

Final Report of:

**STRUCTURE FOUNDATION EXPLORATION FOR
MAIN STREET BRIDGE OVER INTERSTATE 71 (FRA-71-1716C)
PROJECT FRA-70-14.48
I-70/I-71 EAST INTERCHANGE (PID 77370)
COLUMBUS, OHIO**

Prepared for:
**Ohio Department of Transportation
District 6
400 East William Street
Delaware, Ohio 43015**

DLZ Ohio, Inc.
6121 Huntley Road
Columbus, Ohio 43229

November 14, 2013

DLZ Job No. 1021-1005.01



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EXECUTIVE SUMMARY

As part of the FRA-70-14.48 East Interchange project in Columbus, Ohio, a structure foundation exploration was performed for the proposed FRA-71-1716C Bridge. The proposed bridge will replace the existing five-span bridge and will be a three-span structure that will carry Main Street traffic over Interstate 71 (I-71), beginning at approximate Sta. 33+78.38 and ending at approximate Sta. 37+56.77. This exploration was performed to (1) determine the subsurface conditions, (2) determine the characteristics of the subsurface materials, and (3) provide information to assist in the design of the structure foundations.

The subsurface conditions were determined by considering nine borings. Six borings were drilled to depths of between 7.0 and 90.0 feet and three historic borings drilled to depths of between 61.0 and 66.0 feet were also considered. Samples of the subsurface materials were obtained for classification and general index testing.

Concrete cantilever-type wall abutments and piers supported on spread footings are being considered for the proposed FRA-71-1716C Bridge. Based on the drawings provided by ms consultants inc., the maximum proposed wall heights (taken as the difference between the top of the proposed bridge deck elevations and the proposed bottom of footing elevations) are expected to be approximately 33.1 feet at the rear abutment and 31.9 feet at the forward abutment. The bottom of footing elevation at proposed Pier 1 is anticipated to be at elevation 739 (approximately 46.0 feet below the proposed bridge deck), while the bottom of footing elevation at proposed Pier 2 is anticipated to be at elevation 746 (approximately 40.0 feet below the proposed bridge deck). Based on the foundation dimensions and loading information provided by ms consultants inc., the results of the analyses indicated that the provided base widths of 20.25 feet and 20.0 feet were adequate for the stability of the rear and forward abutments, respectively. Furthermore, the provided base width of 14.0 feet for both Pier 1 and Pier 2 was adequate in terms of stability.

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1.0 INTRODUCTION

This report includes the findings of the foundation exploration for the proposed Main Street Bridge (FRA-71-1716C) that will carry Main Street traffic over Interstate 71 (I-71) as part of the I-70/I-71 East Interchange improvements. Based on the information provided, the proposed bridge will replace the existing five-span bridge and will be a three-span structure, beginning at approximate Sta. 33+78.38 and ending at approximate Sta. 37+56.77. Note that all station numbers used in this report are referenced to the baseline of the proposed Main Street Bridge.

According to the most recent bridge profile drawing provided, the maximum proposed wall heights (taken as the difference between the top of the proposed bridge deck elevations and the proposed bottom of footing elevations) are expected to be approximately 33.1 feet at the rear abutment and 31.9 feet at the forward abutment. The bottom of footing elevation at proposed Pier 1 is anticipated to be at elevation 739 (approximately 46.0 feet below the proposed bridge deck), while the bottom of footing elevation at proposed Pier 2 is anticipated to be at elevation 746 (approximately 40.0 feet below the proposed bridge deck). It is understood that concrete cantilever-type wall abutments and piers supported on spread footings are being considered for this structure. The anticipated bottom of footing levels for the cantilever-type wall abutments and piers are expected to vary between elevations 739.0 and 754.5. In order to achieve the proposed final grade immediately in front of the abutment walls, it is anticipated that up to approximately 10.0 feet of excavation will be required. To achieve the final grade immediately behind the rear abutment wall, approximately 16.6 feet of new fill will need to be placed at the abutment. However, the proposed grade behind the forward abutment wall is anticipated to be approximately equal to the existing grade. Consequently, a minimal amount of cut and fill is expected at the forward abutment. To achieve the final grade at the proposed pier locations, excavations between approximately 2.0 and 11.0 feet are anticipated.

The purpose of this exploration was to 1) determine the subsurface conditions to the depths of the borings, 2) evaluate the engineering characteristics of the subsurface materials, and 3) provide information to assist in the design of the structure foundations. The exploration presented in this report was performed essentially in accordance with DLZ Ohio, Inc.'s (DLZ) proposal for the project.

Information about the proposed bridge was obtained from the most recent Site Plan (see Appendices I) and email/phone communications in May and June of 2013. If the proposed locations or structural concepts are changed or differ from that assumed, DLZ should be informed of the changes so that recommendations and conclusions presented in this report may be revised as necessary.

The geotechnical engineer has planned and supervised the performance of the geotechnical engineering services, considered the findings, and prepared this report in accordance with generally accepted geotechnical engineering practices. No other warranties, either expressed or implied, are made as to the professional advice included in this report.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Geology of the Site

Generalized geology information is presented in the sections below. Information regarding the thickness of the glacial deposits and hydrogeology was obtained from general geologic references. A complete list of the geologic references used for the project is presented in the Project FRA-70-8.93 Red Flag Summary report, dated April 11, 2008.

2.1.1 General Geology

The natural ground in the area of this structure is characterized by relatively flat to gently sloping topography. The natural ground surface is interrupted by the 20 to 25 foot deep trench at the existing freeway. The project area is located in the Columbus Lowland of the glaciated Central Lowland Physiographic Region. The Columbus Lowland is characterized by the Wisconsin-age till over Devonian and Mississippian-age carbonate rock, shale and siltstone.

General geological references report the site was covered by the Wisconsin and Illinoian glaciers. The East Interchange area is approximately 1.5 miles east of the Scioto River.

The predominant glacial deposits in Franklin County consist of glacial till, which is a heterogeneous mixture of clay, silt, sand, gravel, cobbles and boulders. Most of the project alignment extends through glacial outwash deposits that fill the preglacial valley in which the Scioto River flows. Fine-grained fill and quaternary (recent) alluvial deposits cover the glacial outwash materials and are present along the length of the project area. The thickness of the glacial deposits in the East Interchange area generally varies between 80 and 120 feet.

Bedrock along the alignment lies on the eastern margin of the Cincinnati Arch and is inclined at approximately 10 feet per mile to the southeast towards the Appalachian Basin. Bedrock units as well as contacts between units display a pronounced north-south orientation or strike. Three different rock formations lie immediately below the soils along the alignment. These include from youngest to oldest: the Olentangy Shale, the Delaware Limestone and the Columbus Limestone, all of the Devonian Age.

2.1.2 Hydrogeology and Groundwater

Most of the project alignment extends through glacial outwash deposits that fill the preglacial valley in which the Scioto River flows. Fine-grained fill and quaternary (recent) alluvial deposits cover the glacial outwash materials and are

present along the length of the project area. These glacial outwash sequences consist primarily of sand and gravel, and have the potential to yield significant amounts (greater than 100 gpm) of ground water.

Groundwater along the corridor is influenced by the presence of manmade subsurface features, including but not limited to: buried utility conduits and pipelines, subsurface foundations and basements and the roadway trench of I-70/71. These features can disrupt the natural groundwater flow by creating barriers and preferential migration pathways. These controlling features can impact near surface groundwater by affecting the flow directions and speed. Additionally, these impacts can be highly localized.

2.2 Field Reconnaissance

A field reconnaissance for the FRA-70-14.48 project area was performed in 2008 as part of the preliminary study for the FRA-70-8.93 project. Findings of the field reconnaissance as well as other information obtained for the project area are presented in the Project FRA-70-8.93 Red Flag Summary report, dated April 11, 2008.

3.0 EXPLORATION

3.1 Field Exploration

The geotechnical investigation consisted of three different explorations, totaling nine borings. Three borings, B-001-M-59, B-007-M-59, and B-012-M-59, were drilled as part of the original geotechnical investigation for the existing Main Street Bridge between June 10, 1959 and June 19, 1959. These borings were drilled adjacent to the existing west abutment, the existing third pier (from west to east), and the existing east abutment, respectively. The borings were drilled to depths of between 61 and 66 feet beneath the ground surface. Note that historic borings, B-001-M-59, B-007-M-59, and B-012-M-59, are identified as 1B, 7B, and 12B on the original boring logs, respectively. It should be noted that elevations presented on the historic documents are referenced to the 1929 datum (NGVD 29). These elevations as presented in this report have been corrected to the 1988 datum (NAVD 88), which is 0.6 feet lower than NGVD 29. It is our understanding that an auger boring was also drilled, adjacent to the existing third pier (from west to east), as part of the original investigation, to a depth of 30 feet. However, no standard penetration data or detailed log was available for the auger boring, so it was not evaluated for this report.

Another three borings, B-172-0-09, B-173-0-08, and B-174-0-08, were drilled as part of a preliminary drilling program for the proposed roadway improvements and are located adjacent to the proposed structure. These borings were drilled between January 22 and March 15, 2009. Borings B-172-0-09 and B-174-0-08 were drilled between the proposed west abutment and the proposed first pier. Boring B-173-0-08 was drilled to the west of the proposed west abutment. The borings were drilled to depths of between 7 and 15 feet.

Three borings, B-175-1-10, B-287-1-10, and B-287-2-10, were drilled for the final design of the proposed replacement structure. Boring B-175-1-10 was drilled on February 3 and 4, 2011, and borings B-287-1-10 and B-287-2-10 were drilled between May 26 and June 2, 2010. Boring B-287-1-10 was drilled adjacent to the proposed Pier 1 to a depth of 70 feet, and borings B-175-1-10 and B-287-2-10 was drilled adjacent to the proposed east abutment to depths of 80.0 and 90.0 feet, respectively.

Borings in the preliminary and final phases of drilling were drilled with a truck mounted rotary drill rig. The as-drilled boring locations are shown on the boring plan presented in Appendix I. Boring logs for the preliminary and final phases of drilling are presented in Appendix I and historical borings logs along with a plan showing their locations on the Structure Plan and Profile drawing are presented Appendix II. Information concerning the drilling procedures is presented in Appendix I. The boring locations were determined by representatives of DLZ and approved by representatives at ODOT. The surveyed locations and ground surface elevations of the borings were determined by representatives of ms consultants for the preliminary drilling program and by representatives of DLZ for the final design phase.

3.2 Laboratory Testing Program

The laboratory testing program consisted of visual classifications and general index tests. The general index tests consisted of grain-size analyses, moisture content, and plasticity determinations. The results of the classifications, grain-size analyses, moisture content, and plasticity determinations are shown on the boring logs in Appendix I.

4.0 FINDINGS

The following sections present the generalized subsurface conditions encountered by the borings. For more detailed information, refer to the boring logs presented in Appendix I.

4.1 Soil Conditions

Based on the boring information, the site is generally underlain with thick and dense to very dense layers of granular materials. Cohesive soils, if present, were typically encountered in the upper portions of the borings, at depths of between 3 feet (elevation 759.8) and 5 feet (elevation 777.3) below the existing ground surface or below a depth of 52.5 feet or an elevation of 718.1. However, borings B-175-1-10 and B-287-2-10, located near the proposed forward abutment, encountered relatively thick cohesive soil layers at shallow depths of between approximately 1 foot (approximately elevation 784) and 21.0 feet (approximately elevation 764.5).

At the ground surface, the preliminary borings, B-172-0-09, B-173-0-08, and B-174-0-08, generally encountered 3 to 8 inches of asphalt concrete. Beneath the asphalt concrete, borings B-172-0-09 and B-173-0-08 both encountered 9 inches of Portland cement concrete. Portland cement concrete was not encountered in boring B-174-0-08. Underneath the Portland cement concrete in boring B-172-0-09 and asphalt concrete in boring B-174-0-08, two to four inches of base aggregates were encountered in these borings. Boring B-172-0-09 and B-174-0-08 were drilled in an existing trench on I-71. Below the surface materials, these borings encountered primarily medium dense to very

dense granular soils consisting of gravel (A-1-a), gravel with sand (A-1-b), and coarse and fine sand (A-3a) to the completion depths of the borings (both 15 feet or elevations between 742.6 and 750.3). Note that boring B-172-0-09 encountered a thin layer of hard sandy silt (A-4a) at depths of between 1.5 and 3.5 feet. Borings B-173-0-08 was drilled outside of the trench. Below the surface materials, the boring encountered hard sandy silt (A-4a) at depths of between 1.0 feet and 5.0 feet, followed by medium dense to dense gravel (A-1-a) to the completion depth of the boring (7.0 feet or elevation 775.3).

Boring B-287-1-10 was drilled in the existing I-71 trench and encountered 8 inches of topsoil at the surface, followed by possible fill consisting of hard silty clay (A-6b) to a depth of 3.5 feet. Below the possible fill, the boring encountered medium dense to very dense granular soils (A-1-b, A-3a, and A-4a) to a depth of 50.5 feet (elevation 712.3). Underlying the granular soils, the boring encountered very dense silt (A-4b) interbedded with a thin layer (approximately 2.5 feet thick) of silty clay (A-6b) at depths of between 50.5 feet (elevation 712.3) and 63.0 feet (elevation 699.8), followed by very stiff clay (A-7-6) to the completion depth of the boring (70 feet or elevation 692.8).

Borings B-175-1-10 and B-287-2 were drilled outside of the I-71 trench and encountered 4 to 8 inches of asphalt concrete overlying 7 to 8 inches of Portland cement concrete. Beneath the Portland cement concrete, approximately 6 inches of base aggregate were encountered in boring B-287-2-10. Beneath these pavement materials, boring B-175-1-10 encountered approximately 5 feet of fill consisting primarily of very stiff to hard silt and clay (A-6a) to a depth of 6.0 feet (elevation 780.2). However, below the pavement materials, boring B-287-2-10 encountered possible fill to a depth of 14.5 feet (elevation 770.6). This possible fill consisted primarily of stiff to very stiff silty clay (A-6b) with varying amounts of brick and rock fragments. Below the fill and possible fill, the borings encountered interbedded layers of loose or hard sandy silt (A-4a), dense to very dense gravel with sand (A-1-b), and stiff to hard silt and clay (A-6a) and silty clay (A-6b) to depths of 21.0 (elevation 765.2) and 23.5 (elevation 761.6) where very dense gravel with sand (A-1-b) was first encountered in the borings. This gravel layer was relatively thick (approximately 43.5 to 51.0 feet) and was present at the depths of between 21.0 feet (elevation 765.2) and 72.0 feet (elevation 714.2). Below the thick gravel layer, very stiff to hard cohesive soils (A-4a and A-6a) were generally encountered to the completion depths of the borings (between 80.0 and 90.0 feet or elevations between 695.1 and 706.2).

The historical borings, B-001-M-59, B-007-M-59, and B-012-M-59, generally encountered sands and gravels with varying amounts of fines to their completion depth. However, borings B-007-M-59 and B-012-M-59 encountered sandy gravelly silt between the ground surface and the approximate depths of 8 feet and 24 feet, respectively. It should be noted that these borings were drilled prior to the construction of the trench. Reportedly, approximately ten to twenty feet of materials in the upper portions of these borings was removed for the construction of the existing I-71.

Heaving sands were first encountered in borings B-175-1-10, B-287-1-10, and B-287-2-10 at depths of 23.5 feet (elevation 739.3) and 43.5 feet (elevation 742.7).

Large gravel pieces, possible cobbles or boulders, or split- spoon refusal were encountered below a depth of 14.0 feet (elevation 772.2) in boring B-175-1-10 and at a

depth of 51.5 feet (elevation 711.3) in boring B-287-1-10. Although boulders were not reported in the historic borings, refusal was encountered below the depth of 50 feet (NAVD 88 elevation 735.4) in boring B-012-M-59. It should be noted that boulders were encountered in borings drilled for nearby bridges along I-71 and I-70.

4.2 Bedrock Conditions

Bedrock was not encountered in any of the borings within the depths of the investigation (depths up to 90 feet or elevation 692.8).

4.3 Groundwater Conditions

Seepage was first encountered in borings B-175-1-10, B-287-1-10, and B-287-2-10 at depths of 38.5 feet (elevation 747.7), 16.0 feet (elevation 746.8), and 14.5 feet (elevation 770.6), respectively. Prior to adding water to wash out heaving sands, groundwater was observed in borings B-175-1-10, B-287-1-10, and B-287-2-10 at depths of 40.8 feet (elevation 745.4), 19.7 feet (elevation 743.1), and 40.2 feet (elevation 744.9), respectively. The final groundwater levels including the drilling water were generally observed in these borings at depths of 39.6 feet (elevation 746.6), 53.7 feet (elevation 709.1), and 39.7 feet (elevation 745.4), respectively. No final groundwater levels were observed in the remaining borings and no groundwater levels were reported on the historic boring logs.

It should be noted that groundwater levels may fluctuate with seasonal variations and following periods of heavy or prolonged precipitation, and therefore, the readings indicated on the boring logs may not be representative of the long-term groundwater level. Long-term monitoring would be needed to obtain a more accurate estimate of the groundwater table elevation.

5.0 ANALYSES AND RECOMMENDATIONS

5.1 General Considerations

According to the most recent bridge profile drawing provided, the maximum proposed wall heights (taken as the difference between the top of the proposed bridge deck elevations and the proposed bottom of footing elevations) are expected to be approximately 33.1 feet at the rear abutment and 31.9 feet at the forward abutment. The bottom of footing elevation at proposed Pier 1 is anticipated to be at elevation 739 (approximately 46.0 feet below the proposed bridge deck), while the bottom of footing elevation at proposed Pier 2 is anticipated to be at elevation 746 (approximately 40.0 feet below the proposed bridge deck). It is understood that concrete cantilever-type wall abutments and piers supported on spread footings are being considered for this structure. The anticipated bottom of footing levels for the cantilever-type wall abutments and piers are expected to vary between elevations 739.0 and 754.5. Table 1 is a summary of the foundation information.

Table 1: Substructure Foundation Assumptions Summary

Type	Approximate Location	Bearing Elevation (ft)	Height (ft)
Rear Abutment	Sta. 33+80.97	750.5	33.1
Pier 1	Sta. 35+32.39	739.0	-
Pier 2	Sta. 36+23.25	746.0	-
Forward Abutment	Sta. 37+54.26	754.5	31.9

Note that spread footing foundations with cast-in-place abutment walls are typically the most economical foundation based on the materials required. However, structure height, fill and excavation quantities, excavation support, maintenance of traffic and right-of-way restrictions, and other construction considerations may make another foundation type more cost effective or feasible. A cost analysis, which is outside the scope of work for the geotechnical engineer on this project, would need to be performed to compare the construction costs of different foundation types.

The subsurface conditions at the site are considered suitable for the use of other foundation types besides spread footings. Note that in general, pile foundations are not recommended for the structures in this project due to the damage that could occur to the pile during driving due to the cobbles and boulders that are typically present in the area. In addition, the vibrations from pile driving could cause damage to adjacent structures.

5.2 Loading

The proposed abutment walls will be subject to loading from several sources. Loading sources include horizontal earth pressure, live load (traffic) surcharge, temperature loads, wind loads, bridge live loads, centrifugal forces, and bridge dead loads. The proposed piers will be subject to loading from the bridge including live loads, bridge dead loads, wind loads, braking loads, centrifugal forces, and temperature loads. Three different live load cases were provided for the analyses of each pier. The anticipated vertical design loads for the abutments and piers are summarized in Tables 2 and 3, respectively. More detailed loading information including horizontal loadings can be found in Appendix III.

Table 2: Abutment Vertical Design Loads

Loading Type	Rear Abutment	Forward Abutment
Maximum unfactored live load reactions (no dynamic load allowance)	5.59 klf	5.43 klf
Total unfactored dead load	34.79 klf*	31.42 klf*
Unfactored dead load of wearing surfaces	2.56 klf	2.24 klf
Maximum factored live load reactions (no dynamic load allowance)	9.78 klf	9.50 klf
Maximum factored dead load	43.49 klf	39.28 klf
Maximum factored dead load of wearing surfaces	3.84 klf	3.36 klf

Unfactored dead loads, including stem wall weight, were provided by ms consultants, inc.

Table 3: Pier Vertical Design Loads

Loading Type	Location	Live Load Case I (kips)	Live Load Case II (kips)	Live Load Case III (kips)
Maximum unfactored live load reactions (no dynamic load allowance)	Pier 1	359	583	457
	Pier 2	329	198	420
Total unfactored dead load*	Pier 1	5,829	5,829	5,829
	Pier 2	5,272	5,272	5,272
Unfactored dead load of wearing surfaces	Pier 1	632	632	632
	Pier 2	549	549	549
Maximum factored live load reactions (no dynamic load allowance)	Pier 1	628	1,020	800
	Pier 2	576	347	735
Maximum factored dead load*	Pier 1	7,286	7,286	7,286
	Pier 2	6,590	6,590	6,590
Maximum factored dead load of wearing surfaces	Pier 1	948	948	948
	Pier 2	824	824	824

* Includes self weight and column weight but not footing weight.

5.3 Soil Conditions and General Soil Properties Assumed

The subsurface conditions encountered in the borings drilled in the vicinity of the proposed abutment walls were generally consistent. The retained soils immediately behind the abutment walls are anticipated to generally consist of medium dense to very dense granular soils or stiff to hard cohesive soil interbedded with very loose to very

dense granular soils. The foundation soils below the proposed abutment walls and piers generally consisted of dense to very dense gravel with sand (A-1-b) and coarse and fine sand (A-3a). The subsurface profile for the soil-structure interaction analyses used the stratigraphy disclosed by the borings and material properties were estimated from Standard Penetration Test blow counts and engineering judgment.

5.4 Cantilever Abutment Wall Foundation Analyses Results

The soils at the foundation elevations and the embankment backfill/retained soils were considered granular. Table 4 summarizes the soil properties assumed for the cantilever abutment wall analyses.

Table 4: Soil Properties Used in Cantilever Abutment Wall Analyses

Location	Soil Type	Unit Weight (pcf)	c (psf)	c' (psf)	ϕ	ϕ'	Representative Boring
Rear Abutment	Embankment Fill/Retained Soil	120	0	0	30	30	B-001-M-59
	Foundation Soil	120	0	0	34	34	
Forward Abutment	Embankment Fill/Retained Soil	120	0	0	30	30	B-175-1-10 & B-287-2-10
	Foundation Soil	120	0	0	36	36	

Bearing capacity and stability analyses (sliding, overturning, and global stability) were performed for the abutment walls, in accordance with ODOT and AASHTO guidelines. The dimensions for the abutment walls were provided by ms consultants inc. It should be noted that analyses for the proposed walls were performed assuming that the groundwater table was at the bottom of the footing. The retained soil behind the wall will be essentially level and subjected to traffic loads.

Results of the analyses indicated that the provided base widths of 20.25 feet and 20.0 feet were adequate for the stability of the rear and forward abutments, respectively. These provided base widths were adequate based on the required factored resistance to bearing, sliding, and overturning.

Results of the stability analyses including the configurations of the wall sections used in the analyses are presented in Appendix III. The analytical results are also summarized in Tables 5 and 6.

Table 5: Rear Cantilever Abutment Wall Analysis Results
(Station 33+80.97)

<u>Sliding along base of cantilever wall</u> $R_R = 61,337$ lbs/ft width, where $\phi_\tau = 1.0$
<u>Factored bearing resistance</u> (Strength I-b Case) Undrained, $q_r = \text{N/A}$ (granular) Drained, $q_r = 19,824$ psf
Max factored uniform bearing pressure, $q_{\text{uni}} = 7,784$ psf (Strength I-b Case) Factored uniform bearing pressure, $q_{\text{uni}} = 5,813$ psf (Service I Case)
Approximate maximum height of abutment wall (including embedment) = 33.1 feet Approximate height of retained soil = 33.1 feet Approximate embedment depth = 4.0 feet (minimum) Width of footing analyzed for external stability = 20.25 feet (for a 33.1-foot high wall with a toe width of 5.0 feet and a stem width of 4.25 feet with no batter)

Table 6: Forward Cantilever Abutment Wall Analysis Results
(Station 37+54.26)

<u>Sliding along base of cantilever wall</u> $R_R = 55,671$ lbs/ft width, where $\phi_\tau = 1.0$
<u>Factored bearing resistance</u> (Strength I-b Case) Undrained, $q_r = \text{N/A}$ (granular) Drained, $q_r = 28,280$ psf
Max factored uniform bearing pressure, $q_{\text{uni}} = 5,971$ psf (Strength I-b Case) Factored uniform bearing pressure, $q_{\text{uni}} = 4,382$ psf (Service I Case)
Approximate maximum height of abutment wall (including embedment) = 31.9 feet Approximate height of retained soil = 31.9 feet Approximate embedment depth = 4.0 feet (minimum) Width of footing analyzed for external stability = 20.0 feet (for a 31.9-foot high wall with a toe width of 7.25 feet and a stem width of 4.25 feet with no batter)

5.5 Pier Foundation Analyses Results

Three different live load (LL) cases were provided for the analyses of each pier. For each of the live load cases, one wind load on vehicle (WL) case was analyzed for Pier 1 and two WL cases were analyzed for Pier 2. However, at the request of ms consultants, inc., two horizontal earth load (EH) cases were also considered for each of the live load cases provided for Pier 1. Table 7 below summarizes the load cases analyzed for Piers 1 and 2.

Table 7: Load Cases Analyzed for Piers 1 and 2

Location	Wind Load on Vehicle (WL) Case	Horizontal Earth Load (EH) Case	Live Load (LL) Case I	Live Load Case II	Live Load Case III	No Live Load (LL) ³
Pier 1	Case I	EH _{resisting} ¹	✓	✓	✓	n/a ⁴
		EH _{driving} ²	✓	✓	✓	✓
Pier 2	Case I	Case I	✓	✓	✓	n/a ⁴
	Case II	Case I	✓	✓	✓	n/a ⁴

¹EH_{resisting}: Horizontal load acts as a resisting force, all other horizontal loads as driving forces, and moments equal to all vertical loads times initial eccentricity (3.5 feet) as driving forces.

²EH_{driving}: Horizontal load acts as a driving force, all other horizontal loads as driving forces, and moments equal to all vertical loads times initial eccentricity (3.5 feet) as resisting forces.

³EH_{driving}: Horizontal load acts as a driving force, all other horizontal loads as driving forces, and moments equal to all vertical loads (excluding LL) times initial eccentricity (3.5 feet) as resisting forces.

⁴n/a: not analyzed.

The soils at the foundation elevation were primarily granular. Table 8 below summarizes the soils properties assumed for the analyses.

Table 8: Soil Properties Used in the Pier Analyses

Soil Type	Unit Weight (pcf)	c (psf)	c' (psf)	ϕ	ϕ'	Representative Boring
Foundation Soil Pier 1	120	0	0	36	36	B-287-1-10
Foundation Soil Pier 2	120	0	0	36	36	B-287-1-10 & B-012-M-59

Bearing capacity and sliding analyses were performed for the proposed piers using the four provided load cases, in accordance with ODOT and AASHTO guidelines. The dimensions for the proposed piers were provided by ms consultants inc. Results of the analyses indicated that the provided base widths of 14.0 feet and 15.0 feet were adequate for the stability of the Pier 1 and Pier 2 in all load cases. These provided base widths were adequate based on the required factored resistance to bearing and sliding.

Results of the analyses including the configurations of the wall sections used in the analyses are presented in Appendix III. The analytical results are also summarized in Tables 9 and 10.

Table 9: Factored Bearing Resistances

	Location	Wind Load on Vehicle (WL) Case	Horizontal Earth Load (EH) Case	Live Load (LL) Case I	Live Load (LL) Case II	Live Load (LL) Case III	No Live Load (LL) ³
Factored Bearing Resistance (qr), ksf (Strength I-b)	Pier 1	Case I	EH _{resisting} ¹	14.3	14.3	14.3	n/a
			EH _{driving} ²	15.1	15.0	15.1	15.2
	Pier 2	Case I	Case I	10.7	10.7	10.7	n/a
		Case II	Case I	10.7	10.7	10.7	n/a
Factored Uniform Bearing Pressure, ksf (Strength I-b)	Pier 1	Case I	EH _{resisting} ¹	13.2	13.7	13.5	n/a
			EH _{driving} ²	11.7	12.2	11.9	10.9
	Pier 2	Case I	Case I	8.3	8.0	8.4	n/a
		Case II	Case I	8.3	8.0	8.4	n/a
Factored Uniform Bearing Pressure, ksf (Service I)	Pier 1	Case I	EH _{resisting} ¹	10.7	11.0	10.9	n/a
			EH _{driving} ²	8.5	8.8	8.7	8.1
	Pier 2	Case I	Case I	6.9	6.7	6.9	n/a
		Case II	Case I	6.9	6.7	7.0	n/a

¹EH_{resisting}: Horizontal load acts as a resisting force, all other horizontal loads as driving forces, and moments equal to all vertical loads times initial eccentricity (3.5 feet) as driving forces.

²EH_{driving}: Horizontal load acts as a driving force, all other horizontal loads as driving forces, and moments equal to all vertical loads times initial eccentricity (3.5 feet) as resisting forces.

³EH_{driving}: Horizontal load acts as a driving force, all other horizontal loads as driving forces, and moments equal to all vertical loads (excluding LL) times initial eccentricity (3.5 feet) as resisting forces. Only the results of the worst case (LL3) are presented.

⁴n/a: not analyzed.

Nominal sliding resistance values and horizontal forces due to the bridge loading for the strength limit state are summarized in Table 10.

Table 10: Factored Sliding Resistances

	Location	Wind Load on Vehicle (WL) Case	Horizontal Earth Load (EH) Case	Live Load (LL) Case I	Live Load (LL) Case II	Live Load (LL) Case III	No Live Load (LL) ³
Resistance	Pier 1	Case I	$EH_{resisting}^1$	$\tan \delta = 0.73$	$\tan \delta = 0.73$	$\tan \delta = 0.73$	$\tan \delta = 0.73$
			$EH_{driving}^2$	$\tan \delta = 0.73$	$\tan \delta = 0.73$	$\tan \delta = 0.73$	$\tan \delta = 0.73$
	Pier 2	Case I	Case I	$\tan \delta = 0.73$	$\tan \delta = 0.73$	$\tan \delta = 0.73$	$\tan \delta = 0.73$
		Case II	Case I	$\tan \delta = 0.73$	$\tan \delta = 0.73$	$\tan \delta = 0.73$	$\tan \delta = 0.73$
ϕ_τ	Pier 1	Case I	$EH_{resisting}^1$	0.80	0.80	0.80	0.80
			$EH_{driving}^2$	0.80	0.80	0.80	0.80
	Pier 2	Case I	Case I	0.80	0.80	0.80	0.80
		Case II	Case I	0.80	0.80	0.80	0.80
R_R , (kips)	Pier 1	Case I	$EH_{resisting}^1$	4,115	4,342	4,214	n/a
			$EH_{driving}^2$	4,115	4,342	4,214	3,749
	Pier 2	Case I	Case I	3,735	3,602	3,828	n/a
		Case II	Case I	3,735	3,602	3,828	n/a
Horizontal Force, (kips)	Pier 1	Case I	$EH_{resisting}^1$	741	741	741	n/a
			$EH_{driving}^2$	1,368	1,368	1,368	1,368
	Pier 2	Case I	Case I	785	785	785	n/a
		Case II	Case I	785	785	785	n/a

¹ $EH_{resisting}$: Horizontal load acts as a resisting force, all other horizontal loads as driving forces, and moments equal to all vertical loads times initial eccentricity (3.5 feet) as driving forces.

² $EH_{driving}$: Horizontal load acts as a driving force, all other horizontal loads as driving forces, and moments equal to all vertical loads times initial eccentricity (3.5 feet) as resisting forces.

³ $EH_{driving}$: Horizontal load acts as a driving force, all other horizontal loads as driving forces, and moments equal to all vertical loads (excluding LL) times initial eccentricity (3.5 feet) as resisting forces. The results reported are for all live load cases.

⁴n/a: not analyzed.

5.6 Global Stability Considerations

Global stability analyses were performed for the cantilever-type wall at the forward and rear abutments. The global stability analyses were performed for the drained and seismic conditions. An analysis of undrained condition was not necessary because of the granular nature of the soil. The drained analyses resulted in critical factors of safety of 1.7 and 1.5 for the rear and forward abutments, respectively. These factors of safety meet or exceed the minimum factor of safety of 1.5 based on a resistance factor of 0.65. A seismic analysis was also performed using a horizontal acceleration of 0.06. This analysis resulted in critical factors of safety of 1.6 and 1.4 for the rear and forward abutments, respectively, which is greater than the minimum acceptable factor of safety of 1.1 for the seismic condition. The analytical results are summarized in Table 11.

Table 11: Global Stability Analyses

<p><u>Global Stability</u> as per AASHTO 11.6.2.3 Evaluating the global stability using the Spencer method</p>
<p><u>Rear Abutment:</u> Undrained Condition, F.S. = N/A (granular soils) Drained Condition, F.S. = 1.7 Seismic Condition, F.S. = 1.6 > required F.S. of 1.1</p>
<p><u>Forward Abutment:</u> Undrained Condition, F.S. = N/A (granular soils) Drained Condition, F.S. = 1.5 Seismic Condition, F.S. = 1.4 > required F.S. of 1.1</p>
<p>F.S. > $1/\Phi_R = 1/0.65 = 1.5$ for drained and undrained conditions</p>

5.7 Settlement Considerations

In order to achieve the proposed final grade immediately in front of the abutment walls, it is anticipated that up to approximately 10.0 feet of excavation will be required. To achieve the final grade immediately behind the rear abutment wall, approximately 16.6 feet of new fill will need to be placed at the abutment. However, the proposed grade behind the forward abutment wall is anticipated to be approximately equal to the existing grade. To achieve the proposed grade at the proposed pier locations, excavations between approximately 2.0 and 11.0 feet are anticipated.

Settlement analyses were performed at the abutments and the piers, evaluating the effects of the bridge loading and the earthwork. The Live Load Case II with the horizontal earth load as a resisting force and the Live Load Case III with the Wind Load (on vehicle) Case II were determined to be the most critical in terms of settlement for proposed Pier 1 and Pier 2, respectively. Results of settlements due to the structure loading were 0.5 inches at the rear abutment, 0.6 inches at Pier 1, 0.6 inches at Pier 2, and 0.6 inches at the forward abutment. Results of the analyses are summarized in Table 12.

Table 12: Estimated Settlement

<u>Estimated Settlement of the Rear Abutment</u> Settlement due to structure loading = 0.5 in. Settlement due to new fill (16.6 feet) = 0.4 in. Total Settlement = 0.9 in.
<u>Estimated Settlement of Pier 1 – Live Load Case II with EH as a resisting force</u> Settlement due to structure loading = 0.6 in. Total Settlement = 0.6 in.
<u>Estimated Settlement of Pier 2 – Live Load Case III with Wind Load on Vehicle Case II</u> Settlement due to structure loading = 0.6 in. Total Settlement = 0.6 in.
<u>Estimated Settlement of the Forward Abutment</u> Settlement due to structure loading = 0.6 in. Total Settlement = 0.6 in.

5.8 Groundwater Considerations

Groundwater seepage was encountered in at approximate elevations 746.8 and 770.6 in borings B-287-1-10 and B-287-2-10, respectively. These elevations are between approximately 0.8 and 31.6 feet above the proposed bottom of footing elevations. Consequently, a significant amount of groundwater seepage is anticipated in the excavations and dewatering will likely be required. The contractor should also be prepared to deal with any unexpected seepage and with surface water that may accumulate in the excavations. It may be desirable to contact a dewatering specialist for the design of the dewatering system prior to construction.

Due to the free-flowing nature of the granular foundation soils, excess water may continue to accumulate along the roadways underneath the proposed bridge after the completion of construction. Oversized underdrains, oversized drain pipes, and a permanent pumping system may be required. It is recommended that a dewatering specialist be consulted for long-term seepage control.

6.0 CLOSING REMARKS

We appreciate having the opportunity to be of service to you on this project. Please do not hesitate to call if you have any questions concerning this report.

Respectfully submitted,

DLZ OHIO, INC.



Eric W. Tse, P.E.
Senior Geotechnical Engineer



Timothy A. Hampshire, P.E.
Geotechnical Engineering Division Manager

cc: file

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APPENDIX I

General Information - Drilling Procedures and Logs of Borings

Legend - Boring Log Terminology

Site Plan/Boring Plan

Boring Logs – Six (6) Borings

GENERAL INFORMATION DRILLING PROCEDURES AND LOGS OF BORINGS

Drilling and sampling were conducted in accordance with procedures generally recognized and accepted as standardized methods of investigation of subsurface conditions concerning geotechnical engineering considerations. Borings were drilled with either a truck-mounted or ATV-mounted drill rig.

Drive split-barrel sampling was performed in 1.5 foot increments at intervals not exceeding 5 feet. In the event the sampler encountered resistance to penetration of 6 inches or less after 50 blows of the drop hammer, the sampling increment was discontinued. Standard penetration data were recorded and one or more representative samples were preserved from each sampling increment.

In borings where rock was cored, NXM or NQ size diamond coring tools were used.

In the laboratory all samples were visually classified by a geotechnical engineer. Moisture contents of representative fine-grained soil samples were determined. A limited number of samples, considered representative of foundation materials present, were selected for performance of grain-size analyses and plasticity characteristics tests. The results of these tests are shown on the boring logs.

The boring logs included in the Appendix have been prepared on the basis of the field record of drilling and sampling, and the results of the laboratory examination and testing of samples. Stratification lines on the boring logs indicating changes in soil stratigraphy represent depths of changes approximated by the driller, by sampling effort and recovery, and by laboratory test results. Actual depths to changes may differ somewhat from the estimated depths, or transitions may occur gradually and not be sharply defined. The boring logs presented in this report therefore contain both factual and interpretative information and are not an exact copy of the field log.

