
Active Earth Pressure	2.814	4.111	11.569	
	3.727		17.197	

Lateral Pressure 'Ps' Summary Between Top of Wall and Top of Decomposed Rock *(z = 0 to z =D)*

	Cooper E80	Const. LL	Active	Case 1	Case 2
z	σ	p_s	p_a	$\Sigma(\sigma + p_a)$	$\Sigma(\sigma + p_s)$
(ft)	(psf)	(psf)	(psf)	(psf)	(psf)
0.000	0.00	74.00	0.00	0.00	74.00
0.500	33.48	74.00	18.50	51.98	92.50
1.000	66.60	74.00	37.00	103.60	111.00
1.500	99.01	74.00	55.50	154.51	129.50
2.000	130.39	74.00	74.00	204.39	148.00
2.500	160.44	74.00	92.50	252.94	166.50
3.000	188.89	74.00	111.00	299.89	185.00
3.500	215.52	74.00	129.50	345.02	203.50
4.000	240.15	74.00	148.00	388.15	222.00
4.500	262.66	74.00	166.50	429.16	240.50
5.000	282.94	74.00	185.00	467.94	259.00
5.500	300.96	74.00	203.50	504.46	277.50
6.000	316.70	74.00	222.00	538.70	296.00
6.500	330.20	74.00	240.50	570.70	314.50
7.000	341.53	74.00	259.00	600.53	333.00
7.500	350.75	74.00	277.50	628.25	351.50
8.000	357.99	74.00	296.00	653.99	370.00
8.500	363.35	74.00	314.50	677.85	388.50
9.000	366.97	74.00	333.00	699.97	407.00
9.500	369.00	74.00	351.50	720.50	425.50
10.000	369.56	74.00	370.00	739.56	444.00
10.500	368.80	74.00	388.50	757.30	462.50
11.000	366.86	74.00	407.00	773.86	481.00
11.500	363.87	74.00	425.50	789.37	499.50
12.000	359.96	74.00	444.00	803.96	518.00
12.333	356.90	74.00	456.33	813.24	530.33

Case 1 - LPile Loads (s = 5-ft c/c)		
ASD		ASD x 1.5
(lb/ft)	(lb/in)	(lb/in)
0.00	0.00	0.00
259.89	21.66	32.49
517.99	43.17	64.75
772.56	64.38	96.57
1,021.96	85.16	127.75
1,264.70	105.39	158.09
1,499.45	124.95	187.43
1,725.11	143.76	215.64
1,940.77	161.73	242.60
2,145.78	178.82	268.22
2,339.69	194.97	292.46
2,522.28	210.19	315.28
2,693.50	224.46	336.69
2,853.52	237.79	356.69
3,002.63	250.22	375.33
3,141.26	261.77	392.66
3,269.93	272.49	408.74
3,389.25	282.44	423.66
3,499.87	291.66	437.48
3,602.48	300.21	450.31
3,697.79	308.15	462.22
3,786.49	315.54	473.31
3,869.29	322.44	483.66
3,946.85	328.90	493.36
4,019.80	334.98	502.48
4,066.18	338.85	508.27

CT Consultants
Temporary Shoring Design
Lagging & Pile

Maximum load on lagging

Pile Spacing = 5.000 ft c/c
H-Pile flange width, b_f = 12.875 in
Net Span, L = Pile Spacing - b_f = 3.927 ft
w = max uniform load at top of shaft = 813.24 lb/ft^2 (psf)

Lagging Design (timber)

-Reference: The National Design Specification for Wood Construction (NSD)

Flexural

b_s = 3.5 in
h_s = 3.5 in (depth) in direction of load
$w' = (b_s/12)w$ = 237.20 lb/ft (plf)
$M_{max} = w' L^2/8$ = 457.25 ft-lb = 5,487.03 in-lb

max. bending stress, F_b =	1100 psi		*(NDS Supplement, Table 4A)*
load duration factor, C_D =	1.000	*duration > 2 month, but < 1 year*	*(NDS Table 2.3.2)*
wet surface factor, C_M =	1.00	*F_bC_F <= 1,150 psi*	*(NDS Table 4B)*
temperature factor, C_t =	1.00	*typ. temperature < 150 ^{o}*	*(NDS 4.3.4)*
beam stability factor, C_L =	1.00	*since b = d*	*(NDS 3.3.3)*
size factor, C_F =	1.00	*incorporated into tabulated design values*	*(NDS Table 4B)*
flat use factor, C_{fu} =	1.00	*since b = d*	*(NDS Table 4B)*
incising factor, C_i =	0.80	*normal preservative incising*	*(NDS 4.3.8)*
repetitive member factor, C_r =	1.00	*mbrs <u>not</u> joined by a distributing element*	*(NDS Table 4B)*
$F'_b = F_bC_DC_MC_tC_LC_FC_{fu}C_iC_r$ =	880 psi		

req'd section modulus, $S_{req'd} = M_{max}/F'_b$ = 6.24 in^3
section modulus, $S = bh^2/6$ = 7.15 in^3 OK

Shear

Check shear in the lagging at distance d from the face of support:

$V = w'L/2$= 465.74 lb
max. bending stress, F_v = 140 psi NDS
wet surface factor, C_M = 0.97 *F_bC_F <= 1,150 psi* *(NDS Table 4B)*
$F'_v = F_vC_DC_MC_tC_i$ = 109 psi
$f_v = 3V/2A$ = 57.03 psi OK

Table 4.3.1 Applicability of Adjustment Factors for Sawn Lumber

		ASD only	ASD and LRFD										LRFD only		
		Load Duration Factor	Wet Service Factor	Temperature Factor	Beam Stability Factor	Size Factor	Flat Use Factor	Incising Factor	Repetitive Member Factor	Column Stability Factor	Buckling Stiffness Factor	Bearing Area Factor	Format Conversion Factor K_F	Resistance Factor ϕ	Time Effect Factor
$F_b' = F_b$	x	C_D	C_M	C_t	C_L	C_F	C_{fu}	C_i	C_r	-	-	-	2.54	0.85	λ
$F_t' = F_t$	x	C_D	C_M	C_t	-	C_F	-	C_i	-	-	-	-	2.70	0.80	λ
$F_v' = F_v$	x	C_D	C_M	C_t	-	-	-	C_i	-	-	-	-	2.88	0.75	λ
$F_c' = F_c$	x	C_D	C_M	C_t	-	C_F	-	C_i	-	C_P	-	-	2.40	0.90	λ
$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	-	-	-	C_i	-	-	-	C_b	1.67	0.90	-
$E' = E$	x	-	C_M	C_t	-	-	-	C_i	-	-	-	-	-	-	-
$E_{min}' = E_{min}$	x	-	C_M	C_t	-	-	-	C_i	-	-	C_T	-	1.76	0.85	-

4.3.5 Beam Stability Factor, C_L

Reference bending design values, F_b, shall be multiplied by the beam stability factor, C_L, specified in 3.3.3.

4.3.6 Size Factor, C_F

4.3.6.1 Reference bending, tension, and compression parallel to grain design values for visually graded dimension lumber 2" to 4" thick shall be multiplied by the size factors specified in Tables 4A and 4B.

4.3.6.2 Where the depth of a rectangular sawn lumber bending member 5" or thicker exceeds 12", the reference bending design values, F_b, in Table 4D shall be multiplied by the following size factor:

$$C_F = (12 / d)^{1/9} \leq 1.0 \tag{4.3-1}$$

4.3.6.3 For beams of circular cross section with a diameter greater than 13.5", or for 12" or larger square beams loaded in the plane of the diagonal, the size factor shall be determined in accordance with 4.3.6.2 on the basis of an equivalent conventionally loaded square beam of the same cross-sectional area.

4.3.6.4 Reference bending design values for all species of 2" thick or 3" thick Decking, except Redwood, shall be multiplied by the size factors specified in Table 4E.

4.3.7 Flat Use Factor, C_{fu}

4.3.7.1 When sawn lumber 2" to 4" thick is loaded on the wide face, multiplying the reference bending design value, F_b, by the flat use factors, C_{fu}, specified in Tables 4A, 4B, 4C, and 4F, shall be permitted.

4.3.7.2 When members classified as Beams and Stringers are loaded on the wide face, the reference bending design value, F_b, and the reference modulus of elasticity, (E or E_{min}), shall be multiplied by the flat use factors, C_{fu}, specified in Table 4D.

Table 4B Reference Design Values for Visually Graded Southern Pine Dimension Lumber (2" - 4" thick)[1,2,3,4,5]

(Tabulated design values are for normal load duration and dry service conditions, unless specified otherwise. See NDS 4.3 for a comprehensive description of design value adjustment factors.)

USE WITH TABLE 4B ADJUSTMENT FACTORS

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)							Specific Gravity[6]	Grading Rules Agency
		Bending	Tension parallel to grain	Shear parallel to grain	Compression perpendicular to grain	Compression parallel to grain	Modulus of Elasticity			
		F_b	F_t	F_v	$F_{c\perp}$	F_c	E	E_{min}	G	
SOUTHERN PINE										
Dense Select Structural	2" - 4" wide	2,700	1,900	175	660	2,050	1,900,000	690,000	0.55	SPIB
Select Structural		2,350	1,650	175	565	1,900	1,800,000	660,000		
Non-Dense Select Structural		2,050	1,450	175	480	1,800	1,600,000	580,000		
No.1 Dense		1,650	1,100	175	660	1,750	1,800,000	660,000		
No.1		1,500	1,000	175	565	1,650	1,600,000	580,000		
No.1 Non-Dense		1,300	875	175	480	1,550	1,400,000	510,000		
No.2 Dense		1,200	750	175	660	1,500	1,600,000	580,000		
No.2		1,100	675	175	565	1,450	1,400,000	510,000		
No.2 Non-Dense		1,050	600	175	480	1,450	1,300,000	470,000		
No.3 and Stud		650	400	175	565	850	1,300,000	470,000		
Construction	4" wide	875	500	175	565	1,600	1,400,000	510,000	0.55	
Standard		475	275	175	565	1,300	1,200,000	440,000		
Utility		225	125	175	565	850	1,200,000	440,000		
Dense Select Structural	5" - 6" wide	2,400	1,650	175	660	1,900	1,900,000	690,000	0.55	
Select Structural		2,100	1,450	175	565	1,800	1,800,000	660,000		
Non-Dense Select Structural		1,850	1,300	175	480	1,700	1,600,000	580,000		
No.1 Dense		1,500	1,000	175	660	1,650	1,800,000	660,000		
No.1		1,350	875	175	565	1,550	1,600,000	580,000		
No.1 Non-Dense		1,200	775	175	480	1,450	1,400,000	510,000		
No.2 Dense		1,050	650	175	660	1,450	1,600,000	580,000		
No.2		1,000	600	175	565	1,400	1,400,000	510,000		
No.2 Non-Dense		950	525	175	480	1,350	1,300,000	470,000		
No.3 and Stud		575	350	175	565	800	1,300,000	470,000		
Dense Select Structural	8" wide	2,200	1,550	175	660	1,850	1,900,000	690,000	0.55	
Select Structural		1,950	1,350	175	565	1,700	1,800,000	660,000		
Non-Dense Select Structural		1,700	1,200	175	480	1,650	1,600,000	580,000		
No.1 Dense		1,350	900	175	660	1,600	1,800,000	660,000		
No.1		1,250	800	175	565	1,500	1,600,000	580,000		
No.1 Non-Dense		1,100	700	175	480	1,400	1,400,000	510,000		
No.2 Dense		975	600	175	660	1,400	1,600,000	580,000		
No.2		925	550	175	565	1,350	1,400,000	510,000		
No.2 Non-Dense		875	500	175	480	1,300	1,300,000	470,000		
No.3 and Stud		525	325	175	565	775	1,300,000	470,000		
Dense Select Structural	10" wide	1,950	1,300	175	660	1,800	1,900,000	690,000	0.55	
Select Structural		1,700	1,150	175	565	1,650	1,800,000	660,000		
Non-Dense Select Structural		1,500	1,050	175	480	1,600	1,600,000	580,000		
No.1 Dense		1,200	800	175	660	1,550	1,800,000	660,000		
No.1		1,050	700	175	565	1,450	1,600,000	580,000		
No.1 Non-Dense		950	625	175	480	1,400	1,400,000	510,000		
No.2 Dense		850	525	175	660	1,350	1,600,000	580,000		
No.2		800	475	175	565	1,300	1,400,000	510,000		
No.2 Non-Dense		750	425	175	480	1,250	1,300,000	470,000		
No.3 and Stud		475	275	175	565	750	1,300,000	470,000		
Dense Select Structural	12" wide	1,800	1,250	175	660	1,750	1,900,000	690,000	0.55	
Select Structural		1,600	1,100	175	565	1,650	1,800,000	660,000		
Non-Dense Select Structural		1,400	975	175	480	1,550	1,600,000	580,000		
No.1 Dense		1,100	750	175	660	1,500	1,800,000	660,000		
No.1		1,000	650	175	565	1,400	1,600,000	580,000		
No.1 Non-Dense		900	575	175	480	1,350	1,400,000	510,000		
No.2 Dense		800	500	175	660	1,300	1,600,000	580,000		
No.2		750	450	175	565	1,250	1,400,000	510,000		
No.2 Non-Dense		700	400	175	480	1,250	1,300,000	470,000		
No.3 and Stud		450	250	175	565	725	1,300,000	470,000		

Table 4C Adjustment Factors

Flat Use Factor, C_{fu}

Bending design values are based on edgewise use (load applied to narrow face). When dimension lumber is used flatwise (load applied to wide face), the bending design value, F_b, shall be multiplied by the following flat use factors:

Flat Use Factors, C_{fu}

Width (depth)	Thickness (breadth) 2"
2" & 3"	1.0
4"	1.1
5"	1.1
6"	1.15
8"	1.15
10" & wider	1.2

Repetitive Member Factor, C_r

Bending design values, F_b, for dimension lumber 2" to 4" thick shall be multiplied by the repetitive member factor, $C_r = 1.15$, when such members are used as joists, truss chords, rafters, studs, planks, decking, or similar members which are in contact or spaced not more than 24" on center, are not less than 3 in number and are joined by floor, roof, or other load distributing elements adequate to support the design load.

Wet Service Factor, C_M

When dimension lumber is used where moisture content will exceed 19% for an extended time period, design values shall be multiplied by the appropriate wet service factors from the following table:

Wet Service Factors, C_M

F_b	F_t	F_v	$F_{c\perp}$	F_c	E and E_{min}
0.85*	1.0	0.97	0.67	0.8**	0.9

* when $F_b \leq 1{,}150$ psi, $C_M = 1.0$

** when $F_c \leq 750$ psi, $C_M = 1.0$

Table 2.3.2 Frequently Used Load Duration Factors, C_D[1]

Load Duration	C_D	Typical Design Loads
Permanent	0.9	Dead Load
Ten years	1.0	Occupancy Live Load
Two months	1.15	Snow Load
Seven days	1.25	Construction Load
Ten minutes	1.6	Wind/Earthquake Load
Impact[2]	2.0	Impact Load

1. Load duration factors shall not apply to reference modulus of elasticity, E, reference modulus of elasticity for beam and column stability, E_{min}, nor to reference compression perpendicular to grain design values, $F_{c\perp}$, based on a deformation limit.
2. Load duration factors greater than 1.6 shall not be used in the design of structural members pressure-treated with water-borne preservatives (see Reference 30), or fire retardant chemicals. Load duration factors greater than 1.6 shall not be used in the design of connections or wood structural panels.