Although it is considered that the borings have disclosed information generally representative of site conditions, it should be expected that between borings conditions may occur which are not precisely represented by any one of the borings. Soil deposition processes and natural geologic forces are such that soil and rock types and conditions may change in short vertical intervals and horizontal distances.

Soil/rock samples will be stored at our laboratory for a period of six months. After this period of time, they will be discarded, unless notified to the contrary by the client.

LEGEND - BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right

1. Depth (in feet) - refers to distance below the ground surface.
2. Elevation (in feet) - is referenced to mean sea level, unless otherwise noted.
3. Standard Penetration (N) - the number of blows required to drive a 2-inch O.D., 1-3/8 inch I.D., split-barrel sampler, using a 140-pound hammer with a 30-inch free fall. The blows are recorded in 6-inch drive increments. Standard penetration resistance is determined from the total number of blows required for one foot of penetration by summing the second and third 6-inch increments of an 18-inch drive.

50/n - indicates number of blows (50) to drive a split-barrel sampler a certain number of inches (n) other than the normal 6-inch increment.

WOR – indicates the split-barrel sampler advanced the 6-inch increment from the weight of the rods alone.

WOH – indicates the split-barrel sampler advanced the 6-inch increment from the combined weight of the hammer and rods alone.
4. The length of the sampler drive is indicated graphically by horizontal lines across the "Standard Penetration" and "Recovery" columns.
5. Sample recovery from each drive is indicated numerically in the column headed "Recovery".
6. The drive sample location is designated by the heavy vertical bar in the "Sample No., Drive" column.
7. The length of hydraulically pressed "Undisturbed" samples is indicated graphically by horizontal lines across the "Press" column.
8. Sample numbers are designated consecutively, increasing in depth.
9. Soil Description

- a. The following terms are used to describe the relative compactness and consistency of soils:

Granular Soils - Compactness

<u>Term</u>	<u>Blows/Foot Standard Penetration</u>
Very Loose	0 – 4
Loose	4 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	over 50

Cohesive Soils – Consistency

<u>Term</u>	<u>Unconfined Compression tons/sq. ft</u>	<u>Blows/Foot Standard Penetration</u>	<u>Hand Manipulation</u>
Very Soft	less than 0.25	below 2	Easily penetrated by fist
Soft	0.25 – 0.50	2 – 4	Easily penetrated by thumb
Medium Stiff	0.50 – 1.0	4 – 8	Penetrated by thumb with moderate pressure
Stiff	1.0 – 2.0	8 – 15	Readily indented by thumb but not penetrated
Very Stiff	2.0 – 4.0	15 – 30	Readily indented by thumb nail
Hard	over 4.0	over 30	Indented with difficulty by thumb nail

- b. Color - If a soil is a uniform color throughout, the term is single, modified by such adjective as light and dark. If the predominant color is shaded by a secondary color, the secondary color precedes the primary color. If two major and distinct colors are swirled throughout the soil, the colors are modified by the term "mottled".
- c. Texture is based on the Unified Classification System. Soil particle size definitions are as follows:

<u>Description</u>	<u>Size</u>	<u>Description</u>	<u>Size</u>
Boulders	Larger than 8"	Sand-Coarse	4.75 mm to 2.00 mm
Cobbles	8" to 3"	-Medium	2.00 mm to 0.42 mm
Gravel-Coarse	3" to 3/4"	-Fine	0.42 mm to 0.074 mm
-Fine	3/4" to 4.76 mm	Silt	0.074 mm to 0.005 mm
		Clay	Smaller than 0.005 mm

- d. The primary soil component is listed first and may include a modifier before and/or after it as indicated by the USCS classification system. The minor components are listed in order of decreasing percentage of particle size.
- | | |
|--|--|
| <u>Coarse Grained Soils</u> | <u>Fine Grained Soils</u> |
| 5% - 12% silt/clay - "with silt/clay" post-modifier | 5% - 12% sand/gravel- "with sand/gravel" post-modifier |
| > 15% sand/gravel – "with sand/gravel" post-modifier | > 30% sand/gravel – "sandy/gravelly" pre-modifier |
| > 12% silt/clay – "silty/clayey" pre-modifier | |

e. Minor modifiers to main soil descriptions are indicated as a percentage by weight of particle sizes.

trace - 0 to 10%
 little - 10 to 20%

f. The moisture content of **cohesive soils** (silts and clays) is expressed relative to plastic properties.

<u>Term</u>	<u>Relative Moisture or Appearance</u>
Dry	Powdery
Damp	Moisture content slightly below plastic limit
Moist	Moisture content above plastic limit, but below liquid limit
Wet	Moisture content above liquid limit

g. Moisture content of **cohesionless soils** (sands and gravels) is described as follows:

<u>Term</u>	<u>Relative Moisture or Appearance</u>
Dry	No moisture present
Damp	Internal moisture, but none to little surface moisture
Moist	Free water on surface
Wet	Voids filled with free water

10. Rock hardness and rock quality description.

a. The following terms are used to describe the relative hardness of the **bedrock**.

<u>Term</u>	<u>Description</u>
Very Soft	Difficult to indent with thumb nails; resembles hard soil but has rock structure
Soft	Resists indentation with thumb nail but can be abraded and pierced to a shallow depth by a pencil point.
Medium Hard	Resists pencil point, but can be scratched with a knife blade.
Hard	Can be deformed or broken by light to moderate hammer blows.
Very Hard	Can be broken only by heavy blows, and in some rocks, by repeated hammer blows.

b. Rock Quality Designation, RQD - This value is expressed in percent and is an indirect measure of rock soundness. It is obtained by summing the total length of all core pieces which are at least four inches long, and then dividing this sum by the total length of the core run.

11. Gradation - when tests are performed, the percentage of each particle size is listed in the appropriate column (defined in Item 9c).

12. When a test is performed to determine the natural moisture content, liquid limit moisture content, or plastic limit moisture content, the moisture content is indicated graphically.

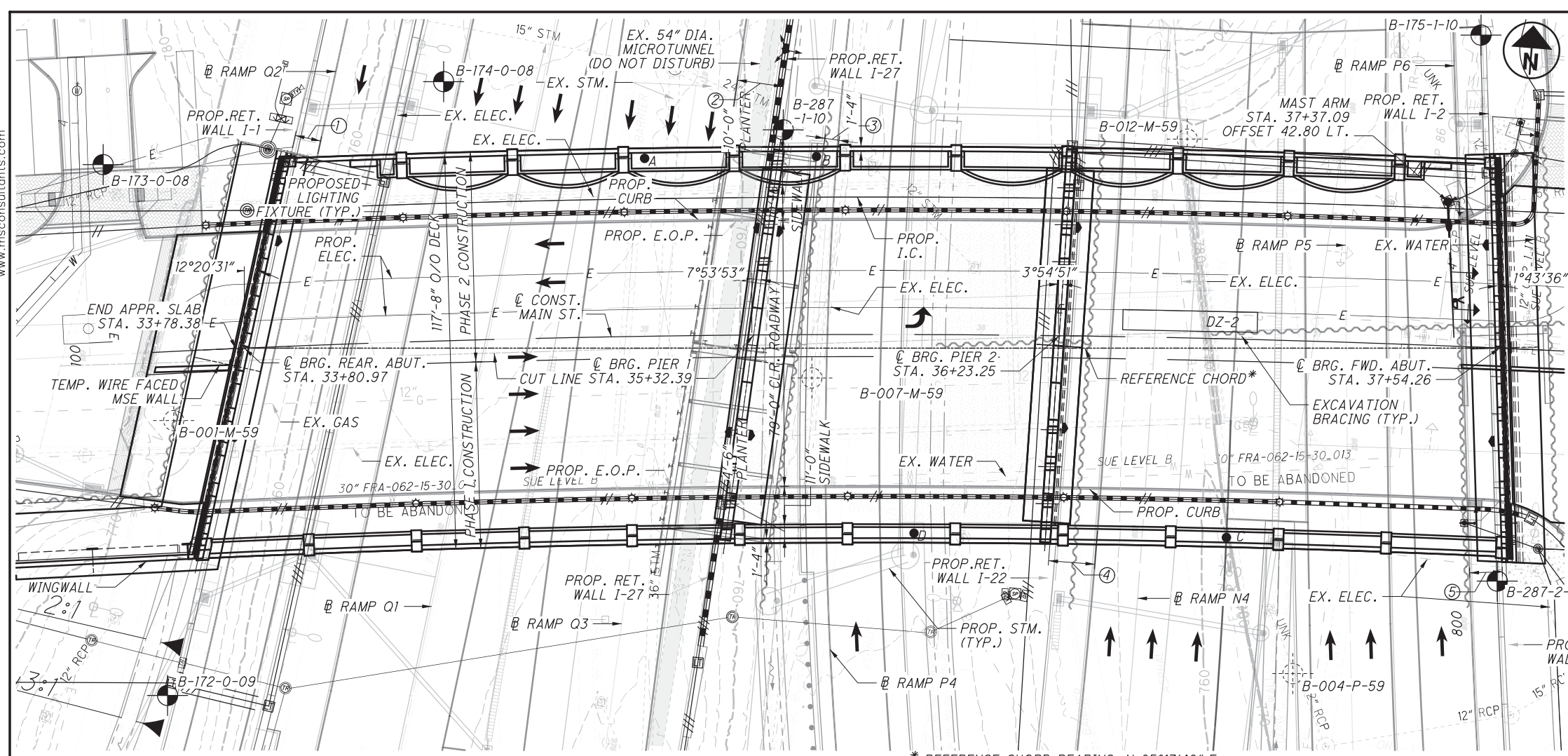
13. The corrected standard penetration (N60) value in blows per foot is indicated graphically.

14. Soil Symbology

	GW	Well-graded Gravel
	GP	Poorly-graded Gravel
	GW-GM	Well-graded Gravel with Silt
	GP-GM	Poorly-graded Gravel with Silt
	GM	Silty Gravel
	SW	Well-graded Sand
	SP	Poorly-graded Sand
	SW-SM	Well-graded Sand with Silt

	SP-SM	Poorly-graded Sand with Silt
	SM	Silty Sand
	SC-SM	Clayey, Silty Sand
	SC	Clayey Sand
	ML	Silt
	CL-ML	Low Plasticity Silty Clay
	CL	Low Plasticity Clay

PLOT.CEL
ms consultants, inc.
msconsultants.com
Ohio DOT Workspace
UCF: ohio181
PCF: 60-06634-20-Columbus_(No_Asc20)71 East Interchange 20
www.msconsultants.com
Batchplot Spec: \\0330share\60\06634-01\standards\plotdrv\batchplot.spc
Pen Table: \\0330share\60\06634-01\standards\lables\77370_Structures_v8_20.tbl
Plot Driver: S:\std\plotting\usin\81\PDF.plt
Model: Sheet
Printed: 11/11/2013 1:08:51 PM By: osenagala
File: \\0330share\60\06634-01\structures\FRA07L-1760\steels\07L-1760SP001.dgn

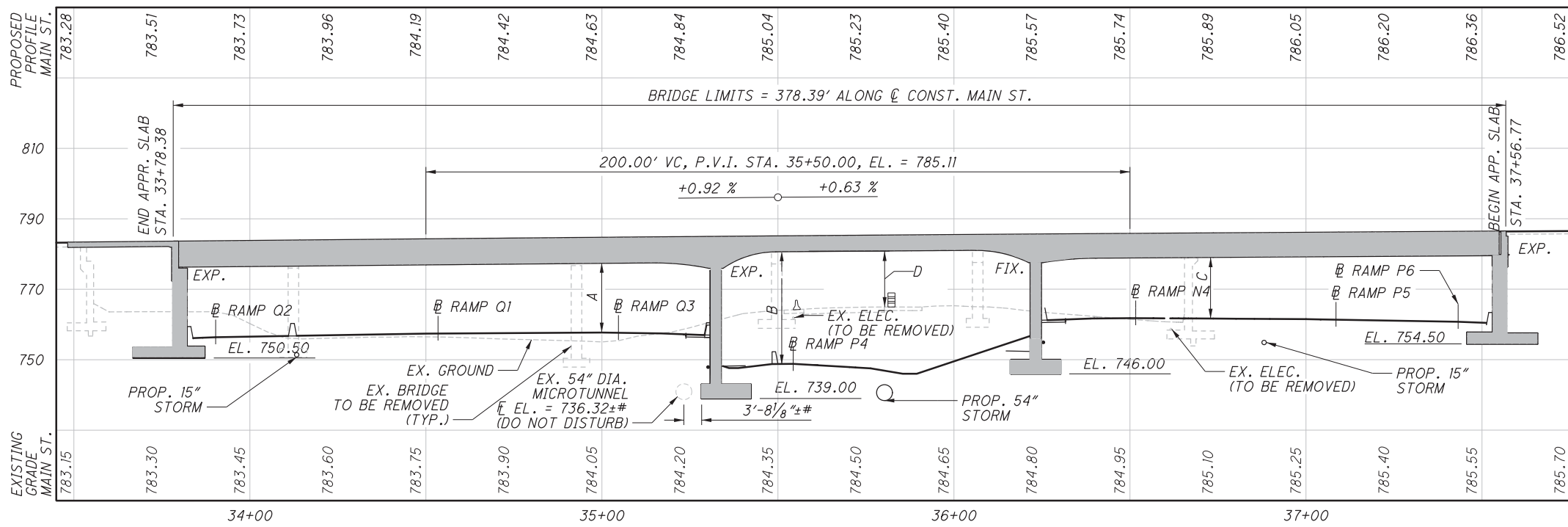


PLAN * REFERENCE CHORD BEARING: N 85°17'48" E
** PROTECTED BY BARRIER

HORIZONTAL CLEARANCES

① PROVIDED = 7'-9"; REQUIRED = 13'-8" **	④ PROVIDED = 13'-9 1/8"; REQUIRED = 13'-8" **
② PROVIDED = 13'-9"; REQUIRED = 13'-8" **	⑤ PROVIDED = 9'-9"; REQUIRED = 13'-8" **
③ PROVIDED = 4'-3 3/8"; REQUIRED = 13'-8" **	

FOR LIGHT POLE LOCATIONS, SEE SHEET 3/114.



AT NORTH END OF FOOTING
BRIDGE PROFILE ALONG C MAIN ST.

BENCHMARK DATA

BM #071-211	N 713282.096	E 1833557.561	ELEV. 796.58
BM #071-212	N 713490.694	E 1833404.934	ELEV. 785.55
BM #071-213	N 713540.567	E 1832977.603	ELEV. 783.24

FOR ADDITIONAL BENCHMARK INFORMATION. SEE ROADWAY PLAN SHEETS 22/4017 AND 23/4017

NOTES

ALL EXISTING UTILITIES TO BE REMOVED/RELOCATED, U.N.O.. EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS. FOR INTERSECTION DETAILS FOR CURB RAMP, SEE SHEET 1748/4017.

DESIGN TRAFFIC:

2010 ADT = 21,500	2010 ADTT = 430
2035 ADT = 25,600	2035 ADTT = 512
DIRECTIONAL DISTRIBUTION = 0.73	

LEGEND

- PROJECT BORING LOCATION
- HISTORIC BORING LOCATION
- 15'-6" REQUIRED MINIMUM VERTICAL CLEARANCE
- PT. A = 18'-2 7/8"
- PT. B = 28'-8 1/16"
- PT. C = 16'-1"
- PT. D = 14'-7 1/2" (MILLED TO 15'-0" MIN. TEMP. MOT CONDITION)

HORIZONTAL CURVE DATA

TANGENT 57	CURVE 16	CURVE 16 (CONT'D)
STA. = 25+00.00	P.I. STA. 36+97.25	E = 14.00'
STA. = 33+50.14	Δ = 9° 14' 11" (RT)	PC = 33+50.14
L = 850.14'	Dc = 1° 20' 00"	PT = 40+42.86
BRG. = N 82°23'49" E	R = 4,297.18	V (des) = 35 MPH
	T = 347.12'	e (max) = NC

EXISTING STRUCTURE

TYPE: CONTINUOUS STEEL BEAM COMPOSITE WITH REINFORCED CONCRETE DECK AND SUBSTRUCTURE
 SPANS: 57'-6", 80'-6", 57'-0", 57'-0" AND 56'-6" C/C BRGS. ALONG C CONST.
 ROADWAY: 68'-0" F/F WITH 5'-2" SIDEWALK
 LOADING: HS20-44 CASE II & ALT. MILITARY LOADING
 SKEW: 15°47'15" LF (R.A.), 13°39'28" LF (PIER 1), 6°03'00" LF (PIERS 2 & 3), NONE (PIER 4), 5°09'00" RF (F.A.)
 WEARING SURFACE: 1" MONOLITHIC CONCRETE
 APPROACH SLABS: 25'-0" LONG
 ALIGNMENT: TANGENT
 CROWN: 3/16" / FT.
 STRUCTURAL FILE NUMBER: 2501562
 DATE BUILT: 1963 (1997 REHAB)
 DISPOSITION: TO BE REMOVED

PROPOSED STRUCTURE

TYPE: CONTINUOUS COMPOSITE CURVED STEEL GIRDERS (A709 GRADE 50) WITH REINFORCED CONCRETE DECK, ABUTMENTS AND PIERS ON SPREAD FOOTINGS.
 SPANS: 150'-10", 91'-1 5/8", 131'-2 1/2" C/C BRGS. ALONG REFERENCE CHORD
 ROADWAY: 79'-0" TOE/TOE CURB WITH 10'-6" NORTH SIDEWALK & 10'-6" SOUTH SIDEWALK
 LOADING: HL-93 AND FUTURE WEARING SURFACE 0.06 KSF
 SKEW: 12°20'31" LF (R.A.), 7°53'53" LF (PIER 1), 3°54'51" LF (PIER 2), 1°43'36" RF (F.A.) TO REFERENCE CHORD
 WEARING SURFACE: 1" MONOLITHIC CONCRETE
 APPROACH SLABS: 25'-0" LONG (AS-1-81) @ R.A. 30'-0" LONG (AS-1-81 MODIFIED) @ F.A.
 ALIGNMENT: 1°20'00" RIGHT CURVE
 CROWN: 0.016 FT/FT
 COORDINATES: LATITUDE N 39°57'29" LONGITUDE W 82°58'58"

DESIGN AGENCY: ms consultants, inc.
2221 Schrock Road, Columbus, Ohio 43229
DATE: 2013 NOV
REVIEWED: GLG
DRAWN: FBW
DESIGNED: FBW
FRANKLIN COUNTY
STA. 33+78.38
STA. 37+56.77
SITE PLAN
BRIDGE NO. FRA-71-1716
RAMPS Q2, Q1, Q3, P4, N5, P5 & P6 UNDER MAIN STREET
FRA-70-14.48
PID No. 77370
1/114
3854
4017
ms consultants, inc.

PROJECT: <u>I-70/I-71 EAST INTERCHANGE</u>	DRILLING FIRM / OPERATOR: <u>DLZ / K. CONRAD</u>	DRILL RIG: <u>CME 75 TRUCK</u>	STATION / OFFSET: _____	EXPLORATION ID B-175-1-10
TYPE: <u>ROADWAY</u>	SAMPLING FIRM / LOGGER: <u>DLZ / M. EVENER</u>	HAMMER: <u>CME AUTOMATIC</u>	ALIGNMENT: _____	
PID: <u>77370</u> BR ID: _____	DRILLING METHOD: <u>3.25" HSA</u>	CALIBRATION DATE: <u>1/7/10</u>	ELEVATION: <u>786.2 (MSL)</u> EOB: <u>80.0 ft.</u>	PAGE 1 OF 4
START: <u>2/3/11</u> END: <u>2/4/11</u>	SAMPLING METHOD: <u>SPT</u>	ENERGY RATIO (%): <u>79</u>	COORD: <u>713630.480 N, 1833367.660 E</u>	

MATERIAL DESCRIPTION AND NOTES	ELEV. 786.2	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG				ODOT CLASS (GI)	BACK FILL	
								GR	CS	FS	SI	CL	LL	PL	PI	WC			
Asphalt - 8" Concrete - 7"	784.9	1																	
FILL: Very stiff to hard gray SILT AND CLAY (A-6a), little fine to coarse sand, little gravel; contains small tile fragments; damp.	780.2	2	4	8	56	SS-1	2.00	-	-	-	-	-	-	-	-	-	19	A-6a (V)	
		3	3																
		4	6	17	100	SS-2	4.5+	-	-	-	-	-	-	-	-	-	14	A-6a (V)	
		5	7																
Hard gray SANDY SILT (A-4a), some fine to coarse sand, trace gravel; damp.	777.7	6	6	25	100	SS-3	4.5+	3	8	18	42	29	26	17	9	15	A-4a (7)		
		7	8	11															
Dense to very dense brown GRAVEL WITH SAND (A-1-b), some fine to coarse sand, some silt; contains rust stains and silty clay pieces; damp.	772.7	8																	
		9	13	45	67	SS-4	-	-	-	-	-	-	-	-	-	8	A-1-b (V)		
		10	18	16															
		11	12	16	51	100	SS-5	-	-	-	-	-	-	-	-	9	A-1-b (V)		
Hard brown SANDY SILT (A-4a), some fine to coarse sand, trace to little gravel; damp.	765.2	12	23																
		13																	
		14	28	50/5"	-	36	SS-6	-	-	-	-	-	-	-	-	-	11	A-4a (V)	
		15																	
NOTE: Hole caved to 21.8 feet when augers removed.	761.2	16	12	42	100	SS-7	4.5+	10	10	21	35	24	24	14	10	9	A-4a (5)		
		17	15	17															
		18																	
		19	11	19	51	100	SS-8	4.5+	-	-	-	-	-	-	-	8	A-4a (V)		
Very dense gray GRAVEL WITH SAND (A-1-b), little silt; damp.	761.2	20																	
		21	10	33	84	100	SS-9	-	-	-	-	-	-	-	4	A-1-b (V)			
		22	31																
		23																	
		24	14	33	88	100	SS-10	-	30	34	24	- 12 -	NP	NP	NP	4	A-1-b (0)		

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - I-70/I-71 East Interchange - 10/27/11 10:42

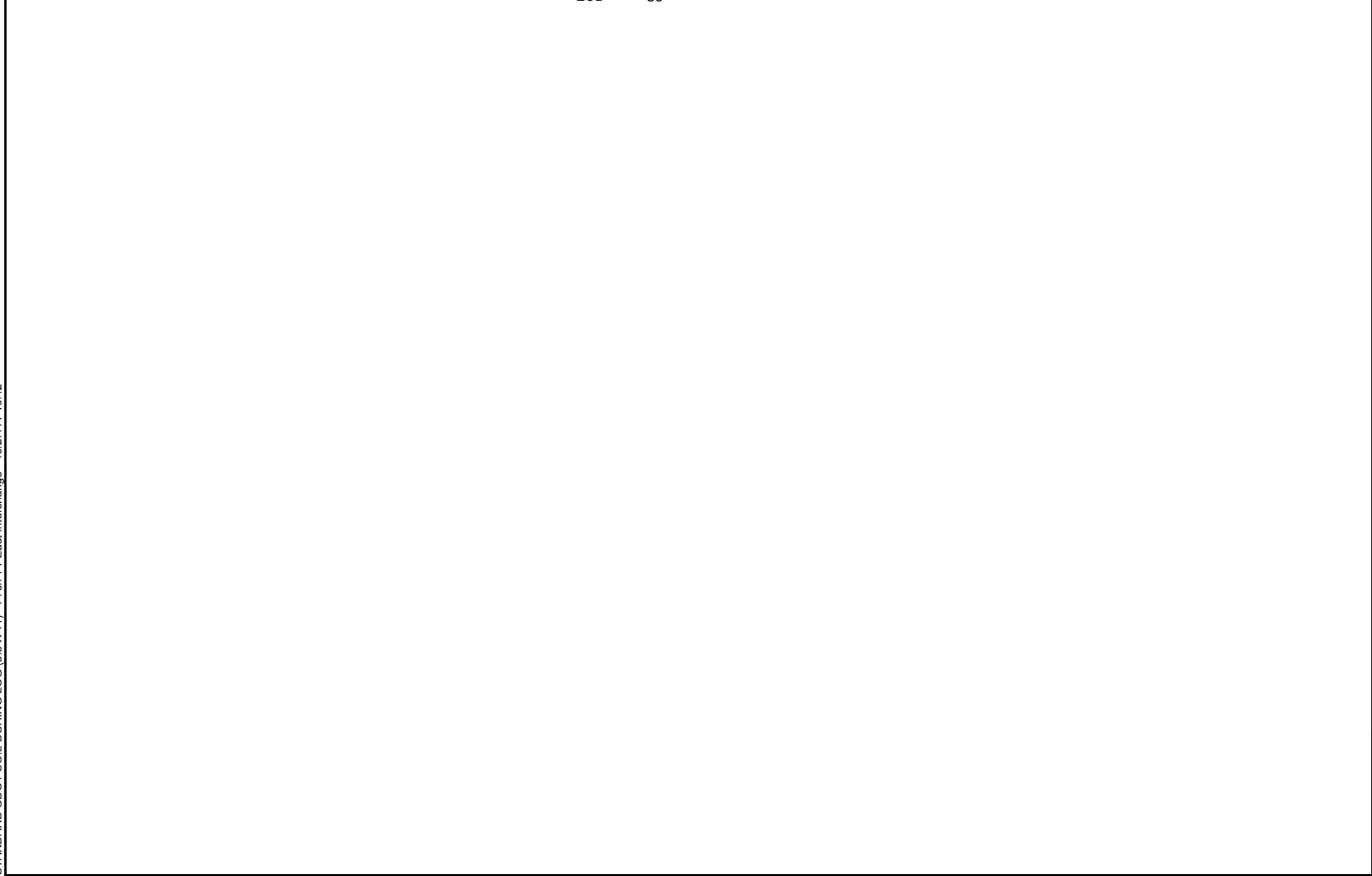
MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
Very dense gray GRAVEL WITH SAND (A-1-b), trace to little silt; wet.	734.4	52																
	734.2	53																
		54	34 29 32	80	100	SS-21	-	27	40	22	- 11 -	NP	NP	NP	13	A-1-b (0)		
		55																
		56																
		57																
		58																
		59	21 19 22	54	67	SS-22	-	-	-	-	- - -	-	-	-	9	A-1-b (V)		
		60																
		61																
Very stiff gray SILT AND CLAY (A-6a), some fine to coarse sand, trace gravel; damp.	714.2	62																
		63																
		64	19 30 29	78	100	SS-23	-	40	37	15	- 8 -	NP	NP	NP	12	A-1-b (0)		
		65																
		66																
		67																
		68																
		69	22 18 22	53	100	SS-24	-	-	-	-	- - -	-	-	-	13	A-1-b (V)		
		70																
		71																
Hard gray SANDY SILT (A-4a), some fine to coarse sand, little gravel; damp.	709.2	72																
		73																
	74	14 19 19	50	100	SS-25	3.00	4	9	22	37	28	25	14	11	13	A-6a (6)		
	75																	
	76																	
	77																	
	78																	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - I-70/I-71 East Interchange - 10/27/11 10:42

PID: 77370 BR ID: PROJECT: I-70/I-71 EAST INTERCHANGE STATION / OFFSET: START: 2/3/11 END: 2/4/11 PG 4 OF 4 B-175-1-10

MATERIAL DESCRIPTION AND NOTES	ELEV. 707.6	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
Hard gray SANDY SILT (A-4a), some fine to coarse sand, little gravel; damp. (continued)	706.2	79 80	40 37 32	91	100	SS-26	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - I-70/I-71 East Interchange - 10/27/11 10:42



NOTES: SEEPAGE AT 38.5 FEET; WATER LEVEL PRIOR TO ADDING WATER = 40.8 FEET; FINAL WATER LEVEL INCLUDING DRILLING WATER = 39.6 FEET.
 ABANDONMENT METHODS, MATERIALS, QUANTITIES: 11 BAGS BENTONITE CHIPS; SOIL CUTTINGS

PROJECT: I-70/I-71 EAST INTERCHANGE TYPE: ROADWAY PID: 77370 BR ID: START: 6/2/10 END: 6/2/10	DRILLING FIRM / OPERATOR: DLZ / K. CONRAD SAMPLING FIRM / LOGGER: DLZ / M. EVENER DRILLING METHOD: 3.25" HSA SAMPLING METHOD: SPT	DRILL RIG: CME 75 TRUCK HAMMER: CME AUTOMATIC CALIBRATION DATE: 1/7/10 ENERGY RATIO (%): 79	STATION / OFFSET: 879+96.81, 16.5 RT ALIGNMENT: RAMP P4 ELEVATION: 762.8 (MSL) EOB: 70.0 ft. COORD: 713578.700 N, 1833164.080 E	EXPLORATION ID B-287-1-10 PAGE 1 OF 3
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MATERIAL DESCRIPTION AND NOTES	ELEV. 762.8	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			ODOT CLASS (GI)	HOLE SEALED	
								GR	CS	FS	SI	CL	LL	PL	PI			WC
Topsoil - 8"	762.1																	
POSSIBLE FILL: Hard brown SILTY CLAY (A-6b), some fine to coarse sand, little gravel; contains trace roots and rock fragments; moist. (CL)		1	4															
		2	5	4	12	56	SS-1	4.5+	-	-	-	-	-	-	-	-	10	A-6b (V)
	759.8	3																
Dense to very dense brown GRAVEL WITH SAND (A-1-b), trace to little silt; damp. @ 3.5'-5.0', (SP-SM). @ 6.0'-10.0', (SP).		4	14	23	21	58	56	SS-2	-	-	-	-	-	-	-	-	5	A-1-b (V)
		5																
		6	11	16	14	40	67	SS-3	-	-	-	-	-	-	-	-	4	A-1-b (V)
		7																
		8																
		9	6	18	22	53	72	SS-4	-	-	-	-	-	-	-	-	4	A-1-b (V)
		10																
@ 11.0'-20.0', (SP-SM).		11	14	15	15	40	100	SS-5	-	18	51	20	- 11 -	NP	NP	NP	4	A-1-b (0)
		12																
		13																
		14	13	20	22	55	100	SS-6	-	-	-	-	-	-	-	-	7	A-1-b (V)
		15																
@ 16.0', becomes wet.		16	17	11	15	34	83	SS-7	-	-	-	-	-	-	-	-	14	A-1-b (V)
		17																
		18																
		19	18	17	17	45	100	SS-8	-	-	-	-	-	-	-	-	23	A-1-b (V)
		20																
@ 21.0'-22.5', (SM).		21	8	19	39	76	56	SS-9	-	21	30	34	- 15 -	NP	NP	NP	14	A-1-b (0)
		22																
		23																
@ 23.5'-25.0', 1.5 feet sand heave; washed out with tricone. (SP-SM)		24	19	26	22	63	56	SS-10	-	-	-	-	-	-	-	-	11	A-1-b (V)

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - I-70/I-71 East Interchange - 3/14/13 11:13

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED	
								GR	CS	FS	SI	CL	LL	PL	PI				
Very dense brown GRAVEL WITH SAND AND SILT (A-2-4), some to "and" fine to coarse sand, little silt, little clay; wet. (SM)	737.8	26	23	61	61	SS-11	-	-	-	-	-	-	-	-	-	-	11	A-2-4 (V)	
	734.8	27	25																21
Very dense brown COARSE AND FINE SAND (A-3a), little gravel, trace to little silt; wet. (SP-SM)	732.3	28	15	57	50	SS-12	-	-	-	-	-	-	-	-	-	-	-	15	A-3a (V)
		29	18																
Dense to very dense brown GRAVEL WITH SAND (A-1-b), trace to little silt; wet. @ 31.0'-35.0', SM.	717.3	30	25	65	78	SS-13	-	49	23	15	-	13	-	NP	NP	NP	10	A-1-b (0)	
		31	26																23
@ 36.0'-37.5', 41.0'-45.0', SP-SM.	714.8	32	23	53	83	SS-14	-	-	-	-	-	-	-	-	-	-	-	10	A-1-b (V)
		33	22																
@ 38.5'-40.0', SW-SM.	712.3	34	24	47	100	SS-15	-	-	-	-	-	-	-	-	-	-	-	9	A-1-b (V)
		35	18																
@ 41.0'-42.5', gray.	717.3	36	18	66	100	SS-16	-	32	36	23	-	9	-	NP	NP	NP	13	A-1-b (0)	
		37	26																24
Very dense gray COARSE AND FINE SAND (A-3a), little gravel, trace silt; ; wet. (SP)	714.8	38	25	117	100	SS-17	-	-	-	-	-	-	-	-	-	-	-	6	A-1-b (V)
		39	46																
Very dense gray SANDY SILT (A-4a), "and" fine sand, trace to little coarse sand, trace gravel; wet. (SM)	712.3	40	19	63	56	SS-18	-	38	38	15	-	9	-	NP	NP	NP	12	A-1-b (0)	
		41	26																22
Very dense gray SILT (A-4b), some fine sand, trace clay, trace gravel; damp. (CL-ML)	712.3	42	10	54	56	SS-19	-	-	-	-	-	-	-	-	-	-	-	8	A-3a (V)
		43	15																
Very dense gray SILT (A-4b), some fine sand, trace clay, trace gravel; damp. (CL-ML)	712.3	44	19	84	44	SS-20	-	-	-	-	-	-	-	-	-	-	-	11	A-4a (V)
		45	31																
Very dense gray SILT (A-4b), some fine sand, trace clay, trace gravel; damp. (CL-ML)	712.3	46	38	-	100	SS-21	-	-	-	-	-	-	-	-	-	-	-	11	A-4b (V)
		47	50/4"																

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - I-70/I-71 East Interchange - 3/14/13 11:13

MATERIAL DESCRIPTION AND NOTES	ELEV. 711.0	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI			
Hard gray SILTY CLAY (A-6b), little fine sand, trace gravel; damp. (CL-ML)	710.3	52																
		53																
		54	50/4"	-	50	SS-22	4.5+	-	-	-	-	-	-	-	8	A-6b (V)		
Very dense gray SILT (A-4b), little to some fine sand, little clay, trace gravel; wet. (ML)	707.8	55																
		56	38 45 50/5"	-	100	SS-23	-	1	0	22	64	13	NP	NP	NP	18	A-4b (8)	
		57																
		58																
		59	50/5"	-	80	SS-24	-	-	-	-	-	-	-	-	-	28	A-4b (V)	
		60																
		61																
Very stiff gray CLAY (A-7-6), trace to little fine sand, trace gravel; damp to moist. (CH)	699.8	62	38 42 32	97	100	SS-25	-	-	-	-	-	-	-	-	17	A-4b (V)		
		63																
		64	11 18 19	49	78	SS-26	3.00	0	0	1	22	77	45	21	24	23	A-7-6 (15)	
		65																
		66																
		67	11 15 18	43	100	SS-27	3.00	-	-	-	-	-	-	-	-	22	A-7-6 (V)	
		68																
	692.8	69	16 16 17	43	100	SS-28	3.00	-	-	-	-	-	-	-	27	A-7-6 (V)		
		70																

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - I-70/I-71 East Interchange - 3/14/13 11:13

EOB

NOTES: SEEPAGE AT 16.0 AND 55.5 FEET; WATER LEVEL PRIOR TO ADDING WATER = 19.7 FEET; FINAL WATER LEVEL INCLUDING DRILLING WATER = 53.7 FEET.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: 1 BAG BENTONITE CHIPS; 3 BAGS QUIKGROUT

PROJECT: I-70/I-71 EAST INTERCHANGE TYPE: ROADWAY PID: 77370 BR ID: START: 5/26/10 END: 5/27/10	DRILLING FIRM / OPERATOR: DLZ / K. CONRAD SAMPLING FIRM / LOGGER: DLZ / M. EVENER DRILLING METHOD: 3.25" HSA SAMPLING METHOD: SPT	DRILL RIG: CME 75 TRUCK HAMMER: CME AUTOMATIC CALIBRATION DATE: 1/7/10 ENERGY RATIO (%): 79	STATION / OFFSET: 800+12.28, 8.11 RT ALIGNMENT: RAMP P6 ELEVATION: 785.1 (MSL) EOB: 90.0 ft. COORD: 713462.500 N, 1833386.600 E	EXPLORATION ID B-287-2-10 PAGE 1 OF 4
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MATERIAL DESCRIPTION AND NOTES	ELEV. 785.1	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED		
								GR	CS	FS	SI	CL	LL	PL	PI					
Asphalt - 4" Concrete - 8" Base - 6"	783.6	1																		
POSSIBLE FILL: Stiff to very stiff brown SILTY CLAY (A-6b), little fine to coarse sand, trace to little gravel; contains trace brick and rock fragments; damp to moist. @3.5' - 5.0', hard.	783.6	2	3	10	29	100	SS-1	2.50	19	10	14	26	31	39	19	20	19	A-6b (8)		
		3																		
		4	6	9	8	22	100	SS-2	4.5+	-	-	-	-	-	-	-	-	16	A-6b (V)	
		5																		
		6																		
		7	1	2	2	5	100	SS-3	1.75	-	-	-	-	-	-	-	-	16	A-6b (V)	
		8																		
		9	3	5	5	13	100	SS-4	1.50	-	-	-	-	-	-	-	-	19	A-6b (V)	
		10																		
		11																		
12																				
13	771.6																			
POSSIBLE FILL: Medium stiff to stiff brown SANDY SILT (A-4a), "and" fine to coarse sand, little gravel; moist.	770.6	14	1	2	4	8	100	SS-6A	1.00	15	15	24	30	16	23	16	7	18	A-4a (2)	
Loose brown SANDY SILT (A-4a), some to "and" fine to coarse sand, trace to little gravel; wet.	769.1	15						SS-6B	-	-	-	-	-	-	-	-	-	17	A-4a (V)	
16																				
Hard gray SILTY CLAY (A-6b), little fine to coarse sand, little silt, trace gravel; damp.	764.1	17	6	11	14	33	100	SS-7	4.5+	-	-	-	-	-	-	-	-	8	A-6b (V)	
18																				
19		10	10	18	37	100	SS-8	4.5+	-	-	-	-	-	-	-	-	9	A-6b (V)		
20																				
Very loose brown SANDY SILT (A-4a), some to "and" fine to coarse sand, little clay, little gravel; wet.	761.6	21	1	1	1	3	44	SS-9	-	-	-	-	-	-	-	-	24	A-4a (V)		
22																				
23																				
Very dense brown GRAVEL WITH SAND (A-1-b), little to some silt; damp. @23.5'-25.0', medium dense.	761.6	24	6	10	11	28	67	SS-10	-	28	35	16	-	21	-	NP	NP	NP	9	A-1-b (0)

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - I-70/I-71 East Interchange - 3/14/13 11:13

MATERIAL DESCRIPTION AND NOTES	ELEV. 760.1	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI			
Very dense brown GRAVEL WITH SAND (A-1-b), little to some silt; damp. (continued)		26	21															
		27	25 16	54	100	SS-11	-	-	-	-	-	-	-	-	8	A-1-b (V)		
		28																
		29	20															
		30	22 16	50	100	SS-12	-	-	-	-	-	-	-	-	6	A-1-b (V)		
		31																
		32																
		33																
		34	20															
		35	31 26	75	100	SS-13	-	43	24	19	- 14 -	NP	NP	NP	6	A-1-b (0)		
		36																
		37																
		38																
		39	19															
	@38.5', becomes wet.		20															
		20																
		40																
		41																
		42																
		43																
		44	26															
@43.5', 1.5 feet sand heave; washed out with tricone.		27																
		31																
		44																
		26																
		27																
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STANDARD ODOT SOIL BORING LOG (8.5 X 11) - I-70/I-71 East Interchange - 3/14/13 11:13

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI			
Very dense brown GRAVEL WITH SAND (A-1-b), little to some silt; wet. @56.0' - 67.0', gray.	733.3	52	25 23	63	56	SS-17	-	-	-	-	-	-	-	-	-	-	13	A-1-b (V)
		53																
		54	20 23 25	63	83	SS-18	-	-	-	-	-	-	-	-	-	-	9	A-1-b (V)
		55																
		56	27															
		57	26 22	63	72	SS-19	-	28	41	20	- 11 -	NP	NP	NP		13	A-1-b (0)	
		58																
		59	20 27 22	65	28	SS-20	-	-	-	-	-	-	-	-	-	-	16	A-1-b (V)
		60																
		61																
	62																	
	63																	
	64	36 36 29	86	72	SS-21	-	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
	65																	
	66																	
	67																	
Hard gray SANDY SILT (A-4a), "and" fine to coarse sand, little gravel; damp.	718.1	68																
		69	22 41 37	103	67	SS-22	4.5+	12	14	30	30	14	17	13	4	10	A-4a (2)	
	70																	
	71																	
	72																	
Very dense gray SANDY SILT (A-4a), some to "and" fine to coarse sand, trace gravel; wet.	713.1	73																
		74	43 29	71	83	SS-23A	-	-	-	-	-	-	-	-	-	15	A-4a (V)	
	75	25	-	-	SS-23B	3.75	-	-	-	-	-	-	-	-	19	A-6a (V)		
Very stiff to hard gray SILT AND CLAY (A-6a), trace to little fine to coarse sand, trace gravel; damp to moist.	710.7	76																
		77																
		78																

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - I-70/I-71 East Interchange - 3/14/13 11:13

MATERIAL DESCRIPTION AND NOTES	ELEV. 706.5	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED	
								GR	CS	FS	SI	CL	LL	PL	PI				
Very stiff to hard gray SILT AND CLAY (A-6a), trace to little fine to coarse sand, trace gravel; damp to moist. (continued)		79	17																
		80	28 30	76	100	SS-24	4.5+	0	1	1	47	51	35	21	14	19	A-6a (10)		
		81																	
		82																	
		83																	
		84		12 13 19	42	100	SS-25	4.5+	-	-	-	-	-	-	-	-	19	A-6a (V)	
		85																	
		86																	
		87																	
		88																	
	89		18 15 13	37	100	SS-26	4.5+	-	-	-	-	-	-	-	-	24	A-6a (V)		
	695.1	EOB																	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - I-70/I-71 East Interchange - 3/14/13 11:13

NOTES: SEEPAGE AT 14.5, 20.5, AND 38.5 FEET; WATER LEVEL PRIOR TO ADDING WATER = 40.2 FEET; FINAL WATER LEVEL INCLUDING DRILLING WATER = 39.7 FEET.
 ABANDONMENT METHODS, MATERIALS, QUANTITIES: 1 BAG ASPHALT PATCH; 2 BAGS BENTONITE CHIPS; 3 BAGS QUIKGROUT

APPENDIX II

Historic Data:
Investigational Procedures
Boring Plan
Site Plan and Profile
Boring Logs – Three (3) Borings
Summary of Soil Test Data

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY
FOUNDATION EXPLORATION SECTION

FOUNDATION INVESTIGATIONAL PROCEDURES

Drive Rod Penetration Tests

Drive rod penetration resistance tests constitute driving a 1.315-inch diameter steel rod, with a 45° cone point, into the ground, using a 122 pound drop-hammer with a free fall of five feet. At one-foot depth intervals, a measurement is taken to determine the amount of penetration achieved in three hammer drops. This reading is converted to an empirical value for capacity "R", in thousands of pounds (which is a measure of both the point resistance and frictional resistance on the rod), by using charts prepared by the Ohio Department of Highways, Bureau of Bridges, on the basis of correlation study of rod penetration with past performance of pile driving. For interpretation, a graph is prepared by plotting the value "R" against the depth at which the reading was taken, and connecting the plotted points. The curve so obtained reflects the density of subsurface materials in a manner that can be readily compared with data from similar tests at other locations on the structure site. From this comparison, the overall uniformity of subsurface conditions may be evaluated.

Drive Sample Borings

Drive sample borings are made by means of a rotary-type drill rig, employing a 2" O. D., 1-3/8" I. D. sampler, at 2-1/2 and/or 5-foot depth intervals, driven by means of a 140 pound drop-hammer, with a free fall of 30 inches. The number of blows required to drive the sampler 12 inches is considered the standard penetration test. The Boring Log sheets show a graphic plot of the information obtained, including depth and elevation of the sample, number of blows for the standard penetration tests in two 6-inch increments, laboratory sample number, sample description - based on laboratory test results and the Casagrande AC classification system - and the results of laboratory testing which consists of gradation, plasticity, and moisture content determinations. At depths where the drift is bouldery or gravelly to the extent that the sampler can not be driven, a wash sample is procured for visual classification, in order to determine the general character of the material. These samples are not considered sufficiently representative to warrant laboratory testing.

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

SHEET 4

P-3

LOG OF BORING

CO., RT. NO., SEC. FRA-40- 13.83 BRIDGE NO. FRA-40
REAR ABUTMENT E. INNERBELT UNDER MAIN STREET
 LOCATION: T.H. 1B STA. 45+78 OFFSET 30' RT. FED. NO. _____

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
784.7	0		NGVD29	
	2			
	4			
779.7	6	9/10	23882	Brown Silty Sandy Gravel
	8			
774.7	10			
	12	8/7	23883	Brown Silty Gravelly Sand
	14			
769.9	16	19/22	23884	Brown Silty Sandy Gravel
	18			
764.7	20			
	22	28/40	23885	Brown and Gray Silty Gravelly Sand
	24			
759.7	26	30/60	23886	Light-Brown Silty Sandy Gravel
757.2	28	19/50	23887	Brown Silty Sandy Gravel
754.7	30			
	32	17/25	23888	Brown Silty Gravelly Sand
752.2	34	20/29	23889	Brown Silty Gravelly Sand
749.7	36	20/43	23890	Brown Silty Sandy Gravel

MB
7-22-59

LOG OF BORING (CONTINUED)

SHEET 5

BRIDGE NO. FRA-40-

T.H. 1B

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
747.2	38	19/38	23891	Brown Silty Gravelly Sand
744.7	40			
	42	10/32	23892	Brown Sandy Gravel
742.2	44	13/29	23893	Brown Silty Sand
739.7	46	27/55	23894	Brown Gravelly Sand
	48			
734.7	50	29/45	23895	Brown Sand
	52			
	54			
729.7	56	15/35	23896	Brown Silty Sand
	58			
724.7	60	25/52	23897	Brown and Gray Silty Sandy Gravel
	62			
	64			
719.7	66	18/34	23898	Brown Silty Sand
718.7	68			BOTTOM OF BORING
	70			
	72			
	74			
	76			
	78			
	80			
	82			

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

SHEET 4

p. 3

LOG OF BORING

CO., RT. NO., SEC. FRA-40- 13.83 BRIDGE NO. FRA-40-
THIRD PIER E. INNERBELT UNDER MAIN STREET
 LOCATION: T.H. 7B STA. 47+68 OFFSET 24' RT FED. NO.

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
785.3	0		NGVD29	
	2			
	4			
780.3	6	5/11	23915	Brown Sandy Gravelly Silt
	8			
775.3	10			
	12	5/11	23916	Brown Silty Sandy Gravel
	14			
770.3	16	3/3	---	Brown Silty Sandy Gravel
	18			
765.3	20			
	22	9/6	23917	Brown Silty Sandy Gravel
	24			
760.3	26	36/42	23918	Brown Silty Sandy Gravel
757.8	28	68/50	23919	Brown Silty Sandy Gravel
755.3	30			
	32	25/25	23920	Brown Silty Gravelly Sand
752.8	34	45/36	23921	Brown Silty Sandy Gravel
750.3	36	37/80	23922	Brown silty Sandy Gravel

MB
7-22-59

LOG OF BORING (CONTINUED)

SHEET 7

BRIDGE NO. ERA-40 T.H. 7B

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
747.8	38	56/50	23923	Brown Silty Gravelly Sand
745.3	40			
	42	28/29	23924	Brown Silty Gravelly Sand
742.8	44	33/50	23925	Brown Silty Gravelly Sand
740.3	46	41/50	23926	Brown Silty Gravelly Sand
737.8	48	26/56	23927	Brown Silty Sandy Gravel
735.3	50			
	52	43/60	23928	Brown Silty Sandy Gravel
	54			
730.3	56	28/56	23929	Brown Silty Sandy Gravel
727.8	58	----	23930	Brown Gravel (Wash sample)
725.3	60			
	62	36/50	23931	Brown Silty Sandy Gravel
722.8	64	----	23932	Brown Sandy Gravel (Wash sample)
720.3	66	----	----	Brown Sandy Gravel
	68			BOTTOM OF BORING
	70			
	72			
	74			
	76			
	78			
	80			
	82			

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY

SHEET 8

p-3

LOG OF BORING

CO., RT. NO., SEC. FRA-40-13.83 BRIDGE NO. FRA-40
FORWARD ABUTMENT E. INNERBELT UNDER MAIN STREET
 LOCATION: T.H. 12 B STA. 48+82 OFFSET 43' LT FED. NO. _____

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
786.0	0			
	2			
	4			
781.0	6	8/8	23899	Brown Sandy Gravelly Silt
	8			
776.0	10			
	12	15/19	23900	Brown Silty Sandy Gravel
	14			
771.0	16	33/36	23901	Gray Gravelly Sandy Silt
	18			
766.0	20			
	22	29/47	23902	Gray Gravelly Sandy Silt
	24			
761.0	26	20/36	23903	Brown Silty Gravelly Sand
	28			
756.0	30			
	32	40/50	23904	Brown Silty Gravelly Sand
753.5	34	23/45	23905	Brown Silty Gravelly Sand
751.0	36	26/55	23906	Brown Silty Sandy Gravel

LOG OF BORING (CONTINUED)

SHEET 9

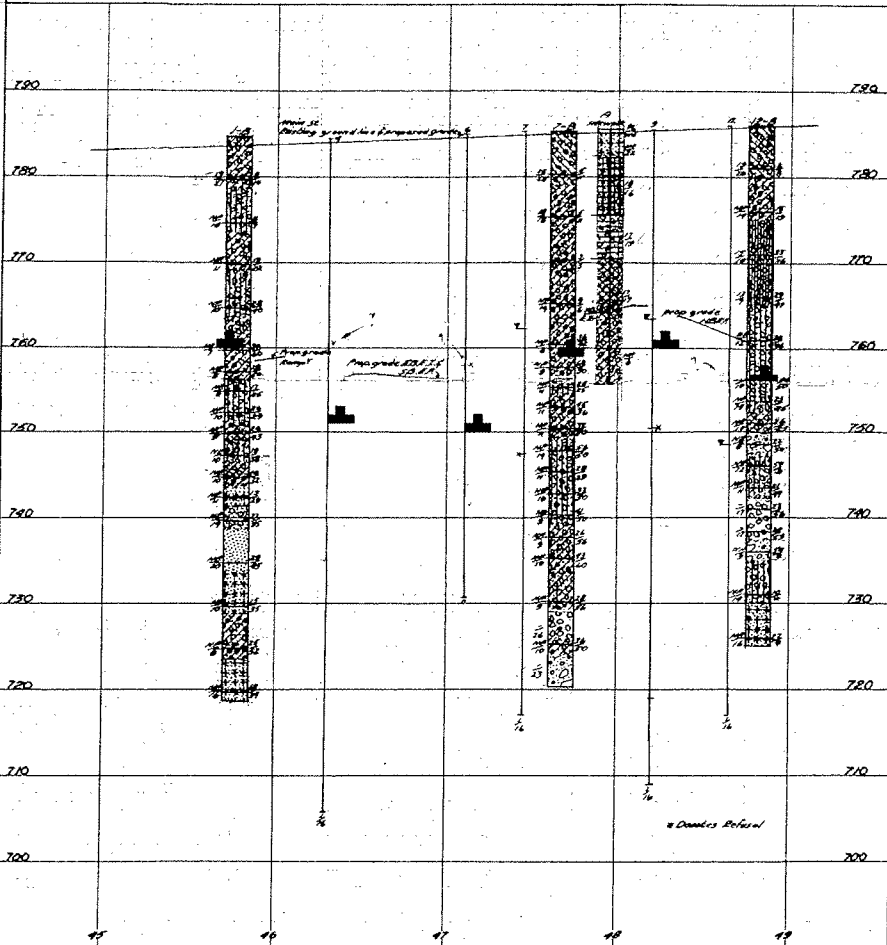
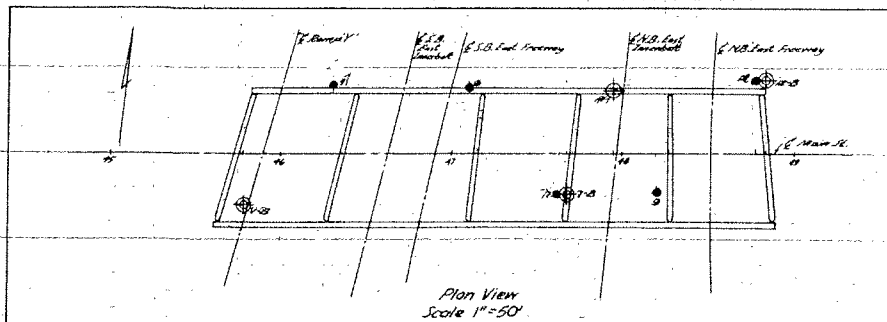
BRIDGE NO. FRA-40

T.H. 12B

ELEV.	DEPTH	NO. BLOWS	SAMPLE NO.	DESCRIPTION
748.5	38	22/50	23907	Brown Sandy Gravel
746.0	40	50/*	23908	Brown Silty Gravelly Sand
	42			
743.5	44	21/47	23909	Brown Silty Gravelly Sand
741.0	46	22/56	23910	Brown Gravel
738.5	48	20/59	23911	Brown Gravel
736.0	50	50/*	23912	Brown Sandy Gravel
	52			
	54			
731.0	56	10/*	23913	Brown Silty Gravelly Sand
	58			
726.0	60			
725.0	62	22/*	23914	Gray Silty Sand
	64			* Refusal
	66			
	68			
	70			
	72			
	74			
	76			
	78			
	80			
	82			

BOTTOM OF BORING

* Refusal



Profile View
 Hor. Scale 1"=30'
 Ver. Scale 1"=10'

- Gravel
- Sandy gravel
- Silty sandy gravel
- Clayey sandy gravel
- Gravelly sand
- Silty gravelly sand
- Clayey gravelly sand
- Sand
- Silty sand
- Sandy gravelly silt
- Gravelly sandy silt
- Sandy gravelly clay

LEGEND

	Excavation		Indicates Resistance of 15' ± 10,000 lbs. in profile view
	Auger boring - Plan View		Figures to the left of boring log in profile view: X = Plastic Limit Y = Moisture Content
	Drive Sample Boring - Plan View		Figures to the right of boring log in profile view indicate number of blows for "Standard Penetration Test" P = First 6 inches G = Second 6 inches
	Drive Rod Penetration Resistance Soundings - Plan View		Indicates final measurement of penetration in inches
	Drive Rod Penetration Resistance Soundings - Depth Penetrated		
	B Indicates Drive Sample Boring		
	A Indicates Auger Boring		
	Indicates water elevation		

BORING PLAN
 &
 SUBSURFACE DATA SKETCH

FRA-40-
 EAST INNERBELT
 E. INNERBELT UNDER MAIN ST.
 FRANKLIN COUNTY

40 13.83

December 3, 1959

J. A. Francis, Personnel Officer

W. E. Mason, Assistant Engineer

Information for Soil Profile and Structure Foundation Investigation Charges

PRA-40-13.83 Columbus Eas Innerbelt

File: 14-18-1
Franklin
Columbus

Transmitted herein is information relative to charges for the soil profile and the structure foundation investigations for the subject project.

I. Project Identification - Job. No. 06306(2)

A. Soil Profile

PRA-40-13.83

B. Structures

1. PRA-40-1404 A. B. E. Innerbelt Over S. B. E. Freeway
2. PRA-40-1447 East Innerbelt Under Oak Street
- 3. PRA-40- East Innerbelt Under Main Street
4. PRA-40- East Innerbelt Under Town Street
5. PRA-40- N. B. East Freeway Under Parsons Ave.
6. Retaining Wall "E", Station 122+35 to Station 123+25, Left of Centerline
7. Retaining Wall "F", Station 56+05 to Station 59+50, Left of Centerline (Parsons Avenue Stationing)
8. Retaining Wall "G", Station 126+40 to Station 132+30, Left of Centerline
9. Retaining Wall "H", Station 133+00 to Station 135+40, Right of Centerline
10. Retaining Wall "I", Station 135+85 N. B. E. Innerbelt to Station 4+70 E. Innerbelt, Right of Centerline

II. Charges

A. Salaries and Expenses (Field Operations)

Pay Periods - April 1-15, 1959
 April 16-30, 1959
 May 1-15, 1959
 May 16-31, 1959
 June 1-15, 1959
 June 16-30, 1959
 July 1-15, 1959
 July 16-31, 1959

J. A. Francis
FRA-40-13.83

-2-

December 3, 1959

B. Equipment
Drill Unit Days - 132

C. Sample Testing
Samples Tested - 454

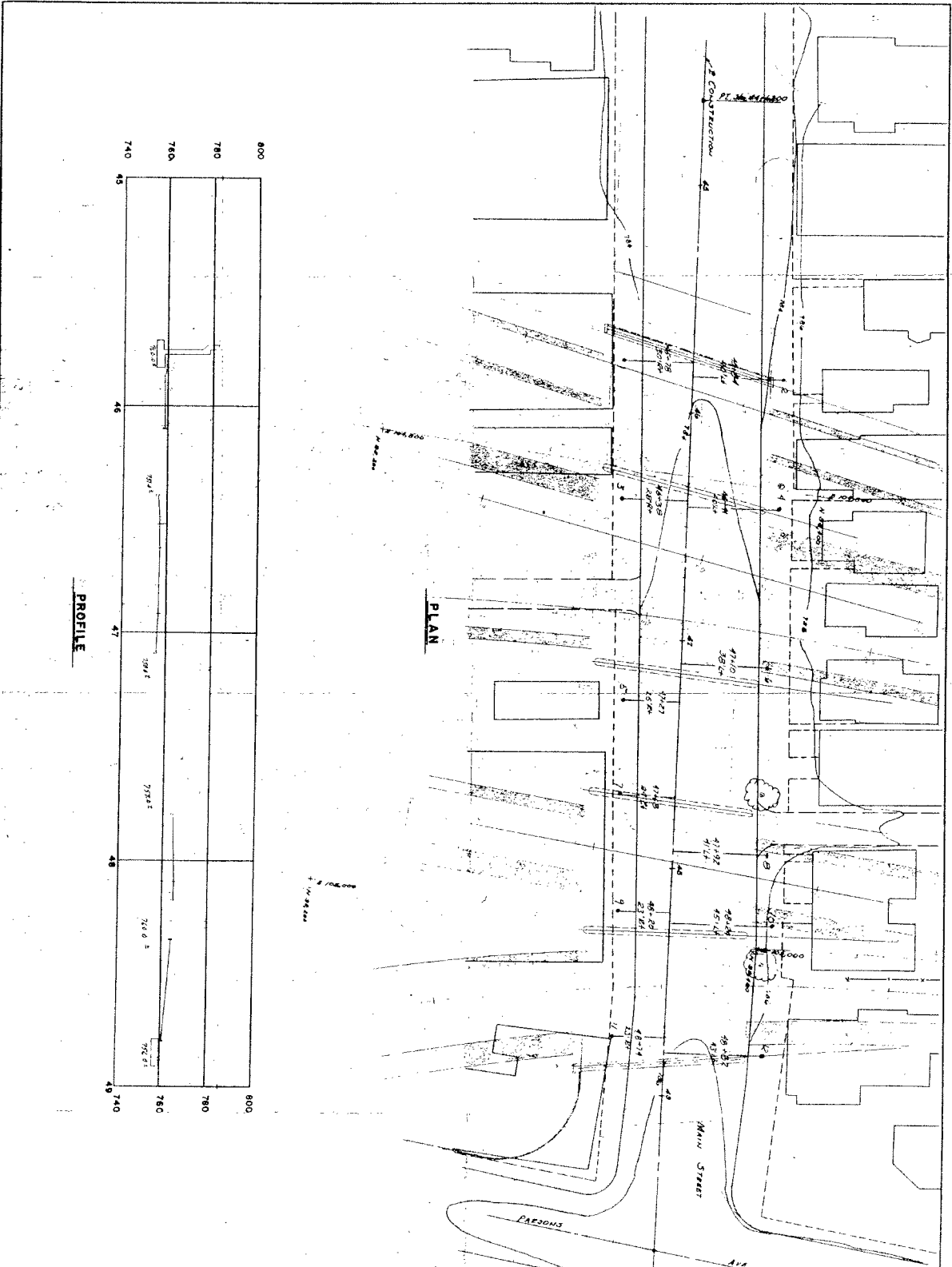
III. Authorization for Billing

See letter in File 14-18-1, from C. W. McCaughey to
Mr. Robert Werner, Columbus City Engineer, dated
March 10, 1959.

W. On the basis of the letter mentioned in Item III, it
is my understanding that the City of Columbus is
responsible for plan preparation.

M. E. Mason
Assistant Engineer

NEN:DJR:GPH:FAH:bjc
cc: J. A. Francis
K. E. Mason (14)



DATE	SCALE	PROJECT	NO.
8	OHIO	FRANKLIN COUNTY	FRA

ALDEN E. SHANNON & ASSOCIATES, LIMITED
 CONSULTING ENGINEERS
 1000 W. MAIN STREET
 COLUMBUS, OHIO 43260

SITE PLAN
 BRIDGE NO. FRA
 EAST INNERBELT UNDER MAIN STREET
 FRANKLIN COUNTY STA.

46 04202 (7)

**STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY**

SUMMARY OF SOIL TEST DATA

CO., RT. NO., SEC. FRANKLIN
FBA-40-13.83
FRA-40
E. INNERBELT UNDER MAIN STREET
 SHEET NO. 3 OF 12 SHEETS

SAMPLE NUMBER	LABORATORY NUMBER SQ-	PHYSICAL CHARACTERISTICS								WATER CONTENT	DESCRIPTION
		% AGGREGATE RET. # 10	% COARSE SAND 2.0MM - 0.42MM	% FINE SAND 0.42MM - 0.075MM	% SILT 0.075MM - 0.005MM	% CLAY < 0.005MM	LIQUID LIMIT	PLASTICITY INDEX			
DRIVE SAMPLES											
1	23915	37	7	10	21	25	43	21	25	BROWN SANDY GRAVELLY SILT	
2	23916	46	17	12	11	14	31	13	18	BROWN SILTY SANDY GRAVEL	
3	23917	52	29	13	-6	---	NP	NP	14	BROWN SILTY SANDY GRAVEL	
4	23918	57	17	10	11	5	NP	NP	8	BROWN SILTY SANDY GRAVEL	
5	23919	57	23	9	-11	---	NP	NP	9	BROWN SILTY SANDY GRAVEL	
6	23920	27	32	18	15	8	NP	NP	11	BROWN SILTY GRAVELLY SAND	
7	23921	38	30	14	-18	--	NP	NP	11	BROWN SILTY SANDY GRAVEL	
8	23922	46	23	17	-14	--	NP	NP	11	BROWN SILTY SANDY GRAVEL	
9	23923	27	22	33	-18	--	NP	NP	14	BROWN SILTY GRAVELLY SAND	
10	23924	30	32	20	-18	--	NP	NP	11	BROWN SILTY GRAVELLY SAND	
11	23925	19	24	37	-20	--	NP	NP	10	BROWN SILTY GRAVELLY SAND	
12	23926	31	33	18	-18	--	NP	NP	9	BROWN SILTY GRAVELLY SAND	
13	23927	56	22	10	-12	--	NP	NP	9	BROWN SILTY SANDY GRAVEL	
14	23928	41	31	14	-14	--	NP	NP	10	BROWN SILTY SANDY GRAVEL	
15	23929	64	16	10	-10	--	NP	NP	9	BROWN SILTY SANDY GRAVEL	
16	23930	V I S U A L								26	BROWN GRAVEL
17	23931	54	23	10	7	6	NP	NP	10	BROWN SILTY SANDY GRAVEL	
18	23932	V I S U A L								23	BROWN SANDY GRAVEL
STA 47+95, 37' LT											
(120+75.7' LT., SURF. EL. 785.5)											
LOG OF WILLIAMS SAUGER BORING											
DEPTH											
V I S U A L											
0.0-0.5 SIDEWALK MATERIAL											
42	18165	8	6	13	43	30	NP	NP	22	0.5-3.0 BROWN SANDY SILT	
43	18166	33	12	12	19	24	33	15	16	3.0-10.0 BR. SANDY GRAVELLY CLAY	
44	18167	47	19	10	13	11	34	17	19	10.0-15.0 BR. CLAYEY SANDY GRAVEL	
45	18168	35	28	14	12	11	28	11	17	15.0-22.0 BR. CLAYEY GRAVELLY SAND	
46	18169	48	28	10	7	7	NP	NP	6	22.0-30.0 BR. SILTY SANDY GRAVEL	

**STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY**

SUMMARY OF SOIL TEST DATA

CO., RT. NO., SEC FRANKLIN
 FRA-40-13.83
 FRA-40-
 E. INNERBELT UNDER MAIN STREET
 SHEET NO. 1 OF 12 SHEETS

SAMPLE NUMBER	LABORATORY NUMBER SQ-	PHYSICAL CHARACTERISTICS							WATER CONTENT	DESCRIPTION
		% AGGREGATE RET. # 10	% COARSE SAND 2.0MM - 0.42MM	% FINE SAND 0.42MM - 0.075MM	% SILT 0.075MM - 0.005MM	% CLAY < 0.005MM	LIQUID LIMIT	PLASTICITY INDEX		
										DRIVE SAMPLES
1	23882	37	17	16	15	15	25	6	21	BROWN SILTY SANDY GRAVEL
2	23883	37	34	11	13	5	NP	NP	10	BROWN SILTY GRAVELLY SAND
3	23884	60	16	8	11	5	NP	NP	11	BROWN SILTY SANDY GRAVEL
4	23885	38	24	18	14	6	NP	NP	10	BROWN and GRAY SILTY GRAVELLY SAND
5	23886	60	16	8	12	4	NP	NP	7	LIGHT BROWN SILTY SANDY GRAVEL
6	23887	43	31	10	16	0	NP	NP	8	BROWN SILTY SANDY GRAVEL
7	23888	30	42	13	-1	-	NP	NP	8	BROWN SILTY GRAVELLY SAND
8	23889	34	41	11	-1	-	NP	NP	10	BROWN SILTY GRAVELLY SAND
9	23890	42	34	12	-1	-	NP	NP	9	BROWN SILTY SANDY GRAVEL
10	23891	24	43	15	-1	-	NP	NP	10	BROWN SILTY GRAVELLY SAND
11	23892	48	36	8	--	8-	NP	NP	10	BROWN SANDY GRAVEL
12	23893	7	62	14	-1	-	NP	NP	11	BROWN SILTY SAND
13	23894	40	51	5	--	4-	NP	NP	14	BROWN GRAVELLY SAND
14	23895	0	74	20	-6	--	NP	NP	20	BROWN SAND
15	23896	6	58	15	-1	--	NP	NP	10	BROWN SILTY SAND
16	23897	48	25	11	-1	--	NP	NP	6	BROWN & GRAY SILTY SANDY GRAVEL
17	23898	0	35	50	-1	--	NP	NP	16	BROWN SILTY SAND

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**STATE OF OHIO
DEPARTMENT OF HIGHWAYS
TESTING LABORATORY**

SUMMARY OF SOIL TEST DATA

CO., RT. NO., SEC. FRANKLIN
FRA-40-13.83
FRA-40
E. INNERBELT UNDER MAIN STREET
 SHEET NO. 3 OF 12 SHEETS

SAMPLE NUMBER	LABORATORY NUMBER SO-	PHYSICAL CHARACTERISTICS								WATER CONTENT	DESCRIPTION	
		% AGGREGATE RET. # 10	% COARSE SAND 2.0MM - 0.42MM	% FINE SAND 0.42MM - 0.074MM	% SILT 0.074MM - 0.006MM	% CLAY < 0.006MM	LIQUID LIMIT	PLASTICITY INDEX				
											DRIVE SAMPLES	
1	23915	37	7	10	21	25	43	21	25		BROWN SANDY GRAVELLY SILT	
2	23916	46	17	12	11	14	31	13	18		BROWN SILTY SANDY GRAVEL	
3	23917	52	29	13	-6	---	NP	NP	14		BROWN SILTY SANDY GRAVEL	
4	23918	57	17	10	11	5	NP	NP	8		BROWN SILTY SANDY GRAVEL	
5	23919	57	23	9	-11	---	NP	NP	9		BROWN SILTY SANDY GRAVEL	
6	23920	27	32	18	15	8	NP	NP	11		BROWN SILTY GRAVELLY SAND	
7	23921	38	30	14	-18	---	NP	NP	11		BROWN SILTY SANDY GRAVEL	
8	23922	46	23	17	-14	---	NP	NP	11		BROWN SILTY SANDY GRAVEL	
9	23923	27	22	33	-18	---	NP	NP	14		BROWN SILTY GRAVELLY SAND	
10	23924	30	32	20	-18	---	NP	NP	11		BROWN SILTY GRAVELLY SAND	
11	23925	19	24	37	-20	---	NP	NP	10		BROWN SILTY GRAVELLY SAND	
12	23926	31	33	18	-18	---	NP	NP	9		BROWN SILTY GRAVELLY SAND	
13	23927	56	22	10	-12	---	NP	NP	9		BROWN SILTY SANDY GRAVEL	
14	23928	41	31	14	-14	---	NP	NP	10		BROWN SILTY SANDY GRAVEL	
15	23929	64	16	10	-10	---	NP	NP	9		BROWN SILTY SANDY GRAVEL	
16	23930		V I S U A L							26		BROWN GRAVEL
17	23931	54	23	10	7	6	NP	NP	10		BROWN SILTY SANDY GRAVEL	
18	23932		V I S U A L							23		BROWN SANDY GRAVEL
											LOG OF WILLIAMS SAUGER BORING	
											DEPTH	
											0.0-0.5 SIDEWALK MATERIAL	
42	18165	8	6	13	43	30	NP	NP	22		0.5-3.0 BROWN SANDY SILT	
43	18166	33	12	12	19	24	33	15	16		3.0-10.0 BR. SANDY GRAVELLY CLAY	
44	18167	47	19	10	13	11	34	17	19		10.0-15.0 BR. CLAYEY SANDY GRAVEL	
45	18168	35	28	14	12	11	28	11	17		15.0-22.0 BR CLAYEY GRAVELLY SAND	
46	18169	48	28	10	7	7	NP	NP	6		22.0-30.0 BR. SILTY SANDY GRAVEL	

APPENDIX III

Loading Information
Calculations



	1	2	3
Spans (ft)	150.83	91.14	130.96

STRUCTURE DEPTH = 18.00 FT SKEW = 12.34 DEG
 REAR ABUMENT WIDTH = 124.00 FT
 SKEW ANGLE OF WIND = 45.00 DEG
 WIND LOAD LONG TO BRDG = 0.016 K/FT² LONG TO R.A. = 0.023 K/FT²
 WIND LOAD TRANS TO BRDG = 0.033 K/FT² TRANS TO R.A. = 0.029 K/FT²
 WIND ON LIVE LONG TO BRDG = 0.032 K/FT LONG TO R.A. = 0.045 K/FT
 WIND ON LIVE TRANS TO BRDG = 0.066 K/FT TRANS TO R.A. = 0.058 K/FT

Vertical Loads * Includes stem weight, but not footing

Substructure	Unfactored LL	Unfactored DL *	Unfactored DW
	k/ft	k/ft	k/ft
R.A.	5.59	34.79	2.56

Transverse Loads (All loads acting at bearing elevation, in the center of the abutment, unless otherwise indicated)

Substructure	Height Beam Seat to Bot/Ftg	Wind Load on LL (WL)	Wind Load on Superstructure (WS1)	Wind Load on Substructure (WS2)	Braking Load (BR)	Centrifugal Force (CF)	Temp. Load (TU)
		kip	kip	kip	kip	kip	kip
R.A.	26.24	4.35	39.12	N/A	N/A	12.59	31.06

Longitudinal Loads (All loads acting at bearing elevation, in the center of the abutment, unless otherwise indicated)

Substructure	Area	Wind Load on LL (WL)	Wind Load on Superstructure (WS1)	Wind Load on Substructure (WS2)	Braking Load (BR)	Centrifugal Force (CF)	Temp. Load (TU)
	ft ²	kip	kip	kip	kip	kip	kip
R.A.	N/A	3.42	30.79	N/A	N/A	2.75	141.96



	1	2	3
Spans (ft)	150.83	91.14	130.96

STRUCTURE DEPTH = 17.00 FT SKEW = 1.73 DEG
 FORWARD ABUMENT WIDTH = 121.25 FT
 SKEW ANGLE OF WIND = 60.00 DEG
 WIND LOAD LONG TO BRDG = 0.019 K/FT² LONG TO F.A. = 0.020 K/FT²
 WIND LOAD TRANS TO BRDG = 0.017 K/FT² TRANS TO F.A. = 0.016 K/FT²
 WIND ON LIVE LONG TO BRDG = 0.038 K/FT LONG TO F.A. = 0.039 K/FT
 WIND ON LIVE TRANS TO BRDG = 0.034 K/FT TRANS TO F.A. = 0.033 K/FT

Vertical Loads * Includes stem weight, but not footing

Substructure	Unfactored LL	Unfactored DL *	Unfactored DW
	k/ft	k/ft	k/ft
F.A.	5.43	31.42	2.24

Transverse Loads (All loads acting at bearing elevation, in the center of the abutment, unless otherwise indicated)

Substructure	Height Beam Seat to Bot/Ftg	Wind Load on LL (WL)	Wind Load on Superstructure (WS1)	Wind Load on Substructure (WS2)	Braking Load (BR)	Centrifugal Force (CF)	Temp. Load (TU)
		kip	kip	kip	kip	kip	kip
F.A.	25.74	2.15	18.28	N/A	N/A	12.23	2.35

Longitudinal Loads (All loads acting at bearing elevation, in the center of the abutment, unless otherwise indicated)

Substructure	Area	Wind Load on LL (WL)	Wind Load on Superstructure (WS1)	Wind Load on Substructure (WS2)	Braking Load (BR)	Centrifugal Force (CF)	Temp. Load (TU)
	ft ²	kip	kip	kip	kip	kip	kip
F.A.	N/A	2.55	21.71	N/A	N/A	0.37	77.96



Loads are taken at top of footing and include the weight of the pier columns. Weight of footing is not included.

Force resultants are at the center of pier.

Vertical and lateral earth pressures, and live load surcharge on footing not applied.

All loads are unfactored service loads.

All force effects may be taken as +/- to maximize effects except upward wind (STR III).

Coordinate Axes:

- Y axis is vertical
- X axis is parallel to the pier cap long direction
- Z axis is perpendicular to the pier cap

Foundation Load Summary

Load Type	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kip-ft)	Mz (kip-ft)
Self Weight	0	1384	0	0	0
DC	0	4445	0	0	6580
DW	0	632	0	0	392
TU	9	0	63	2058	285
WS (STR III only)	134	283	19	527	12780
WS	28	0	703	1476	0
WL	1	0	5	173	45
EH	0	0	703	1476	0
LS	0	0	160	824	0
LL Case 1					
LL	0	359	0	0	14478
BR	0	0	0	0	0
CE	4	0	1	23	167
LL Case 2					
LL	0	583	0	0	14704
BR	0	0	0	0	0
CE	6	0	1	38	271
LL Case 3					
LL	0	457	0	0	16152
BR	0	0	0	0	0
CE	5	0	1	29	213

Eric Tse

From: Gardner, Gary [ggardner@msconsultants.com]
Sent: Thursday, June 13, 2013 7:14 AM
To: Eric Tse
Cc: Brian Afek; Timothy Hampshire; Daniel O'Rorke
Subject: RE: New Loads for 1716 Pier 1

Eric,

Primarily I am concerned about the case where EH is a driving force and eccentricity is a resisting force. If the other direction controlled the argument could always be made that EH should be calculated using passive instead of active pressure, in which case you'd end up with huge resisting forces.