2.3.3 Temperature Factor, C_t

Reference design values shall be multiplied by the temperature factors, C_t, in Table 2.3.3 for structural members that will experience sustained exposure to elevated temperatures up to 150°F (see Appendix C).

2.3.4 Fire Retardant Treatment

The effects of fire retardant chemical treatment on strength shall be accounted for in the design. Adjusted design values, including adjusted connection design values, for lumber and structural glued laminated timber pressure-treated with fire retardant chemicals shall be obtained from the company providing the treatment and redrying service. Load duration factors greater than 1.6 shall not apply to structural members pressure-treated with fire retardant chemicals (see Table 2.3.2).

2.3.5 Format Conversion Factor, K_F (LRFD Only)

For LRFD, reference design values shall be multiplied by the format conversion factor, K_F, specified in Table 2.3.5. The format conversion factor, K_F, shall not apply for designs in accordance with ASD methods specified herein.

2.3.6 Resistance Factor, ϕ (LRFD Only)

For LRFD, reference design values shall be multiplied by the resistance factor, ϕ, specified in Table 2.3.6. The resistance factor, ϕ, shall not apply for designs in accordance with ASD methods specified herein.

2.3.7 Time Effect Factor, λ (LRFD Only)

For LRFD, reference design values shall be multiplied by the time effect factor, λ, specified in Appendix N.3.3. The time effect factor, λ, shall not apply for designs in accordance with ASD methods specified herein.

2

DESIGN VALUES FOR STRUCTURAL MEMBERS

Table 2.3.3 Temperature Factor, C_t

Reference Design Values	In-Service Moisture Conditions[1]	C_t		
		T≤100°F	100°F<T≤125°F	125°F<T≤150°F
F_t, E, E_{min}	Wet or Dry	1.0	0.9	0.9
F_b, F_v, F_c, and $F_{c\perp}$	Dry	1.0	0.8	0.7
	Wet	1.0	0.7	0.5

1. Wet and dry service conditions for sawn lumber, structural glued laminated timber, prefabricated wood I-joists, structural composite lumber, wood structural panels and cross-laminated timber are specified in 4.1.4, 5.1.4, 7.1.4, 8.1.4, 9.3.3, and 10.1.5 respectively.

3.2 Bending Members – General

3.2.1 Span of Bending Members

For simple, continuous and cantilevered bending members, the span shall be taken as the distance from face to face of supports, plus ½ the required bearing length at each end.

3.2.2 Lateral Distribution of Concentrated Load

Lateral distribution of concentrated loads from a critically loaded bending member to adjacent parallel bending members by flooring or other cross members shall be permitted to be calculated when determining design bending moment and vertical shear force (see 15.1).

3.2.3 Notches

3.2.3.1 Bending members shall not be notched except as permitted by 4.4.3, 5.4.5, 7.4.4, and 8.4.1. A gradual taper cut from the reduced depth of the member to the full depth of the member in lieu of a square-cornered notch reduces stress concentrations.

3.2.3.2 The stiffness of a bending member, as determined from its cross section, is practically unaffected by a notch with the following dimensions:

notch depth $\leq$ (1/6) (beam depth)

notch length $\leq$ (1/3) (beam depth)

3.2.3.3 See 3.4.3 for effect of notches on shear strength.

3.3 Bending Members – Flexure

3.3.1 Strength in Bending

The actual bending stress or moment shall not exceed the adjusted bending design value.

3.3.2 Flexural Design Equations

3.3.2.1 The actual bending stress induced by a bending moment, M, is calculated as follows:

$$f_b = \frac{Mc}{I} = \frac{M}{S} \quad (3.3\text{-}1)$$

For a rectangular bending member of breadth, b, and depth, d, this becomes:

$$f_b = \frac{M}{S} = \frac{6M}{bd^2} \quad (3.3\text{-}2)$$

3.3.2.2 For solid rectangular bending members with the neutral axis perpendicular to depth at center:

$$I = \frac{bd^3}{12} = \text{moment of inertia, in.}^4 \quad (3.3\text{-}3)$$

$$S = \frac{I}{c} = \frac{bd^2}{6} = \text{section modulus, in.}^3 \quad (3.3\text{-}4)$$

3.3.3 Beam Stability Factor, C_L

3.3.3.1 When the depth of a bending member does not exceed its breadth, $d \leq b$, no lateral support is required and $C_L = 1.0$.

3.3.3.2 When rectangular sawn lumber bending members are laterally supported in accordance with 4.4.1, $C_L = 1.0$.

3.3.3.3 When the compression edge of a bending member is supported throughout its length to prevent lateral displacement, and the ends at points of bearing have lateral support to prevent rotation, $C_L = 1.0$.

3.3.3.4 Where the depth of a bending member exceeds its breadth, $d > b$, lateral support shall be provided at points of bearing to prevent rotation. When such lateral support is provided at points of bearing, but no additional lateral support is provided throughout the length of the bending member, the unsupported length, ℓ_u, is the distance between such points of end bearing, or the length of a cantilever. When a bending member is provided with lateral support to prevent rotation at intermediate points as well as at the ends, the unsupported length, ℓ_u, is the distance between such points of intermediate lateral support.

3.3.3.5 The effective span length, ℓ_e, for single span or cantilever bending members shall be determined in accordance with Table 3.3.3.

4.3.8 Incising Factor, C_i

Reference design values for dimension lumber shall be multiplied by the incising factor, C_i, in Table 4.3.8 when dimension lumber is incised parallel to grain a maximum depth of 0.4", a maximum length of 3/8", and a density of incisions up to 1100/ft^2. As an alternative, incising factors for specific incising patterns and lumber sizes shall be obtained from the company providing the incising.

Table 4.3.8 Incising Factors, C_i

Design Value	C_i
E, E_{min}	0.95
F_b, F_t, F_c, F_v	0.80
$F_{c\perp}$	1.00

4.3.9 Repetitive Member Factor, C_r

Reference bending design values, F_b, in Tables 4A, 4B, 4C, and 4F for dimension lumber 2" to 4" thick shall be multiplied by the repetitive member factor, C_r = 1.15, where such members are used as joists, truss chords, rafters, studs, planks, decking, or similar members which are in contact or spaced not more than 24" on center, are not less than three in number and are joined by floor, roof or other load distributing elements adequate to support the design load. (A load distributing element is any adequate system that is designed or has been proven by experience to transmit the design load to adjacent members, spaced as described above, without displaying structural weakness or unacceptable deflection. Subflooring, flooring, sheathing, or other covering elements and nail gluing or tongue-and-groove joints, and through nailing generally meet these criteria.) Reference bending design values in Table 4E for visually graded Decking have already been multiplied by C_r = 1.15.

4.3.10 Column Stability Factor, C_P

Reference compression design values parallel to grain, F_c, shall be multiplied by the column stability factor, C_P, specified in 3.7.

4.3.11 Buckling Stiffness Factor, C_T

Reference modulus of elasticity for beam and column stability, E_{min}, shall be permitted to be multiplied by the buckling stiffness factor, C_T, as specified in 4.4.2.

4.3.12 Bearing Area Factor, C_b

Reference compression design values perpendicular to grain, $F_{c\perp}$, shall be permitted to be multiplied by the bearing area factor, C_b, as specified in 3.10.4.

4.3.13 Pressure-Preservative Treatment

Reference design values apply to sawn lumber pressure-treated by an approved process and preservative (see Reference 30). Load duration factors greater than 1.6 shall not apply to structural members pressure-treated with water-borne preservatives.

4.3.14 Format Conversion Factor, K_F (LRFD Only)

For LRFD, reference design values shall be multiplied by the format conversion factor, K_F, specified in Table 4.3.1.

4.3.15 Resistance Factor, ϕ (LRFD Only)

For LRFD, reference design values shall be multiplied by the resistance factor, ϕ, specified in Table 4.3.1.

4.3.16 Time Effect Factor, λ (LRFD Only)

For LRFD, reference design values shall be multiplied by the time effect factor, λ, specified in Appendix N.3.3.

CT Consultants
Temporary Shoring Design
Lagging & Pile

W-pile Check

W24x131 beams

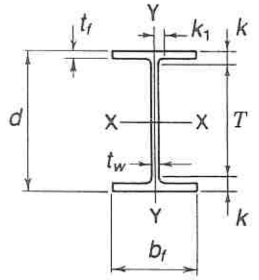
beam weight =		131	**lb/ft**
		10.92	**lb/in**
I_b =	4020	in^4	
S_b =	329	in^3	
d =	24.48 in		
t_w =	0.605 in		
Mmax =		271.12	ft-kip	From Lpile Output
		3,253	in-kip
		3,253,444	in-lb

Section allowable Stresses for 50ksi Steel

E =	29,000	ksi, Modulus of elasticity of steel
fy =	50,000	psi, steel yield stress (Assume worst case)
Fbmax = 0.75fy =	37,500	psi, allowable bending stress (weak axis)
Fv = 0.4fy =	20,000	psi, Allowable shear stress
Fbmin = 0.6fy =	30,000	psi, allowable bending stress (strong axis)

$S_{required}$ = Mmax/Fbmin = 108.45 in^3 OK < Sb

Vmax = 230.91 kips
fv = Vmax/dt_w = 15,591.21 psi OK <Fv

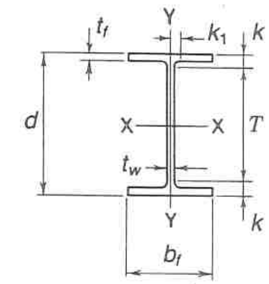


W SHAPES
Dimensions

Designation	Area A	Depth d		Web Thickness t_w		Web $\frac{t_w}{2}$	Flange Width b_f		Flange Thickness t_f		Distance T	Distance k	Distance k_1
	In.^{2}	In.		In.		In.	In.		In.		In.	In.	In.
W 24×492[a]	144.0	29.65	29 5/8	1.970	2	1	14.115	14 1/8	3.540	3 9/16	21	4 5/16	1 9/16
×450[a]	132.0	29.09	29 1/8	1.810	1 13/16	15/16	13.955	14	3.270	3 1/4	21	4 1/16	1 1/2
×408[a]	119.0	28.54	28 1/2	1.650	1 5/8	13/16	13.800	13 3/4	2.990	3	21	3 3/4	1 3/8
×370[a]	108.0	27.99	28	1.520	1 1/2	3/4	13.660	13 5/8	2.720	2 3/4	21	3 1/2	1 5/16
×335[a]	98.4	27.52	27 1/2	1.380	1 3/8	11/16	13.520	13 1/2	2.480	2 1/2	21	3 1/4	1 1/4
×306[a]	89.8	27.13	27 1/8	1.260	1 1/4	5/8	13.405	13 3/8	2.280	2 1/4	21	3 1/16	1 3/16
×279[a]	82.0	26.73	26 3/4	1.160	1 3/16	5/8	13.305	13 1/4	2.090	2 1/16	21	2 7/8	1 1/8
×250[a]	73.5	26.34	26 3/8	1.040	1 1/16	9/16	13.185	13 1/8	1.890	1 7/8	21	2 11/16	1 1/8
×229	67.2	26.02	26	0.960	1	1/2	13.110	13 1/8	1.730	1 3/4	21	2 1/2	1
×207	60.7	25.71	25 3/4	0.870	7/8	7/16	13.010	13	1.570	1 9/16	21	2 3/8	1
×192	56.3	25.47	25 1/2	0.810	13/16	7/16	12.950	13	1.460	1 7/16	21	2 1/4	1
×176	51.7	25.24	25 1/4	0.750	3/4	3/8	12.890	12 7/8	1.340	1 5/16	21	2 1/8	15/16
×162	47.7	25.00	25	0.705	11/16	3/8	12.955	13	1.220	1 1/4	21	2	1 1/16
×146	43.0	24.74	24 3/4	0.650	5/8	5/16	12.900	12 7/8	1.090	1 1/16	21	1 7/8	1 1/16
×131	38.5	24.48	24 1/2	0.605	5/8	5/16	12.855	12 7/8	0.960	15/16	21	1 3/4	1 1/16
×117	34.4	24.26	24 1/4	0.550	9/16	5/16	12.800	12 3/4	0.850	7/8	21	1 5/8	1
×104	30.6	24.06	24	0.500	1/2	1/4	12.750	12 3/4	0.750	3/4	21	1 1/2	1
W 24×103[b]	30.3	24.53	24 1/2	0.550	9/16	5/16	9.000	9	0.980	1	21	1 3/4	13/16
× 94	27.7	24.31	24 1/4	0.515	1/2	1/4	9.065	9 1/8	0.875	7/8	21	1 5/8	1
× 84	24.7	24.10	24 1/8	0.470	1/2	1/4	9.020	9	0.770	3/4	21	1 9/16	15/16
× 76	22.4	23.92	23 7/8	0.440	7/16	1/4	8.990	9	0.680	11/16	21	1 7/16	15/16
× 68	20.1	23.73	23 3/4	0.415	7/16	1/4	8.965	9	0.585	9/16	21	1 3/8	15/16
W 24× 62	18.2	23.74	23 3/4	0.430	7/16	1/4	7.040	7	0.590	9/16	21	1 3/8	15/16
× 55	16.2	23.57	23 5/8	0.395	3/8	3/16	7.005	7	0.505	1/2	21	1 5/16	15/16

[a]For application refer to Notes in Table 2.
[b]Heavier shapes in this series are available from some producers.

W SHAPES
Properties

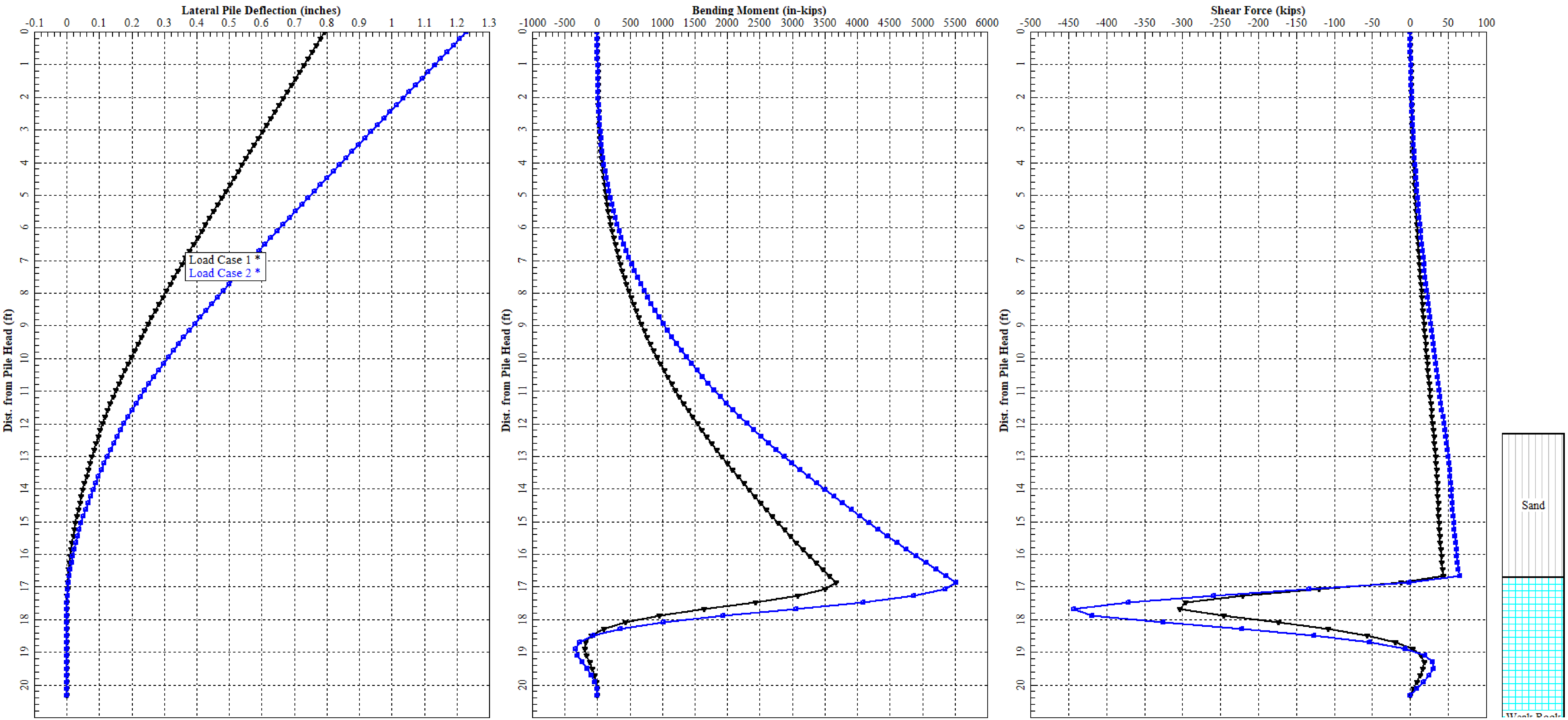


Nominal Wt. per Ft	Compact Section Criteria $\frac{b_f}{2t_f}$	F_y'	$\frac{d}{t_w}$	F_y'''	r_T	$\frac{d}{A_f}$	Elastic Properties Axis X-X I	S	r	Axis Y-Y I	S	r	Plastic Modulus Z_x	Z_y
Lb.		Ksi		Ksi	In.		In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.^{3}	In.^{3}
492	2.0	—	15.1	—	3.80	0.59	19100	1290	11.5	1670	237	3.41	1550	375
450	2.1	—	16.1	—	3.76	0.64	17100	1170	11.4	1490	214	3.36	1410	337
408	2.3	—	17.3	—	3.71	0.69	15100	1060	11.3	1320	191	3.33	1250	300
370	2.5	—	18.4	—	3.67	0.75	13400	957	11.1	1160	170	3.28	1120	267
335	2.7	—	19.9	—	3.63	0.82	11900	864	11.0	1030	152	3.23	1020	238
306	2.9	—	21.5	—	3.60	0.89	10700	789	10.9	919	137	3.20	922	214
279	3.2	—	23.0	—	3.57	0.96	9600	718	10.8	823	124	3.17	835	193
250	3.5	—	25.3	—	3.53	1.06	8490	644	10.7	724	110	3.14	744	171
229	3.8	—	27.1	—	3.51	1.15	7650	588	10.7	651	99.4	3.11	676	154
207	4.1	—	29.6	—	3.48	1.26	6820	531	10.6	578	88.8	3.08	606	137
192	4.4	—	31.4	—	3.46	1.35	6260	491	10.5	530	81.8	3.07	559	126
176	4.8	—	33.7	58.2	3.44	1.46	5680	450	10.5	479	74.3	3.04	511	115
162	5.3	—	35.5	52.5	3.45	1.58	5170	414	10.4	443	68.4	3.05	468	105
146	5.9	—	38.1	45.6	3.43	1.76	4580	371	10.3	391	60.5	3.01	418	93.2
131	6.7	—	40.5	40.3	3.40	1.98	4020	329	10.2	340	53.0	2.97	370	81.5
117	7.5	—	44.1	33.9	3.37	2.23	3540	291	10.1	297	46.5	2.94	327	71.4
104	8.5	58.5	48.1	28.5	3.35	2.52	3100	258	10.1	259	40.7	2.91	289	62.4
103	4.6	—	44.6	33.2	2.33	2.78	3000	245	9.96	119	26.5	1.99	280	41.5
94	5.2	—	47.2	29.6	2.33	3.06	2700	222	9.87	109	24.0	1.98	254	37.5
84	5.9	—	51.3	25.1	2.31	3.47	2370	196	9.79	94.4	20.9	1.95	224	32.6
76	6.6	—	54.4	22.3	2.29	3.91	2100	176	9.69	82.5	18.4	1.92	200	28.6
68	7.7	—	57.2	20.2	2.26	4.52	1830	154	9.55	70.4	15.7	1.87	177	24.5
62	6.0	—	55.2	21.7	1.71	5.72	1550	131	9.23	34.5	9.80	1.38	153	15.7
55	6.9	—	59.7	18.5	1.68	6.66	1350	114	9.11	29.1	8.30	1.34	134	13.3

27/27

SOLDIER PILE EMBEDMENT CALCULATION

(LPile Analysis Output)



Lateral Pile Deflection (inches)
Bending Moment (in-kips)
Shear Force (kips)
Dist. from Pile Head (ft)
Load Case 1 *
Load Case 2 *
Sand

==

LPile for Version 2022-12.012

License ID : 5279320353
License Type : (Office Cloud License)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This software is licensed for exclusive use by:
Verdantas Inc.

==

This model was prepared by:
ihajjar

Files Used for Analysis

Path to file locations:
\2023\232245\PHASE\02 Geotechnical Engineering Services\Project Data\Calculations\L-Pile Analysis\Rear (west) Abutment - Shoring\

Name of input data file:
30-inch Dia - Lateral Loading Stop at 12.33 feet.lp12d

Name of output report file:
30-inch Dia - Lateral Loading Stop at 12.33 feet.lp12o

Name of plot output file:
30-inch Dia - Lateral Loading Stop at 12.33 feet.lp12p

Name of runtime message file:
30-inch Dia - Lateral Loading Stop at 12.33 feet.lp12r

Date and Time of Analysis

Date: January 8, 2026 Time: 14:29:03

Problem Title

Project Name: ATB Old Main Street Bridtge

Job Number: 232245

Client: City of Conneaut

Engineer: MSI

Description: REAR (WEST) ABUTMENT - SHORING PILES

Program Options and Settings

Computational Options:
 - Conventional Analysis
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)

Analysis Control Options:
 - Maximum number of iterations allowed = 500
 - Deflection tolerance for convergence = 1.0000E-05 in
 - Maximum allowable deflection = 100.0000 in
 - Number of pile increments = 100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- Analysis includes loading by multiple distributed lateral loads acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:
- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined	=	2
Total length of pile	=	20.310 ft
Depth of ground surface below top of pile	=	12.3100 ft

Pile diameters used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	12.9000
2	12.310	12.9000
3	12.310	30.0000
4	20.310	30.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a AISC strong axis steel pile

```
   Length of section                                    =    12.310000 ft
   AISC Section Type                                    = W


   AISC Section Name                                    = W24X131


   Pile width                                           =    12.900000 in

Pile Section No. 2:

   Section 2 is a drilled shaft with casing and AISC section core/insert
   Length of section                                    =     8.000000 ft
   Section Diameter                                     =    30.000000 in
   Core/Insert AISC Section Type                        = W


   Core/Insert AISC Section Name                        = W24X131
```

Ground Slope and Pile Batter Angles

```
Ground Slope Angle                                      =        0.000 degrees
                                                        =        0.000 radians

Pile Batter Angle                                       =        0.000 degrees
                                                        =        0.000 radians
```

Soil and Rock Layering Information

The soil profile is modelled using 2 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

```
   Distance from top of pile to top of layer            =    12.310000 ft
   Distance from top of pile to bottom of layer         =    16.710000 ft
   Effective unit weight at top of layer                =    77.600000 pcf
```

Effective unit weight at bottom of layer = 77.600000 pcf
Friction angle at top of layer = 43.000000 deg.
Friction angle at bottom of layer = 43.000000 deg.
Subgrade k at top of layer = 190.000000 pci
Subgrade k at bottom of layer = 190.000000 pci

Layer 2 is weak rock, p-y criteria by Reese, 1997

Distance from top of pile to top of layer = 16.710000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 87.600000 pcf
Effective unit weight at bottom of layer = 87.600000 pcf
Uniaxial compressive strength at top of layer = 2730. psi
Uniaxial compressive strength at bottom of layer = 2860. psi
Initial modulus of rock at top of layer = 1400000. psi
Initial modulus of rock at bottom of layer = 1400000. psi
RQD of rock at top of layer = 10.000000 %
RQD of rock at bottom of layer = 10.000000 %
k rm of rock at top of layer = 0.0000500
k rm of rock at bottom of layer = 0.0000500

(Depth of the lowest soil layer extends 29.690 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Angle of Friction deg.	Uniaxial qu psi	RQD %	E50 or krm	kpy pci	Rock Mass Modulus psi
1	Sand	12.3100	77.6000	43.0000	--	--	--	190.0000	--
	(Reese, et al.)	16.7100	77.6000	43.0000	--	--	--	190.0000	--
2	Weak	16.7100	87.6000	--	2730.	10.0000	5.00E-05	--	1400000.
	Rock	50.0000	87.6000	--	2860.	10.0000	5.00E-05	--	1400000.

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Distributed Lateral Loading for Individual Load Cases

Distributed lateral load intensity for Load Case 1 defined using 26 points

Point No.	Depth X ft	Dist. Load lb/in
1	0.000	0.000
2	0.500	21.660
3	1.000	43.170
4	1.500	64.380
5	2.000	85.160
6	2.500	105.390
7	3.000	124.950
8	3.500	143.760
9	4.000	161.730
10	4.500	178.820
11	5.000	194.940
12	5.500	210.190
13	6.000	224.460
14	6.500	237.790
15	7.000	250.220
16	7.500	261.770
17	8.000	272.490
18	8.500	282.440
19	9.000	291.660
20	9.500	300.210
21	10.000	308.150
22	10.500	315.540
23	11.000	322.440
24	11.500	328.900
25	12.000	334.980
26	12.330	338.850

Distributed lateral load intensity for Load Case 2 defined using 26 points

Point No.	Depth X ft	Dist. Load lb/in
1	0.000	0.000

2	0.500	32.490
3	1.000	64.750
4	1.500	96.570
5	2.000	127.750
6	2.500	158.090
7	3.000	187.430
8	3.500	215.640
9	4.000	242.600
10	4.500	268.220
11	5.000	292.460
12	5.500	315.280
13	6.000	336.690
14	6.500	356.690
15	7.000	375.330
16	7.500	392.660
17	8.000	408.740
18	8.500	423.660
19	9.000	437.480
20	9.500	450.310
21	10.000	462.220
22	10.500	473.310
23	11.000	483.660
24	11.500	493.360
25	12.000	502.480
26	12.330	508.270

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	1	V = 0.0000 lbs	M = 0.0000 in-lbs	0.0000000	Yes	Yes
2	1	V = 0.0000 lbs	M = 0.0000 in-lbs	0.0000000	Yes	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis

```
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.



--------------------------------------------------------------------------------
     Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
--------------------------------------------------------------------------------

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:
-------------------

Dimensions and Properties of Steel AISC Strong Axis:
---------------------------------------------

Length of Section                                       =    12.310000 ft
Flange Width                                            =    12.900000 in
Section Depth                                           =    24.500000 in
Flange Thickness                                        =     0.960000 in
Web Thickness                                           =     0.605000 in
Yield Stress of Pipe                                    =    50.000000 ksi
Elastic Modulus                                         =       29000. ksi
Cross-sectional Area                                    =    38.600000 sq. in.
Moment of Inertia                                       =        4020. in^4
Elastic Bending Stiffness                               =   116580000. kip-in^2
Plastic Modulus, Z                                      =   370.000000in^3
Plastic Moment Capacity = Fy Z                          =       18500.in-kip

Axial Structural Capacities:
---------------------------

Nom. Axial Structural Capacity = Fy As                  =     1930.000 kips
Nominal Axial Tensile Capacity                          =    -1930.000 kips



Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

   Number     Axial Thrust Force
                   kips
   ------     ------------------
      1               0.000
```

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 0.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000568	660.9014790	116349762.	12.2500000	1.9977471	
0.00001136	1322.	116349762.	12.2500000	3.9954942	
0.00001704	1983.	116349762.	12.2500000	5.9932413	
0.00002272	2644.	116349762.	12.2500000	7.9909884	
0.00002840	3305.	116349762.	12.2500000	9.9887355	
0.00003408	3965.	116349762.	12.2500000	11.9864827	
0.00003976	4626.	116349762.	12.2500000	13.9842298	
0.00004544	5287.	116349762.	12.2500000	15.9819769	
0.00005112	5948.	116349762.	12.2500000	17.9797240	
0.00005680	6609.	116349762.	12.2500000	19.9774711	
0.00006248	7270.	116349762.	12.2500000	21.9752182	
0.00006816	7931.	116349762.	12.2500000	23.9729653	
0.00007384	8592.	116349762.	12.2500000	25.9707124	
0.00007952	9253.	116349762.	12.2500000	27.9684595	
0.00008520	9914.	116349762.	12.2500000	29.9662066	
0.00009088	10574.	116349762.	12.2500000	31.9639538	
0.00009657	11235.	116349762.	12.2500000	33.9617009	
0.0001022	11896.	116349762.	12.2500000	35.9594480	
0.0001079	12557.	116349762.	12.2500000	37.9571951	
0.0001136	13218.	116349762.	12.2500000	39.9549422	
0.0001193	13879.	116349762.	12.2500000	41.9526893	
0.0001250	14540.	116349762.	12.2500000	43.9504364	
0.0001306	15201.	116349762.	12.2500000	45.9481835	
0.0001363	15862.	116349762.	12.2500000	47.9459306	
0.0001420	16523.	116349762.	12.2500000	49.9436777	
0.0001477	16968.	114888363.	12.2500000	50.0000000	Y
0.0001534	17154.	111851123.	12.2500000	50.0000000	Y
0.0001590	17244.	108419070.	12.2500000	50.0000000	Y
0.0001647	17324.	105168186.	12.2500000	50.0000000	Y
0.0001704	17397.	102087640.	12.2500000	50.0000000	Y
0.0001761	17462.	99165667.	12.2500000	50.0000000	Y
0.0001818	17522.	96394720.	12.2500000	50.0000000	Y
0.0001874	17576.	93763791.	12.2500000	50.0000000	Y
0.0001931	17625.	91261877.	12.2500000	50.0000000	Y
0.0001988	17671.	88881762.	12.2500000	50.0000000	Y
0.0002045	17712.	86616109.	12.2500000	50.0000000	Y
0.0002102	17751.	84457536.	12.2500000	50.0000000	Y
0.0002159	17786.	82397606.	12.2500000	50.0000000	Y