Gary Gardner, PE

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From: Eric Tse [mailto:etse@dlz.com]
Sent: Wednesday, June 12, 2013 7:52 PM
To: Gardner, Gary
Cc: Brian Afek; Timothy Hampshire; Daniel O'Rorke
Subject: RE: New Loads for 1716 Pier 1

Gary,

Yesterday you wrote: **EH loading can act in only one direction. Moment due to vertical load eccentricity can also act in only one direction. By choosing the direction of the eccentricity we can use these two facts to offset one another, same as we do in an abutment footing. All other applied horizontal loads (WS, WL, etc.) can act in either direction. So one case has EH as a resisting force, vertical loading X eccentricity as a driving force, and all other horizontal loads as driving forces. The other case has EH as a driving force, all other horizontal loads as a driving force, and vertical loads X eccentricity as a resisting force.**

Do you want us to consider two load cases for EH as described above, one is resisting and the other driving, for all three LL cases?

By the way, I sent you an e-mail earlier asking if WL should have been zero. If it is in fact a zero, please check if all other loads are still valid.

Eric

From: Gardner, Gary [<mailto:ggardner@msconsultants.com>]
Sent: Wednesday, June 12, 2013 7:42 PM
To: Eric Tse
Cc: Brian Afek; Timothy Hampshire; Daniel O'Rorke
Subject: Re: New Loads for 1716 Pier 1

The 14' footing has 3.5' eccentricity. I'd like to use smaller footing if we can though. In general the footing has a 2' heel, a 3' stem, and "x" toe. The eccentricity is (toe minus heel)/2.

Apply same as before. Eh acts one way, eccentrics acts the other way, and all other horizontal loads are reversible

Sent from my iPhone

On Jun 12, 2013, at 6:20 PM, "Eric Tse" <etse@dlz.com> wrote:

Gary,

Please let me know what the new eccentricity for the 14' wide footing is and how I should apply the EHmoment and the moments associated with the new eccentricity.

Eric

From: Gardner, Gary [<mailto:ggardner@msconsultants.com>]
Sent: Wednesday, June 12, 2013 6:09 PM
To: Eric Tse
Subject: New Loads for 1716 Pier 1

Eric,

Small changes to most loadings, but the EH loading is changed quite a bit.

Gary Gardner, PE

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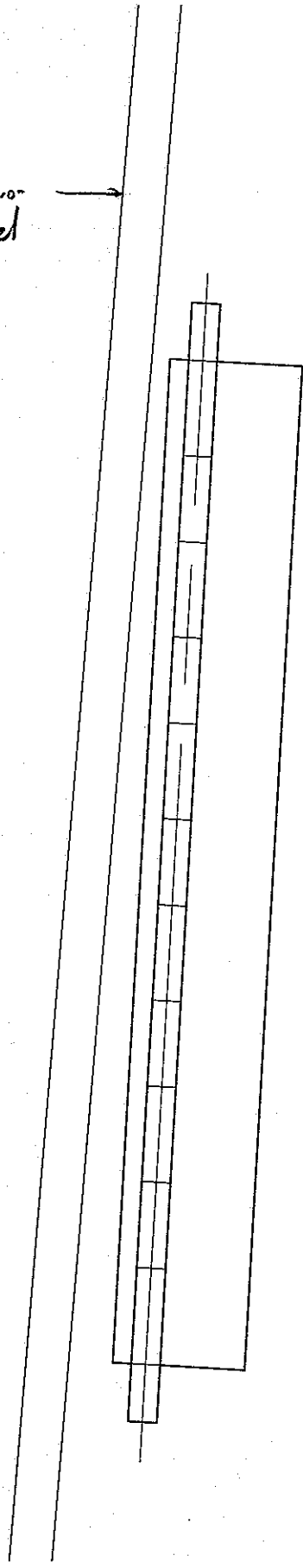
FRA-71-1716C

PIER 1

6-12-13

NITIS

Micro-Tunnel



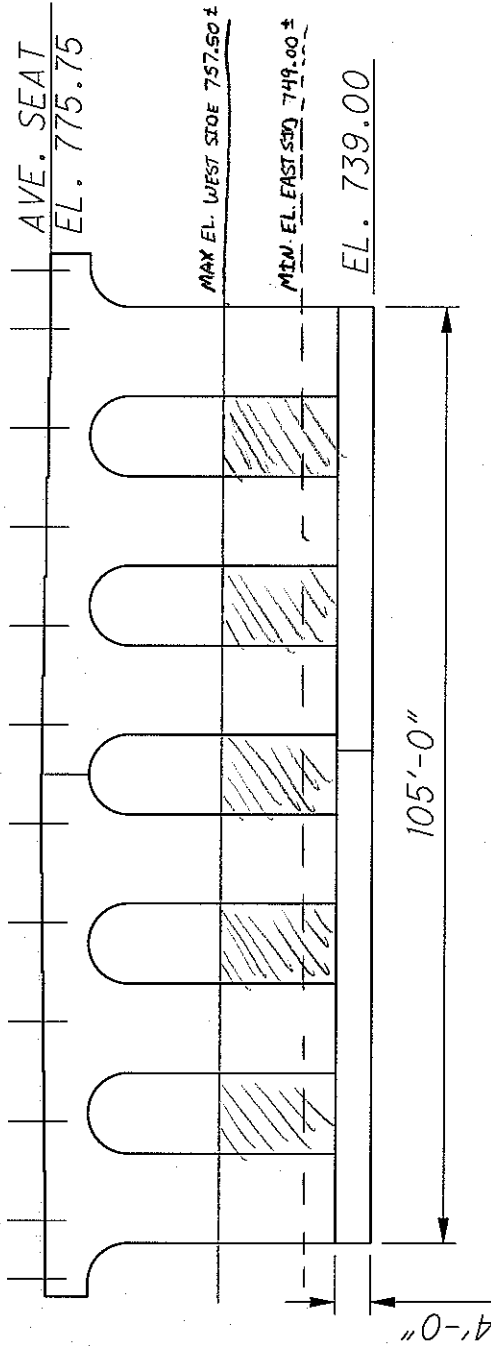
FRA-71-1716C

PIER 1

6-12-13

N.T.S.

AVE. DECK EL. 784.45





Loads are taken at top of footing and include the weight of the pier columns. Weight of footing is not included.

Force resultants are at the center of pier.

Vertical and lateral earth pressures, and live load surcharge on footing not applied.

All loads are unfactored service loads.

All force effects may be taken as +/- to maximize effects except upward wind (STR III).

Coordinate Axes:

- Y axis is vertical
- X axis is parallel to the pier cap long direction
- Z axis is perpendicular to the pier cap

Foundation Load Summary

Load Type	Fx (kips)	Fy (kips)	Fz (kips)	Mx (kip-ft)	Mz (kip-ft)
Self Weight	0	1456	0	0	0
DC	0	3816	0	0	5138
DW	0	549	0	0	630
TU	15	0	216	6272	430
WS (STR III only)	86	259	6	171	11964
WS	20	0	60	1704	704
WL (Case 1)	11	0	1	22	488
WL (Case 2)	1	0	5	138	53
EH	0	0	268	1009	0
LS	0	0	99	558	0
LL Case 1					
LL	0	329	0	0	11548
BR	4	0	58	2574	176
CE	4	0	0	10	161
LL Case 2					
LL	0	198	0	0	7919
BR	4	0	58	2574	176
CE	2	0	0	10	161
LL Case 3					
LL	0	420	0	0	12621
BR	4	0	58	2574	176
CE	5	0	0	10	161

Eric Tse

From: Brian Afek
Sent: Thursday, May 30, 2013 4:49 PM
To: 'Gardner, Gary'; Eric Tse
Cc: Daniel O'Rorke; Timothy Hampshire
Subject: RE: Main Street Pier 1

Gary,

As per our conversation on the phone:

Based on an assumed footing width of 15.0' and using the existing loading information; enough bearing resistance is achieved at the bearing elevation of 746.0.

Brian

From: Gardner, Gary [mailto:ggardner@msconsultants.com]
Sent: Thursday, May 30, 2013 3:24 PM
To: Brian Afek; Eric Tse
Cc: Daniel O'Rorke; Timothy Hampshire
Subject: RE: Main Street Pier 1

I take it back. That width causes us problems on MOT.

Can we narrow up the footing width by dropping the elevation to the bottom of ditch elevation? (747.0)

Gary Gardner, PE

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From: Brian Afek [mailto:bafek@dlz.com]
Sent: Thursday, May 30, 2013 2:42 PM
To: Eric Tse; Gardner, Gary
Cc: Daniel O'Rorke; Timothy Hampshire
Subject: RE: Main Street Pier 1

Gary,

Based on the bearing calculations, an estimated footing width of 17.0' will be required for Pier 2 if left at the same bearing elevation.

Thanks,

Brian

From: Eric Tse
Sent: Wednesday, May 29, 2013 5:15 PM
To: 'Gardner, Gary'
Cc: Daniel O'Rorke; Timothy Hampshire; Brian Afek
Subject: RE: Main Street Pier 1

We will get back to you with an estimated footing width tomorrow.

From: Gardner, Gary [<mailto:ggardner@msconsultants.com>]
Sent: Wednesday, May 29, 2013 5:22 PM
To: Eric Tse
Cc: Daniel O'Rorke; Timothy Hampshire; Brian Afek
Subject: RE: Main Street Pier 1

We agree the loads will be revised. Due to the sloping ground we need an initial estimate of size to determine the location based on cover requirements.

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From: Eric Tse [<mailto:etse@dlz.com>]
Sent: Wednesday, May 29, 2013 5:01 PM
To: Gardner, Gary
Cc: Daniel O'Rorke; Timothy Hampshire; Brian Afek
Subject: RE: Main Street Pier 1

Gary,

I forgot to mention in my e-mail earlier that the loading would change should we lower the footing elevation. The footing will only need to be lowered to an elevation as same as the toe elevation of the sloping ground shown on the site plan.

Brian will give you an estimated footing width with a sloping ground later (probably tomorrow).

From: Gardner, Gary [<mailto:ggardner@msconsultants.com>]
Sent: Wednesday, May 29, 2013 4:40 PM
To: Eric Tse
Cc: Daniel O'Rorke; Timothy Hampshire; Brian Afek
Subject: RE: Main Street Pier 1

Responses in red below.

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From: Eric Tse [<mailto:etse@dlz.com>]
Sent: Wednesday, May 29, 2013 3:55 PM
To: Gardner, Gary
Cc: Daniel O'Rorke; Timothy Hampshire; Brian Afek
Subject: RE: Main Street Pier 1

Gary,

I left you a voice message earlier regarding your May 23rd response e-mail (see the very bottom of the attached e-mails) concerning the negative dead load moment. I wanted to confirm with you the following:

1. Isn't it true that the negative dead load moment should be equal to ALL axial loads (dead and live loads) times the initial eccentricity of 15"? **This is correct. For a given load case, all axial loads in the columns would have to be multiplied by the initial eccentricity.**
2. When we analyzed the footing at Pier 1, we assumed all the moments provided in the loading sheet were in the same direction. Isn't it true that the negative dead load moment in question should be in a direction opposite to all other moments being applied? It means that the negative dead load moment is the only resisting moment in our analysis.

Moment due to initial eccentricity is in a single direction. Moment due to EH loading is also in a single (opposite to initial eccentricity) direction. All other moments due to braking, thermal, etc. can be in either direction depending on which direction governs.

For pier 2, there will be a sloping ground surface on the west side of the pier. According to AASHTO, we should ignore the embedment surcharge effect (Df) and reduce the shear resistance of the foundation soil for bearing resistance. At pier 2, the foundation soil is granular. Using the chart in AASHTO for footings on or near slopes, we came up with a factored bearing resistance of approx 4 ksf, which is only 1/3 of required bearing resistance for a 9.5 feet wide footing. According to GEC 6, the sloping effect can be ignored if the sloping ground is beyond the footing a distance of between 2 to 6 times the footing width. Given the close proximity of a ditch to pier 2, having a flat ground around the pier is not feasible. The only option will be to lower the footing to an elevation to match the toe elevation of the sloping ground. We are not governed on width for this footing. Please let us know what footing size will work for this location, and we'll check it against the grading plan to determine if the footing elevation needs to be dropped. The ditch elevation is 746+/- . Could this be our bottom of footing elevation, or do we need to have the footing completely below this elevation to increase the bearing capacity? Lowering the footing by 3.5' isn't too bad, but lowering by 7.5' is not preferred.

FYI only. Temporary sheeting will be used at piers 1 and 2. According ODOT BDM, the design of the temporary sheeting is required by the structural engineer for the project.

We are aware of the need for temporary shoring at several locations for the construction of this bridge in ½ width phases. Where we are greater than 8' and supporting traffic, our in-house geotechnical engineer is providing temporary shoring design for our plan sheets.

If you have any questions, please let me know.

Eric

From: Gardner, Gary [<mailto:ggardner@msconsultants.com>]
Sent: Wednesday, May 29, 2013 10:49 AM
To: Brian Afek
Cc: Daniel O'Rorke; Timothy Hampshire; Eric Tse
Subject: RE: Main Street Pier 1

Provided we get Pier 1 to work as a spread footing, we're going to revise Pier 2 as well. We won't need eccentricity on that one because the EH loading is much less. It will just be a regular concentric spread.

Using the loadings we previously sent, can you please provide the minimum footing size required for Pier 2?

Once we have an approximate size we may need to refine slightly the footing elevation just to make sure the toe of footing has sufficient cover due to the sloping ground. Use 749.5 as the bottom of spread footing.

Thanks,

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From: Brian Afek [<mailto:bafek@dlz.com>]
Sent: Wednesday, May 29, 2013 10:21 AM
To: Gardner, Gary
Cc: Daniel O'Rorke; Timothy Hampshire; Eric Tse
Subject: RE: Main Street Pier 1

Gary,

Another question:

For Main Street we only looked at Pier 1 using the spread footing design. Should we analyze pier 2 as a 9.5' wide spread footing with the 15" eccentricity as well?

Brian

From: Gardner, Gary [<mailto:ggardner@msconsultants.com>]
Sent: Wednesday, May 29, 2013 10:23 AM
To: Brian Afek
Cc: Daniel O'Rorke; Timothy Hampshire; Eric Tse
Subject: RE: Main Street Pier 1

For Grant Avenue I thought we had already discussed using a single layer cage and an embedded WF section. At the 5/15 meeting with ODOT they concurred with this. Please verify if I sent something previously about exactly what reinforcement and WF section to use. I'll look on my end as well.

For Main Street, just multiply the previously provided loadings by the eccentricity.

Gary Gardner, PE

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From: Brian Afek [<mailto:bafek@dlz.com>]
Sent: Wednesday, May 29, 2013 9:30 AM
To: Gardner, Gary
Cc: Daniel O'Rorke; Timothy Hampshire; Eric Tse
Subject: RE: Main Street Pier 1

Gary,

I have a few questions in regard to the Main Street and Grant Ave Bridges.

Main Street - 1716C

In your previous email, you suggest to design the footing as 9.5' and to shift it 15" to the east. Will you be providing an updated loading sheet or should we just multiply the vertical components by 15" in order to get the new moments?

Grant Ave – 1456

Our LPILE analyses indicated an increase in rebar to approximately 4.25% will be needed to control the deflection on the north abutment. Has there been any talks on whether this amount of rebar is acceptable for the design?

Thank You,

Brian Afek

From: Gardner, Gary [<mailto:ggardner@msconsultants.com>]
Sent: Thursday, May 23, 2013 8:31 AM
To: Eric Tse
Cc: Brian Afek; Daniel O'Rorke; Timothy Hampshire
Subject: RE: Main Street Pier 1

Using your loadings spreadsheet for the 9.5' wide footing with EH force, I optimize the effective footing size to almost exactly 9.5' if I introduce an initial eccentricity of 15" (therefore resulting in a negative dead load moment). My suggestion is to design this as a 9.5' spread footing shifted 15" to the east. This gives us about 20" clearance to the bore hole for the microtunnel, and it's much, much, much cheaper than the DS foundations. After accounting for the larger effective footing size, the bearing resistance works easily.

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From: Eric Tse [<mailto:etse@dlz.com>]
Sent: Wednesday, May 22, 2013 6:40 PM
To: Gardner, Gary
Cc: Brian Afek; Daniel O'Rorke; Timothy Hampshire
Subject: RE: Main Street Pier 1

Gary,

Attached are the spreadsheets for the calcs.

See results in red below.

From: Gardner, Gary [<mailto:ggardner@msconsultants.com>]
Sent: Wednesday, May 22, 2013 1:22 PM
To: Eric Tse; Brian Afek
Subject: Main Street Pier 1

Eric and Brian,

Question: At Main Steet Pier 1, using the loads we sent a few weeks ago, what would be the minimum spread footing size that would work? I figure you guys should be able to plug this in pretty quickly to your spreadsheet and get an answer. **Min. Footing width = 9.5'**

Question Part 2: What does that minimum footing size become if you eliminate the "EH" force? **Min. Footing with = 9.0'**

Please forward your calcs when you get these answers so we can see what controls.

I think this takes precedence over the question about piles because nobody can put a number on how close to the existing microtunnel we can drive piles.

Assume the spread footing elevation is 740.0.

Thanks,

Gary Gardner, PE

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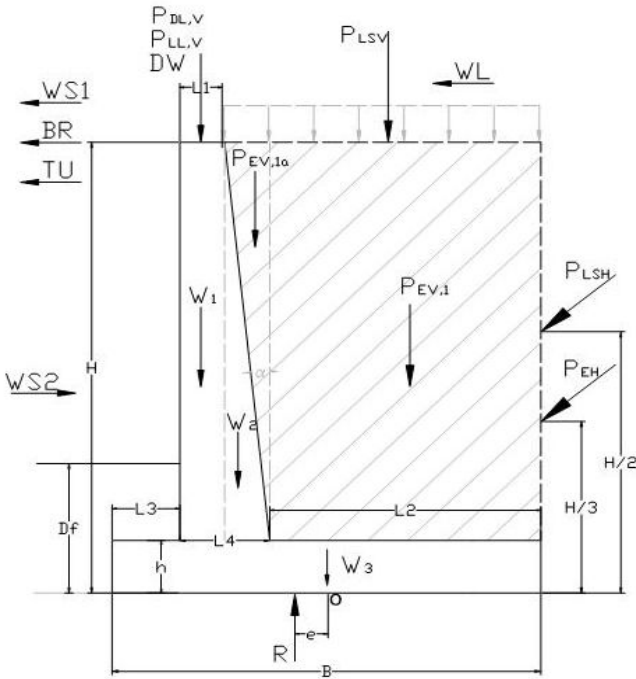
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Forward Abutment

LRFD

BEARING RESISTANCE CALCULATION FOR CANTILEVER WALL

Ref: {AASHTO; LRFD BRIDGE DESIGN SPECIFICATIONS}



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	0	psf	Cohesion	Foundation soil
ϕ	=	36	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	36	deg.	Friction ang.	Foundation soil

Unfactored Loads and Parameters

TU	=	643.0	lb/ft	Temperature Load
WS1	=	179.1	lb/ft	Wind Loads on Superstructure
WS2	=	0.0	lb/ft	Wind Load on Substructure
WL	=	21.0	lb/ft	Wind on Live Load
BR	=	0.0	lb/ft	Braking Force
P_{LSV}/P_{LSH}	=	250	psf	Traffic Load
$P_{DL,V}$	=	31,420	lb/ft	Bridge Dead Load + Stem Wght (W1)
$P_{LL,V}$	=	5,430	lb/ft	Bridge Live Load
DW	=	2,240.0	lb/ft	Dead Load of Wearing Surf./Utilities
B	=	20.0	ft	Width of footing
L1	=	4.3	ft	Width of stem
L2	=	8.5	ft	Length of heel
L3	=	7.3	ft	Length of toe
L4	=	4.3	ft	Base of stem width
h	=	3.0	ft	Thickness of footing
D_f	=	4.0	ft	Embedment depth
Dw	=	4.0	ft	Groundwater depth
H-D	=	27.9	ft	Height Above Ground Surface
H	=	31.9	ft	Total height of wall
K_a	=	0.30		
B'	=	19.7	ft	
γ'	=	57.6	pcf	
α	=	0.0	degree	Back face batter angle
δ	=	20.0	degree	Embankment Interface Friction Ang.

Max Factored Uniform Bearing Pressure

From Table 2 on following page,

$$q_{uni} = \underline{\underline{5,971 \text{ psf}}}$$

Factored Unit Bearing Resistance, q_r (Undrained)

$$q_{ULT} = cN_c + \gamma D_f N_q C_{wq} + 0.5 \gamma B' N_\gamma C_{w\gamma}$$

$$q_{ULT} = \text{N/A psf}$$

$$q_R = \phi q_{ULT}$$

$$^1 \phi = 0.55$$

$$q_r = \text{N/A psf}$$

Undrained Bearing resistance is N/A

Factored Unit Bearing Resistance, q_r (Drained)

$$q_{ULT} = cN_c + \gamma D_f N_q C_{wq} + 0.5 \gamma B' N_\gamma C_{w\gamma}$$

$$q_{ULT} = 51,419 \text{ psf}$$

$$q_R = \phi q_{ULT}$$

$$^1 \phi = 0.55$$

$$q_r = 28,280 \text{ psf}$$

Drained Bearing resistance is OK

Bearing Capacity Factors for Equations

	Undrained	Drained (AASHTO Table 10.6.3.1.2a-1)
N_c	50.60	50.60
N_q	37.80	37.80
N_γ	56.30	56.30

AASHTO Table 10.6.3.1.2a-2

$$C_{wq} = 1.0 \quad C_{w\gamma} = 0.5$$

¹ Resistance factors from AASHTO LRFD, Table 11.5.7-1

Client ms consultants/ ODOT D-6
 Project FRA-70-14.48
 Item Stability Calculations
 Bridge 1716C Forward Abutment

JOB NUMBER 1021-1005.01
 SHEET NO. 2 of 2
 COMP. BY EWT Date 11/14/2013
 CHECKED BY TAH Date 11/14/2013

LRFD

STABILITY CALCULATIONS FOR CANTILEVER WALL
 Ref: (AASHTO; LRFD BRIDGE DESIGN SPECIFICATIONS)

Summary of Unfactored Loads

Item	Force (lb/ft)	Arm (ft)	M (ft-lb/ft)
P _{LSV}	2,125	5.75	12,219
P _{LSH,b}	2,228	-15.95	-35,538
P _{LSH,v}	811	10	8,110
P _{EV,I}	29,478	5.75	169,499
P _{EV,la}	0	1.50	0
P _{LL,V}	5,430	-0.63	-3,394
P _{DL,V}	31,420	-0.63	-19,638
P _{EH,b}	17,058	-10.63	-181,386
P _{EH,v}	6,209	10.00	62,087
W1	0	-0.63	0
W2	0	1.50	0
W3	9,000	0.00	0
WS1	179	-34.20	-6,125
WS2	0	12.85	0
WL	21	-37.90	-796
TU	643	-25.70	-16,525
DW	2,240	-0.63	-1,400
BR	0	-31.90	0

Load Factors and Load Combinations

Group	EV	EH	LS	DC	TU	WS	DW	BR	WL
Strength I-a	1.00	1.50	1.75	0.90	0.50	0.00	0.65	1.75	0.00
Strength I-b	1.35	1.50	1.75	1.25	0.50	0.00	1.50	1.75	0.00
Service I	1.00	1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00
Strength IV	1.35	1.50	0.00	1.50	0.50	0.00	1.50	0.00	0.00

Vertical Factored Loads (lbs/ft width)			Vertical Factored Moments, M _v (ft-lbs/ft)		
Group	P _{LSH,v}	P _{EH,v}	Group	P _{LSH,v}	P _{EH,v}
Strength I-a	1,419	9,313	Strength I-a	14,192	93,130
Strength I-b	1,419	9,313	Strength I-b	14,192	93,130
Service I	811	6,209	Service I	8,110	62,087
Strength IV	0	9,313	Strength IV	0	93,130

Vertical Factored Loads (lbs per ft width)

Group	P _{EV,I}	P _{EV,la}	P _{LL,V} , P _{LSV}	W1 (DC1)	W2 (DC2)	W3 (DC3)	P _{DL,V}	DW	Total
Strength I-a	29,478	0	13,221	0	0	8,100	28,278	1,456	91,265
Strength I-b	39,795	0	13,221	0	0	11,250	39,275	3,360	117,634
Service I	29,478	0	7,555	0	0	9,000	31,420	2,240	86,713
Strength IV	39,795	0	0	0	0	13,500	47,130	3,360	113,098

Horizontal Factored Loads (lbs per ft width)

Group	P _{EH,b}	P _{LSH,b}	TU	WS1	WS2	BR	WL	Total
Strength I-a	25,587	3,899	322	0	0	0	0	29,808
Strength I-b	25,587	3,899	322	0	0	0	0	29,808
Service I	17,058	2,228	643	54	0	0	21	20,004
Strength IV	25,587	0	322	0	0	0	0	25,909

Vertical Factored Moments, M_v (ft-lbs/ft)

Group	P _{EV,I}	P _{EV,la}	P _{LL,V} , P _{LSV}	W1 (DC1)	W2 (DC2)	W3 (DC3)	P _{DL,V}	DW	Total
Strength I-a	169,499	0	15,444	0	0	0	-17,674	-910	273,680
Strength I-b	228,823	0	15,444	0	0	0	-24,547	-2,100	324,942
Service I	169,499	0	8,825	0	0	0	-19,638	-1,400	227,482
Strength IV	228,823	0	0	0	0	0	-29,456	-2,100	290,397

Horizontal Factored Moments, M_h (ft-lbs/ft)

Group	P _{EH,b}	P _{LSH,b}	TU	WS1	WS2	BR	WL	Total
Strength I-a	-272,079	-62,191	-8,263	0	0	0	0	-342,532
Strength I-b	-272,079	-62,191	-8,263	0	0	0	0	-342,532
Service I	-181,386	-35,538	-16,525	-1,838	0	0	-1,400	-236,686
Strength IV	-272,079	0	-8,263	0	0	0	0	-280,341

Check Overturning (Eccentricity)³

Table 1

Group	V _{TOT} (lb/ft)	M _v (ft-lbs/ft)	M _h (ft-lbs/ft)	e (ft)	B/3 (ft)
Strength I-a	86,128	238,106	-342,532	1.21	6.67
*Strength IV	113,098	290,397	-280,341	0.09	6.67

* Strength IV does not apply (Superstructure DL/LL<.7.0)

$e_{all\ cases}$ is less than Cal. $e_{max} = 1.21$ B/3= 6.67 $e_{max} = B/3$ Overturning is **OK**
 Check if Cal. $e_{max} \leq B/3 =$ **YES** Inside middle 2/3

Check Sliding*

Factored resistance against failure by sliding

Because of the potential for loss of soil in front of wall, assume:

Check sliding using Strength I-a loading

$R_R = \phi R_n = \phi (R_t + \phi_{ep} R_{ep})$
 $\phi_{ep} R_{ep} = 0$

Cohesionless Soil:

$R_t = V \tan \phi_r$
 $\tan \phi_r = 0.73$ $R_t = 55,671$ lbs/ft

Cohesive Soil: $R_t =$ area under q_s diagram (pg3)

$R_t =$ N/A lbs/ft

$\phi_r = 1$ $R_R = 55,671$ lbs/ft

For $R_R = 55,671$ lbs/ft Active Driving Horizontal Force, $H_{TOTAL} = 29,808$ lbs/ft Sliding is **OK**

Check Bearing Pressure

Table 2

Group	V _{TOT} (lb/ft)	M _v (ft-lbs/ft)	M _h (ft-lbs/ft)	e (ft)	B'	q _{uniform} (psf)	e (ft)
Strength I-b	117,634	324,942	-342,532	0.15	19.70	5,971	0.15
Service I	86,713	227,482	-236,686	0.11	19.79	4,382	0.11
*Strength IV	113,098	290,397	-280,341	0.09	19.82	5,706	0.09

Max q_{uniform} (psf) = 5,971

¹ Resistance factors from AASHTO LRFD, Table 11.5.7-1

² Load factors and load combinations from AASHTO LRFD, Table 3.4.1-1, Table 3.4.1-2 & Section C3.4.1 pg 3-11 & 3-12

³ Vertical Traffic Loads (P_{LSV} & P_{LSH,v}) & horizontal wind loads (WS) are not included in overturning analysis

* Live Vertical Loads (P_{LL,V}, P_{LSV}, & P_{LSH,v}) & horizontal wind loads (WS) are not included in sliding analysis

* Strength IV does not apply (Superstructure DL/LL<.7.0)

CLIENT ODOT D-6/ms consultants
PROJECT East Interchange
SUBJECT Bridge 1716
Settlement - Forward Abutment

PROJECT NO. 1021-1005.01
SHEET NO. 1 OF 1
COMP. BY BTA DATE 6-5-13
CHECKED BY SWT DATE 6-12-13

Load on forward Abutment

$$LL = 5,430 \text{ lbs/ft}$$

$$DL = 31,420 \text{ lbs/ft}$$

$$DW = 2,240 \text{ lbs/ft}$$

$$\underline{\underline{39,090 \text{ lbs/ft}}}$$

Stem weight

$$\text{per spreadsheet} = 18,424 \text{ lbs/ft}$$

$$\text{Total load} = 39,090 - 18,424 = \underline{\underline{20,666 \text{ lbs/ft}}}$$

$$B' = 19.79$$

$$\underline{\underline{\text{Pressure} = 1,044 \text{ psf}}}$$

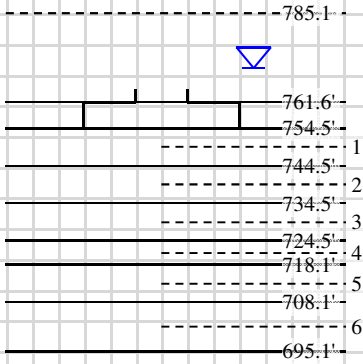
Fill height = 0.8' → negligible settlement

$$\text{Settlement from loadings} = 0.6''$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-71-1716- Forward Abutment
 Boring Overburden

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 5
 COMP. BY BTA DATE 06/05/13
 CHECKED BY EWT DATE 06/06/13

Based on Boring B-287-2-10	Existing Fndn Load ^a	psf	Bearing Depth	754.5 ft	Water Elev.	770.6 ft
-785.1'	Layer	Zone (ft)	Mid-Pt Elev (ft)	γ_m (pcf)	Hc (ft)	
	A	785.1 to 761.6	-	130	-	
-761.6'	B	761.6 to 754.5	-	120	-	
-754.5'	C	754.5 to 744.5	749.5	120	10	
-744.5'	D	744.5 to 734.5	739.5	120	10	
-734.5'	E	734.5 to 724.5	729.5	120	10	
-724.5'	F	724.5 to 718.1	721.3	120	6.4	
-718.1'	G	718.1 to 708.1	713.1	130	10	
-708.1'	H	708.1 to 695.1	701.6	130	13	
-695.1'						

Calculate Existing Load

Point 1

$$\begin{aligned} \sigma'_{o1} &= 2902 \text{ psf} + (754.5' - 749.5')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2902 \text{ psf} + (5.0')(57.6 \text{ pcf}) \\ &= 3190 \text{ psf} \end{aligned}$$

Point 2

$$\begin{aligned} \sigma'_{o2} &= 3190 \text{ psf} + (749.5' - 739.5')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3190 \text{ psf} + (10.0')(57.6 \text{ pcf}) \\ &= 3766 \text{ psf} \end{aligned}$$

Point 3

$$\begin{aligned} \sigma'_{o3} &= 3766 \text{ psf} + (739.5' - 729.5')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3766 \text{ psf} + (10.0')(57.6 \text{ pcf}) \\ &= 4342 \text{ psf} \end{aligned}$$

Point 4

$$\begin{aligned} \sigma'_{o4} &= 4342 \text{ psf} + (729.5' - 721.3')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4342 \text{ psf} + (8.2')(57.6 \text{ pcf}) \\ &= 4814 \text{ psf} \end{aligned}$$

Point 5

$$\begin{aligned} &= 4814 \text{ psf} + (721.3' - 718.1')(120 \text{ pcf} - 62.4 \text{ pcf}) + (718.1' - 713.1')(130 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4814 \text{ psf} + (3.2')(57.6 \text{ pcf}) + (5.0')(67.6 \text{ pcf}) \\ &= 5336 \text{ psf} \end{aligned}$$

Point 6

$$\begin{aligned} &= 5336 \text{ psf} + (713.1' - 701.6')(130 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 5336 \text{ psf} + (11.5')(67.6 \text{ pcf}) \\ &= 6113 \text{ psf} \end{aligned}$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-71-1716- Forward Abutment
 Existing Overburden

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 5
 COMP. BY BTA DATE 06/05/13
 CHECKED BY EWT DATE 06/06/13

Based on Boring B-287-2-10	Existing Fndn Load"		Bearing Depth	754.5 ft	Water Elev.	770.6 ft
	Layer	Zone (ft)	Mid-Pt Elev (ft)	γ_m (pcf)	Hc (ft)	
-----785.6'	A	785.6 to 761.6	-	130	-	
-----761.6'	B	761.6 to 754.5	-	120	-	
-----754.5'	C	754.5 to 744.5	749.5	120	10	
-----744.5'	D	744.5 to 734.5	739.5	120	10	
-----734.5'	E	734.5 to 724.5	729.5	120	10	
-----724.5'	F	724.5 to 718.1	721.3	120	6.4	
-----718.1'	G	718.1 to 708.1	713.1	130	10	
-----708.1'	H	708.1 to 695.1	701.6	130	13	
-----695.1'						

Calculate Existing Load

Point 1

$$\begin{aligned} \sigma'_{o1} &= 2967 \text{ psf} + (754.5' - 749.5')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2967 \text{ psf} + (5.0')(57.6 \text{ pcf}) \\ &= 3255 \text{ psf} \end{aligned}$$

Point 2

$$\begin{aligned} \sigma'_{o2} &= 3255 \text{ psf} + (749.5' - 739.5')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3255 \text{ psf} + (10.0')(57.6 \text{ pcf}) \\ &= 3831 \text{ psf} \end{aligned}$$

Point 3

$$\begin{aligned} \sigma'_{o3} &= 3831 \text{ psf} + (739.5' - 729.5')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3831 \text{ psf} + (10.0')(57.6 \text{ pcf}) \\ &= 4407 \text{ psf} \end{aligned}$$

Point 4

$$\begin{aligned} \sigma'_{o4} &= 4407 \text{ psf} + (729.5' - 721.3')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4407 \text{ psf} + (8.2')(57.6 \text{ pcf}) \\ &= 4879 \text{ psf} \end{aligned}$$

Point 5

$$\begin{aligned} &= 4879 \text{ psf} + (721.3' - 718.1')(120 \text{ pcf} - 62.4 \text{ pcf}) + (718.1' - 713.1')(130 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4879 \text{ psf} + (3.2')(57.6 \text{ pcf}) + (5.0')(67.6 \text{ pcf}) \\ &= 5401 \text{ psf} \end{aligned}$$

Point 6

$$\begin{aligned} &= 5401 \text{ psf} + (713.1' - 701.6')(130 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 5401 \text{ psf} + (11.5')(67.6 \text{ pcf}) \\ &= 6178 \text{ psf} \end{aligned}$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-71-1716- Forward Abutment
 Proposed Overburden

JOB NUMBER 1021-1005.01
 SHEET NO. 3 OF 5
 COMP. BY BTA DATE 06/05/13
 CHECKED BY EWT DATE 06/06/13

Based on Boring B-287-2-10	Foundation Load ^a	1044 psf	Bearing Depth	754.5 ft	Water Elev.	754.5 ft
785.6	Layer	Zone (ft)	Mid-Pt Elev (ft)	γ_m (pcf)	Hc (ft)	i^*
	A	785.6 to 754.5	-	120	-	
	B	754.5 to 744.5	749.5	120	10	1.00
	C	744.5 to 734.5	739.5	120	10	0.65
	D	734.5 to 724.5	729.5	120	10	0.45
	E	724.5 to 718.1	721.3	120	6.4	0.36
	F	718.1 to 708.1	713.1	130	10	0.30
	G	708.1 to 695.1	701.6	130	13	0.24

Calculate Proposed Load

^a - *fdn load at center of footing*

^{*} - *see settlement calcs for determination of i values*

Point 1

$$\begin{aligned} \sigma'_{o1} &= 3732 \text{ psf} + (754.5' - 749.5')(120 \text{ pcf} - 62.4 \text{ pcf}) + (1044 \text{ psf})(1) \\ &= 3732 \text{ psf} + (5.0')(57.6 \text{ pcf}) + (1044 \text{ psf}) \\ &= 5064 \text{ psf} \end{aligned}$$

Point 2

$$\begin{aligned} \sigma'_{o2} &= 4020 \text{ psf} + (749.5' - 739.5')(120 \text{ pcf} - 62.4 \text{ pcf}) + (1044 \text{ psf})(0.65) \\ &= 4020 \text{ psf} + (10.0')(57.6 \text{ pcf}) + (679 \text{ psf}) \\ &= 5274.6 \text{ psf} \end{aligned}$$

Point 3

$$\begin{aligned} \sigma'_{o3} &= 4596 \text{ psf} + (739.5' - 729.5')(120 \text{ pcf} - 62.4 \text{ pcf}) + (1044 \text{ psf})(0.45) \\ &= 4596 \text{ psf} + (10.0')(57.6 \text{ pcf}) + (470 \text{ psf}) \\ &= 5641.8 \text{ psf} \end{aligned}$$