0.0002215	17818.	80431012.	12.2500000	50.0000000	Y
0.0002329	17876.	76757122.	12.2500000	50.0000000	Y
0.0002443	17926.	73392499.	12.2500000	50.0000000	Y
0.0002556	17970.	70302757.	12.2500000	50.0000000	Y
0.0002670	18008.	67453674.	12.2500000	50.0000000	Y
0.0002783	18042.	64820970.	12.2500000	50.0000000	Y
0.0002897	18072.	62381700.	12.2500000	50.0000000	Y
0.0003011	18098.	60115855.	12.2500000	50.0000000	Y
0.0003124	18122.	58005893.	12.2500000	50.0000000	Y
0.0003238	18143.	56035116.	12.2500000	50.0000000	Y
0.0003351	18162.	54192109.	12.2500000	50.0000000	Y
0.0003465	18179.	52465506.	12.2500000	50.0000000	Y
0.0003579	18195.	50843593.	12.2500000	50.0000000	Y
0.0003692	18209.	49317062.	12.2500000	50.0000000	Y
0.0003806	18222.	47880019.	12.2500000	50.0000000	Y
0.0003919	18234.	46521248.	12.2500000	50.0000000	Y
0.0004033	18245.	45238568.	12.2500000	50.0000000	Y
0.0004147	18254.	44022289.	12.2500000	50.0000000	Y
0.0004260	18264.	42870877.	12.2500000	50.0000000	Y
0.0004374	18272.	41775785.	12.2500000	50.0000000	Y
0.0004487	18280.	40735990.	12.2500000	50.0000000	Y
0.0004601	18287.	39745903.	12.2500000	50.0000000	Y
0.0004715	18294.	38802144.	12.2500000	50.0000000	Y
0.0004828	18300.	37902797.	12.2500000	50.0000000	Y
0.0004942	18306.	37042855.	12.2500000	50.0000000	Y
0.0005055	18311.	36221025.	12.2500000	50.0000000	Y
0.0005169	18317.	35435319.	12.2500000	50.0000000	Y
0.0005283	18321.	34682162.	12.2500000	50.0000000	Y
0.0005396	18326.	33959956.	12.2500000	50.0000000	Y
0.0005510	18330.	33267531.	12.2500000	50.0000000	Y
0.0005623	18334.	32603083.	12.2500000	50.0000000	Y
0.0005737	18338.	31963442.	12.2500000	50.0000000	Y
0.0005851	18341.	31348555.	12.2500000	50.0000000	Y
0.0005964	18344.	30757091.	12.2500000	50.0000000	Y
0.0006078	18348.	30187738.	12.2500000	50.0000000	Y
0.0006192	18351.	29638484.	12.2500000	50.0000000	Y
0.0006305	18353.	29108574.	12.2500000	50.0000000	Y
0.0006419	18356.	28597422.	12.2500000	50.0000000	Y
0.0006532	18359.	28104049.	12.2500000	50.0000000	Y
0.0006646	18361.	27627543.	12.2500000	50.0000000	Y
0.0006760	18363.	27166572.	12.2500000	50.0000000	Y
0.0007214	18371.	25466100.	12.2500000	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load	Axial	Nominal Moment

No.	Thrust kips	Capacity in-kips
1	0.00000000	18371.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Pile Section No. 2:

Dimensions and Properties of Drilled Shaft (Bored Pile) with Casing and AISC Strong Axis Core/Insert:

Length of Section	=	8.000000 ft
Outside Diameter of Casing	=	30.000000 in
Casing Wall Thickness	=	0.0000 in
Moment of Inertia of Steel Casing	=	0.0000 in^4
Width Flange of Core/Insert	=	12.900000 in
Depth of Core/Insert	=	24.500000 in
Flange Thickness of Core/Insert	=	0.960000 in
Web Thickness of Core/Insert	=	0.605000 in
Moment of Inertia of Steel Core/Insert	=	4020. in^4
Yield Stress of Casing	=	50000. psi
Elastic Modulus of Casing	=	29000000. psi
Yield Stress of Core/Insert	=	50000. psi
Elastic Modulus of Core/Insert	=	29000000. psi
Number of Reinforcing Bars	=	0 bars
Gross Area of Pile	=	706.858347 sq. in.
Area of Concrete	=	668.429447 sq. in.
Cross-sectional Area of Steel Casing	=	0.0000 sq. in.
Cross-sectional Area of Steel Core/Insert	=	38.600000 sq. in.
Area of All Steel (Casing, Core/Insert, and Bars)	=	38.428900 sq. in.
Area Ratio of All Steel to Gross Area	=	5.44 percent

Note that the core is assumed to be void of concrete.

Axial Structural Capacities:

```
Nom. Axial Structural Capacity = 0.85 Fc Ac + Fy As    =     4202.078 kips
Tensile Load for Cracking of Concrete                  =     -407.081 kips
Nominal Axial Tensile Capacity                         =    -1930.000 kips
```

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	0.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 0.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Comp Strain in/in	Max Tens Strain in/in	Max Conc Stress ksi	Max Steel Stress ksi	Max Casing Stress ksi	Max Core Stress ksi	Run Msg
0.00000568	1509.	265671716.	15.0000000	0.00008520	-0.00008520	0.3497541	0.00000	0.00000	2.0014535	
0.00001136	1809.	159223537.	11.2233269	0.0001275	-0.000213	0.5157465	0.00000	0.00000	-5.247160	C
0.00001704	2708.	158905804.	11.2439970	0.0001916	-0.000320	0.7616205	0.00000	0.00000	-7.860525	C
0.00002272	3603.	158584589.	11.2646717	0.0002559	-0.000426	0.9993619	0.00000	0.00000	-10.467077	C
0.00002840	4495.	158259829.	11.2855083	0.0003205	-0.000532	1.2288861	0.00000	0.00000	-13.066684	C
0.00003408	5383.	157931424.	11.3066199	0.0003853	-0.000637	1.4501127	0.00000	0.00000	-15.659155	C
0.00003976	6266.	157599282.	11.3280141	0.0004504	-0.000742	1.6629516	0.00000	0.00000	-18.244344	C
0.00004544	7146.	157263308.	11.3496987	0.0005158	-0.000848	1.8673100	0.00000	0.00000	-20.822102	C
0.00005112	8022.	156923402.	11.3716820	0.0005814	-0.000952	2.0630924	0.00000	0.00000	-23.392274	C
0.00005680	8894.	156579461.	11.3939725	0.0006472	-0.001057	2.2502001	0.00000	0.00000	-25.954696	C

0.00006248	9762.	156231376.	11.4165790	0.0007133	-0.001161
2.4285314	0.00000	0.00000	-28.509203 C		
0.00006816	10625.	155879035.	11.4395110	0.0007798	-0.001265
2.5979811	0.00000	0.00000	-31.055618 C		
0.00007384	11484.	155522321.	11.4627780	0.0008465	-0.001369
2.7584407	0.00000	0.00000	-33.593760 C		
0.00007952	12339.	155161110.	11.4863903	0.0009134	-0.001472
2.9097979	0.00000	0.00000	-36.123441 C		
0.00008520	13189.	154795274.	11.5103586	0.0009807	-0.001575
3.0519365	0.00000	0.00000	-38.644463 C		
0.00009088	14035.	154424679.	11.5346377	0.0010483	-0.001678
3.1847364	0.00000	0.00000	-41.156621 C		
0.00009657	14876.	154049328.	11.5590359	0.0011162	-0.001781
3.3080420	0.00000	0.00000	-43.660208 C		
0.0001022	15712.	153669243.	11.5836657	0.0011844	-0.001883
3.4217064	0.00000	0.00000	-46.155465 C		
0.0001079	16543.	153284069.	11.6086854	0.0012529	-0.001985
3.5256493	0.00000	0.00000	-48.641396 C		
0.0001136	17301.	152285245.	11.6121663	0.0013192	-0.002089
3.6164460	0.00000	0.00000	-50.000000 CY		
0.0001193	17678.	148200310.	11.4988206	0.0013717	-0.002207
3.6812396	0.00000	0.00000	-50.000000 CY		
0.0001250	17934.	143511082.	11.3584827	0.0014194	-0.002330
3.7349166	0.00000	0.00000	-50.000000 CY		
0.0001306	18168.	139058926.	11.2237146	0.0014663	-0.002453
3.7828208	0.00000	0.00000	-50.000000 CY		
0.0001363	18380.	134819788.	11.0921432	0.0015122	-0.002578
3.8249846	0.00000	0.00000	-50.000000 CY		
0.0001420	18575.	130801899.	10.9666551	0.0015573	-0.002703
3.8621058	0.00000	0.00000	-50.000000 CY		
0.0001477	18754.	126981427.	10.8440559	0.0016015	-0.002829
3.8941195	0.00000	0.00000	-50.000000 CY		
0.0001534	18919.	123357398.	10.7266132	0.0016451	-0.002956
3.9215540	0.00000	0.00000	-50.000000 CY		
0.0001590	19073.	119917353.	10.6135707	0.0016881	-0.003083
3.9445656	0.00000	0.00000	-50.000000 CY		
0.0001647	19214.	116639586.	10.5031797	0.0017302	-0.003212
3.9632495	0.00000	0.00000	-50.000000 CY		
0.0001704	19347.	113530408.	10.3979449	0.0017719	-0.003340
3.9779879	0.00000	0.00000	-50.000000 CY		
0.0001761	19470.	110566627.	10.2955448	0.0018129	-0.003470
3.9888142	0.00000	0.00000	-50.000000 CY		
0.0001818	19585.	107745255.	10.1963645	0.0018534	-0.003600
3.9959329	0.00000	0.00000	-50.000000 CY		
0.0001874	19693.	105057473.	10.1013000	0.0018935	-0.003730
3.9995089	0.00000	0.00000	-50.000000 CY		
0.0001931	19794.	102491661.	10.0089741	0.0019330	-0.003861
3.9981883	0.00000	0.00000	-50.000000 CY		
0.0001988	19889.	100040034.	9.9189767	0.0019720	-0.003992
3.9999843	0.00000	0.00000	-50.000000 CY		

```
     0.0002045         19979.      97699960.     9.8327588     0.0020107     -0.004124
    3.9990590       0.00000        0.00000    -50.000000 CY
     0.0002102         20064.      95464917.     9.7500239     0.0020492     -0.004256
    3.9983465       0.00000        0.00000    -50.000000 CY
     0.0002159         20143.      93319255.     9.6679311     0.0020868     -0.004389
    3.9992354       0.00000        0.00000    -50.000000 CY
     0.0002215         20218.      91266487.     9.5889868     0.0021243     -0.004522
    3.9983114       0.00000        0.00000    -50.000000 CY
     0.0002329         20359.      87416902.     9.4397521     0.0021984     -0.004788
    3.9999944       0.00000        0.00000    -50.000000 CY
     0.0002443         20484.      83865201.     9.2969165     0.0022708     -0.005057
    3.9996157       0.00000        0.00000    -50.000000 CY
     0.0002556         20599.      80586989.     9.1636719     0.0023424     -0.005326
    3.9979828       0.00000        0.00000    -50.000000 CY
     0.0002670         20703.      77547549.     9.0354550     0.0024122     -0.005597
    3.9983189       0.00000        0.00000    -50.000000 CY
     0.0002783         20799.      74727495.     8.9149733     0.0024813     -0.005869
    3.9989079       0.00000        0.00000    -50.000000 CY
     0.0002897         20887.      72101615.     8.7996138     0.0025492     -0.006142
    3.9964740       0.00000        0.00000    -50.000000 CY
     0.0003011         20958.      69614304.     8.7036151     0.0026203     -0.006411
    3.9987241       0.00000        0.00000    50.0000000 CY
     0.0003124         21024.      67293740.     8.6127454     0.0026908     -0.006682
    3.9975449       0.00000        0.00000    50.0000000 CY
     0.0003238         21079.      65102702.     8.5354310     0.0027636     -0.006950
    3.9976768       0.00000        0.00000    50.0000000 CY
     0.0003351         21123.      63027967.     8.4684619     0.0028381     -0.007216
    3.9999877       0.00000        0.00000    50.0000000 CY
     0.0003465         21165.      61081260.     8.4046354     0.0029122     -0.007483
    3.9951696       0.00000        0.00000    50.0000000 CY
     0.0003579         21192.      59219231.     8.3616151     0.0029923     -0.007743
    3.9994924       0.00000        0.00000    50.0000000 CY
     0.0003692         21215.      57459914.     8.3257527     0.0030740     -0.008003
    3.9922504       0.00000        0.00000    50.0000000 CY
     0.0003806         21236.      55799219.     8.2922793     0.0031559     -0.008262
    3.9978895       0.00000        0.00000    50.0000000 CY
     0.0003919         21250.      54216759.     8.2703406     0.0032415     -0.008517
    3.9990541       0.00000        0.00000    50.0000000 CY
     0.0004033         21262.      52720432.     8.2493817     0.0033270     -0.008772
    3.9942298       0.00000        0.00000    50.0000000 CY


-----------------------------------------------------------------------------------
   Summary of Results for Nominal Moment Capacity for Section 2
-----------------------------------------------------------------------------------

                                          Nominal
Load                 Axial                Moment
 No.                Thrust               Capacity
                     kips                 in-kips
```

1	0.00000000	21262.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	12.3100	0.00	N.A.	No	0.00	43799.
2	16.7100	4.4000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head = 0.0 lbs
Applied moment at pile head = 0.0 in-lbs
Axial thrust load on pile head = 0.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.2518	-2.28E-05	0.00	-0.00157	3.66E-08	1.16E+11	0.00	0.00	2.1996
0.2031	0.2480	6.5327	13.4020	-0.00157	0.01048	1.16E+11	0.00	0.00	8.7983
0.4062	0.2442	65.3267	45.5667	-0.00157	0.1048	1.16E+11	0.00	0.00	17.5965
0.6093	0.2404	228.6432	99.1347	-0.00157	0.3669	1.16E+11	0.00	0.00	26.3621
0.8124	0.2365	548.5490	174.0317	-0.00157	0.8801	1.16E+11	0.00	0.00	35.0994
1.0155	0.2327	1077.	270.1988	-0.00157	1.7279	1.16E+11	0.00	0.00	43.8166
1.2186	0.2289	1866.	387.5007	-0.00157	2.9933	1.16E+11	0.00	0.00	52.4430
1.4217	0.2250	2966.	525.8123	-0.00157	4.7585	1.16E+11	0.00	0.00	61.0574
1.6248	0.2212	4429.	684.9908	-0.00157	7.1056	1.16E+11	0.00	0.00	69.5667
1.8279	0.2174	6305.	864.8247	-0.00157	10.1157	1.16E+11	0.00	0.00	78.0075
2.0310	0.2135	8644.	1065.	-0.00157	13.8693	1.16E+11	0.00	0.00	86.4008
2.2341	0.2097	11497.	1286.	-0.00157	18.4463	1.16E+11	0.00	0.00	94.6317
2.4372	0.2059	14912.	1526.	-0.00157	23.9252	1.16E+11	0.00	0.00	102.8442
2.6403	0.2021	18937.	1787.	-0.00157	30.3843	1.16E+11	0.00	0.00	110.8785
2.8434	0.1982	23621.	2067.	-0.00157	37.9000	1.16E+11	0.00	0.00	118.8238
3.0465	0.1944	29011.	2366.	-0.00157	46.5483	1.16E+11	0.00	0.00	126.6881
3.2496	0.1906	35154.	2684.	-0.00157	56.4039	1.16E+11	0.00	0.00	134.3400
3.4527	0.1868	42095.	3021.	-0.00157	67.5399	1.16E+11	0.00	0.00	141.9684
3.6558	0.1829	49878.	3376.	-0.00157	80.0289	1.16E+11	0.00	0.00	149.3595
3.8589	0.1791	58549.	3749.	-0.00157	93.9413	1.16E+11	0.00	0.00	156.6589
4.0620	0.1753	68151.	4139.	-0.00156	109.3468	1.16E+11	0.00	0.00	163.8424

4.2651	0.1715	78726.	4547.	-0.00156	126.3138	1.16E+11	0.00	0.00	170.7911
4.4682	0.1677	90315.	4972.	-0.00156	144.9086	1.16E+11	0.00	0.00	177.7098
4.6713	0.1639	102960.	5413.	-0.00156	165.1970	1.16E+11	0.00	0.00	184.3427
4.8744	0.1601	116700.	5870.	-0.00156	187.2422	1.16E+11	0.00	0.00	190.8907
5.0775	0.1563	131574.	6343.	-0.00155	211.1068	1.16E+11	0.00	0.00	197.3013
5.2806	0.1525	147619.	6832.	-0.00155	236.8517	1.16E+11	0.00	0.00	203.4983
5.4837	0.1488	164874.	7335.	-0.00155	264.5361	1.16E+11	0.00	0.00	209.6578
5.6868	0.1450	183373.	7853.	-0.00154	294.2186	1.16E+11	0.00	0.00	215.5213
5.8899	0.1412	203153.	8386.	-0.00154	325.9552	1.16E+11	0.00	0.00	221.3177
6.0930	0.1375	224248.	8932.	-0.00154	359.8010	1.16E+11	0.00	0.00	226.9390
6.2961	0.1337	246691.	9492.	-0.00153	395.8097	1.16E+11	0.00	0.00	232.3540
6.4992	0.1300	270513.	10064.	-0.00153	434.0328	1.16E+11	0.00	0.00	237.7237
6.7023	0.1263	295748.	10650.	-0.00152	474.5216	1.16E+11	0.00	0.00	242.8192
6.9054	0.1226	322426.	11248.	-0.00151	517.3245	1.16E+11	0.00	0.00	247.8680
7.1085	0.1189	350575.	11858.	-0.00151	562.4897	1.16E+11	0.00	0.00	252.7263
7.3116	0.1153	380226.	12480.	-0.00150	610.0636	1.16E+11	0.00	0.00	257.4180
7.5147	0.1116	411405.	13113.	-0.00149	660.0908	1.16E+11	0.00	0.00	262.0543
7.7178	0.1080	444142.	13757.	-0.00148	712.6155	1.16E+11	0.00	0.00	266.4396
7.9209	0.1044	478461.	14411.	-0.00147	767.6795	1.16E+11	0.00	0.00	270.7922
8.1240	0.1008	514388.	15076.	-0.00146	825.3242	1.16E+11	0.00	0.00	274.9576
8.3271	0.09729	551949.	15751.	-0.00145	885.5895	1.16E+11	0.00	0.00	278.9993
8.5302	0.09377	591167.	16436.	-0.00144	948.5138	1.16E+11	0.00	0.00	282.9786
8.7333	0.09028	632066.	17130.	-0.00142	1014.	1.16E+11	0.00	0.00	286.7421
8.9364	0.08682	674668.	17834.	-0.00141	1082.	1.16E+11	0.00	0.00	290.4825
9.1395	0.08340	718995.	18546.	-0.00140	1154.	1.16E+11	0.00	0.00	294.0454

9.3426	0.08002	765069.	19267.	-0.00138	1228.	1.16E+11	0.00	0.00	297.5185
9.5457	0.07667	812910.	19996.	-0.00136	1304.	1.16E+11	0.00	0.00	300.9263
9.7488	0.07337	862539.	20734.	-0.00135	1384.	1.16E+11	0.00	0.00	304.1609
9.9519	0.07010	913975.	21479.	-0.00133	1466.	1.16E+11	0.00	0.00	307.3784
10.1550	0.06689	967236.	22232.	-0.00131	1552.	1.16E+11	0.00	0.00	310.4409
10.3581	0.06373	1022341.	22992.	-0.00129	1640.	1.16E+11	0.00	0.00	313.4427
10.5612	0.06061	1079309.	23760.	-0.00127	1732.	1.16E+11	0.00	0.00	316.3806
10.7643	0.05756	1138155.	24534.	-0.00124	1826.	1.16E+11	0.00	0.00	319.1873
10.9674	0.05456	1198897.	25315.	-0.00122	1924.	1.16E+11	0.00	0.00	321.9798
11.1705	0.05162	1261552.	26103.	-0.00119	2024.	1.16E+11	0.00	0.00	324.6429
11.3736	0.04874	1326136.	26898.	-0.00117	2128.	1.16E+11	0.00	0.00	327.2669
11.5767	0.04594	1392663.	27699.	-0.00114	2234.	1.16E+11	0.00	0.00	329.8315
11.7798	0.04320	1461149.	28505.	-0.00111	2344.	1.16E+11	0.00	0.00	332.3024
11.9829	0.04054	1531610.	29318.	-0.00108	2457.	1.16E+11	0.00	0.00	334.7645
12.1860	0.03796	1604058.	30137.	-0.00104	2574.	1.16E+11	0.00	0.00	337.1613
12.3891	0.03546	1678510.	30626.	-0.00102	0.00	1.89E+11	-6.395	439.5441	70.7711
12.5922	0.03301	1753344.	30679.	-9.92E-04	0.00	1.70E+11	-21.241	1568.	0.00
12.7953	0.03063	1828051.	30612.	-9.65E-04	0.00	1.59E+11	-33.888	2697.	0.00
12.9984	0.02831	1902558.	30516.	-9.37E-04	0.00	1.59E+11	-44.431	3825.	0.00
13.2015	0.02606	1976800.	30398.	-9.07E-04	0.00	1.59E+11	-52.972	4954.	0.00
13.4046	0.02389	2050728.	30260.	-8.76E-04	0.00	1.59E+11	-59.616	6082.	0.00
13.6077	0.02179	2124302.	30109.	-8.44E-04	0.00	1.59E+11	-64.473	7211.	0.00
13.8108	0.01977	2197493.	29948.	-8.11E-04	0.00	1.59E+11	-67.660	8340.	0.00
14.0139	0.01784	2270282.	29781.	-7.77E-04	0.00	1.59E+11	-69.297	9468.	0.00
14.2170	0.01599	2342659.	29612.	-7.41E-04	0.00	1.59E+11	-69.511	10597.	0.00

14.4201	0.01422	2414623.	29444.	-7.05E-04	0.00	1.59E+11	-68.431	11725.	0.00
14.6232	0.01255	2486181.	29280.	-6.67E-04	0.00	1.59E+11	-66.194	12854.	0.00
14.8263	0.01097	2557346.	29123.	-6.29E-04	0.00	1.59E+11	-62.941	13983.	0.00
15.0294	0.00949	2628137.	28974.	-5.89E-04	0.00	1.59E+11	-58.817	15111.	0.00
15.2325	0.00810	2698578.	28837.	-5.48E-04	0.00	1.59E+11	-53.973	16240.	0.00
15.4356	0.00681	2768699.	28712.	-5.06E-04	0.00	1.59E+11	-48.564	17368.	0.00
15.6387	0.00563	2838531.	28601.	-4.63E-04	0.00	1.59E+11	-42.750	18497.	0.00
15.8418	0.00456	2908110.	28504.	-4.19E-04	0.00	1.59E+11	-36.696	19626.	0.00
16.0449	0.00359	2977470.	28422.	-3.74E-04	0.00	1.59E+11	-30.572	20754.	0.00
16.2480	0.00273	3046649.	28355.	-3.28E-04	0.00	1.59E+11	-24.552	21883.	0.00
16.4511	0.00199	3115682.	28302.	-2.80E-04	0.00	1.59E+11	-18.817	23011.	0.00
16.6542	0.00137	3184604.	28262.	-2.32E-04	0.00	1.59E+11	-13.549	24140.	0.00
16.8573	8.62E-04	3253444.	-15671.	-1.83E-04	0.00	1.59E+11	-36039.	1.02E+08	0.00
17.0604	4.78E-04	3108216.	-101483.	-1.34E-04	0.00	1.59E+11	-34380.	1.75E+08	0.00
17.2635	2.10E-04	2758773.	-180761.	-8.87E-05	0.00	1.59E+11	-30677.	3.55E+08	0.00
17.4666	4.58E-05	2227113.	-229116.	-5.05E-05	0.00	1.59E+11	-9004.	4.79E+08	0.00
17.6697	-3.56E-05	1641968.	-230912.	-2.35E-05	0.00	2.01E+11	7531.	5.16E+08	0.00
17.8728	-6.85E-05	1101555.	-202804.	-8.45E-06	0.00	2.66E+11	15535.	5.53E+08	0.00
18.0759	-7.68E-05	653422.	-161232.	-4.02E-07	0.00	2.66E+11	18579.	5.90E+08	0.00
18.2790	-7.04E-05	315645.	-116516.	4.04E-06	0.00	2.66E+11	18116.	6.27E+08	0.00
18.4821	-5.71E-05	85477.	-75502.	5.88E-06	0.00	2.66E+11	15540.	6.64E+08	0.00
18.6852	-4.18E-05	-52382.	-41931.	6.03E-06	0.00	2.66E+11	12009.	7.01E+08	0.00
18.8883	-2.76E-05	-118910.	-17099.	5.25E-06	0.00	2.66E+11	8368.	7.38E+08	0.00
19.0914	-1.62E-05	-135731.	-633.373	4.08E-06	0.00	2.66E+11	5144.	7.75E+08	0.00
19.2945	-7.76E-06	-121997.	8782.	2.90E-06	0.00	2.66E+11	2583.	8.12E+08	0.00

```
   19.4976  -2.05E-06    -92923.     12801.   1.91E-06       0.00   2.66E+11
715.1490   8.48E+08       0.00
   19.7007   1.57E-06    -59601.     12977.   1.21E-06       0.00   2.66E+11
-570.434   8.85E+08       0.00
   19.9038   3.86E-06    -29667.     10501.   8.04E-07       0.00   2.66E+11
-1462.   9.22E+08       0.00
   20.1069   5.49E-06     -8415.      6086.   6.30E-07       0.00   2.66E+11
-2161.   9.59E+08       0.00
   20.3100   6.93E-06       0.00       0.00   5.91E-07       0.00   2.66E+11
-2833.   4.98E+08       0.00
```

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

```
Pile-head deflection              =     0.25184473 inches
Computed slope at pile head       =     -0.0015714 radians
Maximum bending moment            =       3253444. inch-lbs
Maximum shear force               =       -230912. lbs
Depth of maximum bending moment   =    16.85730000 feet below pile head
Depth of maximum shear force      =    17.66970000 feet below pile head
Number of iterations              =             18
Number of zero deflection points  =              2
Pile deflection at ground         =     0.03643411 inches
```

Pile-head Deflection vs. Pile Length for Load Case 1

Boundary Condition Type 1, Shear and Moment

```
Shear      =             0. lbs
Moment     =             0. in-lbs
Axial Load =             0. lbs
```

Pile Length feet	Pile Head Deflection inches	Maximum Moment ln-lbs	Maximum Shear lbs
20.31000	0.25184473	3253444.	-230912.
19.29450	0.24873805	3231694.	-230824.