Point 4

$$\begin{aligned} \sigma'_{o4} &= 5172 \text{ psf} + (729.5' - 721.3')(120 \text{ pcf} - 62.4 \text{ pcf}) + (1044 \text{ psf})(0.36) \\ &= 5172 \text{ psf} + (8.2')(57.6 \text{ pcf}) + (376 \text{ psf}) \\ &= 6019.84 \text{ psf} \end{aligned}$$

Point 5

$$\begin{aligned} &= 5644 \text{ psf} + (721.3' - 718.1')(120 \text{ pcf} - 62.4 \text{ pcf}) + (718.1' - 713.1')(130 \text{ pcf} - 62.4 \text{ pcf}) + (1044 \text{ psf})(0.3) \\ &= 5644 \text{ psf} + (3.2')(57.6 \text{ pcf}) + (5.0')(67.6 \text{ pcf}) + (313 \text{ psf}) \\ &= 6479.2 \text{ psf} \end{aligned}$$

Point 6

$$\begin{aligned} &= 6166 \text{ psf} + (713.1' - 701.6')(130 \text{ pcf} - 62.4 \text{ pcf}) + (1044 \text{ psf})(0.24) \\ &= 6166 \text{ psf} + (11.5')(67.6 \text{ pcf}) + (251 \text{ psf}) \\ &= 7193.56 \text{ psf} \end{aligned}$$

σ'_{o7}



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-71-1716- Forward Abutment
 Settlement

JOB NUMBER 1021-1005.01
 SHEET NO. 4 OF 5
 COMP. BY BTA DATE 06/05/13
 CHECKED BY EWT DATE 06/06/13

Proposed Loading										
Assume loading =		1044 psf								
Assume width =		19.8 ft								
0										
Point 1	(749.5')	Depth=	(754.5' - 749.5')	= 5.0'	→	$\frac{5.0'}{19.8'}$	= 0.25B	→	i = 1.00	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{3.2}\right) = 0.85$	$N_{60} = 64$	$N_{160} = 64 * 0.85 = 54.1$					
		$C' = 185$	Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand and gravel"					
		$\Delta H = 10.0' \frac{1}{185} \log\left(\frac{5.1 \text{ ksf}}{3.3 \text{ ksf}}\right) = 0.010 \text{ ft} = 0.12 \text{ in}$	$\Sigma H = 0.12 \text{ in}$							
0										
Point 2	(739.5')	Depth=	(754.5' - 739.5')	= 15.0'	→	$\frac{15.0'}{19.8'}$	= 0.76B	→	i = 0.65	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{3.8}\right) = 0.79$	$N_{60} = 73$	$N_{160} = 73 * 0.79 = 57.6$					
		$C' = 202$	Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand and gravel"					
		$\Delta H = 10.0' \frac{1}{202} \log\left(\frac{5.3 \text{ ksf}}{3.8 \text{ ksf}}\right) = 0.007 \text{ ft} = 0.08 \text{ in}$	$\Sigma H = 0.21 \text{ in}$							
0										
Point 3	(729.5')	Depth=	(754.5' - 729.5')	= 25.0'	→	$\frac{25.0'}{19.8'}$	= 1.26B	→	i = 0.45	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{4.3}\right) = 0.74$	$N_{60} = 63$	$N_{160} = 63 * 0.74 = 46.7$					
		$C' = 157$	Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand and gravel"					
		$\Delta H = 10.0' \frac{1}{157} \log\left(\frac{5.6 \text{ ksf}}{4.4 \text{ ksf}}\right) = 0.007 \text{ ft} = 0.08 \text{ in}$	$\Sigma H = 0.29 \text{ in}$							
0										
Point 4	(721.3')	Depth=	(754.5' - 721.3')	= 33.2'	→	$\frac{33.2'}{19.8'}$	= 1.68B	→	i = 0.36	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{4.8}\right) = 0.71$	$N_{60} = 86$	$N_{160} = 86 * 0.71 = 60.9$					
		$C' = 217$	Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand and gravel"					
		$\Delta H = 6.4' \frac{1}{217} \log\left(\frac{6.0 \text{ ksf}}{4.9 \text{ ksf}}\right) = 0.003 \text{ ft} = 0.03 \text{ in}$	$\Sigma H = 0.32 \text{ in}$							



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-71-1716- Forward Abutment
 Settlement

JOB NUMBER 1021-1005.01
 SHEET NO. 5 OF 5
 COMP. BY BTA DATE 06/05/13
 CHECKED BY EWT DATE 06/06/13

Proposed Loading

Assume loading = 1044 psf
 Assume width = 19.8 ft

Point 5 (713.1') Depth= (754.5' - 713.1') = 41.4' $\rightarrow \frac{41.4'}{19.8'} = 2.09B \rightarrow i = 0.30$ Fig 10.6.2.4.1-1a (10-49)

Cohesive $\rightarrow S_c = \frac{H_c}{1 + e_o} C_r \log\left(\frac{\sigma'_f}{\sigma'_o}\right) \quad C_r = \frac{WC}{1000} = \frac{15}{1000} = 0.015 \quad e_o = \frac{2.75 * WC}{100} = \frac{2.75 * 15}{100} = 0.413$

C_r, e_o - FHWA Soils and Foundations (PG 152)

$S_c = \frac{10.0 \text{ ft}}{1 + 0.4125} \cdot 0.015 \cdot \log\left(\frac{6.5 \text{ ksf}}{5.4 \text{ ksf}}\right) = 0.008 \text{ ft} = 0.10 \text{ in} \quad \Sigma H = 0.42 \text{ in}$

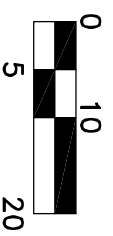
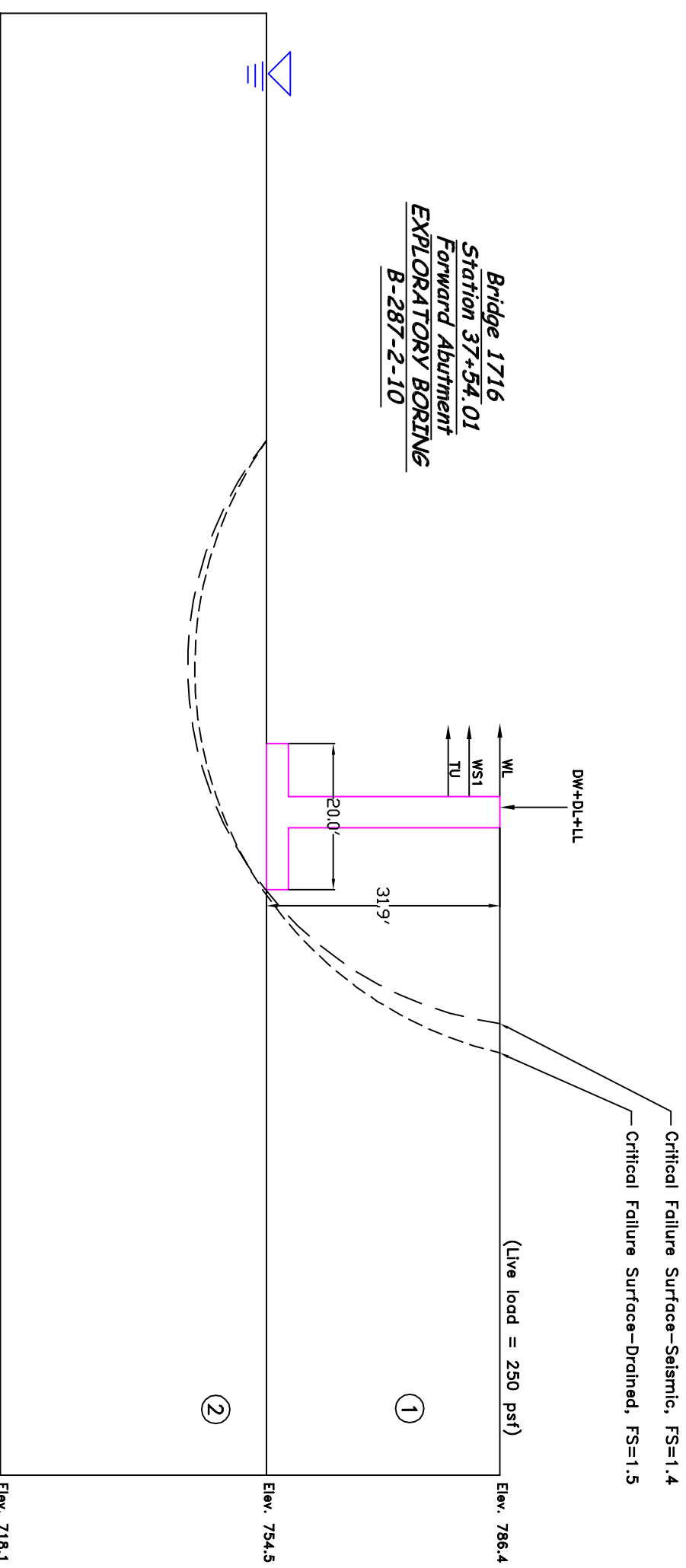
Point 6 (701.6') Depth= (754.5' - 701.6') = 52.9' $\rightarrow \frac{52.9'}{19.8'} = 2.67B \rightarrow 0.24$ Fig 10.6.2.4.1-1a (10-49)

Cohesive $\rightarrow S_c = \frac{H_c}{1 + e_o} C_r \log\left(\frac{\sigma'_f}{\sigma'_o}\right) \quad C_r = \frac{WC}{1000} = \frac{20}{1000} = 0.020 \quad e_o = \frac{2.75 * WC}{100} = \frac{2.75 * 20}{100} = 0.550$

C_r, e_o - FHWA Soils and Foundations (PG 152)

$S_c = \frac{13.0 \text{ ft}}{1 + 0.55} \cdot 0.02 \cdot \log\left(\frac{7.2 \text{ ksf}}{6.2 \text{ ksf}}\right) = 0.011 \text{ ft} = 0.13 \text{ in} \quad \Sigma H = 0.55 \text{ in}$

Material	Soil Type	c (psf)	ϕ (deg)	c' (psf)	ϕ' (deg)	γ (pcf)
Material 1	Embankment Fill	0	30	0	30	120
Material 2	V. Dense A-1-b	0	36	0	36	120



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GLOBAL STABILITY EVALUATION

FRA-70-14.48
 FRA-71-1716 BRIDGE

STATION 37+54.01
 Global Stability – Forward Abutment

PROJECT NO. 1021-1005.01 | CALC: BTA | DATE 06/11/13

Pier 1

13-Jun-13

Bridge 1716

PIER #1

EHmoment acts as a driving force, all other horizontal loads as driving forces, and vertical loads times eccentricity as resisting forces per ms e-mail dated June 11, 2013

Footing dimensions provided by ms:

Service Case Factors	
WL	1
WS	0.3
TU	1
LL	1
LS	1
DC	1
EH	1
CE	1
DW	1
BR	1

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	359	4,445	632	1,384	0	882	359	4,445	632	1384	0	882
LL Case 2	583	4,445	632	1,384	0	882	583	4,445	632	1384	0	882
LL Case 3	457	4,445	632	1,384	0	882	457	4,445	632	1384	0	882

Initial eccentricity provided by ms=	3.5 ft
Footing length=	105.0 ft
Footing width=	14.0 ft
Toe width=	9.0 ft
Heel width=	2.0 ft
Pier column width=	3.0 ft

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment	Transverse Loading
LL Case 1	45	0	0	167	285	14,478	6,580	392	0	0	
LL Case 2	45	0	0	271	285	14,704	6,580	392	0	0	
LL Case 3	45	0	0	213	285	16,152	6,580	392	0	0	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	Factored Transverse Loading
LL Case 1	45	0	0	167	285	14,478	6,580	392	0	0	
LL Case 2	45	0	0	271	285	14,704	6,580	392	0	0	
LL Case 3	45	0	0	213	285	16,152	6,580	392	0	0	

LL Case 1	W =	14.00
	L =	105.00
LL Case 2	W =	14.00
	L =	105.00
LL Case 3	W =	14.00
	L =	105.00

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment	Longitudinal Loading
LL Case 1	173	1,476	1,476	23	2,058	824	0	0	0	0	
LL Case 2	173	1,476	1,476	38	2,058	824	0	0	0	0	
LL Case 3	173	1,476	1,476	29	2,058	824	0	0	0	0	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	Factored Longitudinal Loading
LL Case 1	173	443	1,476	23	2,058	824	0	0	0	0	
LL Case 2	173	443	1,476	38	2,058	824	0	0	0	0	
LL Case 3	173	443	1,476	29	2,058	824	0	0	0	0	

LL Case 1	W' =	9.10
	L' =	99.30
LL Case 2	W' =	9.04
	L' =	99.38
LL Case 3	W' =	9.07
	L' =	98.93

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	Factored Transverse Loading
LL Case 1	21,947	7,702	2.85	
LL Case 2	22,277	7,926	2.81	
LL Case 3	23,667	7,800	3.03	

Loading Pressure
Based on Effective Footing Size

LL Case 1	8,524 psf
LL Case 2	8,819 psf
LL Case 3	8,689 psf

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	Factored Longitudinal Loading
LL Case 1	-18,873	7,702	-2.45	
LL Case 2	-19,642	7,926	-2.48	
LL Case 3	-19,210	7,800	-2.46	

Footing thickness h= 4.00

UNFACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment	Longitudinal Loading
LL Case 1	-1,257	-15,558	-2,212	-4,844	
LL Case 2	-2,041	-15,558	-2,212	-4,844	
LL Case 3	-1,600	-15,558	-2,212	-4,844	

FACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment	Longitudinal Loading
LL Case 1	-1,257	-15,558	-2,212	-4,844	
LL Case 2	-2,041	-15,558	-2,212	-4,844	
LL Case 3	-1,600	-15,558	-2,212	-4,844	

13-Jun-13

Bridge 1716

PIER #1

EHmoment acts as a driving force, all other horizontal loads as driving forces, and vertical loads times eccentricity as resisting forces per ms e-mail dated June 11, 2013

Footing dimensions provided by ms:

Strength I-a Case Factors

WL	0
WS	0
TU	0.5
LL	1.75
LS	1.75
DC	0.9
EH	1.5
CE	1.75
DW	0.65
BR	1.75

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	359	4,445	632	1,384	0	882	628	4,001	410.8	1245.6	0	793.8
LL Case 2	583	4,445	632	1,384	0	882	1,020	4,001	410.8	1245.6	0	793.8
LL Case 3	457	4,445	632	1,384	0	882	800	4,001	410.8	1245.6	0	793.8

Initial eccentricity provided by ms=	3.5 ft
Footing length=	105.0 ft
Footing width=	14.0 ft
Toe width=	9.0 ft
Heel width=	2.0 ft
Pier column width=	3.0 ft

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	45	0	0	167	285	14,478	6,580	392	0	0
LL Case 2	45	0	0	271	285	14,704	6,580	392	0	0
LL Case 3	45	0	0	213	285	16,152	6,580	392	0	0

Transverse Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	0	0	0	292	143	25,337	5,922	255	0	0
LL Case 2	0	0	0	474	143	25,732	5,922	255	0	0
LL Case 3	0	0	0	373	143	28,266	5,922	255	0	0

Factored Transverse Loading

LL Case 1	W =	14.00
	L =	105.00
LL Case 2	W =	14.00
	L =	105.00
LL Case 3	W =	14.00
	L =	105.00

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	173	1,476	1,476	23	2,058	824	0	0	0	0
LL Case 2	173	1,476	1,476	38	2,058	824	0	0	0	0
LL Case 3	173	1,476	1,476	29	2,058	824	0	0	0	0

Longitudinal Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	0	0	2,214	40	1,029	1,442	0	0	0	0
LL Case 2	0	0	2,214	67	1,029	1,442	0	0	0	0
LL Case 3	0	0	2,214	51	1,029	1,442	0	0	0	0

Factored Longitudinal Loading

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	e _{max} *	e < e _{max}
LL Case 1	31,948	7,079	4.51	35.00	Yes
LL Case 2	32,526	7,471	4.35	35.00	Yes
LL Case 3	34,958	7,250	4.82	35.00	Yes

Factored Transverse Loading

**Loading Pressure
Based on Effective Footing Size**

LL Case 1	8,088 psf
LL Case 2	8,606 psf
LL Case 3	8,381 psf

LL Case 1	W' =	9.12
	L' =	95.97
LL Case 2	W' =	9.02
	L' =	96.29
LL Case 3	W' =	9.07
	L' =	95.36

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	e _{max} *	ABS(e) < e _{max}
LL Case 1	-17,273	7,079	-2.44	4.67	Yes
LL Case 2	-18,619	7,471	-2.49	4.67	Yes
LL Case 3	-17,863	7,250	-2.46	4.67	Yes

Factored Longitudinal Loading

*Eccentricity Check AASHTO 10.6.3.3, 6th Edition

Footing thickness h= 4.00

UNFACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	-1,257	-15,558	-2,212	-4,844
LL Case 2	-2,041	-15,558	-2,212	-4,844
LL Case 3	-1,600	-15,558	-2,212	-4,844

Longitudinal Loading

FACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	-2,199	-14,002	-1,438	-4,360
LL Case 2	-3,571	-14,002	-1,438	-4,360
LL Case 3	-2,799	-14,002	-1,438	-4,360

Longitudinal Loading

13-Jun-13

Bridge 1716

PIER #1

EHmoment acts as a driving force, all other horizontal loads as driving forces, and vertical loads times eccentricity as resisting forces per ms e-mail dated June 11, 2013

Footing dimensions provided by ms:

Strength I-b Case Factors

WL	0
WS	0
TU	0.5
LL	1.75
LS	1.75
DC	1.25
EH	1.5
CE	1.75
DW	1.5
BR	1.75

LL Case 1	W =	14.00
	L =	105.00
LL Case 2	W =	14.00
	L =	105.00
LL Case 3	W =	14.00
	L =	105.00

LL Case 1	W' =	8.72
	L' =	98.06
LL Case 2	W' =	8.66
	L' =	98.21
LL Case 3	W' =	8.70
	L' =	97.58

Footing thickness h= 4.00

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	359	4,445	632	1,384	0	882	628	5,556	948	1730	0	1102.5
LL Case 2	583	4,445	632	1,384	0	882	1,020	5,556	948	1730	0	1102.5
LL Case 3	457	4,445	632	1,384	0	882	800	5,556	948	1730	0	1102.5

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	45	0	0	167	285	14,478	6,580	392	0	0
LL Case 2	45	0	0	271	285	14,704	6,580	392	0	0
LL Case 3	45	0	0	213	285	16,152	6,580	392	0	0

Transverse Loading

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	173	1,476	1,476	23	2,058	824	0	0	0	0
LL Case 2	173	1,476	1,476	38	2,058	824	0	0	0	0
LL Case 3	173	1,476	1,476	29	2,058	824	0	0	0	0

Longitudinal Loading

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e
LL Case 1	34,584	9,965	3.47
LL Case 2	35,162	10,357	3.39
LL Case 3	37,594	10,137	3.71

Factored Transverse Loading

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e
LL Case 1	-26,294	9,965	-2.64
LL Case 2	-27,639	10,357	-2.67
LL Case 3	-26,883	10,137	-2.65

Factored Longitudinal Loading

UNFACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	-1,257	-15,558	-2,212	-4,844
LL Case 2	-2,041	-15,558	-2,212	-4,844
LL Case 3	-1,600	-15,558	-2,212	-4,844

Longitudinal Loading

Initial eccentricity provided by ms= 3.5 ft
 Footing length= 105.0 ft
 Footing width= 14.0 ft
 Toe width= 9.0 ft
 Heel width= 2.0 ft
 Pier column width= 3.0 ft

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	0	0	0	292	143	25,337	8,225	588	0	0
LL Case 2	0	0	0	474	143	25,732	8,225	588	0	0
LL Case 3	0	0	0	373	143	28,266	8,225	588	0	0

Factored Transverse Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	0	0	2,214	40	1,029	1,442	0	0	0	0
LL Case 2	0	0	2,214	67	1,029	1,442	0	0	0	0
LL Case 3	0	0	2,214	51	1,029	1,442	0	0	0	0

Factored Longitudinal Loading

Loading Pressure Based on Effective Footing Size

LL Case 1	11,650 psf
LL Case 2	12,174 psf
LL Case 3	11,946 psf

FACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	-2,199	-19,447	-3,318	-6,055
LL Case 2	-3,571	-19,447	-3,318	-6,055
LL Case 3	-2,799	-19,447	-3,318	-6,055

Longitudinal Loading



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 1 (EHd & LL1)

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	739.0 ft	Foot. Width (B') =	8.7 ft	Foot. Length (L') =	98.1 ft			
Unfactored Dead Load =	6711.00 kips	Unfactored Live Load =	359.00 kips	Load =	78.54	k/lf		
Unfactored DW =	632.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)						
Max Factored Dead Load =	8388.75 kips	Max Factored Live Load =	628.25 kips	F. Load =	101.62	k/lf		
Max Factored DW =	948.00 kips							
Calculate factored bearing resistance (q_R)				Reference: AASHTO LRFD Bridge Design Specifications				
q _R = φ _b q _n				Eq. 10.6.3.1.1-1				
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2α-1, 2,3,4				
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0				
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.37}$) = 0.95				Eq. 10.4.6.2.4-1				
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 63 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3				
N ₁₆₀ = 59.5 → φ _f = 38° to 43° Use φ _f = 36°				Table 10.4.6.2.4-1				
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2α-1				
Groundwater at 739.0 ft → (739' - 739') = 0.0'				Water is within 1.5B of bearing depth				
C _{wq} = 1.0, C _{wy} = 0.5				Table 10.6.3.1.2α-2				
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{8.7'}{98.1'}$) = 0.96				s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{8.7'}{98.1'}$ tan 36°) = 1.06				
dq = 1.0				Table 10.6.3.1.2α-4				
q _n = 0 + (120.0 pcf) (4.0 ft) (37.8) (1.06) (1.0) (1.0) + (0.5) (120.0 pcf) (8.7 ft) (56.3) (0.96) (0.5)								
q _n = 19316 + 14204				For a 8.7 ft wide footing (eff. width 8.7 ft) q _n = 33.5 ksf				
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2				
				Table 10.5.5.2.2-1				
q _R = φ _b q _n = (0.45) (33.5 ksf) = <u>15.1 ksf</u> > 11.7 ksf				(= 101.62 k/lf / 8.7 ft)				



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 1 (EHd & LL1)

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 *Eq. 10.6.3.4-1*

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ *Eq. 10.6.3.4-2*

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil *Section 10.6.3.4*
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):
 concrete on sand → $\phi_t = 0.80$ *Table 10.5.5.2.2-1*

Foot. Length (L') = 96.0 ft → Effective Footing Length based on Strength I-a
 Foot. Width (W') = 9.1 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	11.5	kips
Sum of Factored Longitudinal Loads=	1367.8	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	1367.8	kips
Sum of Factored Vertical loads, V_{TOT} =	7079.0	kips

$R_t = V_{TOT} \tan \phi_t$
 $R_t = 5143$ kips

For $R_R = 4,115$ kips

Active Driving Horizontal Force, $H_{TOT} = 1367.8$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a	Load Factors:
WL=	1.0 kips	WL=	5.0 kips	0	
WS=	28.0 kips	WS=	703.0 kips	0	
EH=	0.0 kips	EH=	703.0 kips (Driving force like	1.5	
CE+LS=	4.0 kips	CE+LS=	161.0 kips others)	1.75	CE and LS
TU=	9.0 kips	TU=	63.0 kips	0.5	
BR=	0 kips	BR=	0 kips	1.75	
Vertical Unfactored LL=	359.0 ft	kips		1.75	
Vertical Unfactored DL=	6711.0 ft	kips		0.9	
Verical Unfactored DW=	632.0 ft	kips		0.65	



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 1 (EHd & LL2)

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	739.0 ft	Foot. Width (B') =	8.7 ft	Foot. Length (L') =	98.2 ft			
Unfactored Dead Load =	6711.00 kips	Unfactored Live Load =	583.00 kips	Load =	80.70	k/lf		
Unfactored DW =	632.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)						
Max Factored Dead Load =	8388.75 kips	Max Factored Live Load =	1020.25 kips	F. Load =	105.46	k/lf		
Max Factored DW =	948.00 kips							
Calculate factored bearing resistance (q_R)				<i>Reference: AASHTO LRFD Bridge Design Specifications</i>				
q _R = φ _b q _n				Eq. 10.6.3.1.1-1				
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2α-1, 2,3,4				
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0				
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.37}$) = 0.95				Eq. 10.4.6.2.4-1				
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 63 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3				
N ₁₆₀ = 59.5 → φ _f = 38° to 43° Use φ _f = 36°				Table 10.4.6.2.4-1				
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2α-1				
Groundwater at 739.0 ft → (739' - 739') = 0.0'				Water is within 1.5B of bearing depth				
				C _{wq} = 1.0 , C _{wy} = 0.5		Table 10.6.3.1.2α-2		
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{8.7'}{98.2'}$) = 0.96				s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{8.7'}{98.2'}$ tan 36°) = 1.06				
dq = 1.0				Table 10.6.3.1.2α-4				
q _n = 0 + (120.0 pcf) (4.0 ft) (37.8) (1.06) (1.0) (1.0) + (0.5) (120.0 pcf) (8.7 ft) (56.3) (0.96) (0.5)								
q _n = 19306 + 14111				For a 8.7 ft wide footing (eff. width 8.7 ft) q _n = 33.4 ksf				
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2				
				Table 10.5.5.2.2-1				
q _R = φ _b q _n = (0.45) (33.4 ksf) = <u>15.0 ksf</u> > 12.2 ksf				(= 105.46 k/lf / 8.7 ft)				



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 1 (EHd & LL2)

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 Eq. 10.6.3.4-1

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ Eq. 10.6.3.4-2

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil Section 10.6.3.4
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):

concrete on sand → $\phi_t = 0.80$ Table 10.5.5.2.2-1

Foot. Length (L') = 96.3 ft → Effective Footing Length based on Strength I-a
 Foot. Width (W') = 9.0 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	15.0	kips
Sum of Factored Longitudinal Loads=	1367.8	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	1367.8	kips
Sum of Factored Vertical loads, V_{TOT} =	7471.0	kips

$R_t = V_{TOT} \tan \phi_t$
 $R_t = 5428$ kips

For $R_R = 4,342$ kips

Active Driving Horizontal Force, $H_{TOT} = 1367.8$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a Load Factors:	
WL=	1.0 kips	WL=	5.0 kips		0
WS=	28.0 kips	WS=	703.0 kips		0
EH=	0.0 kips	EH=	703.0 kips (Driving force like others)		1.5
CE+LS=	6.0 kips	CE+LS=	161.0 kips		1.75
TU=	9.0 kips	TU=	63.0 kips		0.5
BR=	0 kips	BR=	0 kips		1.75
Vertical Unfactored LL=	583.0 ft kips				1.75
Vertical Unfactored DL=	6711.0 ft kips				0.9
Vertical Unfactored DW=	632.0 ft kips				0.65



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 1 (EHd & LL3)

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	739.0 ft	Foot. Width (B') =	8.7 ft	Foot. Length (L') =	97.6 ft		
Unfactored Dead Load =	6711.00 kips	Unfactored Live Load =	457.00 kips	Load =	79.93 k/lf		
Unfactored DW =	632.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)					
Max Factored Dead Load =	8388.75 kips	Max Factored Live Load =	799.75 kips	F. Load =	103.88 k/lf		
Max Factored DW =	948.00 kips						
Calculate factored bearing resistance (q_R)				Reference: AASHTO LRFD Bridge Design Specifications			
q _R = φ _b q _n				Eq. 10.6.3.1.1-1			
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2α-1,2,3,4			
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0			
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.37}$) = 0.95				Eq. 10.4.6.2.4-1			
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 63 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3			
N ₁₆₀ = 59.5 → φ _f = 38° to 43° Use φ _f = 36°				Table 10.4.6.2.4-1			
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2α-1			
Groundwater at 739.0 ft → (739' - 739') = 0.0'				Water is within 1.5B of bearing depth			
C _{wq} = 1.0, C _{wy} = 0.5				Table 10.6.3.1.2α-2			
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{8.7'}{97.6'}$) = 0.96				s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{8.7'}{97.6'}$ tan 36°) = 1.06			
dq = 1.0				Table 10.6.3.1.2α-4			
q _n = 0 + (120.0 pcf) (4.0 ft) (37.8) (1.06) (1.0) (1.0) + (0.5) (120.0 pcf) (8.7 ft) (56.3) (0.96) (0.5)							
q _n = 19319 + 14170				For a 8.7 ft wide footing (eff. width 8.7 ft) q _n = 33.5 ksf			
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2			
				Table 10.5.5.2.2-1			
q _R = φ _b q _n = (0.45) (33.5 ksf) = <u>15.1 ksf</u> > 11.9 ksf				(= 103.88 k/lf / 8.7 ft)			



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 1 (EHd & LL3)

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 Eq. 10.6.3.4-1

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ Eq. 10.6.3.4-2

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil Section 10.6.3.4
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):
 concrete on sand → $\phi_t = 0.80$ Table 10.5.5.2.2-1

Foot. Length (L') = 95.4 ft → Effective Footing Length based on Strength I-a
 Foot. Width (W') = 9.1 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	13.3	kips
Sum of Factored Longitudinal Loads=	1367.8	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	1367.8	kips
Sum of Factored Vertical loads, V_{TOT} =	7250.5	kips

$R_t = V_{TOT} \tan \phi_t$
 $R_t = 5268$ kips

For $R_R = 4,214$ kips Active Driving Horizontal Force, $H_{TOT} = 1367.8$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a Load Factors:	
WL=	1.0 kips	WL=	5.0 kips		0
WS=	28.0 kips	WS=	703.0 kips		0
EH=	0.0 kips	EH=	703.0 kips (Driving force like others)		1.5
CE+LS=	5.0 kips	CE+LS=	161.0 kips		1.75
TU=	9.0 kips	TU=	63.0 kips		0.5
BR=	0 kips	BR=	0 kips		1.75
Vertical Unfactored LL=	457.0 ft kips				1.75
Vertical Unfactored DL=	6711.0 ft kips				0.9
Vertical Unfactored DW=	632.0 ft kips				0.65

13-Jun-13

Bridge 1716

PIER #1

EHmoment acts as a resisting force, all other horizontal loads as driving forces, and vertical loads times eccentricity as driving forces per ms e-mail dated June 11, 2013

Footing dimensions provided by ms:

Service Case Factors

WL	1
WS	0.3
TU	1
LL	1
LS	1
DC	1
EH	1
CE	1
DW	1
BR	1

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	359	4,445	632	1,384	0	882	359	4,445	632	1384	0	882
LL Case 2	583	4,445	632	1,384	0	882	583	4,445	632	1384	0	882
LL Case 3	457	4,445	632	1,384	0	882	457	4,445	632	1384	0	882

Initial eccentricity provided by ms= 3.5 ft
 Footing length= 105.0 ft
 Footing width= 14.0 ft
 Toe width= 9.0 ft
 Heel width= 2.0 ft
 Pier column width= 3.0 ft

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment	Transverse Loading
LL Case 1	45	0	0	167	285	14,478	6,580	392	0	0	
LL Case 2	45	0	0	271	285	14,704	6,580	392	0	0	
LL Case 3	45	0	0	213	285	16,152	6,580	392	0	0	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	Factored Transverse Loading
LL Case 1	45	0	0	167	285	14,478	6,580	392	0	0	
LL Case 2	45	0	0	271	285	14,704	6,580	392	0	0	
LL Case 3	45	0	0	213	285	16,152	6,580	392	0	0	

LL Case 1	W =	14.00
	L =	105.00
LL Case 2	W =	14.00
	L =	105.00
LL Case 3	W =	14.00
	L =	105.00

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment	Longitudinal Loading
LL Case 1	173	1,476	-1,476	23	2,058	824	0	0	0	0	
LL Case 2	173	1,476	-1,476	38	2,058	824	0	0	0	0	
LL Case 3	173	1,476	-1,476	29	2,058	824	0	0	0	0	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	Factored Longitudinal Loading
LL Case 1	173	443	-1,476	23	2,058	824	0	0	0	0	
LL Case 2	173	443	-1,476	38	2,058	824	0	0	0	0	
LL Case 3	173	443	-1,476	29	2,058	824	0	0	0	0	

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	Factored Transverse Loading
LL Case 1	21,947	7,702	2.85	
LL Case 2	22,277	7,926	2.81	
LL Case 3	23,667	7,800	3.03	

Loading Pressure
Based on Effective Footing Size

LL Case 1	10,668 psf
LL Case 2	10,987 psf
LL Case 3	10,851 psf

LL Case 1	W' =	7.27
	L' =	99.30
LL Case 2	W' =	7.26
	L' =	99.38
LL Case 3	W' =	7.27
	L' =	98.93

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	Factored Longitudinal Loading
LL Case 1	25,915	7,702	3.36	
LL Case 2	26,714	7,926	3.37	
LL Case 3	26,264	7,800	3.37	

Footing thickness h= 4.00

UNFACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment	Longitudinal Loading
LL Case 1	1,257	15,558	2,212	4,844	
LL Case 2	2,041	15,558	2,212	4,844	
LL Case 3	1,600	15,558	2,212	4,844	

FACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment	Longitudinal Loading
LL Case 1	1,257	15,558	2,212	4,844	
LL Case 2	2,041	15,558	2,212	4,844	
LL Case 3	1,600	15,558	2,212	4,844	

13-Jun-13

Bridge 1716

PIER #1

EHmoment acts as a resisting force, all other horizontal loads as driving forces, and vertical loads times eccentricity as driving forces per ms e-mail dated June 11, 2013

Footing dimensions provided by ms:

Strength I-a Case Factors

WL	0
WS	0
TU	0.5
LL	1.75
LS	1.75
DC	0.9
EH	1.5
CE	1.75
DW	0.65
BR	1.75

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	359	4,445	632	1,384	0	882	628	4,001	410.8	1245.6	0	793.8
LL Case 2	583	4,445	632	1,384	0	882	1,020	4,001	410.8	1245.6	0	793.8
LL Case 3	457	4,445	632	1,384	0	882	800	4,001	410.8	1245.6	0	793.8

Initial eccentricity provided by ms=	3.5 ft
Footing length=	105.0 ft
Footing width=	14.0 ft
Toe width=	9.0 ft
Heel width=	2.0 ft
Pier column width=	3.0 ft

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	45	0	0	167	285	14,478	6,580	392	0	0
LL Case 2	45	0	0	271	285	14,704	6,580	392	0	0
LL Case 3	45	0	0	213	285	16,152	6,580	392	0	0

Transverse Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	0	0	0	292	143	25,337	5,922	255	0	0
LL Case 2	0	0	0	474	143	25,732	5,922	255	0	0
LL Case 3	0	0	0	373	143	28,266	5,922	255	0	0

Factored Transverse Loading

LL Case 1	W =	14.00
	L =	105.00
LL Case 2	W =	14.00
	L =	105.00
LL Case 3	W =	14.00
	L =	105.00

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	173	1,476	-1,476	23	2,058	824	0	0	0	0
LL Case 2	173	1,476	-1,476	38	2,058	824	0	0	0	0
LL Case 3	173	1,476	-1,476	29	2,058	824	0	0	0	0

Longitudinal Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	0	0	-2,214	40	1,029	1,442	0	0	0	0
LL Case 2	0	0	-2,214	67	1,029	1,442	0	0	0	0
LL Case 3	0	0	-2,214	51	1,029	1,442	0	0	0	0

Factored Longitudinal Loading

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	e _{max} *	e < e _{max}
LL Case 1	31,948	7,079	4.51	35.00	Yes
LL Case 2	32,526	7,471	4.35	35.00	Yes
LL Case 3	34,958	7,250	4.82	35.00	Yes

Factored Transverse Loading

**Loading Pressure
Based on Effective Footing Size**

LL Case 1	9,578 psf
LL Case 2	10,132 psf
LL Case 3	9,898 psf

LL Case 1	W' =	7.70
	L' =	95.97
LL Case 2	W' =	7.66
	L' =	96.29
LL Case 3	W' =	7.68
	L' =	95.36

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	e _{max} *	e < e _{max}
LL Case 1	22,295	7,079	3.15	4.67	Yes
LL Case 2	23,694	7,471	3.17	4.67	Yes
LL Case 3	22,906	7,250	3.16	4.67	Yes

Factored Longitudinal Loading

*Eccentricity Check AASHTO 10.6.3.3, 6th Edition

Footing thickness h= 4.00

UNFACTORED driving moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	1,257	15,558	2,212	4,844
LL Case 2	2,041	15,558	2,212	4,844
LL Case 3	1,600	15,558	2,212	4,844

Longitudinal Loading

FACTORED driving moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	2,199	14,002	1,438	4,360
LL Case 2	3,571	14,002	1,438	4,360
LL Case 3	2,799	14,002	1,438	4,360

Longitudinal Loading

13-Jun-13

Bridge 1716

PIER #1

EHmoment acts as a resisting force, all other horizontal loads as driving forces, and vertical loads times eccentricity as driving forces per ms e-mail dated June 11, 2013

Footing dimensions provided by ms:

Strength I-b Case Factors

WL	0
WS	0
TU	0.5
LL	1.75
LS	1.75
DC	1.25
EH	1.5
CE	1.75
DW	1.5
BR	1.75

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	359	4,445	632	1,384	0	882	628	5,556	948	1730	0	1102.5
LL Case 2	583	4,445	632	1,384	0	882	1,020	5,556	948	1730	0	1102.5
LL Case 3	457	4,445	632	1,384	0	882	800	5,556	948	1730	0	1102.5

Initial eccentricity provided by ms=	3.5 ft
Footing length=	105.0 ft
Footing width=	14.0 ft
Toe width=	9.0 ft
Heel width=	2.0 ft
Pier column width=	3.0 ft

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	45	0	0	167	285	14,478	6,580	392	0	0
LL Case 2	45	0	0	271	285	14,704	6,580	392	0	0
LL Case 3	45	0	0	213	285	16,152	6,580	392	0	0