18.27900 0.25790505 3234880. -295253.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 2

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head = 0.0 lbs
Applied moment at pile head = 0.0 in-lbs
Axial thrust load on pile head = 0.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	0.3890	6.42E-05	0.00	-0.00241	1.03E-07	1.16E+11	0.00	0.00	3.2994
0.2031	0.3831	9.7990	20.1030	-0.00241	0.01572	1.16E+11	0.00	0.00	13.1974
0.4062	0.3772	97.9901	68.3501	-0.00241	0.1572	1.16E+11	0.00	0.00	26.3948
0.6093	0.3713	342.9648	148.7007	-0.00241	0.5503	1.16E+11	0.00	0.00	39.5420
0.8124	0.3655	822.8170	261.0411	-0.00241	1.3202	1.16E+11	0.00	0.00	52.6460
1.0155	0.3596	1615.	405.2825	-0.00241	2.5918	1.16E+11	0.00	0.00	65.7204
1.2186	0.3537	2798.	581.2265	-0.00241	4.4899	1.16E+11	0.00	0.00	78.6617
1.4217	0.3478	4449.	788.6895	-0.00241	7.1375	1.16E+11	0.00	0.00	91.5853
1.6248	0.3419	6643.	1027.	-0.00241	10.6581	1.16E+11	0.00	0.00	104.3525
1.8279	0.3361	9457.	1297.	-0.00241	15.1732	1.16E+11	0.00	0.00	117.0178
2.0310	0.3302	12966.	1598.	-0.00241	20.8035	1.16E+11	0.00	0.00	129.6105
2.2341	0.3243	17245.	1929.	-0.00241	27.6690	1.16E+11	0.00	0.00	141.9552
2.4372	0.3184	22367.	2290.	-0.00241	35.8875	1.16E+11	0.00	0.00	154.2719
2.6403	0.3125	28406.	2680.	-0.00241	45.5763	1.16E+11	0.00	0.00	166.3228

2.8434	0.3067	35432.	3100.	-0.00241	56.8502	1.16E+11	0.00	0.00	178.2407
3.0465	0.3008	43517.	3549.	-0.00241	69.8228	1.16E+11	0.00	0.00	190.0367
3.2496	0.2949	52732.	4026.	-0.00241	84.6066	1.16E+11	0.00	0.00	201.5124
3.4527	0.2891	63143.	4531.	-0.00241	101.3109	1.16E+11	0.00	0.00	212.9532
3.6558	0.2832	74819.	5064.	-0.00241	120.0447	1.16E+11	0.00	0.00	224.0407
3.8589	0.2773	87825.	5623.	-0.00240	140.9138	1.16E+11	0.00	0.00	234.9919
4.0620	0.2715	102228.	6209.	-0.00240	164.0224	1.16E+11	0.00	0.00	245.7666
4.2651	0.2656	118090.	6821.	-0.00240	189.4734	1.16E+11	0.00	0.00	256.1837
4.4682	0.2598	135475.	7458.	-0.00240	217.3659	1.16E+11	0.00	0.00	266.5575
4.6713	0.2539	154442.	8119.	-0.00239	247.7988	1.16E+11	0.00	0.00	276.5246
4.8744	0.2481	175052.	8805.	-0.00239	280.8672	1.16E+11	0.00	0.00	286.3709
5.0775	0.2423	197363.	9515.	-0.00239	316.6648	1.16E+11	0.00	0.00	295.9931
5.2806	0.2365	221432.	10248.	-0.00238	355.2833	1.16E+11	0.00	0.00	305.2666
5.4837	0.2307	247315.	11003.	-0.00238	396.8112	1.16E+11	0.00	0.00	314.4856
5.6868	0.2249	275065.	11780.	-0.00237	441.3364	1.16E+11	0.00	0.00	323.2788
5.8899	0.2191	304736.	12579.	-0.00237	488.9425	1.16E+11	0.00	0.00	331.9755
6.0930	0.2134	336379.	13398.	-0.00236	539.7125	1.16E+11	0.00	0.00	340.4095
6.2961	0.2076	370044.	14238.	-0.00235	593.7268	1.16E+11	0.00	0.00	348.5340
6.4992	0.2019	405779.	15097.	-0.00234	651.0628	1.16E+11	0.00	0.00	356.5900
6.7023	0.1962	443632.	15975.	-0.00233	711.7973	1.16E+11	0.00	0.00	364.2317
6.9054	0.1905	483648.	16872.	-0.00233	776.0031	1.16E+11	0.00	0.00	371.8030
7.1085	0.1849	525874.	17787.	-0.00231	843.7524	1.16E+11	0.00	0.00	379.0906
7.3116	0.1792	570350.	18720.	-0.00230	915.1146	1.16E+11	0.00	0.00	386.1301
7.5147	0.1736	617121.	19669.	-0.00229	990.1568	1.16E+11	0.00	0.00	393.0863
7.7178	0.1681	666226.	20635.	-0.00228	1069.	1.16E+11	0.00	0.00	399.6644

7.9209	0.1625	717706.	21617.	-0.00226	1152.	1.16E+11	0.00	0.00	406.1933
8.1240	0.1570	771598.	22615.	-0.00225	1238.	1.16E+11	0.00	0.00	412.4402
8.3271	0.1516	827940.	23628.	-0.00223	1328.	1.16E+11	0.00	0.00	418.5007
8.5302	0.1462	886768.	24655.	-0.00221	1423.	1.16E+11	0.00	0.00	424.4672
8.7333	0.1408	948117.	25696.	-0.00219	1521.	1.16E+11	0.00	0.00	430.1084
8.9364	0.1355	1012021.	26751.	-0.00217	1624.	1.16E+11	0.00	0.00	435.7151
9.1395	0.1302	1078513.	27820.	-0.00215	1730.	1.16E+11	0.00	0.00	441.0596
9.3426	0.1250	1147625.	28901.	-0.00213	1841.	1.16E+11	0.00	0.00	446.2711
9.5457	0.1198	1219388.	29995.	-0.00210	1956.	1.16E+11	0.00	0.00	451.3844
9.7488	0.1147	1293832.	31101.	-0.00208	2076.	1.16E+11	0.00	0.00	456.2364
9.9519	0.1097	1370986.	32219.	-0.00205	2200.	1.16E+11	0.00	0.00	461.0627
10.1550	0.1048	1450879.	33348.	-0.00202	2328.	1.16E+11	0.00	0.00	465.6579
10.3581	0.09987	1533538.	34488.	-0.00199	2461.	1.16E+11	0.00	0.00	470.1627
10.5612	0.09507	1618989.	35640.	-0.00195	2598.	1.16E+11	0.00	0.00	474.5709
10.7643	0.09034	1707260.	36801.	-0.00192	2739.	1.16E+11	0.00	0.00	478.7810
10.9674	0.08571	1798374.	37973.	-0.00188	2885.	1.16E+11	0.00	0.00	482.9700
11.1705	0.08117	1892357.	39155.	-0.00184	3036.	1.16E+11	0.00	0.00	486.9677
11.3736	0.07672	1989233.	40347.	-0.00180	3192.	1.16E+11	0.00	0.00	490.9078
11.5767	0.07237	2089024.	41548.	-0.00176	3352.	1.16E+11	0.00	0.00	494.7572
11.7798	0.06814	2191755.	42758.	-0.00172	3517.	1.16E+11	0.00	0.00	498.4636
11.9829	0.06401	2297446.	43978.	-0.00167	3686.	1.16E+11	0.00	0.00	502.1559
12.1860	0.06000	2406120.	45206.	-0.00162	3861.	1.16E+11	0.00	0.00	505.7435
12.3891	0.05611	2517799.	45942.	-0.00158	0.00	1.59E+11	-8.137	353.4176	106.1558
12.5922	0.05232	2630059.	46024.	-0.00154	0.00	1.59E+11	-30.161	1405.	0.00
12.7953	0.04863	2742140.	45923.	-0.00149	0.00	1.59E+11	-53.367	2675.	0.00

12.9984	0.04504	2853905.	45772.	-0.00145	0.00	1.59E+11	-70.687	3825.	0.00
13.2015	0.04155	2965249.	45582.	-0.00141	0.00	1.59E+11	-84.458	4954.	0.00
13.4046	0.03818	3076092.	45363.	-0.00136	0.00	1.59E+11	-95.279	6082.	0.00
13.6077	0.03492	3186369.	45121.	-0.00131	0.00	1.59E+11	-103.316	7211.	0.00
13.8108	0.03178	3296032.	44863.	-0.00126	0.00	1.59E+11	-108.743	8340.	0.00
14.0139	0.02876	3405049.	44594.	-0.00121	0.00	1.59E+11	-111.742	9468.	0.00
14.2170	0.02587	3513402.	44321.	-0.00116	0.00	1.59E+11	-112.502	10597.	0.00
14.4201	0.02312	3621087.	44048.	-0.00110	0.00	1.59E+11	-111.219	11725.	0.00
14.6232	0.02050	3728112.	43781.	-0.00105	0.00	1.59E+11	-108.098	12854.	0.00
14.8263	0.01801	3834494.	43524.	-9.89E-04	0.00	1.58E+11	-103.351	13983.	0.00
15.0294	0.01568	3940263.	43279.	-9.29E-04	0.00	1.58E+11	-97.197	15111.	0.00
15.2325	0.01349	4045454.	43051.	-8.68E-04	0.00	1.58E+11	-89.862	16240.	0.00
15.4356	0.01145	4150111.	42842.	-8.05E-04	0.00	1.58E+11	-81.579	17368.	0.00
15.6387	0.00956	4254284.	42654.	-7.40E-04	0.00	1.58E+11	-72.590	18497.	0.00
15.8418	0.00784	4358026.	42489.	-6.74E-04	0.00	1.58E+11	-63.141	19626.	0.00
16.0449	0.00628	4461393.	42347.	-6.06E-04	0.00	1.58E+11	-53.490	20754.	0.00
16.2480	0.00489	4564441.	42228.	-5.36E-04	0.00	1.58E+11	-43.897	21883.	0.00
16.4511	0.00367	4667230.	42132.	-4.65E-04	0.00	1.58E+11	-34.632	23011.	0.00
16.6542	0.00262	4769812.	42059.	-3.92E-04	0.00	1.58E+11	-25.973	24140.	0.00
16.8573	0.00176	4872240.	-10437.	-3.18E-04	0.00	1.58E+11	-43053.	5.98E+07	0.00
17.0604	0.00107	4718937.	-114171.	-2.44E-04	0.00	1.58E+11	-42073.	9.56E+07	0.00
17.2635	5.66E-04	4315724.	-213315.	-1.75E-04	0.00	1.58E+11	-39286.	1.69E+08	0.00
17.4666	2.21E-04	3679155.	-302366.	-1.13E-04	0.00	1.59E+11	-33791.	3.72E+08	0.00
17.6697	1.48E-05	2841870.	-347350.	-6.30E-05	0.00	1.59E+11	-3123.	5.16E+08	0.00
17.8728	-8.56E-05	1986034.	-327509.	-2.59E-05	0.00	1.59E+11	19405.	5.53E+08	0.00

```
   18.0759  -1.12E-04   1245462.   -270914.  -5.03E-06       0.00   2.66E+11
27037.   5.90E+08       0.00
   18.2790  -1.10E-04    665488.   -203478.   3.74E-06       0.00   2.66E+11
28302.   6.27E+08       0.00
   18.4821  -9.35E-05    253629.   -137957.   7.95E-06       0.00   2.66E+11
25465.   6.64E+08       0.00
   18.6852  -7.13E-05     -6969.    -81949.   9.08E-06       0.00   2.66E+11
20496.   7.01E+08       0.00
   18.8883  -4.92E-05   -145822.    -38816.   8.38E-06       0.00   2.66E+11
14900.   7.38E+08       0.00
   19.0914  -3.04E-05   -196172.     -8874.   6.81E-06       0.00   2.66E+11
9671.   7.75E+08       0.00
   19.2945  -1.60E-05   -189079.      9408.   5.05E-06       0.00   2.66E+11
5332.   8.12E+08       0.00
   19.4976  -5.82E-06   -150313.     18377.   3.49E-06       0.00   2.66E+11
2028.   8.48E+08       0.00
   19.7007   1.00E-06    -99503.     20403.   2.35E-06       0.00   2.66E+11
-364.708   8.85E+08       0.00
   19.9038   5.61E-06    -50860.     17373.   1.66E-06       0.00   2.66E+11
-2122.   9.22E+08       0.00
   20.1069   9.07E-06    -14822.     10434.   1.35E-06       0.00   2.66E+11
-3572.   9.59E+08       0.00
   20.3100   1.22E-05       0.00       0.00   1.29E-06       0.00   2.66E+11
-4991.   4.98E+08       0.00
```

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 2:

```
Pile-head deflection              =     0.38898515 inches
Computed slope at pile head       =     -0.0024129 radians
Maximum bending moment            =       4872240. inch-lbs
Maximum shear force               =       -347350. lbs
Depth of maximum bending moment   =    16.85730000 feet below pile head
Depth of maximum shear force      =    17.66970000 feet below pile head
Number of iterations              =             13
Number of zero deflection points =              2
Pile deflection at ground         =     0.05762766 inches
```

Pile-head Deflection vs. Pile Length for Load Case 2

Boundary Condition Type 1, Shear and Moment

Shear = 0. lbs
Moment = 0. in-lbs
Axial Load = 0. lbs

Pile Length feet	Pile Head Deflection inches	Maximum Moment ln-lbs	Maximum Shear lbs
20.31000	0.38898515	4872240.	-347350.
19.29450	0.38375903	4840496.	-332448.
18.27900	0.44160439	4810544.	-448039.

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	0.00	M, in-lb	0.00	0.00	0.2518	-0.00157	-230912.	3253444.
2	V, lb	0.00	M, in-lb	0.00	0.00	0.3890	-0.00241	-347350.	4872240.

Maximum pile-head deflection = 0.3889851469 inches
Maximum pile-head rotation = -0.0024129073 radians = -0.138249 deg.

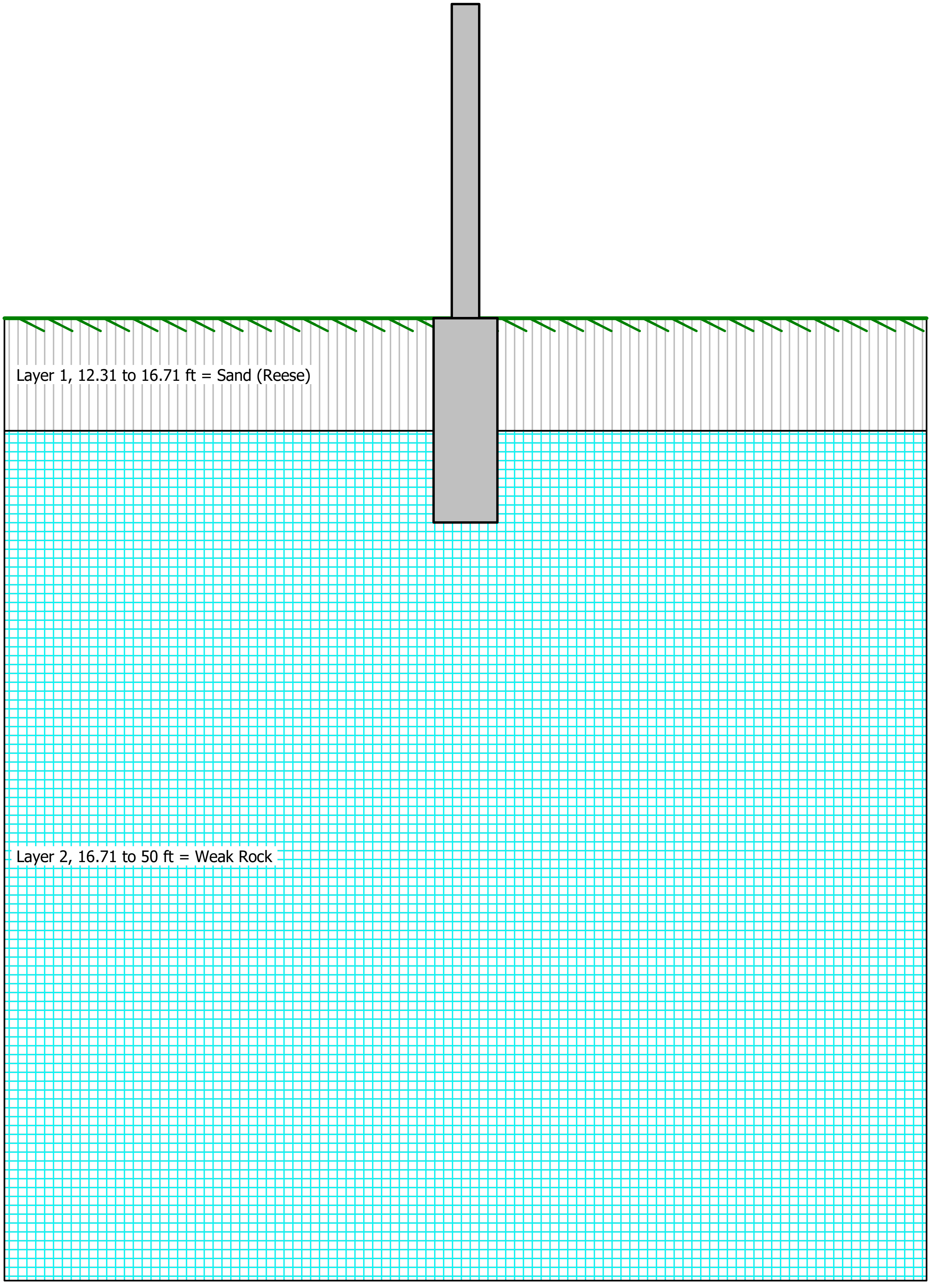
Summary of Warning Messages

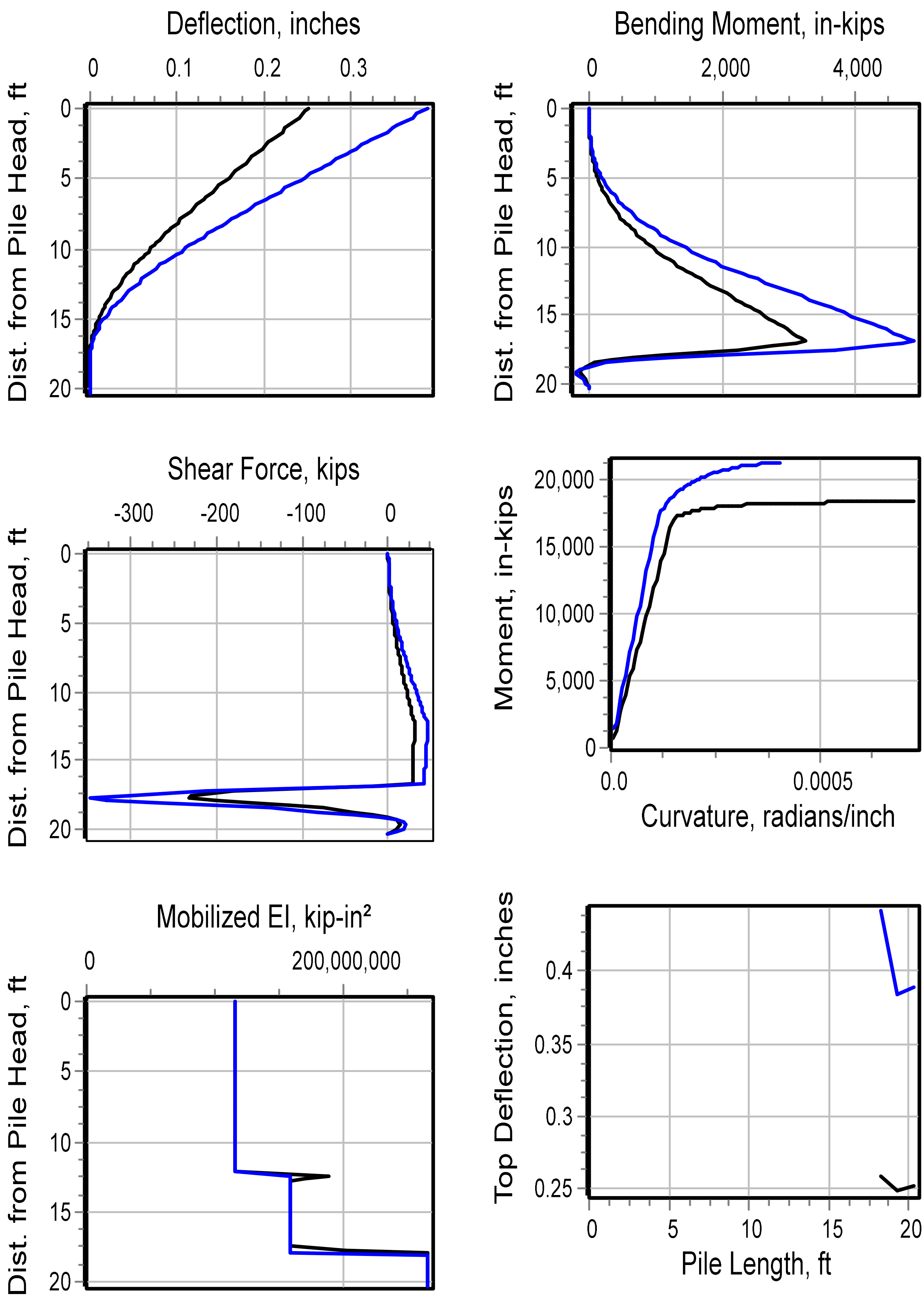
The following warning was reported 1010 times

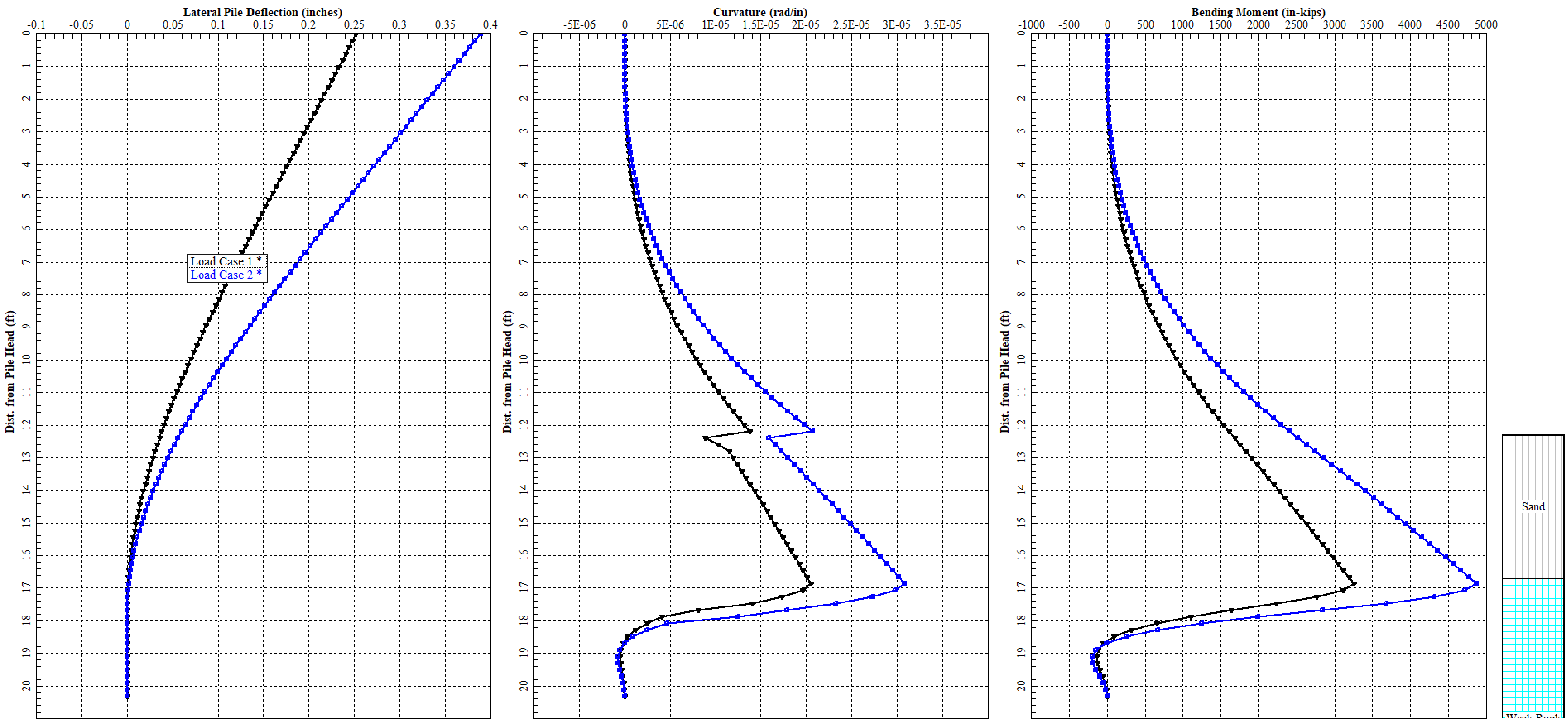
**** Warning ****

An unreasonable input value for unconfined compressive strength has been specified for a soil defined using the weak rock criteria. The input value is greater than 500 psi. Please check your input data for correctness.

The analysis ended normally.


Layer 1, 12.31 to 16.71 ft = Sand (Reese)
Layer 2, 16.71 to 50 ft = Weak Rock


Deflection, inches
0
0.1
0.2
0.3
Dist. from Pile Head, ft
0
5
10
15
20
Bending Moment, in-kips
0
2,000
4,000
Dist. from Pile Head, ft
0
5
10
15
20
Shear Force, kips
-300
-200
-100
0
Dist. from Pile Head, ft
0
5
10
15
20
Moment, in-kips
20,000
15,000
10,000
5,000
0
0.0
0.0005
Curvature, radians/inch
Mobilized EI, kip-in²
0
200,000,000
Dist. from Pile Head, ft
0
5
10
15
20
Top Deflection, inches
0.4
0.35
0.3
0.25
0
5
10
15
20
Pile Length, ft



Lateral Pile Deflection (inches)
-0.1 -0.05 0 0.05 0.1 0.15 0.2 0.25 0.3 0.35 0.4
Load Case 1 *
Load Case 2 *
Dist. from Pile Head (ft)
Curvature (rad/in)
-5E-06 0 5E-06 1E-05 1.5E-05 2E-05 2.5E-05 3E-05 3.5E-05
Bending Moment (in-kips)
-1000 -500 0 500 1000 1500 2000 2500 3000 3500 4000 4500 5000
Sand

Appendix D

Geotechnical Engineering Design Checklists

I. Geotechnical Design Checklists

Project: ATB Old Main St. Bridge (Conneaut)	**PDP Path:**
PID: 119471	**Review Stage:**

Checklist	Included in This Submission
II. Reconnaissance and Planning	✓
III. A. Centerline Cuts	
III. B. Embankments	
III. C. Subgrade	✓
IV. A. Foundations of Structures	✓
IV. B. Retaining Wall	
V. A. Landslide Remediation	
V. B. Rockfall Remediation	
V. C. Wetland or Peat Remediation	
V. D. Underground Mine Remediation	
V. E. Surface Mine Remediation	
V. F. Karst Remediation	
VI. A. Geotechnical Profile	
VI. D. Geotechnical Reports	✓

II. Reconnaissance and Planning Checklist

C-R-S:	(Conneaut)	PID:	119471	Reviewer:	IEH	Date:	3/17/2025

Reconnaissance		(Y/N/X)	Notes:
1	Based on Section 302.1 in the SGE, have the necessary plans been developed in the following areas prior to the commencement of the subsurface exploration reconnaissance:	X	Plans to be prepared by others
	Roadway plans		
	Structures plans		
	Geohazards plans		
2	Have the resources listed in Section 302.2.1 of the SGE been reviewed as part of the office reconnaissance?	Y	
3	Have all the features listed in Section 302.3 of the SGE been observed and evaluated during the field reconnaissance?	Y	
4	If notable features were discovered in the field reconnaissance, were the GPS coordinates of these features recorded?	X	

Planning - General		(Y/N/X)	Notes:
5	In planning the geotechnical exploration program for the project, have the specific geologic conditions, the proposed work, and historic subsurface exploration work been considered?	Y	
6	Has the ODOT Transportation Information Mapping System (TIMS) been accessed to find all available historic boring information and inventoried geohazards?	Y	No historic boring were found at the project location.
7	Have the borings been located to develop the maximum subsurface information while using a minimum number of borings, utilizing historic geotechnical explorations to the fullest extent possible?	Y	
8	Have the topography, geologic origin of materials, surface manifestation of soil conditions, and any other special design considerations been utilized in determining the spacing and depth of borings?	Y	
9	Have the borings been located so as to provide adequate overhead clearance for the equipment, clearance of underground utilities, minimize damage to private property, and minimize disruption of traffic, without compromising the quality of the exploration?	Y	

II. Reconnaissance and Planning Checklist

Planning - General		(Y/N/X)	Notes:
10	Have the scaled boring plans, showing all project and historic borings, and a schedule of borings in tabular format, been submitted to the District Geotechnical Engineer?	Y	Included with proposal.
	The schedule of borings should present the following information for each boring:		
a.	exploration identification number	Y	
b.	location by station and offset	Y	
c.	estimated amount of rock and soil, including the total for each for the entire program.	Y	
Planning – Exploration Number		(Y/N/X)	Notes:
11	Have the coordinates, stations and offsets of all explorations (borings, soundings, test pits, etc.) been identified?	Y	
12	Has each exploration been assigned a unique identification number, in the following format X-ZZZ-W-YY, as per Section 303.2 of the SGE?	Y	
13	When referring to historic explorations that did not use the identification scheme in 12 above, have the historic explorations been assigned identification numbers according to Section 303.2 of the SGE?	X	

II. Reconnaissance and Planning Checklist

Planning – Boring Types		(Y/N/X)	Notes:
14	Based on Sections 303.3 to 303.7.6 of the SGE, have the location, depth, and sampling requirements for the following boring types been determined for the project?	Y	
	Check all boring types utilized for this project:		
	Existing Subgrades (Type A)	✓	
	Roadway Borings (Type B)		
	Embankment Foundations (Type B1)		
	Cut Sections (Type B2)		
	Sidehill Cut Sections (Type B3)		
	Sidehill Cut-Fill Sections (Type B4)		
	Sidehill Fill Sections on Unstable Slopes (Type B5)		
	Geohazard Borings (Type C)		
	Lakes, Ponds, and Low-Lying Areas (Type C1)		
	Peat Deposits, Compressible Soils, and Low Strength Soils (Type C2)		
	Uncontrolled Fills, Waste Pits, and Reclaimed Surface Mines (Type C3)		
	Underground Mines (C4)		
	Landslides (Type C5)		
	Rock Slope (Type C6)		
	Karst (Type C7)		
	Proposed Underground Utilities (Type D)		
	Structure Borings (Type E)		
	Bridges (Type E1)	✓	
	Culverts (Type E2 a,b,c)		
	Retaining Walls (Type E3 a and b)		
	Noise Barrier (Type E4)		
	CCTV & High Mast Lighting Towers (Type E5)		
	Buildings and Salt Domes (Type E6)		

III.C. Subgrade Checklist

C-R-S: (Conneaut)	PID: 119471	Reviewer: IEH	Date: 3/17/2025

Use this Checklist in conjunction with the Subgrade design guidance in GDM Section 600
If you do not have any subgrade work on the project, you do not have to fill out this checklist.