Transverse Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	0	0	0	292	143	25,337	8,225	588	0	0
LL Case 2	0	0	0	474	143	25,732	8,225	588	0	0
LL Case 3	0	0	0	373	143	28,266	8,225	588	0	0

Factored Transverse Loading

LL Case 1	W =	14.00
	L =	105.00
LL Case 2	W =	14.00
	L =	105.00
LL Case 3	W =	14.00
	L =	105.00

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	173	1,476	-1,476	23	2,058	824	0	0	0	0
LL Case 2	173	1,476	-1,476	38	2,058	824	0	0	0	0
LL Case 3	173	1,476	-1,476	29	2,058	824	0	0	0	0

Longitudinal Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	0	0	-2,214	40	1,029	1,442	0	0	0	0
LL Case 2	0	0	-2,214	67	1,029	1,442	0	0	0	0
LL Case 3	0	0	-2,214	51	1,029	1,442	0	0	0	0

Factored Longitudinal Loading

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e
LL Case 1	34,584	9,965	3.47
LL Case 2	35,162	10,357	3.39
LL Case 3	37,594	10,137	3.71

Factored Transverse Loading

**Loading Pressure
Based on Effective Footing Size**

LL Case 1	13,172 psf
LL Case 2	13,727 psf
LL Case 3	13,489 psf

LL Case 1	W' =	7.71
	L' =	98.06
LL Case 2	W' =	7.68
	L' =	98.21
LL Case 3	W' =	7.70
	L' =	97.58

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e
LL Case 1	31,316	9,965	3.14
LL Case 2	32,714	10,357	3.16
LL Case 3	31,927	10,137	3.15

Factored Longitudinal Loading

Footing thickness h= 4.00

UNFACTORED driving moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	1,257	15,558	2,212	4,844
LL Case 2	2,041	15,558	2,212	4,844
LL Case 3	1,600	15,558	2,212	4,844

Longitudinal Loading

FACTORED driving moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	2,199	19,447	3,318	6,055
LL Case 2	3,571	19,447	3,318	6,055
LL Case 3	2,799	19,447	3,318	6,055

Longitudinal Loading



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 1 (Ehr & LL1)

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	739.0 ft	Foot. Width (B') =	7.7 ft	Foot. Length (L') =	98.1 ft			
Unfactored Dead Load =	6711.00 kips	Unfactored Live Load =	359.00 kips	Load =	78.54	k/lf		
Unfactored DW =	632.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)						
Max Factored Dead Load =	8388.75 kips	Max Factored Live Load =	628.25 kips	F. Load =	101.62	k/lf		
Max Factored DW =	948.00 kips							
Calculate factored bearing resistance (q_R)				Reference: AASHTO LRFD Bridge Design Specifications				
q _R = φ _b q _n				Eq. 10.6.3.1.1-1				
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2α-1,2,3,4				
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0				
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.37}$) = 0.95				Eq. 10.4.6.2.4-1				
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 63 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3				
N ₁₆₀ = 59.5 → φ _f = 38° to 43° Use φ _f = 36°				Table 10.4.6.2.4-1				
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2α-1				
Groundwater at 739.0 ft → (739' - 739') = 0.0'				Water is within 1.5B of bearing depth				
C _{wq} = 1.0, C _{wy} = 0.5				Table 10.6.3.1.2α-2				
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{7.7'}{98.1'}$) = 0.97				s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{7.7'}{98.1'}$ tan 36°) = 1.06				
dq = 1.0				Table 10.6.3.1.2α-4				
q _n = 0 + (120.0 pcf) (4.0 ft) (37.8) (1.06) (1.0) (1.0) + (0.5) (120.0 pcf) (7.7 ft) (56.3) (0.97) (0.5)								
q _n = 19180 + 12613				For a 7.7 ft wide footing (eff. width 7.7 ft) q _n = 31.8 ksf				
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2				
				Table 10.5.5.2.2-1				
q _R = φ _b q _n = (0.45) (31.8 ksf) = <u>14.3 ksf</u> > 13.2 ksf				(= 101.62 k/lf / 7.7 ft)				



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 1 (Ehr & LL1)

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 Eq. 10.6.3.4-1

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ Eq. 10.6.3.4-2

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil Section 10.6.3.4
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):
 concrete on sand → $\phi_t = 0.80$ Table 10.5.5.2.2-1

Foot. Length (L') = 96.0 ft → Effective Footing Length based on Strength I-a
 Foot. Width (W') = 7.7 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	11.5	kips
Sum of Factored Longitudinal Loads=	-741.3	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	741.3	kips
Sum of Factored Vertical loads, V_{TOT} =	7079.0	kips

$R_t = V_{TOT} \tan \phi_t$
 $R_t = 5143$ kips

For $R_R = 4,115$ kips Active Driving Horizontal Force, $H_{TOT} = 741.3$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a
				Load Factors:
WL=	1.0 kips	WL=	5.0 kips	0
WS=	28.0 kips	WS=	703.0 kips	0
EH=	0.0 kips	EH=	-703.0 kips (resisting force)	1.5
CE+LS=	4.0 kips	CE+LS=	161.0 kips	1.75
TU=	9.0 kips	TU=	63.0 kips	0.5
BR=	0 kips	BR=	0 kips	1.75
Vertical Unfactored LL=	359.0 ft	kips		1.75
Vertical Unfactored DL=	6711.0 ft	kips		0.9
Vertical Unfactored DW=	632.0 ft	kips		0.65

CE and LS



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 1 (Ehr & LL2)

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	739.0 ft	Foot. Width (B') =	7.7 ft	Foot. Length (L') =	98.2 ft			
Unfactored Dead Load =	6711.00 kips	Unfactored Live Load =	583.00 kips	Load =	80.70	k/lf		
Unfactored DW =	632.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)						
Max Factored Dead Load =	8388.75 kips	Max Factored Live Load =	1020.25 kips	F. Load =	105.46	k/lf		
Max Factored DW =	948.00 kips							
Calculate factored bearing resistance (q_R)				Reference: AASHTO LRFD Bridge Design Specifications				
q _R = φ _b q _n				Eq. 10.6.3.1.1-1				
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2α-1,2,3,4				
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0				
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.37}$) = 0.95				Eq. 10.4.6.2.4-1				
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 63 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3				
N ₁₆₀ = 59.5 → φ _f = 38° to 43° Use φ _f = 36°				Table 10.4.6.2.4-1				
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2α-1				
Groundwater at 739.0 ft → (739' - 739') = 0.0'				Water is within 1.5B of bearing depth				
C _{wq} = 1.0, C _{wy} = 0.5				Table 10.6.3.1.2α-2				
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{7.7'}{98.2'}$) = 0.97				s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{7.7'}{98.2'}$ tan 36°) = 1.06				
dq = 1.0				Table 10.6.3.1.2α-4				
q _n = 0 + (120.0 pcf) (4.0 ft) (37.8) (1.06) (1.0) (1.0) + (0.5) (120.0 pcf) (7.7 ft) (56.3) (0.97) (0.5)								
q _n = 19175 + 12566				For a 7.7 ft wide footing (eff. width 7.7 ft) q _n = 31.7 ksf				
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2				
				Table 10.5.5.2.2-1				
q _R = φ _b q _n = (0.45) (31.7 ksf) = <u>14.3 ksf</u> > 13.7 ksf				(= 105.46 k/lf / 7.7 ft)				



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 1 (Ehr & LL2)

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 Eq. 10.6.3.4-1

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ Eq. 10.6.3.4-2

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil Section 10.6.3.4
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):

concrete on sand → $\phi_t = 0.80$ Table 10.5.5.2.2-1

Foot. Length (L') = 96.3 ft → Effective Footing Length based on Strength I-a
 Foot. Width (W') = 7.7 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	15.0	kips
Sum of Factored Longitudinal Loads=	-741.3	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	741.4	kips
Sum of Factored Vertical loads, V_{TOT} =	7471.0	kips

$R_t = V_{TOT} \tan \phi_t$
 $R_t = 5428$ kips

For $R_R = 4,342$ kips

Active Driving Horizontal Force, $H_{TOT} = 741.4$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a	Load Factors:
WL=	1.0 kips	WL=	5.0 kips	0	
WS=	28.0 kips	WS=	703.0 kips	0	
EH=	0.0 kips	EH=	-703.0 kips (resisting force)	1.5	
CE+LS=	6.0 kips	CE+LS=	161.0 kips	1.75	CE and LS
TU=	9.0 kips	TU=	63.0 kips	0.5	
BR=	0 kips	BR=	0 kips	1.75	
Vertical Unfactored LL=	583.0 ft	kips		1.75	
Vertical Unfactored DL=	6711.0 ft	kips		0.9	
Vertical Unfactored DW=	632.0 ft	kips		0.65	



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 1 (Ehr & LL3)

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	739.0 ft	Foot. Width (B') =	7.7 ft	Foot. Length (L') =	97.6 ft			
Unfactored Dead Load =	6711.00 kips	Unfactored Live Load =	457.00 kips	Load =	79.93	k/lf		
Unfactored DW =	632.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)						
Max Factored Dead Load =	8388.75 kips	Max Factored Live Load =	799.75 kips	F. Load =	103.88	k/lf		
Max Factored DW =	948.00 kips							
Calculate factored bearing resistance (q_R)				Reference: AASHTO LRFD Bridge Design Specifications				
q _R = φ _b q _n				Eq. 10.6.3.1.1-1				
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2a-1, 2,3,4				
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0				
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.37}$) = 0.95				Eq. 10.4.6.2.4-1				
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 63 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3				
N ₁₆₀ = 59.5 → φ _f = 38° to 43° Use φ _f = 36°				Table 10.4.6.2.4-1				
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2a-1				
Groundwater at 739.0 ft → (739' - 739') = 0.0'				Water is within 1.5B of bearing depth				
C _{wq} = 1.0, C _{wy} = 0.5				Table 10.6.3.1.2a-2				
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{7.7'}{97.6'}$) = 0.97				s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{7.7'}{97.6'}$ tan 36°) = 1.06				
dq = 1.0				Table 10.6.3.1.2a-4				
q _n = 0 + (120.0 pcf) (4.0 ft) (37.8) (1.06) (1.0) (1.0) + (0.5) (120.0 pcf) (7.7 ft) (56.3) (0.97) (0.5)								
q _n = 19184 + 12595				For a 7.7 ft wide footing (eff. width 7.7 ft) q _n = 31.8 ksf				
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2				
				Table 10.5.5.2.2-1				
q _R = φ _b q _n = (0.45) (31.8 ksf) = <u>14.3 ksf</u> > 13.5 ksf				(= 103.88 k/lf / 7.7 ft)				



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 1 (Ehr & LL3)

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 *Eq. 10.6.3.4-1*

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ *Eq. 10.6.3.4-2*

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil *Section 10.6.3.4*
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):
 concrete on sand → $\phi_t = 0.80$ *Table 10.5.5.2.2-1*

Foot. Length (L') = 95.4 ft → Effective Footing Length based on Strength I-a
 Foot. Width (W') = 7.7 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	13.3	kips
Sum of Factored Longitudinal Loads=	-741.3	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	741.4	kips
Sum of Factored Vertical loads, V_{TOT} =	7250.5	kips

$R_t = V_{TOT} \tan \phi_t$
 $R_t = 5268$ kips

For $R_R = 4,214$ kips Active Driving Horizontal Force, $H_{TOT} = 741.4$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a
				Load Factors:
WL=	1.0 kips	WL=	5.0 kips	0
WS=	28.0 kips	WS=	703.0 kips	0
EH=	0.0 kips	EH=	-703.0 kips (resisting force)	1.5
CE+LS=	5.0 kips	CE+LS=	161.0 kips	1.75
TU=	9.0 kips	TU=	63.0 kips	0.5
BR=	0 kips	BR=	0 kips	1.75
Vertical Unfactored LL=	457.0 ft	kips		1.75
Vertical Unfactored DL=	6711.0 ft	kips		0.9
Vertical Unfactored DW=	632.0 ft	kips		0.65

CE and LS

PIER #1 EHmoment acts as a driving force, all other horizontal loads as driving forces, and vertical loads times eccentricity as resisting forces per ms e-mail dated June 11, 2013 (WITHOUT LL)

Footing dimensions provided by ms:

Service Case Factors

WL	1
WS	0.3
TU	1
LL	1
LS	1
DC	1
EH	1
CE	1
DW	1
BR	1

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	0	4,445	632	1,384	0	882	0	4,445	632	1384	0	882
LL Case 2	0	4,445	632	1,384	0	882	0	4,445	632	1384	0	882
LL Case 3	0	4,445	632	1,384	0	882	0	4,445	632	1384	0	882

Initial eccentricity provided by ms= 3.5 ft
 Footing length= 105.0 ft
 Footing width= 14.0 ft
 Toe width= 9.0 ft
 Heel width= 2.0 ft
 Pier column width= 3.0 ft

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	45	0	0	167	285	14,478	6,580	392	0	0
LL Case 2	45	0	0	271	285	14,704	6,580	392	0	0
LL Case 3	45	0	0	213	285	16,152	6,580	392	0	0

Transverse Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	45	0	0	167	285	14,478	6,580	392	0	0
LL Case 2	45	0	0	271	285	14,704	6,580	392	0	0
LL Case 3	45	0	0	213	285	16,152	6,580	392	0	0

Factored Transverse Loading

LL Case 1	W =	14.00
	L =	105.00
LL Case 2	W =	14.00
	L =	105.00
LL Case 3	W =	14.00
	L =	105.00

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	173	1,476	1,476	23	2,058	824	0	0	0	0
LL Case 2	173	1,476	1,476	38	2,058	824	0	0	0	0
LL Case 3	173	1,476	1,476	29	2,058	824	0	0	0	0

Longitudinal Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	173	443	1,476	23	2,058	824	0	0	0	0
LL Case 2	173	443	1,476	38	2,058	824	0	0	0	0
LL Case 3	173	443	1,476	29	2,058	824	0	0	0	0

Factored Longitudinal Loading

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e
LL Case 1	21,947	7,343	2.99
LL Case 2	22,277	7,343	3.03
LL Case 3	23,667	7,343	3.22

Factored Transverse Loading

Loading Pressure Based on Effective Footing Size

LL Case 1	8,059 psf
LL Case 2	8,063 psf
LL Case 3	8,096 psf

LL Case 1	W' =	9.20
	L' =	99.02
LL Case 2	W' =	9.21
	L' =	98.93
LL Case 3	W' =	9.20
	L' =	98.55

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e
LL Case 1	-17,617	7,343	-2.40
LL Case 2	-17,602	7,343	-2.40
LL Case 3	-17,611	7,343	-2.40

Factored Longitudinal Loading

Footing thickness h= 4.00

UNFACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	0	-15,558	-2,212	-4,844
LL Case 2	0	-15,558	-2,212	-4,844
LL Case 3	0	-15,558	-2,212	-4,844

Longitudinal Loading

FACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	0	-15,558	-2,212	-4,844
LL Case 2	0	-15,558	-2,212	-4,844
LL Case 3	0	-15,558	-2,212	-4,844

Longitudinal Loading

13-Jun-13

Bridge 1716

PIER #1

EHmoment acts as a driving force, all other horizontal loads as driving forces, and vertical loads times eccentricity as resisting forces per ms e-mail dated June 11, 2013 (WITHOUT LL)

Footing dimensions provided by ms:

Strength I-a Case Factors

WL	0
WS	0
TU	0.5
LL	1.75
LS	1.75
DC	0.9
EH	1.5
CE	1.75
DW	0.65
BR	1.75

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	0	4,445	632	1,384	0	882	0	4,001	410.8	1245.6	0	793.8
LL Case 2	0	4,445	632	1,384	0	882	0	4,001	410.8	1245.6	0	793.8
LL Case 3	0	4,445	632	1,384	0	882	0	4,001	410.8	1245.6	0	793.8

Initial eccentricity provided by ms=	3.5 ft
Footing length=	105.0 ft
Footing width=	14.0 ft
Toe width=	9.0 ft
Heel width=	2.0 ft
Pier column width=	3.0 ft

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	45	0	0	167	285	14,478	6,580	392	0	0
LL Case 2	45	0	0	271	285	14,704	6,580	392	0	0
LL Case 3	45	0	0	213	285	16,152	6,580	392	0	0

Transverse Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	0	0	0	292	143	25,337	5,922	255	0	0
LL Case 2	0	0	0	474	143	25,732	5,922	255	0	0
LL Case 3	0	0	0	373	143	28,266	5,922	255	0	0

Factored Transverse Loading

LL Case 1	W =	14.00
	L =	105.00
LL Case 2	W =	14.00
	L =	105.00
LL Case 3	W =	14.00
	L =	105.00

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	173	1,476	1,476	23	2,058	824	0	0	0	0
LL Case 2	173	1,476	1,476	38	2,058	824	0	0	0	0
LL Case 3	173	1,476	1,476	29	2,058	824	0	0	0	0

Longitudinal Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	0	0	2,214	40	1,029	1,442	0	0	0	0
LL Case 2	0	0	2,214	67	1,029	1,442	0	0	0	0
LL Case 3	0	0	2,214	51	1,029	1,442	0	0	0	0

Factored Longitudinal Loading

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	e _{max} *	e < e _{max}
LL Case 1	31,948	6,451	4.95	35.00	Yes
LL Case 2	32,526	6,451	5.04	35.00	Yes
LL Case 3	34,958	6,451	5.42	35.00	Yes

Factored Transverse Loading

**Loading Pressure
Based on Effective Footing Size**

LL Case 1	7,273 psf
LL Case 2	7,281 psf
LL Case 3	7,343 psf

LL Case 1	W' =	9.33
	L' =	95.09
LL Case 2	W' =	9.33
	L' =	94.92
LL Case 3	W' =	9.33
	L' =	94.16

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	e _{max} *	ABS(e) < e _{max}
LL Case 1	-15,074	6,451	-2.34	4.67	Yes
LL Case 2	-15,048	6,451	-2.33	4.67	Yes
LL Case 3	-15,063	6,451	-2.34	4.67	Yes

Factored Longitudinal Loading

Footing thickness h= 4.00

UNFACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	0	-15,558	-2,212	-4,844
LL Case 2	0	-15,558	-2,212	-4,844
LL Case 3	0	-15,558	-2,212	-4,844

Longitudinal Loading

FACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	0	-14,002	-1,438	-4,360
LL Case 2	0	-14,002	-1,438	-4,360
LL Case 3	0	-14,002	-1,438	-4,360

Longitudinal Loading

PIER #1 EHmoment acts as a driving force, all other horizontal loads as driving forces, and vertical loads times eccentricity as resisting forces per ms e-mail dated June 11, 2013 (WITHOUT LL)

Footing dimensions provided by ms:

Strength I-b Case Factors

WL	0
WS	0
TU	0.5
LL	1.75
LS	1.75
DC	1.25
EH	1.5
CE	1.75
DW	1.5
BR	1.75

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	0	4,445	632	1,384	0	882	0	5,556	948	1730	0	1102.5
LL Case 2	0	4,445	632	1,384	0	882	0	5,556	948	1730	0	1102.5
LL Case 3	0	4,445	632	1,384	0	882	0	5,556	948	1730	0	1102.5

Initial eccentricity provided by ms= 3.5 ft
 Footing length= 105.0 ft
 Footing width= 14.0 ft
 Toe width= 9.0 ft
 Heel width= 2.0 ft
 Pier column width= 3.0 ft

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	45	0	0	167	285	14,478	6,580	392	0	0
LL Case 2	45	0	0	271	285	14,704	6,580	392	0	0
LL Case 3	45	0	0	213	285	16,152	6,580	392	0	0

Transverse Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	0	0	0	292	143	25,337	8,225	588	0	0
LL Case 2	0	0	0	474	143	25,732	8,225	588	0	0
LL Case 3	0	0	0	373	143	28,266	8,225	588	0	0

Factored Transverse Loading

LL Case 1	W =	14.00
	L =	105.00
LL Case 2	W =	14.00
	L =	105.00
LL Case 3	W =	14.00
	L =	105.00

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment
LL Case 1	173	1,476	1,476	23	2,058	824	0	0	0	0
LL Case 2	173	1,476	1,476	38	2,058	824	0	0	0	0
LL Case 3	173	1,476	1,476	29	2,058	824	0	0	0	0

Longitudinal Loading

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment
LL Case 1	0	0	2,214	40	1,029	1,442	0	0	0	0
LL Case 2	0	0	2,214	67	1,029	1,442	0	0	0	0
LL Case 3	0	0	2,214	51	1,029	1,442	0	0	0	0

Factored Longitudinal Loading

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e
LL Case 1	34,584	9,337	3.70
LL Case 2	35,162	9,337	3.77
LL Case 3	37,594	9,337	4.03

Factored Transverse Loading

Loading Pressure
Based on Effective Footing Size

LL Case 1	10,824 psf
LL Case 2	10,831 psf
LL Case 3	10,893 psf

LL Case 1	W' =	8.84
	L' =	97.59
LL Case 2	W' =	8.84
	L' =	97.47
LL Case 3	W' =	8.84
	L' =	96.95

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e
LL Case 1	-24,095	9,337	-2.58
LL Case 2	-24,068	9,337	-2.58
LL Case 3	-24,084	9,337	-2.58

Factored Longitudinal Loading

Footing thickness h= 4.00

UNFACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	0	-15,558	-2,212	-4,844
LL Case 2	0	-15,558	-2,212	-4,844
LL Case 3	0	-15,558	-2,212	-4,844

Longitudinal Loading

FACTORED resisting moments from vertical loads times initial eccentricity

Units in kip-ft	LL moment	DL moment	DW moment	Self moment
LL Case 1	0	-19,447	-3,318	-6,055
LL Case 2	0	-19,447	-3,318	-6,055
LL Case 3	0	-19,447	-3,318	-6,055

Longitudinal Loading



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Capacity - Pier 1 (EHd & LL1) WITHOUT LL

Bearing Depth =	739.0 ft	Foot. Width (B') =	8.8 ft	Foot. Length (L') =	97.6 ft		
Unfactored Dead Load =	6711.00 kips	Unfactored Live Load =	.00 kips	Load =	75.24	k/lf	
Unfactored DW =	632.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)					
Max Factored Dead Load =	8388.75 kips	Max Factored Live Load =	.00 kips	F. Load =	95.67	k/lf	
Max Factored DW =	948.00 kips						
Calculate factored bearing resistance (q_R)				<i>Reference: AASHTO LRFD Bridge Design Specifications</i>			
q _R = φ _b q _n				<i>Eq. 10.6.3.1.1-1</i>			
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				<i>Eq. 10.6.3.1.2a-1, 2,3,4</i>			
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				<i>Assume inclination factors (i_c, i_q, i_y) = 1.0</i>			
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.37}$) = 0.95				<i>Eq. 10.4.6.2.4-1</i>			
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 63 (Boring B-287-1-10)				<i>Eq. 10.4.6.2.4-3</i>			
N ₁₆₀ = 59.5 → φ _f = 38° to 43° Use φ _f = 36°				<i>Table 10.4.6.2.4-1</i>			
φ _f = 36° → N _q = 37.8 N _y = 56.3				<i>Table 10.6.3.1.2a-1</i>			
Groundwater at 739.0 ft → (739' - 739') = 0.0'				Water is within 1.5B of bearing depth			
				C _{wq} = 1.0 , C _{wy} = 0.5 <i>Table 10.6.3.1.2a-2</i>			
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{8.8'}{97.6'}$) = 0.96				s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{8.8'}{97.6'}$ tan 36°) = 1.07			
dq = 1.0				<i>Table 10.6.3.1.2a-4</i>			
q _n = 0 + (120.0 pcf) (4.0 ft) (37.8) (1.07) (1.0) (1.0) + (0.5) (120.0 pcf) (8.8 ft) (56.3) (0.96) (0.5)							
q _n = 19338 + 14390				For a 8.8 ft wide footing (eff. width 8.8 ft) q _n = 33.7 ksf			
Resistance Factor (φ _b) = 0.45 strength limit state				<i>Section 10.5.5.2</i>			
				<i>Table 10.5.5.2.2-1</i>			
q _R = φ _b q _n = (0.45) (33.7 ksf) = <u>15.2 ksf</u> > 10.8 ksf				(= 95.67 k/lf / 8.8 ft)			



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 1 (EHd & LL1) WITHOUT LL

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 Eq. 10.6.3.4-1

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ Eq. 10.6.3.4-2

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil Section 10.6.3.4
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):
 concrete on sand → $\phi_t = 0.80$ Table 10.5.5.2.2-1

Foot. Length (L') = 95.1 ft → Effective Footing Length based on Strength I-a
 Foot. Width (W') = 9.3 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	11.5	kips
Sum of Factored Longitudinal Loads=	1367.8	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	1367.8	kips
Sum of Factored Vertical loads, V_{TOT} =	6450.7	kips

$R_t = V_{TOT} \tan \phi_t$
 $R_t = 4687$ kips

For $R_R = 3,749$ kips Active Driving Horizontal Force, $H_{TOT} = 1367.8$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a Load Factors:	
WL=	1.0 kips	WL=	5.0 kips		0
WS=	28.0 kips	WS=	703.0 kips		0
EH=	0.0 kips	EH=	703.0 kips (Driving force like others)		1.5
CE+LS=	4.0 kips	CE+LS=	161.0 kips		1.75
TU=	9.0 kips	TU=	63.0 kips		0.5
BR=	0 kips	BR=	0 kips		1.75
Vertical Unfactored LL=	.0 ft kips				1.75
Vertical Unfactored DL=	6711.0 ft kips				0.9
Vertical Unfactored DW=	632.0 ft kips				0.65



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Capacity - Pier 1 (EHd & LL2) WITHOUT LL

Bearing Depth =	739.0 ft	Foot. Width (B') =	8.8 ft	Foot. Length (L') =	97.5 ft		
Unfactored Dead Load =	6711.00 kips	Unfactored Live Load =	.00 kips	Load =	75.34	k/lf	
Unfactored DW =	632.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)					
Max Factored Dead Load =	8388.75 kips	Max Factored Live Load =	.00 kips	F. Load =	95.79	k/lf	
Max Factored DW =	948.00 kips						
Calculate factored bearing resistance (q_R)				<i>Reference: AASHTO LRFD Bridge Design Specifications</i>			
q _R = φ _b q _n				<i>Eq. 10.6.3.1.1-1</i>			
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				<i>Eq. 10.6.3.1.2a-1, 2,3,4</i>			
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				<i>Assume inclination factors (i_c, i_q, i_y) = 1.0</i>			
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.37}$) = 0.95				<i>Eq. 10.4.6.2.4-1</i>			
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 63 (Boring B-287-1-10)				<i>Eq. 10.4.6.2.4-3</i>			
N ₁₆₀ = 59.5 → φ _f = 38° to 43° Use φ _f = 36°				<i>Table 10.4.6.2.4-1</i>			
φ _f = 36° → N _q = 37.8 N _y = 56.3				<i>Table 10.6.3.1.2a-1</i>			
Groundwater at 739.0 ft → (739' - 739') = 0.0' Water is within 1.5B of bearing depth							
C _{wq} = 1.0 , C _{wy} = 0.5				<i>Table 10.6.3.1.2a-2</i>			
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{8.8'}{97.5'}$) = 0.96 , s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{8.8'}{97.5'}$ tan 36°) = 1.07							
dq = 1.0				<i>Table 10.6.3.1.2a-4</i>			
q _n = 0 + (120.0 pcf) (4.0 ft) (37.8) (1.07) (1.0) (1.0) + (0.5) (120.0 pcf) (8.8 ft) (56.3) (0.96) (0.5)							
q _n = 19340 + 14389				For a 8.8 ft wide footing (eff. width 8.8 ft) q _n = 33.7 ksf			
Resistance Factor (φ _b) = 0.45 strength limit state				<i>Section 10.5.5.2</i>			
				<i>Table 10.5.5.2.2-1</i>			
q _R = φ _b q _n = (0.45) (33.7 ksf) = <u>15.2 ksf</u> > 10.8 ksf (= 95.79 k/lf / 8.8 ft)							



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 1 (EHd & LL2) WITHOUT LL

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 Eq. 10.6.3.4-1

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ Eq. 10.6.3.4-2

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil Section 10.6.3.4
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):
 concrete on sand → $\phi_t = 0.80$ Table 10.5.5.2.2-1

Foot. Length (L') = 94.9 ft → Effective Footing Length based on Strength I-a
 Foot. Width (W') = 9.3 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	15.0	kips
Sum of Factored Longitudinal Loads=	1367.8	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	1367.8	kips
Sum of Factored Vertical loads, V_{TOT} =	6450.7	kips

$R_t = V_{TOT} \tan \phi_t$
 $R_t = 4687$ kips

For $R_R = 3,749$ kips Active Driving Horizontal Force, $H_{TOT} = 1367.8$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a Load Factors:	
WL=	1.0 kips	WL=	5.0 kips		0
WS=	28.0 kips	WS=	703.0 kips		0
EH=	0.0 kips	EH=	703.0 kips (Driving force like others)		1.5
CE+LS=	6.0 kips	CE+LS=	161.0 kips		1.75
TU=	9.0 kips	TU=	63.0 kips		0.5
BR=	0 kips	BR=	0 kips		1.75
Vertical Unfactored LL=	.0 ft kips				1.75
Vertical Unfactored DL=	6711.0 ft kips				0.9
Vertical Unfactored DW=	632.0 ft kips				0.65



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 1 (EHd & LL3) WITHOUT LL

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	739.0 ft	Foot. Width (B') =	8.8 ft	Foot. Length (L') =	97.0 ft		
Unfactored Dead Load =	6711.00 kips	Unfactored Live Load =	.00 kips	Load =	75.74	k/lf	
Unfactored DW =	632.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)					
Max Factored Dead Load =	8388.75 kips	Max Factored Live Load =	.00 kips	F. Load =	96.30	k/lf	
Max Factored DW =	948.00 kips						
Calculate factored bearing resistance (q_R)				Reference: AASHTO LRFD Bridge Design Specifications			
q _R = φ _b q _n				Eq. 10.6.3.1.1-1			
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2a-1, 2,3,4			
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0			
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.37}$) = 0.95				Eq. 10.4.6.2.4-1			
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 63 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3			
N ₁₆₀ = 59.5 → φ _f = 38° to 43° Use φ _f = 36°				Table 10.4.6.2.4-1			
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2a-1			
Groundwater at 739.0 ft → (739' - 739') = 0.0'				Water is within 1.5B of bearing depth			
C _{wq} = 1.0, C _{wy} = 0.5				Table 10.6.3.1.2a-2			
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{8.8'}{97.0'}$) = 0.96				s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{8.8'}{97.0'}$ tan 36°) = 1.07			
dq = 1.0				Table 10.6.3.1.2a-4			
q _n = 0 + (120.0 pcf) (4.0 ft) (37.8) (1.07) (1.0) (1.0) + (0.5) (120.0 pcf) (8.8 ft) (56.3) (0.96) (0.5)							
q _n = 19346 + 14386				For a 8.8 ft wide footing (eff. width 8.8 ft) q _n = 33.7 ksf			
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2			
				Table 10.5.5.2.2-1			
q _R = φ _b q _n = (0.45) (33.7 ksf) = <u>15.2 ksf</u> > 10.9 ksf				(= 96.30 k/lf / 8.8 ft)			



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 1 (EHd & LL3) WITHOUT LL

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 Eq. 10.6.3.4-1

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ Eq. 10.6.3.4-2

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil Section 10.6.3.4
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):
 concrete on sand → $\phi_t = 0.80$ Table 10.5.5.2.2-1

Foot. Length (L') = 94.2 ft → Effective Footing Length based on Strength I-a
 Foot. Width (W') = 9.3 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	13.3	kips
Sum of Factored Longitudinal Loads=	1367.8	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	1367.8	kips
Sum of Factored Vertical loads, V_{TOT} =	6450.7	kips

$R_t = V_{TOT} \tan \phi_t$
 $R_t = 4687$ kips

For $R_R = 3,749$ kips Active Driving Horizontal Force, $H_{TOT} = 1367.8$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a Load Factors:	
WL=	1.0 kips	WL=	5.0 kips		0
WS=	28.0 kips	WS=	703.0 kips		0
EH=	0.0 kips	EH=	703.0 kips (Driving force like others)		1.5
CE+LS=	5.0 kips	CE+LS=	161.0 kips		1.75
TU=	9.0 kips	TU=	63.0 kips		0.5
BR=	0 kips	BR=	0 kips		1.75
Vertical Unfactored LL=	.0 ft kips				1.75
Vertical Unfactored DL=	6711.0 ft kips				0.9
Vertical Unfactored DW=	632.0 ft kips				0.65



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716 Pier Load Case EHr & LL2
 Boring Overburden

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 5
 COMP. BY EWT DATE 06/11/13
 CHECKED BY BTA DATE 06/11/13

Based on boring B-287-1-10	Existing Fndn Load ^a	psf	Bearing Depth	739.0 ft	Water Elev.	746.8 ft
	Layer	Zone (ft)	Mid-Pt Elev (ft)	γ_m (pcf)	Hc (ft)	
	A	762.8 to 739	-	120	-	
	B	739 to 736	737.5	120	3	
	C	736 to 726	731	120	10	
	D	726 to 714.8	720.4	120	11.2	
	E	714.8 to 712.3	713.55	120	2.5	
	F	712.3 to 710.3	711.3	120	2	
	G	710.3 to 707.8	709.05	130	2.5	
	H	707.8 to 699.8	703.8	120	8	
	I	699.8 to 692.8	696.3	130	7	

Calculate Existing Load

Point 1

$$\begin{aligned} \sigma'_{o1} &= 2369 \text{ psf} + (739' - 737.5')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2369 \text{ psf} + (1.5')(57.6 \text{ pcf}) \\ &= 2456 \text{ psf} \end{aligned}$$

Point 2

$$\begin{aligned} \sigma'_{o2} &= 2456 \text{ psf} + (737.5' - 731')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2456 \text{ psf} + (6.5')(57.6 \text{ pcf}) \\ &= 2830 \text{ psf} \end{aligned}$$

Point 3

$$\begin{aligned} \sigma'_{o3} &= 2830 \text{ psf} + (731' - 720.4')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2830 \text{ psf} + (10.6')(57.6 \text{ pcf}) \\ &= 3441 \text{ psf} \end{aligned}$$

Point 4

$$\begin{aligned} \sigma'_{o4} &= 3441 \text{ psf} + (720.4' - 713.55')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3441 \text{ psf} + (6.9')(57.6 \text{ pcf}) \\ &= 3836 \text{ psf} \end{aligned}$$

Point 5

$$\begin{aligned} &= 3836 \text{ psf} + (713.55' - 711.3')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3836 \text{ psf} + (2.3')(57.6 \text{ pcf}) \\ &= 3966 \text{ psf} \end{aligned}$$

Point 6

$$\begin{aligned} &= 3966 \text{ psf} + (711.3' - 710.3')(120 \text{ pcf} - 62.4 \text{ pcf}) + (710.3' - 709.05')(130 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3966 \text{ psf} + (1.0')(57.6 \text{ pcf}) + (1.3')(67.6 \text{ pcf}) \\ &= 4108 \text{ psf} \end{aligned}$$

Point 7

$$\begin{aligned} &= 4108 \text{ psf} + (709.05' - 707.8')(130 \text{ pcf} - 62.4 \text{ pcf}) + (707.8' - 703.8')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4108 \text{ psf} + (1.3')(67.6 \text{ pcf}) + (4.0')(57.6 \text{ pcf}) \\ &= 4423 \text{ psf} \end{aligned}$$

Point 8

$$\begin{aligned} &= 4423 \text{ psf} + (703.8' - 699.8')(120 \text{ pcf} - 62.4 \text{ pcf}) + (699.8' - 696.3')(130 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4423 \text{ psf} + (4.0')(57.6 \text{ pcf}) + (3.5')(67.6 \text{ pcf}) \\ &= 4890 \text{ psf} \end{aligned}$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716 Pier Load Case EHR & LL2
 Existing Overburden

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 5
 COMP. BY EWT DATE 06/11/13
 CHECKED BY BTA DATE 06/11/13

Based on boring B-287-1-10	Existing Fndn Load ^a	Bearing Depth	739.0 ft	Water Elev.	746.8 ft
			Mid-Pt		
	Layer	Zone (ft)	Elev (ft)	γ_m (pcf)	Hc (ft)
	A	760 to 739	-	120	-
	B	739 to 736	737.5	120	3
	C	736 to 726	731	120	10
	D	726 to 714.8	720.4	120	11.2
	E	714.8 to 712.3	713.55	120	2.5
	F	712.3 to 710.3	711.3	120	2
	D	710.3 to 707.8	709.05	130	2.5
	E	707.8 to 699.8	703.8	120	8
	F	699.8 to 692.8	696.3	130	7

Calculate Existing Load

Point 1

$$\begin{aligned} \sigma'_{o1} &= 2033 \text{ psf} + (739' - 737.5')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2033 \text{ psf} + (1.5')(57.6 \text{ pcf}) \\ &= 2120 \text{ psf} \end{aligned}$$

Point 2

$$\begin{aligned} \sigma'_{o2} &= 2120 \text{ psf} + (737.5' - 731')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2120 \text{ psf} + (6.5')(57.6 \text{ pcf}) \\ &= 2494 \text{ psf} \end{aligned}$$

Point 3

$$\begin{aligned} \sigma'_{o3} &= 2494 \text{ psf} + (731' - 720.4')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2494 \text{ psf} + (10.6')(57.6 \text{ pcf}) \\ &= 3105 \text{ psf} \end{aligned}$$

Point 4

$$\begin{aligned} \sigma'_{o4} &= 3105 \text{ psf} + (720.4' - 713.55')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3105 \text{ psf} + (6.9')(57.6 \text{ pcf}) \\ &= 3500 \text{ psf} \end{aligned}$$

Point 5

$$\begin{aligned} &= 3500 \text{ psf} + (713.55' - 711.3')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3500 \text{ psf} + (2.3')(57.6 \text{ pcf}) \\ &= 3630 \text{ psf} \end{aligned}$$

Point 6

$$\begin{aligned} &= 3630 \text{ psf} + (711.3' - 710.3')(120 \text{ pcf} - 62.4 \text{ pcf}) + (710.3' - 709.05')(130 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3630 \text{ psf} + (1.0')(57.6 \text{ pcf}) + (1.3')(67.6 \text{ pcf}) \\ &= 3772 \text{ psf} \end{aligned}$$

Point 7

$$\begin{aligned} &= 3772 \text{ psf} + (709.05' - 707.8')(130 \text{ pcf} - 62.4 \text{ pcf}) + (707.8' - 703.8')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3772 \text{ psf} + (1.3')(67.6 \text{ pcf}) + (4.0')(57.6 \text{ pcf}) \\ &= 4087 \text{ psf} \end{aligned}$$

Point 8

$$\begin{aligned} &= 4087 \text{ psf} + (703.8' - 699.8')(120 \text{ pcf} - 62.4 \text{ pcf}) + (699.8' - 696.3')(130 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4087 \text{ psf} + (4.0')(57.6 \text{ pcf}) + (3.5')(67.6 \text{ pcf}) \\ &= 4554 \text{ psf} \end{aligned}$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716 Pier Load Case EHR & LL2
 Proposed Overburden

JOB NUMBER 1021-1005.01
 SHEET NO. 3 OF 5
 COMP. BY EWT DATE 06/11/13
 CHECKED BY BTA DATE 06/11/13

Based on boring B-287-1-10	Foundation Load ^a	9378 psf	Bearing Depth	739.0 ft	Water Elev.	746.8 ft
753.3	Layer	Zone (ft)	Mid-Pt Elev (ft)	γ_m (pcf)	Hc (ft)	i*
739.0	A	753.3 to 739	-	120	-	
736.0	B	739 to 736	737.5	120	3	1.00
726.0	C	736 to 726	731	120	10	0.51
726.0	D	726 to 714.8	720.4	120	11.2	0.25
714.8	E	714.8 to 712.3	713.55	120	2.5	0.18
712.3	F	712.3 to 710.3	711.3	120	2	0.16
710.3	D	710.3 to 707.8	709.05	130	2.5	0.15
707.8	E	707.8 to 699.8	703.8	120	8	0.13
699.8	F	699.8 to 692.8	696.3	130	7	0.11
692.8						

Calculate Proposed Load

^a - *fdn load at center of footing*

* - *see settlement calcs for determination of i values*

Point 1

$$\begin{aligned} \sigma'_{o1} &= 1229 \text{ psf} + (739' - 737.5')(120 \text{ pcf} - 62.4 \text{ pcf}) + (9378 \text{ psf})(1) \\ &= 1229 \text{ psf} + (1.5')(57.6 \text{ pcf}) + (9378 \text{ psf}) \\ &= 10694 \text{ psf} \end{aligned}$$

Point 2

$$\begin{aligned} \sigma'_{o2} &= 1316 \text{ psf} + (737.5' - 731')(120 \text{ pcf} - 62.4 \text{ pcf}) + (9378 \text{ psf})(0.51) \\ &= 1316 \text{ psf} + (6.5')(57.6 \text{ pcf}) + (4783 \text{ psf}) \\ &= 6472.78 \text{ psf} \end{aligned}$$

Point 3

$$\begin{aligned} \sigma'_{o3} &= 1690 \text{ psf} + (731' - 720.4')(120 \text{ pcf} - 62.4 \text{ pcf}) + (9378 \text{ psf})(0.25) \\ &= 1690 \text{ psf} + (10.6')(57.6 \text{ pcf}) + (2345 \text{ psf}) \\ &= 4645.5 \text{ psf} \end{aligned}$$

Point 4

$$\begin{aligned} \sigma'_{o4} &= 2301 \text{ psf} + (720.4' - 713.55')(120 \text{ pcf} - 62.4 \text{ pcf}) + (9378 \text{ psf})(0.18) \\ &= 2301 \text{ psf} + (6.9')(57.6 \text{ pcf}) + (1688 \text{ psf}) \\ &= 4384.04 \text{ psf} \end{aligned}$$

Point 5

$$\begin{aligned} &= 2696 \text{ psf} + (713.55' - 711.3')(120 \text{ pcf} - 62.4 \text{ pcf}) + (9378 \text{ psf})(0.16) \\ &= 2696 \text{ psf} + (2.3')(57.6 \text{ pcf}) + (1500 \text{ psf}) \\ &= 4326.48 \text{ psf} \end{aligned}$$

Point 6

$$\begin{aligned} &= 2826 \text{ psf} + (711.3' - 710.3')(120 \text{ pcf} - 62.4 \text{ pcf}) + (710.3' - 709.05')(130 \text{ pcf} - 62.4 \text{ pcf}) + (9378 \text{ psf})(0.15) \\ &= 2826 \text{ psf} + (1.0')(57.6 \text{ pcf}) + (1.3')(67.6 \text{ pcf}) + (1407 \text{ psf}) \\ &= 4374.7 \text{ psf} \end{aligned}$$

Point 7

$$\begin{aligned} &= 2968 \text{ psf} + (709.05' - 707.8')(130 \text{ pcf} - 62.4 \text{ pcf}) + (707.8' - 703.8')(120 \text{ pcf} - 62.4 \text{ pcf}) + (9378 \text{ psf})(0.13) \\ &= 2968 \text{ psf} + (1.3')(67.6 \text{ pcf}) + (4.0')(57.6 \text{ pcf}) + (1219 \text{ psf}) \\ &= 4502.14 \text{ psf} \end{aligned}$$

Point 8

$$\begin{aligned} &= 3283 \text{ psf} + (703.8' - 699.8')(120 \text{ pcf} - 62.4 \text{ pcf}) + (699.8' - 696.3')(130 \text{ pcf} - 62.4 \text{ pcf}) + (9378 \text{ psf})(0.11) \\ &= 3283 \text{ psf} + (4.0')(57.6 \text{ pcf}) + (3.5')(67.6 \text{ pcf}) + (1032 \text{ psf}) \\ &= 4781.58 \text{ psf} \end{aligned}$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716 Pier Load Case EHR & LL2
 Settlement - Pier Load Case EHR&LL2

JOB NUMBER 1021-1005.01
 SHEET NO. 4 OF 5
 COMP. BY EWT DATE 06/11/13
 CHECKED BY BTA DATE 06/11/13

Proposed Loading										
Assume loading =		9378 psf								
Assume width =		7.3 ft								
0										
Point 1	(737.5')	Depth=	(739.0' - 737.5') = 1.5'	→	$\frac{1.5'}{7.3'}$	=	0.21B	→	i = 1.00	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{2.5}\right) = 0.93$	$N_{60} = 62$	$N_{160} = 62 * 0.93 = 57.8$					
		$C' = 200$	Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand and gravel"					
$\Delta H = 3.0'$		$\frac{1}{200} \log\left(\frac{10.7 \text{ ksf}}{2.1 \text{ ksf}}\right)$	= 0.011 ft	= 0.13 in	$\Sigma H = 0.13 \text{ in}$					
Point 2	(731.0')	Depth=	(739.0' - 731.0') = 8.0'	→	$\frac{8.0'}{7.3'}$	=	1.10B	→	i = 0.51	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{2.8}\right) = 0.89$	$N_{60} = 59$	$N_{160} = 59 * 0.89 = 52.2$					
		$C' = 175$	Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand and gravel"					
$\Delta H = 10.0'$		$\frac{1}{175} \log\left(\frac{6.5 \text{ ksf}}{2.5 \text{ ksf}}\right)$	= 0.024 ft	= 0.28 in	$\Sigma H = 0.41 \text{ in}$					
Point 3	(720.4')	Depth=	(739.0' - 720.4') = 18.6'	→	$\frac{18.6'}{7.3'}$	=	2.56B	→	i = 0.25	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{3.4}\right) = 0.82$	$N_{60} = 69$	$N_{160} = 69 * 0.82 = 56.6$					
		$C' = 195$	Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand and gravel"					
$\Delta H = 11.2'$		$\frac{1}{195} \log\left(\frac{4.6 \text{ ksf}}{3.1 \text{ ksf}}\right)$	= 0.010 ft	= 0.12 in	$\Sigma H = 0.53 \text{ in}$					
Point 4	(713.6')	Depth=	(739.0' - 713.6') = 25.5'	→	$\frac{25.5'}{7.3'}$	=	3.51B	→	i = 0.18	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{3.8}\right) = 0.78$	$N_{60} = 84$	$N_{160} = 84 * 0.78 = 65.8$					
		$C' = 155$	Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand"					
$\Delta H = 2.5'$		$\frac{1}{155} \log\left(\frac{4.4 \text{ ksf}}{3.5 \text{ ksf}}\right)$	= 0.002 ft	= 0.02 in	$\Sigma H = 0.55 \text{ in}$					
Point 5	(711.3')	Depth=	(739.0' - 711.3') = 27.7'	→	$\frac{27.7'}{7.3'}$	=	3.82B	→	i = 0.16	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{4.0}\right) = 0.77$	$N_{60} = 50+$	$N_{160} = 50+ - 50$					
		$C' = 160$	Fig 10.6.2.4.2-1 (10-52)		"Inorganic silt"					
$\Delta H = 2.0'$		$\frac{1}{160} \log\left(\frac{4.3 \text{ ksf}}{3.6 \text{ ksf}}\right)$	= 0.001 ft	= 0.01 in	$\Sigma H = 0.56 \text{ in}$					



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716 Pier Load Case EHR & LL2
 Settlement - Pier Load Case EHR&LL2

JOB NUMBER 1021-1005.01
 SHEET NO. 4 OF 5
 COMP. BY EWT DATE 06/11/13
 CHECKED BY BTA DATE 06/11/13

Proposed Loading

Assume loading = 9378 psf

Assume width = 7.3 ft

Point 6 (709.1') Depth= (739.0' - 709.1') = 30.0' → $\frac{30.0'}{7.3'} = 4.13B$ → $i = 0.15$ Fig 10.6.2.4.1-1a (10-49)

Cohesive → $S_c = \frac{H_c}{1 + e_o} C_r \log\left(\frac{\sigma'_f}{\sigma'_o}\right)$ $C_r = \frac{WC}{1000} = \frac{8}{1000} = 0.008$ $e_o = \frac{2.75*WC}{100} = \frac{2.75 \times 8}{100} = 0.220$

C_r, e_o - FHWA Soils and Foundations (PG 152)

$S_c = \frac{2.5 \text{ ft}}{1 + 0.22} 0.008 \log\left(\frac{4.4 \text{ ksf}}{3.8 \text{ ksf}}\right) = 0.001 \text{ ft} = 0.01 \text{ in}$ $\Sigma H = 0.57 \text{ in}$

Point 7 (703.8') Depth= (739.0' - 703.8') = 35.2' → $\frac{35.2'}{7.3'} = 4.85B$ → $i = 0.13$ Fig 10.6.2.4.1-1a (10-49)

Granular → $\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$ $C_N = 0.77 \log\left(\frac{40}{4.4}\right) = 0.74$ $N_{60} = 50+$ $N_{160} = 50+ - 50$

$C' = 160$ Fig 10.6.2.4.2-1 (10-52) "Inorganic silt"

$\Delta H = 8.0' \frac{1}{160} \log\left(\frac{4.5 \text{ ksf}}{4.1 \text{ ksf}}\right) = 0.002 \text{ ft} = 0.03 \text{ in}$ $\Sigma H = 0.60 \text{ in}$

Point 8 (696.3') Depth= (739.0' - 696.3') = 42.7' → $\frac{42.7'}{7.3'} = 5.88B$ → $i = 0.11$ Fig 10.6.2.4.1-1a (10-49)

Cohesive → $S_c = \frac{H_c}{1 + e_o} C_r \log\left(\frac{\sigma'_f}{\sigma'_o}\right)$ $C_r = \frac{WC}{1000} = \frac{24}{1000} = 0.024$ $e_o = \frac{2.75*WC}{100} = \frac{2.75 \times 24}{100} = 0.660$

C_r, e_o - FHWA Soils and Foundations (PG 152)

$S_c = \frac{7.0 \text{ ft}}{1 + 0.66} 0.024 \log\left(\frac{4.8 \text{ ksf}}{4.6 \text{ ksf}}\right) = 0.002 \text{ ft} = 0.03 \text{ in}$ $\Sigma H = 0.63 \text{ in}$

13-Jun-13

Bridge 1716: Critical Case Determination for Settlement Analysis

Footing thickness= 4 ft
 Weight of footing based on design dimensions= 882 kips
 Bearing Elevation= 739 ft
 Top of footing= 743 ft
 Proposed grade elevation = 753.3 ft#

Load Case	W' =	L' =	#Estimated finished grade at Pier 1
EHr&LL1	7.27	99.30	
EHr&LL2	7.26	99.38	
EHr&LL3	7.27	98.93	
EHd&LL1	9.10	99.30	
EHd&LL2	9.04	99.38	
EHd&LL3	9.07	98.93	

EHD: Horizontal loads as a driving force
 EHR: Horizontal loads as a resisting force
 LL: Live Load
 W': effective footing width, ft
 L': effective footing length, ft

Load Case	Vertical Load from Loading sheet*	
EHr&LL1	7,702	kips
EHr&LL2	7,926	kips
EHr&LL3	7,800	kips
EHd&LL1	7,702	kips
EHd&LL2	7,926	kips
EHd&LL3	7,800	kips

*Vert Load=LL+DL+DW+Self WT+Footing

Vertical Load minus footing weight	
6820.00	kips
7044.00	kips
6918.00	kips
6820.00	kips
7044.00	kips
6918.00	kips

Load Case	Vertical Load excluding concrete weight below proposed grade**	
EHr&LL1	6,542	kips
EHr&LL2	6,766	kips
EHr&LL3	6,640	kips
EHd&LL1	6,542	kips
EHd&LL2	6,766	kips
EHd&LL3	6,640	kips

**Pier (concrete weight) between proposed grade & top of footing:

Pier height between proposed grade & top of footing= 10.3 feet
 Pier Column width= 3 feet
 Net Length of Pier= 60 feet
 Total length of void in pier column = 45 feet
 Length of Pier= 105 feet

Load Case	Pressure due to vertical Load*	
EHr&LL1	9,062	psf
EHr&LL2	9,378	psf
EHr&LL3	9,232	psf
EHd&LL1	7,240	psf
EHd&LL2	7,531	psf
EHd&LL3	7,400	psf

*Pressure due to vertical load, excluding concrete weight below proposed grade, on effective footing dimensions.

Load Case	I value at midpoint of layer							
	1	2	3	4	5	6	7	8
EHr&LL1	1	0.51	0.25	0.18	0.16	0.15	0.13	0.11
EHr&LL2	1	0.51	0.25	0.18	0.16	0.15	0.13	0.11
EHr&LL3	1	0.51	0.25	0.18	0.16	0.15	0.13	0.11
EHd&LL1	1	0.6	0.3	0.23	0.2	0.19	0.16	0.14
EHd&LL2	1	0.6	0.3	0.22	0.2	0.19	0.16	0.13
EHd&LL3	1	0.6	0.3	0.22	0.2	0.19	0.16	0.13

i: influence factor

Load Case	change in pressure due to vertical loads**							
	1	2	3	4	5	6	7	8
EHr&LL1	9,062	4,622	2,265	1,631	1,450	1,359	1,178	997
EHr&LL2	9,378	4,783	2,344	1,688	1,500	1,407	1,219	1,032
EHr&LL3	9,232	4,708	2,308	1,662	1,477	1,385	1,200	1,016
EHd&LL1	7,240	4,344	2,172	1,665	1,448	1,376	1,158	1,014
EHd&LL2	7,531	4,519	2,259	1,657	1,506	1,431	1,205	979
EHd&LL3	7,400	4,440	2,220	1,628	1,480	1,406	1,184	962

Yellow-highlighted is the critical case.

**Vertical loads excluding concrete weight below proposed grade.

Pier 2

Service Case Factors

WL	1
WS	0.3
TU	1
LL	1
LS	1
DC	1
EH	1
CE	1
DW	1
BR	1

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	329	3,816	549	1,456	0	832	329	3,816	549	1456	0	832.44
LL Case 2	198	3,816	549	1,456	0	832	198	3,816	549	1456	0	832.44
LL Case 3	420	3,816	549	1,456	0	832	420	3,816	549	1456	0	832.44

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment	Transverse Loading
LL Case 1	488	704	0	161	430	11,548	5,138	630	0	176	
LL Case 2	488	704	0	161	430	7,919	5,138	630	0	176	
LL Case 3	488	704	0	161	430	12,621	5,138	630	0	176	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	Factored Transverse Loading
LL Case 1	488	211	0	161	430	11,548	5,138	630	0	176	
LL Case 2	488	211	0	161	430	7,919	5,138	630	0	176	
LL Case 3	488	211	0	161	430	12,621	5,138	630	0	176	

LL Case 1	W =	14.00
	L =	99.10
LL Case 2	W =	14.00
	L =	99.10
LL Case 3	W =	14.00
	L =	99.10

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment	Longitudinal Loading
LL Case 1	22	1,704	1,009	10	6,272	558	0	0	0	2,574	
LL Case 2	22	1,704	1,009	10	6,272	558	0	0	0	2,574	
LL Case 3	22	1,704	1,009	10	6,272	558	0	0	0	2,574	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	Factored Longitudinal Loading
LL Case 1	22	511	1,009	10	6,272	558	0	0	0	2,574	
LL Case 2	22	511	1,009	10	6,272	558	0	0	0	2,574	
LL Case 3	22	511	1,009	10	6,272	558	0	0	0	2,574	

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	Factored Transverse Loading
LL Case 1	18,782	6,982	2.69	
LL Case 2	15,153	6,851	2.21	
LL Case 3	19,855	7,073	2.81	

Loading Pressure
Based on Effective Footing Size

LL Case 1	6,859 psf
LL Case 2	6,700 psf
LL Case 3	6,940 psf

LL Case 1	W' =	10.86
	L' =	93.72
LL Case 2	W' =	10.80
	L' =	94.68
LL Case 3	W' =	10.90
	L' =	93.49

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	Factored Longitudinal Loading
LL Case 1	10,956	6,982	1.57	
LL Case 2	10,956	6,851	1.60	
LL Case 3	10,956	7,073	1.55	

Footing thickness h= 4.00

PIER #2
WL Case 1

Strength I-a Case Factors

WL	0
WS	0
TU	0.5
LL	1.75
LS	1.75
DC	0.9
EH	1.5
CE	1.75
DW	0.65
BR	1.75

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	329	3,816	549	1,456	0	832	576	3,434	356.85	1310.4	0	749.196
LL Case 2	198	3,816	549	1,456	0	832	347	3,434	356.85	1310.4	0	749.196
LL Case 3	420	3,816	549	1,456	0	832	735	3,434	356.85	1310.4	0	749.196

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment	Transverse Loading
LL Case 1	488	704	0	161	430	11,548	5,138	630	0	176	
LL Case 2	488	704	0	161	430	7,919	5,138	630	0	176	
LL Case 3	488	704	0	161	430	12,621	5,138	630	0	176	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	Factored Transverse Loading
LL Case 1	0	0	0	282	215	20,209	4,624	410	0	308	
LL Case 2	0	0	0	282	215	13,858	4,624	410	0	308	
LL Case 3	0	0	0	282	215	22,087	4,624	410	0	308	

LL Case 1	W =	14.00
	L =	99.10
LL Case 2	W =	14.00
	L =	99.10
LL Case 3	W =	14.00
	L =	99.10

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment	Longitudinal Loading
LL Case 1	22	1,704	1,009	10	6,272	558	0	0	0	2,574	
LL Case 2	22	1,704	1,009	10	6,272	558	0	0	0	2,574	
LL Case 3	22	1,704	1,009	10	6,272	558	0	0	0	2,574	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	Factored Longitudinal Loading
LL Case 1	0	0	1,514	18	3,136	977	0	0	0	4,505	
LL Case 2	0	0	1,514	18	3,136	977	0	0	0	4,505	
LL Case 3	0	0	1,514	18	3,136	977	0	0	0	4,505	

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	e _{max} *	e < e _{max}	Factored Transverse Loading
LL Case 1	26,047	6,427	4.05	33.03	Yes	
LL Case 2	19,697	6,197	3.18	33.03	Yes	
LL Case 3	27,925	6,586	4.24	33.03	Yes	

*Eccentricity Check AASHTO 10.6.3.3, 6th Edition

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	e _{max} *	e < e _{max}	Factored Longitudinal Loading
LL Case 1	10,148	6,427	1.58	4.67	Yes	
LL Case 2	10,148	6,197	1.64	4.67	Yes	
LL Case 3	10,148	6,586	1.54	4.67	Yes	

*Eccentricity Check AASHTO 10.6.3.3, 6th Edition

LL Case 1	W' =	10.84
	L' =	90.99
LL Case 2	W' =	10.73
	L' =	92.74
LL Case 3	W' =	10.92
	L' =	90.62

Footing thickness h= 4.00

Loading Pressure
Based on Effective Footing Size

LL Case 1	6,514 psf
LL Case 2	6,231 psf
LL Case 3	6,656 psf

PIER #2
WL Case 1

Strength I-b Case Factors

WL	0
WS	0
TU	0.5
LL	1.75
LS	1.75
DC	1.25
EH	1.5
CE	1.75
DW	1.5
BR	1.75

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	329	3,816	549	1,456	0	832	576	4,770	823.5	1820	0	1040.55
LL Case 2	198	3,816	549	1,456	0	832	347	4,770	823.5	1820	0	1040.55
LL Case 3	420	3,816	549	1,456	0	832	735	4,770	823.5	1820	0	1040.55

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment	Transverse Loading
LL Case 1	488	704	0	161	430	11,548	5,138	630	0	176	
LL Case 2	488	704	0	161	430	7,919	5,138	630	0	176	
LL Case 3	488	704	0	161	430	12,621	5,138	630	0	176	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	Factored Transverse Loading
LL Case 1	0	0	0	282	215	20,209	6,423	945	0	308	
LL Case 2	0	0	0	282	215	13,858	6,423	945	0	308	
LL Case 3	0	0	0	282	215	22,087	6,423	945	0	308	

LL Case 1	W =	14.00
	L =	99.10
LL Case 2	W =	14.00
	L =	99.10
LL Case 3	W =	14.00
	L =	99.10

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment	Longitudinal Loading
LL Case 1	22	1,704	1,009	10	6,272	558	0	0	0	2,574	
LL Case 2	22	1,704	1,009	10	6,272	558	0	0	0	2,574	
LL Case 3	22	1,704	1,009	10	6,272	558	0	0	0	2,574	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	Factored Longitudinal Loading
LL Case 1	0	0	1,514	18	3,136	977	0	0	0	4,505	
LL Case 2	0	0	1,514	18	3,136	977	0	0	0	4,505	
LL Case 3	0	0	1,514	18	3,136	977	0	0	0	4,505	

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	Factored Transverse Loading
LL Case 1	28,381	9,030	3.14	
LL Case 2	22,031	8,801	2.50	
LL Case 3	30,259	9,189	3.29	

Loading Pressure
Based on Effective Footing Size

LL Case 1	8,278 psf
LL Case 2	7,998 psf
LL Case 3	8,424 psf

LL Case 1	W' =	11.75
	L' =	92.81
LL Case 2	W' =	11.69
	L' =	94.09
LL Case 3	W' =	11.79
	L' =	92.51

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	Factored Longitudinal Loading
LL Case 1	10,148	9,030	1.12	
LL Case 2	10,148	8,801	1.15	
LL Case 3	10,148	9,189	1.10	

Footing thickness h= 4.00



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 2 (WL1 & LL1)

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	746.0 ft	Foot. Width (B') =	11.8 ft	Foot. Length (L') =	92.8 ft		
Unfactored Dead Load =	6104.00 kips	Unfactored Live Load =	329.00 kips	Load =	75.23 k/lf		
Unfactored DW =	549.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)					
Max Factored Dead Load =	7630.00 kips	Max Factored Live Load =	575.75 kips	F. Load =	97.29 k/lf		
Max Factored DW =	823.50 kips						
Calculate factored bearing resistance (q_R)				Reference: AASHTO LRFD Bridge Design Specifications			
q _R = φ _b q _n				Eq. 10.6.3.1.1-1			
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2a-1,2,3,4			
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0			
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.02}$) = 1.00				Eq. 10.4.6.2.4-1			
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 34 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3			
N ₁₆₀ = 34.0 → φ _f = 35° to 40° Use φ _f = 36°				Table 10.4.6.2.4-1			
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2a-1			
Groundwater at 746.0 ft → (746' - 746') = 0.0' Water is within 1.5B of bearing depth							
C _{wq} = 1.0, C _{wy} = 0.5				Table 10.6.3.1.2a-2			
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{11.8'}{92.8'}$) = 0.95, s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{11.8'}{92.8'}$ tan 36°) = 1.09							
dq = 1.0				Table 10.6.3.1.2a-4			
q _n = 0 + (120.0 pcf) (1.0 ft) (37.8) (1.09) (1.0) (1.0) + (0.5) (120.0 pcf) (11.8 ft) (56.3) (0.95) (0.5)							
q _n = 4953.2 + 18841				For a 11.8 ft wide footing (eff. width 11.8 ft) q _n = 23.8 ksf			
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2			
				Table 10.5.5.2.2-1			
q _R = φ _b q _n = (0.45) (23.8 ksf) = <u>10.7 ksf</u> > 8.3 ksf (= 97.29 k/lf / 11.8 ft)							
Note on D_f = 1.0 foot:							
The ground surface on the west side of the footing will slope to a ditch (elevation 747), approx. 30' from the footing. For bearing resistance calculation, the bottom of the footing was set at elevation 746, one foot below the toe of the sloping ground to avoid any reduction in N _y . D _f was measured from the nearest level ground to the bottom of the footing.							



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 2 (WL1 & LL1)

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 Eq. 10.6.3.4-1

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ Eq. 10.6.3.4-2

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil Section 10.6.3.4
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):

concrete on sand → $\phi_t = 0.80$ Table 10.5.5.2.2-1

Foot. Length (L') = 91.0 ft → Effective Footing Length based on Strength I-a
 Foot. Width (W') = 10.8 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	21.5	kips
Sum of Factored Longitudinal Loads=	784.8	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	785.0	kips
Sum of Factored Vertical loads, V_{TOT} =	6426.2	kips

$R_t = V_{TOT} \tan \phi_f$
 $R_t = 4669$ kips

For $R_R = 3,735$ kips

Active Driving Horizontal Force, $H_{TOT} = 785.0$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a	Load Factors:
WL=	11.0 kips	WL=	1.0 kips	0	
WS=	20.0 kips	WS=	60.0 kips	0	
EH=	0.0 kips	EH=	268.0 kips	1.5	
CE+LS=	4.0 kips	CE+LS=	99.0 kips	1.75	CE and LS
TU=	15.0 kips	TU=	216.0 kips	0.5	
BR=	4 kips	BR=	58 kips	1.75	
Vertical Unfactored LL=	329.0 ft	kips		1.75	
Vertical Unfactored DL=	6104.0 ft	kips		0.9	
Vertical Unfactored DW=	549.0 ft	kips		0.65	



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 2 (WL1 & LL2)

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	746.0 ft	Foot. Width (B') =	11.7 ft	Foot. Length (L') =	94.1 ft			
Unfactored Dead Load =	6104.00 kips	Unfactored Live Load =	198.00 kips	Load =	72.81	k/lf		
Unfactored DW =	549.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)						
Max Factored Dead Load =	7630.00 kips	Max Factored Live Load =	346.50 kips	F. Load =	93.53	k/lf		
Max Factored DW =	823.50 kips							
Calculate factored bearing resistance (q_R)				Reference: AASHTO LRFD Bridge Design Specifications				
q _R = φ _b q _n				Eq. 10.6.3.1.1-1				
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2a-1,2,3,4				
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0				
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.02}$) = 1.00				Eq. 10.4.6.2.4-1				
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 34 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3				
N ₁₆₀ = 34.0 → φ _f = 35° to 40° Use φ _f = 36°				Table 10.4.6.2.4-1				
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2a-1				
Groundwater at 746.0 ft → (746' - 746') = 0.0' Water is within 1.5B of bearing depth								
C _{wq} = 1.0, C _{wy} = 0.5				Table 10.6.3.1.2a-2				
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{11.7'}{94.1'}$) = 0.95, s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{11.7'}{94.1'}$ tan 36°) = 1.09								
dq = 1.0				Table 10.6.3.1.2a-4				
q _n = 0 + (120.0 pcf) (1.0 ft) (37.8) (1.09) (1.0) (1.0) + (0.5) (120.0 pcf) (11.7 ft) (56.3) (0.95) (0.5)								
q _n = 4945.5 + 18763				For a 11.7 ft wide footing (eff. width 11.7 ft) q _n = 23.7 ksf				
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2				
				Table 10.5.5.2.2-1				
q _R = φ _b q _n = (0.45) (23.7 ksf) = <u>10.7 ksf</u> > 8.0 ksf (= 93.53 k/lf / 11.7 ft)								
Note on D_f = 1.0 foot:								
The ground surface on the west side of the footing will slope to a ditch (elevation 747), approx. 30' from the footing. For bearing resistance calculation, the bottom of the footing was set at elevation 746, one foot below the toe of the sloping ground to avoid any reduction in N _y . D _f was measured from the nearest level ground to the bottom of the footing.								



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 2 (WL1 & LL2)

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 Eq. 10.6.3.4-1

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ Eq. 10.6.3.4-2

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil Section 10.6.3.4
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):
 concrete on sand → $\phi_t = 0.80$ Table 10.5.5.2.2-1

Foot. Length (L') = 92.7 ft → Effective Footing Length based on Strength I-a
 Foot. Width (W') = 10.7 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	18.0	kips
Sum of Factored Longitudinal Loads=	784.8	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	785.0	kips
Sum of Factored Vertical loads, V_{TOT} =	6197.0	kips

$R_t = V_{TOT} \tan \phi_f$
 $R_t = 4502$ kips

For $R_R = 3,602$ kips Active Driving Horizontal Force, $H_{TOT} = 785.0$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a	Load Factors:
WL=	11.0 kips	WL=	1.0 kips	0	
WS=	20.0 kips	WS=	60.0 kips	0	
EH=	0.0 kips	EH=	268.0 kips	1.5	
CE+LS=	2.0 kips	CE+LS=	99.0 kips	1.75	CE and LS
TU=	15.0 kips	TU=	216.0 kips	0.5	
BR=	4 kips	BR=	58 kips	1.75	
Vertical Unfactored LL=	198.0 ft	kips		1.75	
Vertical Unfactored DL=	6104.0 ft	kips		0.9	
Vertical Unfactored DW=	549.0 ft	kips		0.65	



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 2 (WL1 & LL3)

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	746.0 ft	Foot. Width (B') =	11.8 ft	Foot. Length (L') =	92.5 ft			
Unfactored Dead Load =	6104.00 kips	Unfactored Live Load =	420.00 kips	Load =	76.46	k/lf		
Unfactored DW =	549.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)						
Max Factored Dead Load =	7630.00 kips	Max Factored Live Load =	735.00 kips	F. Load =	99.32	k/lf		
Max Factored DW =	823.50 kips							
Calculate factored bearing resistance (q_R)				Reference: AASHTO LRFD Bridge Design Specifications				
q _R = φ _b q _n				Eq. 10.6.3.1.1-1				
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2a-1,2,3,4				
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0				
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.02}$) = 1.00				Eq. 10.4.6.2.4-1				
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 34 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3				
N ₁₆₀ = 34.0 → φ _f = 35° to 40° Use φ _f = 36°				Table 10.4.6.2.4-1				
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2a-1				
Groundwater at 746.0 ft → (746' - 746') = 0.0'				Water is within 1.5B of bearing depth				
				C _{wq} = 1.0 , C _{wy} = 0.5 Table 10.6.3.1.2a-2				
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{11.8'}{92.5'}$) = 0.95				s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{11.8'}{92.5'}$ tan 36°) = 1.09				
dq = 1.0				Table 10.6.3.1.2a-4				
q _n = 0 + (120.0 pcf) (1.0 ft) (37.8) (1.09) (1.0) (1.0) + (0.5) (120.0 pcf) (11.8 ft) (56.3) (0.95) (0.5)								
q _n = 4956 + 18898				For a 11.8 ft wide footing (eff. width 11.8 ft) q _n = 23.9 ksf				
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2				
				Table 10.5.5.2.2-1				
q _R = φ _b q _n = (0.45) (23.9 ksf) = <u>10.7 ksf</u> > 8.4 ksf				(= 99.32 k/lf / 11.8 ft)				
Note on D_f = 1.0 foot:								
The ground surface on the west side of the footing will slope to a ditch (elevation 747), approx. 30' from the footing. For bearing resistance calculation, the bottom of the footing was set at elevation 746, one foot below the toe of the sloping ground to avoid any reduction in N _y . D _f was measured from the nearest level ground to the bottom of the footing.								