Subgrade		(Y/N/X)	Notes:
1	Has the subsurface exploration adequately characterized the soil or rock according to GDM Section 600?	Y	
a.	Has each sample been visually classified and inspected for the presence of gypsum? Has a moisture content been performed on each sample?	Y	
b.	Has mechanical classification (Plastic Limit (PL), Liquid Limit (LL), and gradation testing) been done on at least two samples from each boring within six feet of the proposed subgrade?	Y	
c.	Has the sulfate content of at least one sample from each boring within 3 feet of the proposed subgrade been determined, per Supplement 1122, Determining Sulfate Content in Soils?	Y	
d.	Has the sulfate content of all samples that exhibit gypsum crystals been determined?	X	No gypsum observed in samples.
e.	Have A-2-5, A-4b, A-5, A-7-5, A-8a, or A-8b soils within the top 3 feet of the proposed subgrade been mechanically classified?	X	None present.
2	If soils classified as A-2-5, A-4b, A-5, A-7-5, A-8a, or A-8b, or having a LL>65, are present at the proposed subgrade (geotechnical profile), do the plans specify that these materials need to be removed and replaced or chemically stabilized?	X	None present.
a.	If these materials are to be removed and replaced, have the station limits, depth, and lateral limits for the planned removal been provided?	X	
3	If there is any rock, shale, or coal present at the proposed subgrade (C&MS 204.05), do the plans specify the removal of the material?	X	
a.	If removal of any rock, shale, or coal is required, have the station limits, depth, and lateral limits for the planned removal of the material at proposed subgrade been provided?		

III.C. Subgrade Checklist

Subgrade		(Y/N/X)	Notes:
4	In accordance with GDM Section 600, do the SPT (N_{60})/HP values and existing moisture contents for the proposed subgrade soils indicate the need for subgrade stabilization?	N	
a.	If removal and replacement is applicable, has the detail of subgrade removal been shown on the plans, including depth of removal, station limits, lateral extent, replacement material, and plan notes (Item 204 - Subgrade Compaction and Proof Rolling)?	Y	Removal and replacement is anticipated. Extent of Removal and replacement is shown in the report. Plans to be prepared by others.
b.	If chemical stabilization is applicable, has the detail of this treatment been shown on the plans, including depth, percentage of chemical, station limits, lateral extent, and plan notes?	X	Chemical stabilization not anticipated to be economical. Plans to be prepared by others.
	Indicate type of chemcial stabilization specified:		
	cement stabilization		
	lime stabilization		
5	If removal and replacement has been specified, do the plans include Plan Note G121 from L&D3?	X	Plans to be prepared by others.
6	If drainage or groundwater is an issue with the proposed subgrade, has an appropriate drainage system (e.g., pipe, underdrains) been provided?	X	Plans to be prepared by others.
7	Has an appropriate quantity of Proof Rolling (C&MS 204.06) and has Plan Note G111 from L&D3 been included in the plans?	X	Plans to be prepared by others.
8	Has a design CBR value been provided?	Y	

IV.A Foundations of Structures Checklist

C-R-S: (Conneaut)	PID: 119471	Reviewer: IEH	3/17/2025

Use this Checklist in conjunction with the bridge foundation design guidance in GDM Section 1300
If you do not have such a foundation or structure on the project, you do not have to fill out this checklist.

		(Y/N/X)	Notes:
Soil and Bedrock Strength Data		(Y/N/X)	Notes:
1	Has the shear strength of the foundation soils been determined?	Y	
	Check method used:		
	laboratory shear tests	✓	
	estimation from SPT or field tests	✓	
2	Have sufficient soil shear strength, consolidation, and other parameters been determined so that the required allowable loads for the foundation/structure can be designed?	Y	
3	Has the shear strength of the foundation bedrock been determined?	Y	
	Check method used:	UCS	
	laboratory shear tests	✓	
	other (describe other methods)		
Spread Footings		(Y/N/X)	Notes:
4	Are there spread footings on the project? If no, go to Question 11	Y	
5	Have the recommended bottom of footing elevation and reason for this recommendation been provided?	Y	
a.	Has the recommended bottom of footing elevation taken scour from streams or other water flow into account?	N	Scour is not aticipated at that footing elevation.
6	Were representative sections analyzed for the entire length of the structure for the following:	Y	
a.	factored bearing resistance?	Y	
b.	factored sliding resistance?	N	Recommended soil parameters provided.
c.	eccentric load limitations (overturning)?	N	
d.	predicted settlement?	Y	
e.	overall (global) stability?	N	
7	Has the need for a shear key been evaluated?	N	
a.	If needed, have the details been included in the plans?	X	Plans to be prepared by others.
8	If special conditions exist (e.g. geometry, sloping rock, varying soil conditions), was the bottom of footing "stepped" to accommodate them?	X	Conditions not present.
9	Have the Service I and Maximum Strength Limit States for bearing pressure on soil or rock been provided?	Y	

IV.A Foundations of Structures Checklist

Spread Footings		(Y/N/X)	Notes:
10	If weak soil is present at the proposed foundation level, has the removal / treatment of this soil been developed and included in the plans?	X	Conditions not present
a.	Have the procedure and quantities related to this removal / treatment been included in the plans?	X	See response from Item 10, above.
Pile Structures		(Y/N/X)	Notes:
11	Are there piles on the project? If no, go to Question 17	N	
12	Has an appropriate pile type been selected?		
	Check the type selected:		
	H-pile (driven)		
	H-pile (prebored)		
	Cast In-place Reinforced Concrete Pipe		
	Micropile		
	Continuous Flight Auger (CFA)		
	other (describe other types)		
13	Have the estimated pile length or tip elevation and section (diameter) based on either the Ultimate Bearing Value (UBV) or the depth to top of bedrock been specified? Indicate method used.		
14	If scour is predicted, has pile resistance in the scour zone been neglected?		
15	Has a wave equation drivability analysis been performed as per BDM 305.3.1.2 to determine whether the pile can be driven to either the UBV, the pile tip elevation, or refusal on bedrock without overstressing the pile?		
16	If required for design, have sufficient soil parameters been provided and calculations performed to evaluate the:		
a.	Nominal unit tip resistance and maximum settlement of the piles?		
b.	Nominal unit side resistance for each contributing soil layer and maximum deflection of the piles?		
c.	Downdrag load on piles driven through new embankment or compressible soil layers, as per BDM 305.3.2.2?		
d.	Potential for and impact of lateral squeeze from soft foundation soils?		

IV.A Foundations of Structures Checklist

Pile Structures		(Y/N/X)	Notes:
17	If piles are to be driven to strong bedrock (Q_u >7.5 ksi) or through very dense granular soils or overburden containing boulders, have "pile points" been recommended in order to protect the tips of the steel piling, as per BDM 305.3.5.6?		
18	If subsurface obstacles exist, has preboring been recommended to avoid these obstructions?		
19	If piles will be driven through 15 feet or more of new embankment, has preboring been specified as per BDM 305.3.5.7?		

IV.A Foundations of Structures Checklist

Drilled Shafts		(Y/N/X)	Notes:
20	Are there drilled shafts on the project? If no, go to the next checklist.	Y	
21	Have the drilled shaft diameter and embedment length been specified?	Y	
22	Have the recommended drilled shaft diameter and embedment been developed based on the nominal unit side resistance and nominal unit tip resistance for vertical loading situations?	Y	
23	For shafts undergoing lateral loading, have the following been determined:	Y	
a.	total factored lateral shear?	Y	
b.	total factored bending moment?	Y	
c.	maximum deflection?	Y	
d.	reinforcement design?	X	
24	If a bedrock socket is required, has a minimum rock socket length equal to 1.5 times the rock socket diameter been used, as per BDM 305.4.2?	Y	
25	Generally, bedrock sockets are 6" smaller in diameter than the soil embedment section of the drilled shaft. Has this factor been accounted for in the drilled shaft design?	Y	
26	If scour is predicted, has shaft resistance in the scour zone been neglected?	✓	See response from Item 4a, above.
27	Has the site been assessed for groundwater influence?	N	
a.	If yes, and if artesian flow is a potential concern, does the design address control of groundwater flow during construction?	X	
28	Have all the proper items been included in the plans for integrity testing?	N	Plans to be prepared by others.
29	If special construction features (e.g., slurry, casing, load tests) are required, have all the proper items been included in the plans?	N	
30	If necessary, have wet construction methods been specified?	N	
General		(Y/N/X)	Notes:
31	Has the need for load testing of the foundations been evaluated?	N	
a.	If needed, have details and plan notes for load testing been included in the plans?		

VI.B. Geotechnical Reports

C-R-S: (Conneaut)	PID: 119471	Reviewer: IEH	Date: 3/17/2025

General		(Y/N/X)	Notes:
1	Has an electronic copy of all geotechnical submissions been provided to the District Geotechnical Engineer (DGE)?	Y	
2	Has the first complete version of a geotechnical report being submitted been labeled as 'Draft'?	Y	
3	Subsequent to ODOT's review and approval, has the complete version of the revised geotechnical report being submitted been labeled 'Final'?	X	This is a draft submittal.
4	Has the boring data been submitted in a native format that is DIGGS (Data Interchange for Geotechnical and Geoenvironmental) compatable? gINT files meet this demand?	Y	gINT project file will be sent with final report.
5	Does the report cover format follow ODOT's Brand and Identity Guidelines Report Standards found at http://www.dot.state.oh.us/brand/Pages/default.aspx ?	Y	
6	Have all geotechnical reports being submitted been titled correctly as prescribed in Section 706.1 of the SGE?	Y	
Report Body		(Y/N/X)	Notes:
7	Do all geotechnical reports being submitted contain the following:	Y	
a.	an Executive Summary as described in Section 706.2 of the SGE?	Y	
b.	an Introduction as described in Section 706.3 of the SGE?	Y	
c.	a section titled "Geology and Observations of the Project," as described in Section 706.4 of the SGE?	Y	
d.	a section titled "Exploration," as described in Section 706.5 of the SGE?	Y	
e.	a section titled "Findings," as described in Section 706.6 of the SGE?	Y	
f.	a section titled "Analyses and Recommendations," as described in Section 706.7 of the SGE?	Y	
Appendices		(Y/N/X)	Notes:
8	Do all geotechnical reports being submitted contain all applicable Appendices as described in Section 706.8 of the SGE?	Y	
9	Do the Appendices present a site Boring Plan showing all boring locations as described in Section 706.8.1 of the SGE?	Y	

VI.B. Geotechnical Reports

Appendices		(Y/N/X)	Notes:
10	Do the Appendices include boring logs and color pictures of rock, if applicable, as described in Section 706.8.2 of the SGE?	Y	
11	Do the Appendices include reports of undisturbed test data as described in Section 706.8.3 of the SGE?	Y	
12	Do the Appendices include calculations in a logical format to support recommendations as described in Section 706.8.4 of the SGE?	Y	

VII. References

Publications - FHWA

Advanced Course on Slope Stability, Volume 1 and 2, Abramson, Lee, Boyce, Glenn, et al., Publication No. FHWA-SA-94-005 and 006

Corrosion/Degradation of Soil Reinforcement for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Elias, Publication No. FHWA-NHI-09-087

Geotechnical Engineering Circular No. 2 - Earth Retaining Systems, Sabitini, Elias, et al., Publication No. FHWA-SA-96-038

Geotechnical Engineering Circular No. 3 - LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Kavazanjian, Publication No. FHWA-NHI-11-032

Geotechnical Engineering Circular No. 4 - Ground Anchors and Anchor Systems, Sabitini, Pass and Bachus, Publication No. FHWA-IF-99-015

Geotechnical Engineering Circular No. 5 – Geotechnical Site Characterization, Loehr, et. al., Publication No. FHWA-NHI-16-072

Geotechnical Engineering Circular No. 6 – Shallow Foundations, Kimmerling, Publication No. FHWA-IF-02-054

Geotechnical Engineering Circular No. 7 – Soil Nail Walls Reference Manual, Lazarte, et. al., Publication No. FHWA-NHI-14-007

Geotechnical Engineering Circular No. 8 – Design and Construction of Continuous Flight Auger Piles, Brown, et. al., Publication No. FHWA-HIF-07-039

Geotechnical Engineering Circular No. 9 – Design and Analysis of Laterally Loaded Deep Foundations, Parkes, et. al., Publication No. FHWA-HIF-18-031

Geotechnical Engineering Circular No. 10 - Drilled Shafts: Construction Procedures and Design Methods, Brown, et. al., Publication No. FHWA-NHI-18-024

Geotechnical Engineering Circular No. 11 - Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Volume I and II, Berg, Christopher, and Samtani, Publication No. FHWA-NHI-10-024 and 025

Geotechnical Engineering Circular No. 12 - Design and Construction of Driven Pile Foundations, Volume I and II, Hannigan, Rausche, Likins, Robinson, and Becker, Publication No. FHWA-NHI-16-009 and 010

Geotechnical Engineering Circular No. 13 – Ground Modification Methods Reference Manual, Volume I and II, Schaefer, et. al., Publication No. FHWA-NHI-16-027 and 028

Geotechnical Engineering Circular No. 15 – Acceptance Procedures for Structural Foundations, Loehr, et. al., Publication No. FHWA-HIF-22-024

Geotechnical Instrumentation Reference Manual, Dunnicliff, NHI Course No. 13241 - Module 11

Prefabricated Vertical Drains: Volume 1: Engineering Guidelines, Rixner, Kraemer, and Smith, Publication No. FHWA-RD-86-168

Soils and Foundations Workshop, Reference Manual and Participant Workbook, Cheney and Chassie, Publication No. NHI-00-045

Soils and Foundations Reference Manual, Volume I and II, Samtani and Nowatzki, Publication No. NHI-06-088 and 089

Highway Subdrainage Design, Moulton, Publication No. FHWA-TS-80-224

Tiebacks, Weatherby, Publication No. FHWA/RD-82/047

VII. References

Publications - ODOT (www.dot.state.oh.us/drrc/)
Bridge Design Manual, Office of Structural Engineering CADD Engineering Standards Manual, Office of CADD and Mapping Construction and Material Specifications, Office of Construction Administration Geotechnical Design Manual, Office of Geotechnical Engineering Location and Design Manual: Volume 1 - Roadway Design, Office of Roadway Engineering Location and Design Manual: Volume 3 - Highway Plans, Office of CADD and Mapping Manual for Abandoned Underground Mine Inventory and Risk Assessment (AUMIRA), Office of Geotechnical Engineering Pavement Design Manual, Office of Pavement Engineering Specifications for Geotechnical Explorations, Office of Geotechnical Engineering
Publications - ODNR (www.dnr.state.oh.us/)
Bedrock Geology Map, DGS Bedrock Structure Map, DGS Geologic Map of Ohio, DGS Quaternary Geology of Ohio, DGS Bedrock Topography Map, DGS Known and Probable Karst in Ohio, DGS USGS Open File Map Series #78-1057 Landslides and Related Features, DGS Other publications or information available from ODNR: Bulletins Information Circulars Boring logs Water well logs Measured geologic section(s) Report of Investigations
Publications – Other Organizations
AASHTO LRFD Bridge Design Specifications, Highway Subcommittee on Bridges and Structures, latest edition Soil Survey, Natural Resources Conservation Service (https://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/survey/) Wetlands Mapper, National Wetlands Inventory (https://www.fws.gov/wetlands/data/Mapper.html)