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 2 (WL1 & LL3)

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 *Eq. 10.6.3.4-1*

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ *Eq. 10.6.3.4-2*

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil *Section 10.6.3.4*
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):

concrete on sand → $\phi_t = 0.80$ *Table 10.5.5.2.2-1*

Foot. Length (L') = 90.6 ft → Effective Footing Length based on Strength I-a

Foot. Width (W') = 10.9 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	23.3	kips
Sum of Factored Longitudinal Loads=	784.8	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	785.1	kips
Sum of Factored Vertical loads, V_{TOT} =	6585.5	kips

$R_t = V_{TOT} \tan \phi_f$

$R_t = 4785$ kips

For $R_R = 3,828$ kips

Active Driving Horizontal Force, $H_{TOT} = 785.1$ kips

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

SLIDING IS: **OK**

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a	Load Factors:
WL=	11.0 kips	WL=	1.0 kips	0	
WS=	20.0 kips	WS=	60.0 kips	0	
EH=	0.0 kips	EH=	268.0 kips	1.5	
CE+LS=	5.0 kips	CE+LS=	99.0 kips	1.75	CE and LS
TU=	15.0 kips	TU=	216.0 kips	0.5	
BR=	4 kips	BR=	58 kips	1.75	
Vertical Unfactored LL=	420.0 ft	kips		1.75	
Vertical Unfactored DL=	6104.0 ft	kips		0.9	
Vertical Unfactored DW=	549.0 ft	kips		0.65	

Service Case Factors

WL	1
WS	0.3
TU	1
LL	1
LS	1
DC	1
EH	1
CE	1
DW	1
BR	1

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	329	3,816	549	1,456	0	832	329	3,816	549	1456	0	832.44
LL Case 2	198	3,816	549	1,456	0	832	198	3,816	549	1456	0	832.44
LL Case 3	420	3,816	549	1,456	0	832	420	3,816	549	1456	0	832.44

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment	Transverse Loading
LL Case 1	53	704	0	161	430	11,548	5,138	630	0	176	
LL Case 2	53	704	0	161	430	7,919	5,138	630	0	176	
LL Case 3	53	704	0	161	430	12,621	5,138	630	0	176	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	Factored Transverse Loading
LL Case 1	53	211	0	161	430	11,548	5,138	630	0	176	
LL Case 2	53	211	0	161	430	7,919	5,138	630	0	176	
LL Case 3	53	211	0	161	430	12,621	5,138	630	0	176	

LL Case 1	W =	14.00
	L =	99.10
LL Case 2	W =	14.00
	L =	99.10
LL Case 3	W =	14.00
	L =	99.10

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment	Longitudinal Loading
LL Case 1	138	1,704	1,009	10	6,272	558	0	0	0	2,574	
LL Case 2	138	1,704	1,009	10	6,272	558	0	0	0	2,574	
LL Case 3	138	1,704	1,009	10	6,272	558	0	0	0	2,574	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	Factored Longitudinal Loading
LL Case 1	138	511	1,009	10	6,272	558	0	0	0	2,574	
LL Case 2	138	511	1,009	10	6,272	558	0	0	0	2,574	
LL Case 3	138	511	1,009	10	6,272	558	0	0	0	2,574	

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	Factored Transverse Loading
LL Case 1	18,347	6,982	2.63	
LL Case 2	14,718	6,851	2.15	
LL Case 3	19,420	7,073	2.75	

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	Factored Longitudinal Loading
LL Case 1	11,072	6,982	1.59	
LL Case 2	11,072	6,851	1.62	
LL Case 3	11,072	7,073	1.57	

Loading Pressure
Based on Effective Footing Size

LL Case 1	6,871 psf
LL Case 2	6,712 psf
LL Case 3	6,952 psf

LL Case 1	W' =	10.83
	L' =	93.84
LL Case 2	W' =	10.77
	L' =	94.80
LL Case 3	W' =	10.87
	L' =	93.61

Footing thickness h= 4.00

PIER #2
WL Case 2

Strength I-a Case Factors

WL	0
WS	0
TU	0.5
LL	1.75
LS	1.75
DC	0.9
EH	1.5
CE	1.75
DW	0.65
BR	1.75

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	329	3,816	549	1,456	0	832	576	3,434	356.85	1310.4	0	749.196
LL Case 2	198	3,816	549	1,456	0	832	347	3,434	356.85	1310.4	0	749.196
LL Case 3	420	3,816	549	1,456	0	832	735	3,434	356.85	1310.4	0	749.196

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment	
LL Case 1	53	704	0	161	430	11,548	5,138	630	0	176	Transverse Loading
LL Case 2	53	704	0	161	430	7,919	5,138	630	0	176	
LL Case 3	53	704	0	161	430	12,621	5,138	630	0	176	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	
LL Case 1	0	0	0	282	215	20,209	4,624	410	0	308	Factored Transverse Loading
LL Case 2	0	0	0	282	215	13,858	4,624	410	0	308	
LL Case 3	0	0	0	282	215	22,087	4,624	410	0	308	

LL Case 1	W =	14.00
	L =	99.10
LL Case 2	W =	14.00
	L =	99.10
LL Case 3	W =	14.00
	L =	99.10

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment	
LL Case 1	138	1,704	1,009	10	6,272	558	0	0	0	2,574	Longitudinal Loading
LL Case 2	138	1,704	1,009	10	6,272	558	0	0	0	2,574	
LL Case 3	138	1,704	1,009	10	6,272	558	0	0	0	2,574	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	
LL Case 1	0	0	1,514	18	3,136	977	0	0	0	4,505	Factored Longitudinal Loading
LL Case 2	0	0	1,514	18	3,136	977	0	0	0	4,505	
LL Case 3	0	0	1,514	18	3,136	977	0	0	0	4,505	

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	e _{max} *	e < e _{max}	
LL Case 1	26,047	6,427	4.05	33.03	Yes	Factored Transverse Loading
LL Case 2	19,697	6,197	3.18	33.03	Yes	
LL Case 3	27,925	6,586	4.24	33.03	Yes	

*Eccentricity Check AASHTO 10.6.3.3, 6th Edition

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	e _{max} *	e < e _{max}	
LL Case 1	10,148	6,427	1.58	4.67	Yes	Factored Longitudinal Loading
LL Case 2	10,148	6,197	1.64	4.67	Yes	
LL Case 3	10,148	6,586	1.54	4.67	Yes	

*Eccentricity Check AASHTO 10.6.3.3, 6th Edition

LL Case 1	W' =	10.84
	L' =	90.99
LL Case 2	W' =	10.73
	L' =	92.74
LL Case 3	W' =	10.92
	L' =	90.62

Footing thickness h= 4.00

Loading Pressure
Based on Effective Footing Size

LL Case 1	6,514 psf
LL Case 2	6,231 psf
LL Case 3	6,656 psf

PIER #2
WL Case 2

Strength I-b Case Factors

WL	0
WS	0
TU	0.5
LL	1.75
LS	1.75
DC	1.25
EH	1.5
CE	1.75
DW	1.5
BR	1.75

Units in kips	Unfactored LL	Unfactored DL	Unfactored DW	Self Weight	Unfactored WS	Footing	Factored LL	Factored DL	Factored DW	F. Self Weight	Factored WS	F. Footing
LL Case 1	329	3,816	549	1,456	0	832	576	4,770	823.5	1820	0	1040.55
LL Case 2	198	3,816	549	1,456	0	832	347	4,770	823.5	1820	0	1040.55
LL Case 3	420	3,816	549	1,456	0	832	735	4,770	823.5	1820	0	1040.55

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LL moment	DL moment	DW moment	Self moment	BR moment	
LL Case 1	53	704	0	161	430	11,548	5,138	630	0	176	Transverse Loading
LL Case 2	53	704	0	161	430	7,919	5,138	630	0	176	
LL Case 3	53	704	0	161	430	12,621	5,138	630	0	176	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	
LL Case 1	0	0	0	282	215	20,209	6,423	945	0	308	Factored Transverse Loading
LL Case 2	0	0	0	282	215	13,858	6,423	945	0	308	
LL Case 3	0	0	0	282	215	22,087	6,423	945	0	308	

LL Case 1	W =	14.00
	L =	99.10
LL Case 2	W =	14.00
	L =	99.10
LL Case 3	W =	14.00
	L =	99.10

Units in kip-ft	WL moment	WS moment	EH moment	CE moment	TU moment	LS moment	DL moment	DW moment	Self moment	BR moment	
LL Case 1	138	1,704	1,009	10	6,272	558	0	0	0	2,574	Longitudinal Loading
LL Case 2	138	1,704	1,009	10	6,272	558	0	0	0	2,574	
LL Case 3	138	1,704	1,009	10	6,272	558	0	0	0	2,574	

	F. WL moment	F. WS moment	F. EH moment	F. CE moment	F. TU moment	F. LL moment	F. DL moment	F. DW moment	F. Self moment	F. BR moment	
LL Case 1	0	0	1,514	18	3,136	977	0	0	0	4,505	Factored Longitudinal Loading
LL Case 2	0	0	1,514	18	3,136	977	0	0	0	4,505	
LL Case 3	0	0	1,514	18	3,136	977	0	0	0	4,505	

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	
LL Case 1	28,381	9,030	3.14	Factored Transverse Loading
LL Case 2	22,031	8,801	2.50	
LL Case 3	30,259	9,189	3.29	

	Factored Moments Sum (kip-ft)	Sum Factored Vertical (kips)	e	
LL Case 1	10,148	9,030	1.12	Factored Longitudinal Loading
LL Case 2	10,148	8,801	1.15	
LL Case 3	10,148	9,189	1.10	

LL Case 1	W' =	11.75
	L' =	92.81
LL Case 2	W' =	11.69
	L' =	94.09
LL Case 3	W' =	11.79
	L' =	92.51

Loading Pressure
Based on Effective Footing Size

LL Case 1	8,278 psf
LL Case 2	7,998 psf
LL Case 3	8,424 psf

Footing thickness h= 4.00



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 2 (WL2 & LL1)

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	746.0 ft	Foot. Width (B') =	11.8 ft	Foot. Length (L') =	92.8 ft		
Unfactored Dead Load =	6104.00 kips	Unfactored Live Load =	329.00 kips	Load =	75.23 k/lf		
Unfactored DW =	549.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)					
Max Factored Dead Load =	7630.00 kips	Max Factored Live Load =	575.75 kips	F. Load =	97.29 k/lf		
Max Factored DW =	823.50 kips						
Calculate factored bearing resistance (q_R)				Reference: AASHTO LRFD Bridge Design Specifications			
q _R = φ _b q _n				Eq. 10.6.3.1.1-1			
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2a-1,2,3,4			
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0			
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.02}$) = 1.00				Eq. 10.4.6.2.4-1			
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 34 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3			
N ₁₆₀ = 34.0 → φ _f = 35° to 40° Use φ _f = 36°				Table 10.4.6.2.4-1			
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2a-1			
Groundwater at 746.0 ft → (746' - 746') = 0.0'				Water is within 1.5B of bearing depth			
C _{wq} = 1.0, C _{wy} = 0.5				Table 10.6.3.1.2a-2			
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{11.8'}{92.8'}$) = 0.95				s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{11.8'}{92.8'}$ tan 36°) = 1.09			
dq = 1.0				Table 10.6.3.1.2a-4			
q _n = 0 + (120.0 pcf) (1.0 ft) (37.8) (1.09) (1.0) (1.0) + (0.5) (120.0 pcf) (11.8 ft) (56.3) (0.95) (0.5)							
q _n = 4953.2 + 18841				For a 11.8 ft wide footing (eff. width 11.8 ft) q _n = 23.8 ksf			
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2			
				Table 10.5.5.2.2-1			
q _R = φ _b q _n = (0.45) (23.8 ksf) = <u>10.7 ksf</u> > 8.3 ksf				(= 97.29 k/lf / 11.8 ft)			
Note on D_f = 1.0 foot:							
The ground surface on the west side of the footing will slope to a ditch (elevation 747), approx. 30' from the footing. For bearing resistance calculation, the bottom of the footing was set at elevation 746, one foot below the toe of the sloping ground to avoid any reduction in N _y . D _f was measured from the nearest level ground to the bottom of the footing.							



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 2 (WL2 & LL1)

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 Eq. 10.6.3.4-1

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ Eq. 10.6.3.4-2

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil Section 10.6.3.4
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):

concrete on sand → $\phi_t = 0.80$ Table 10.5.5.2.2-1

Foot. Length (L') = 91.0 ft → Effective Footing Length based on Strength I-a

Foot. Width (W') = 10.8 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	21.5	kips
Sum of Factored Longitudinal Loads=	784.8	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	785.0	kips
Sum of Factored Vertical loads, V_{TOT} =	6426.2	kips

$R_t = V_{TOT} \tan \phi_f$
 $R_t = 4669$ kips

For $R_R = 3,735$ kips

Active Driving Horizontal Force, $H_{TOT} = 785.0$ kips

SLIDING IS: **OK**

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a	Load Factors:
WL=	1.0 kips	WL=	5.0 kips	0	
WS=	20.0 kips	WS=	60.0 kips	0	
EH=	0.0 kips	EH=	268.0 kips	1.5	
CE+LS=	4.0 kips	CE+LS=	99.0 kips	1.75	CE and LS
TU=	15.0 kips	TU=	216.0 kips	0.5	
BR=	4 kips	BR=	58 kips	1.75	
Vertical Unfactored LL=	329.0 ft	kips		1.75	
Vertical Unfactored DL=	6104.0 ft	kips		0.9	
Vertical Unfactored DW=	549.0 ft	kips		0.65	



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 2 (WL2 & LL2)

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	746.0 ft	Foot. Width (B') =	11.7 ft	Foot. Length (L') =	94.1 ft			
Unfactored Dead Load =	6104.00 kips	Unfactored Live Load =	198.00 kips	Load =	72.81	k/lf		
Unfactored DW =	549.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)						
Max Factored Dead Load =	7630.00 kips	Max Factored Live Load =	346.50 kips	F. Load =	93.53	k/lf		
Max Factored DW =	823.50 kips							
Calculate factored bearing resistance (q_R)				Reference: AASHTO LRFD Bridge Design Specifications				
q _R = φ _b q _n				Eq. 10.6.3.1.1-1				
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2a-1,2,3,4				
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0				
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.02}$) = 1.00				Eq. 10.4.6.2.4-1				
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 34 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3				
N ₁₆₀ = 34.0 → φ _f = 35° to 40° Use φ _f = 36°				Table 10.4.6.2.4-1				
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2a-1				
Groundwater at 746.0 ft → (746' - 746') = 0.0' Water is within 1.5B of bearing depth								
C _{wq} = 1.0, C _{wy} = 0.5				Table 10.6.3.1.2a-2				
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{11.7'}{94.1'}$) = 0.95				s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{11.7'}{94.1'}$ tan 36°) = 1.09				
dq = 1.0				Table 10.6.3.1.2a-4				
q _n = 0 + (120.0 pcf) (1.0 ft) (37.8) (1.09) (1.0) (1.0) + (0.5) (120.0 pcf) (11.7 ft) (56.3) (0.95) (0.5)								
q _n = 4945.5 + 18763				For a 11.7 ft wide footing (eff. width 11.7 ft) q _n = 23.7 ksf				
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2				
				Table 10.5.5.2.2-1				
q _R = φ _b q _n = (0.45) (23.7 ksf) = <u>10.7 ksf</u> > 8.0 ksf (= 93.53 k/lf / 11.7 ft)								
Note on D_f = 1.0 foot:								
The ground surface on the west side of the footing will slope to a ditch (elevation 747), approx. 30' from the footing. For bearing resistance calculation, the bottom of the footing was set at elevation 746, one foot below the toe of the sloping ground to avoid any reduction in N _γ . D _f was measured from the nearest level ground to the bottom of the footing.								



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 2 (WL2 & LL2)

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 Eq. 10.6.3.4-1

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ Eq. 10.6.3.4-2

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil Section 10.6.3.4
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):

concrete on sand → $\phi_t = 0.80$ Table 10.5.5.2.2-1

Foot. Length (L') = 92.7 ft → Effective Footing Length based on Strength I-a

Foot. Width (W') = 10.7 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	18.0	kips
Sum of Factored Longitudinal Loads=	784.8	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	785.0	kips
Sum of Factored Vertical loads, V_{TOT} =	6197.0	kips

$R_t = V_{TOT} \tan \phi_f$
 $R_t = 4502$ kips

For $R_R = 3,602$ kips

Active Driving Horizontal Force, $H_{TOT} = 785.0$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a Load Factors:	
WL=	1.0 kips	WL=	5.0 kips		0
WS=	20.0 kips	WS=	60.0 kips		0
EH=	0.0 kips	EH=	268.0 kips		1.5
CE+LS=	2.0 kips	CE+LS=	99.0 kips		1.75
TU=	15.0 kips	TU=	216.0 kips		0.5
BR=	4 kips	BR=	58 kips		1.75
Vertical Unfactored LL=	198.0 ft	kips			1.75
Vertical Unfactored DL=	6104.0 ft	kips			0.9
Vertical Unfactored DW=	549.0 ft	kips			0.65



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Bearing Capacity - Pier 2 (WL2 & LL3)

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Bearing Depth =	746.0 ft	Foot. Width (B') =	11.8 ft	Foot. Length (L') =	92.5 ft			
Unfactored Dead Load =	6104.00 kips	Unfactored Live Load =	420.00 kips	Load =	76.46	k/lf		
Unfactored DW =	549.00 kips	Dead Load=(DL)+(Self Wt)+(Footing Wt)						
Max Factored Dead Load =	7630.00 kips	Max Factored Live Load =	735.00 kips	F. Load =	99.32	k/lf		
Max Factored DW =	823.50 kips							
Calculate factored bearing resistance (q_R)				Reference: AASHTO LRFD Bridge Design Specifications				
q _R = φ _b q _n				Eq. 10.6.3.1.1-1				
q _n = cN _{cm} + γD _f N _{qm} C _{wq} + 0.5γBN _{ym} C _{wy}				Eq. 10.6.3.1.2a-1,2,3,4				
(N _{cm} = N _c S _c i _c N _{qm} = N _q S _q d _q i _q N _{ym} = N _y S _y i _y)				Assume inclination factors (i _c , i _q , i _y) = 1.0				
Calc. C _N → 0.77 log ($\frac{40}{\sigma'_{vo}}$) → 0.77 log ($\frac{40}{2.02}$) = 1.00				Eq. 10.4.6.2.4-1				
Calc. N ₁₆₀ → C _N N ₆₀ N ₆₀ = 34 (Boring B-287-1-10)				Eq. 10.4.6.2.4-3				
N ₁₆₀ = 34.0 → φ _f = 35° to 40° Use φ _f = 36°				Table 10.4.6.2.4-1				
φ _f = 36° → N _q = 37.8 N _y = 56.3				Table 10.6.3.1.2a-1				
Groundwater at 746.0 ft → (746' - 746') = 0.0'				Water is within 1.5B of bearing depth				
				C _{wq} = 1.0, C _{wy} = 0.5 Table 10.6.3.1.2a-2				
s _γ = 1 - 0.4 ($\frac{B}{L}$) = 1 - 0.4 ($\frac{11.8'}{92.5'}$) = 0.95				s _q = 1 + ($\frac{B}{L}$ tan φ _f) = 1 + ($\frac{11.8'}{92.5'}$ tan 36°) = 1.09				
dq = 1.0				Table 10.6.3.1.2a-4				
q _n = 0 + (120.0 pcf) (1.0 ft) (37.8) (1.09) (1.0) (1.0) + (0.5) (120.0 pcf) (11.8 ft) (56.3) (0.95) (0.5)								
q _n = 4956 + 18898				For a 11.8 ft wide footing (eff. width 11.8 ft) q _n = 23.9 ksf				
Resistance Factor (φ _b) = 0.45 strength limit state				Section 10.5.5.2				
				Table 10.5.5.2.2-1				
q _R = φ _b q _n = (0.45) (23.9 ksf) = <u>10.7 ksf</u> > 8.4 ksf				(= 99.32 k/lf / 11.8 ft)				
Note on D_f = 1.0 foot:								
The ground surface on the west side of the footing will slope to a ditch (elevation 747), approx. 30' from the footing. For bearing resistance calculation, the bottom of the footing was set at elevation 746, one foot below the toe of the sloping ground to avoid any reduction in N _γ . D _f was measured from the nearest level ground to the bottom of the footing.								



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716
 Sliding - Pier 2 (WL2 & LL3)

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 2
 COMP. BY EWT DATE 06/10/13
 CHECKED BY BTA DATE 06/10/13

Sliding (R_R)

$R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep}$ assume passive resistance, R_{ep} , is 0 *Eq. 10.6.3.4-1*

Cohesionless foundation soil → nominal sliding resistance $R_t = V \tan \delta$ *Eq. 10.6.3.4-2*

$\tan \delta = 1.0 * \tan \phi_f$ for concrete cast against soil *Section 10.6.3.4*
 $= 1.0 * \tan 36 = 0.73$

Resistance factor (ϕ_t):

concrete on sand → $\phi_t = 0.80$ *Table 10.5.5.2.2-1*

Foot. Length (L') = 90.6 ft → Effective Footing Length based on Strength I-a
 Foot. Width (W') = 10.9 ft → Effective Footing Width based on Strength I-a

Sum of Factored Transverse Loads=	23.3	kips
Sum of Factored Longitudinal Loads=	784.8	kips
Resultant of Total Factored Transverse and Longitudinal Loads=	785.1	kips
Sum of Factored Vertical loads, V_{TOT} =	6585.5	kips

$R_t = V_{TOT} \tan \phi_f$
 $R_t = 4785$ kips

For $R_R = 3,828$ kips

Active Driving Horizontal Force, $H_{TOT} = 785.1$ kips

SLIDING IS: OK

Conservatively, the absolute magnitude of the horizontal forces were used to determine the active driving horizontal force.

Note: No active or passive soil forces were included in the analysis

Loading Information Provided:

Transverse Loads at Pier Location:		Longitudinal Loads at Pier Location:		Strength I-a Load Factors:
WL=	1.0 kips	WL=	5.0 kips	0
WS=	20.0 kips	WS=	60.0 kips	0
EH=	0.0 kips	EH=	268.0 kips	1.5
CE+LS=	5.0 kips	CE+LS=	99.0 kips	1.75
TU=	15.0 kips	TU=	216.0 kips	0.5
BR=	4 kips	BR=	58 kips	1.75
Vertical Unfactored LL=	420.0 ft	kips		1.75
Vertical Unfactored DL=	6104.0 ft	kips		0.9
Vertical Unfactored DW=	549.0 ft	kips		0.65



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716 Pier Load Case WL2&LL3
 Boring Overburden

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 5
 COMP. BY EWT DATE 06/11/13
 CHECKED BY BTA DATE 06/11/13

Based on boring B-287-1-10	Existing Fndn Load ^a	psf	Bearing Depth	746.0 ft	Water Elev.	746.0 ft
	Layer	Zone (ft)	Mid-Pt Elev (ft)	γ_m (pcf)	Hc (ft)	
-----762.8'	A	762.8 to 746	-	120	-	
-----746.0'	B	746 to 736	741	120	10	
-----736.0'	C	736 to 726	731	120	10	
-----736.0'	D	726 to 714.8	720.4	120	11.2	
-----726.0'	E	714.8 to 712.3	713.55	120	2.5	
-----726.0'	F	712.3 to 710.3	711.3	120	2	
-----714.8'	G	710.3 to 707.8	709.05	130	2.5	
-----714.8'	H	707.8 to 699.8	703.8	120	8	
-----707.8'	I	699.8 to 692.8	696.3	130	7	
-----699.8'						
-----692.8'						

Calculate Existing Load

Point 1

$$\begin{aligned} \sigma'_{o1} &= 2016 \text{ psf} + (746' - 741')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2016 \text{ psf} + (5.0')(57.6 \text{ pcf}) \\ &= 2304 \text{ psf} \end{aligned}$$

Point 2

$$\begin{aligned} \sigma'_{o2} &= 2304 \text{ psf} + (741' - 731')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2304 \text{ psf} + (10.0')(57.6 \text{ pcf}) \\ &= 2880 \text{ psf} \end{aligned}$$

Point 3

$$\begin{aligned} \sigma'_{o3} &= 2880 \text{ psf} + (731' - 720.4')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2880 \text{ psf} + (10.6')(57.6 \text{ pcf}) \\ &= 3491 \text{ psf} \end{aligned}$$

Point 4

$$\begin{aligned} \sigma'_{o4} &= 3491 \text{ psf} + (720.4' - 713.55')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3491 \text{ psf} + (6.9')(57.6 \text{ pcf}) \\ &= 3886 \text{ psf} \end{aligned}$$

Point 5

$$\begin{aligned} &= 3886 \text{ psf} + (713.55' - 711.3')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3886 \text{ psf} + (2.3')(57.6 \text{ pcf}) \\ &= 4016 \text{ psf} \end{aligned}$$

Point 6

$$\begin{aligned} &= 4016 \text{ psf} + (711.3' - 710.3')(120 \text{ pcf} - 62.4 \text{ pcf}) + (710.3' - 709.05')(130 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4016 \text{ psf} + (1.0')(57.6 \text{ pcf}) + (1.3')(67.6 \text{ pcf}) \\ &= 4158 \text{ psf} \end{aligned}$$

Point 7

$$\begin{aligned} &= 4158 \text{ psf} + (709.05' - 707.8')(130 \text{ pcf} - 62.4 \text{ pcf}) + (707.8' - 703.8')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4158 \text{ psf} + (1.3')(67.6 \text{ pcf}) + (4.0')(57.6 \text{ pcf}) \\ &= 4473 \text{ psf} \end{aligned}$$

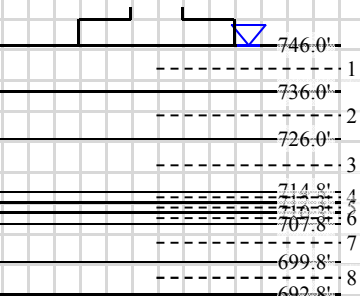
Point 8

$$\begin{aligned} &= 4473 \text{ psf} + (703.8' - 699.8')(120 \text{ pcf} - 62.4 \text{ pcf}) + (699.8' - 696.3')(130 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4473 \text{ psf} + (4.0')(57.6 \text{ pcf}) + (3.5')(67.6 \text{ pcf}) \\ &= 4940 \text{ psf} \end{aligned}$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716 Pier Load Case WL2&LL3
 Existing Overburden

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 5
 COMP. BY EWT DATE 06/11/13
 CHECKED BY BTA DATE 06/11/13

Based on boring B-287-1-10	Existing Fndn Load ^u		Bearing Depth		746.0 ft	Water Elev.	746.0 ft
	Layer	Zone (ft)		Mid-Pt Elev (ft)	γ_m (pcf)	Hc (ft)	
-----765.0	A	765	to 746	-	120	-	
	B	746	to 736	741	120	10	
-----736.0	C	736	to 726	731	120	10	
-----736.0	D	726	to 714.8	720.4	120	11.2	
-----726.0	E	714.8	to 712.3	713.55	120	2.5	
-----726.0	F	712.3	to 710.3	711.3	120	2	
-----714.0	D	710.3	to 707.8	709.05	130	2.5	
-----714.0	E	707.8	to 699.8	703.8	120	8	
-----707.8	F	699.8	to 692.8	696.3	130	7	
-----699.8							
-----692.8							

Calculate Existing Load

Point 1

$$\begin{aligned} \sigma'_{o1} &= 2280 \text{ psf} + (746' - 741')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2280 \text{ psf} + (5.0')(57.6 \text{ pcf}) \\ &= 2568 \text{ psf} \end{aligned}$$

Point 2

$$\begin{aligned} \sigma'_{o2} &= 2568 \text{ psf} + (741' - 731')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2568 \text{ psf} + (10.0')(57.6 \text{ pcf}) \\ &= 3144 \text{ psf} \end{aligned}$$

Point 3

$$\begin{aligned} \sigma'_{o3} &= 3144 \text{ psf} + (731' - 720.4')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3144 \text{ psf} + (10.6')(57.6 \text{ pcf}) \\ &= 3755 \text{ psf} \end{aligned}$$

Point 4

$$\begin{aligned} \sigma'_{o4} &= 3755 \text{ psf} + (720.4' - 713.55')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 3755 \text{ psf} + (6.9')(57.6 \text{ pcf}) \\ &= 4150 \text{ psf} \end{aligned}$$

Point 5

$$\begin{aligned} &= 4150 \text{ psf} + (713.55' - 711.3')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4150 \text{ psf} + (2.3')(57.6 \text{ pcf}) \\ &= 4280 \text{ psf} \end{aligned}$$

Point 6

$$\begin{aligned} &= 4280 \text{ psf} + (711.3' - 710.3')(120 \text{ pcf} - 62.4 \text{ pcf}) + (710.3' - 709.05')(130 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4280 \text{ psf} + (1.0')(57.6 \text{ pcf}) + (1.3')(67.6 \text{ pcf}) \\ &= 4422 \text{ psf} \end{aligned}$$

Point 7

$$\begin{aligned} &= 4422 \text{ psf} + (709.05' - 707.8')(130 \text{ pcf} - 62.4 \text{ pcf}) + (707.8' - 703.8')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4422 \text{ psf} + (1.3')(67.6 \text{ pcf}) + (4.0')(57.6 \text{ pcf}) \\ &= 4737 \text{ psf} \end{aligned}$$

Point 8

$$\begin{aligned} &= 4737 \text{ psf} + (703.8' - 699.8')(120 \text{ pcf} - 62.4 \text{ pcf}) + (699.8' - 696.3')(130 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4737 \text{ psf} + (4.0')(57.6 \text{ pcf}) + (3.5')(67.6 \text{ pcf}) \\ &= 5204 \text{ psf} \end{aligned}$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716 Pier Load Case WL2&LL3
 Proposed Overburden

JOB NUMBER 1021-1005.01
 SHEET NO. 3 OF 5
 COMP. BY EWT DATE 06/11/13
 CHECKED BY BTA DATE 06/11/13

Based on boring B-287-1-10	Foundation Load ^a	5894 psf	Bearing Depth	746.0 ft	Water Elev.	746.0 ft
	Layer	Zone (ft)	Mid-Pt Elev (ft)	γ_m (pcf)	Hc (ft)	i*
760.0'	A	760.0 to 746	-	120	-	
746.0'	B	746 to 736	741	120	10	0.80
736.0'	C	736 to 726	731	120	10	0.41
726.0'	D	726 to 714.8	720.4	120	11.2	0.27
	E	714.8 to 712.3	713.55	120	2.5	0.20
	F	712.3 to 710.3	711.3	120	2	0.19
714.8'	D	710.3 to 707.8	709.05	130	2.5	0.18
707.8'	E	707.8 to 699.8	703.8	120	8	0.16
699.8'	F	699.8 to 692.8	696.3	130	7	0.14
692.8'						

Calculate Proposed Load

^a - *fndn load at center of footing*

* - *see settlement calcs for determination of i values*

Point 1

$$\begin{aligned} \sigma'_{o1} &= 1680 \text{ psf} + (746' - 741')(120 \text{ pcf} - 62.4 \text{ pcf}) + (5894 \text{ psf})(0.8) \\ &= 1680 \text{ psf} + (5.0')(57.6 \text{ pcf}) + (4715 \text{ psf}) \\ &= 6683.2 \text{ psf} \end{aligned}$$

Point 2

$$\begin{aligned} \sigma'_{o2} &= 1968 \text{ psf} + (741' - 731')(120 \text{ pcf} - 62.4 \text{ pcf}) + (5894 \text{ psf})(0.41) \\ &= 1968 \text{ psf} + (10.0')(57.6 \text{ pcf}) + (2417 \text{ psf}) \\ &= 4960.54 \text{ psf} \end{aligned}$$

Point 3

$$\begin{aligned} \sigma'_{o3} &= 2544 \text{ psf} + (731' - 720.4')(120 \text{ pcf} - 62.4 \text{ pcf}) + (5894 \text{ psf})(0.27) \\ &= 2544 \text{ psf} + (10.6')(57.6 \text{ pcf}) + (1591 \text{ psf}) \\ &= 4746.38 \text{ psf} \end{aligned}$$

Point 4

$$\begin{aligned} \sigma'_{o4} &= 3155 \text{ psf} + (720.4' - 713.55')(120 \text{ pcf} - 62.4 \text{ pcf}) + (5894 \text{ psf})(0.2) \\ &= 3155 \text{ psf} + (6.9')(57.6 \text{ pcf}) + (1179 \text{ psf}) \\ &= 4728.8 \text{ psf} \end{aligned}$$

Point 5

$$\begin{aligned} &= 3550 \text{ psf} + (713.55' - 711.3')(120 \text{ pcf} - 62.4 \text{ pcf}) + (5894 \text{ psf})(0.19) \\ &= 3550 \text{ psf} + (2.3')(57.6 \text{ pcf}) + (1120 \text{ psf}) \\ &= 4799.86 \text{ psf} \end{aligned}$$

Point 6

$$\begin{aligned} &= 3680 \text{ psf} + (711.3' - 710.3')(120 \text{ pcf} - 62.4 \text{ pcf}) + (710.3' - 709.05')(130 \text{ pcf} - 62.4 \text{ pcf}) + (5894 \text{ psf})(0.18) \\ &= 3680 \text{ psf} + (1.0')(57.6 \text{ pcf}) + (1.3')(67.6 \text{ pcf}) + (1061 \text{ psf}) \\ &= 4882.92 \text{ psf} \end{aligned}$$

Point 7

$$\begin{aligned} &= 3822 \text{ psf} + (709.05' - 707.8')(130 \text{ pcf} - 62.4 \text{ pcf}) + (707.8' - 703.8')(120 \text{ pcf} - 62.4 \text{ pcf}) + (5894 \text{ psf})(0.16) \\ &= 3822 \text{ psf} + (1.3')(67.6 \text{ pcf}) + (4.0')(57.6 \text{ pcf}) + (943 \text{ psf}) \\ &= 5080.04 \text{ psf} \end{aligned}$$

Point 8

$$\begin{aligned} &= 4137 \text{ psf} + (703.8' - 699.8')(120 \text{ pcf} - 62.4 \text{ pcf}) + (699.8' - 696.3')(130 \text{ pcf} - 62.4 \text{ pcf}) + (5894 \text{ psf})(0.14) \\ &= 4137 \text{ psf} + (4.0')(57.6 \text{ pcf}) + (3.5')(67.6 \text{ pcf}) + (825 \text{ psf}) \\ &= 5429.16 \text{ psf} \end{aligned}$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716 Pier Load Case WL2&LL3
 Settlement - Pier Load Case WL2&LL3

JOB NUMBER 1021-1005.01
 SHEET NO. 4 OF 5
 COMP. BY EWT DATE 06/11/13
 CHECKED BY BTA DATE 06/11/13

Proposed Loading										
Assume loading =		5894 psf								
Assume width =		10.9 ft								
Point 1	(741.0')	Depth=	(746.0' - 741.0') = 5.0'	→	$\frac{5.0'}{10.9'}$	=	0.46B	→	i = 0.80	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{2.3}\right) = 0.95$	$N_{60} = 56$	$N_{160} = 56 * 0.95 = 53.4$					
		$C' = 180$		Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand and gravel"				
$\Delta H = 10.0'$		$\frac{1}{180} \log\left(\frac{6.7 \text{ ksf}}{2.6 \text{ ksf}}\right) = 0.023 \text{ ft} = 0.28 \text{ in}$		$\Sigma H = 0.28 \text{ in}$						
Point 2	(731.0')	Depth=	(746.0' - 731.0') = 15.0'	→	$\frac{15.0'}{10.9'}$	=	1.38B	→	i = 0.41	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{2.9}\right) = 0.88$	$N_{60} = 59$	$N_{160} = 59 * 0.88 = 51.9$					
		$C' = 178$		Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand and gravel"				
$\Delta H = 10.0'$		$\frac{1}{178} \log\left(\frac{5.0 \text{ ksf}}{3.1 \text{ ksf}}\right) = 0.011 \text{ ft} = 0.13 \text{ in}$		$\Sigma H = 0.41 \text{ in}$						
Point 3	(720.4')	Depth=	(746.0' - 720.4') = 25.6'	→	$\frac{25.6'}{10.9'}$	=	2.36B	→	i = 0.27	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{3.5}\right) = 0.82$	$N_{60} = 69$	$N_{160} = 69 * 0.82 = 56.2$					
		$C' = 195$		Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand and gravel"				
$\Delta H = 11.2'$		$\frac{1}{195} \log\left(\frac{4.7 \text{ ksf}}{3.8 \text{ ksf}}\right) = 0.006 \text{ ft} = 0.07 \text{ in}$		$\Sigma H = 0.48 \text{ in}$						
Point 4	(713.6')	Depth=	(746.0' - 713.6') = 32.5'	→	$\frac{32.5'}{10.9'}$	=	2.99B	→	i = 0.20	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{3.9}\right) = 0.78$	$N_{60} = 84$	$N_{160} = 84 * 0.78 = 65.4$					
		$C' = 152$		Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand"				
$\Delta H = 2.5'$		$\frac{1}{152} \log\left(\frac{4.7 \text{ ksf}}{4.2 \text{ ksf}}\right) = 0.001 \text{ ft} = 0.01 \text{ in}$		$\Sigma H = 0.49 \text{ in}$						
Point 5	(711.3')	Depth=	(746.0' - 711.3') = 34.7'	→	$\frac{34.7'}{10.9'}$	=	3.19B	→	i = 0.19	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{4.0}\right) = 0.77$	$N_{60} = 50+$	$N_{160} = 50+ - 50$					
		$C' = 160$		Fig 10.6.2.4.2-1 (10-52)		"Inorganic silt"				
$\Delta H = 2.0'$		$\frac{1}{160} \log\left(\frac{4.8 \text{ ksf}}{4.3 \text{ ksf}}\right) = 0.001 \text{ ft} = 0.01 \text{ in}$		$\Sigma H = 0.50 \text{ in}$						



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-70-1716 Pier Load Case WL2&LL3
 Settlement - Pier Load Case WL2&LL3

JOB NUMBER 1021-1005.01
 SHEET NO. 4 OF 5
 COMP. BY EWT DATE 06/11/13
 CHECKED BY BTA DATE 06/11/13

Proposed Loading

Assume loading = 5894 psf

Assume width = 10.9 ft

Point 6 (709.1') Depth= (746.0' - 709.1') = 37.0' → $\frac{37.0'}{10.9'} = 3.40B$ → $i = 0.18$ Fig 10.6.2.4.1-1a (10-49)

Cohesive → $S_c = \frac{H_c}{1 + e_o} C_r \log\left(\frac{\sigma'_f}{\sigma'_o}\right)$ $C_r = \frac{WC}{1000} = \frac{8}{1000} = 0.008$ $e_o = \frac{2.75*WC}{100} = \frac{2.75 \times 8}{100} = 0.220$

C_r, e_o - FHWA Soils and Foundations (PG 152)

$S_c = \frac{2.5 \text{ ft}}{1 + 0.22} 0.008 \log\left(\frac{4.9 \text{ ksf}}{4.4 \text{ ksf}}\right) = 0.001 \text{ ft} = 0.01 \text{ in}$ $\Sigma H = 0.51 \text{ in}$

Point 7 (703.8') Depth= (746.0' - 703.8') = 42.2' → $\frac{42.2'}{10.9'} = 3.88B$ → $i = 0.16$ Fig 10.6.2.4.1-1a (10-49)

Granular → $\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$ $C_N = 0.77 \log\left(\frac{40}{4.5}\right) = 0.73$ $N_{60} = 50+$ $N_{160} = 50+ - 50$

$C' = 160$ Fig 10.6.2.4.2-1 (10-52) "Inorganic silt"

$\Delta H = 8.0' \frac{1}{160} \log\left(\frac{5.1 \text{ ksf}}{4.7 \text{ ksf}}\right) = 0.002 \text{ ft} = 0.02 \text{ in}$ $\Sigma H = 0.53 \text{ in}$

Point 8 (696.3') Depth= (746.0' - 696.3') = 49.7' → $\frac{49.7'}{10.9'} = 4.57B$ → $i = 0.14$ Fig 10.6.2.4.1-1a (10-49)

Cohesive → $S_c = \frac{H_c}{1 + e_o} C_r \log\left(\frac{\sigma'_f}{\sigma'_o}\right)$ $C_r = \frac{WC}{1000} = \frac{24}{1000} = 0.024$ $e_o = \frac{2.75*WC}{100} = \frac{2.75 \times 24}{100} = 0.660$

C_r, e_o - FHWA Soils and Foundations (PG 152)

$S_c = \frac{7.0 \text{ ft}}{1 + 0.66} 0.024 \log\left(\frac{5.4 \text{ ksf}}{5.2 \text{ ksf}}\right) = 0.002 \text{ ft} = 0.02 \text{ in}$ $\Sigma H = 0.55 \text{ in}$

19-Aug-13

Bridge 1716: Critical Case Determination for Settlement Analysis

Footing thickness= 4 ft
 Weight of footing based on design dimensions= 832 kips
 Bearing Elevation= 746 ft
 Top of footing= 750 ft
 Proposed grade elevation = 760 ft#

Load Case	W' =	L' =	#Estimated finished grade at Pier 2
WL1&LL1	10.86	93.72	
WL1&LL2	10.80	94.68	
WL1&LL3	10.90	93.49	
WL2&LL1	10.83	93.84	
WL2&LL2	10.77	94.80	
WL2&LL3	10.87	93.61	

WL: Wind Load on traffic
 LL: Live Load
 W': effective footing width, ft
 L': effective footing length, ft

Load Case	Vertical Load from Loading sheet*	
WL1&LL1	6,982	kips
WL1&LL2	6,851	kips
WL1&LL3	7,073	kips
WL2&LL1	6,982	kips
WL2&LL2	6,851	kips
WL2&LL3	7,073	kips

*Vert Load=LL+DL+DW+Self WT+Footing

Vertical Load minus footing weight

6150.00	kips
6019.00	kips
6241.00	kips
6150.00	kips
6019.00	kips
6241.00	kips

Load Case	Vertical Load excluding concrete weight below proposed grade**	
WL1&LL1	5,907	kips
WL1&LL2	5,776	kips
WL1&LL3	5,998	kips
WL2&LL1	5,907	kips
WL2&LL2	5,776	kips
WL2&LL3	5,998	kips

**Pier (concrete weight) between proposed grade & top of footing:

Pier height between proposed grade & top of footing= 10 feet
 Pier Column width= 3 feet
 Net Length of Pier= 54.1 feet
 Total length of void in pier column = 45 feet
 Length of Pier= 99.1 feet

Load Case	Pressure due to vertical Load*	
WL1&LL1	5,803	psf
WL1&LL2	5,648	psf
WL1&LL3	5,885	psf
WL2&LL1	5,812	psf
WL2&LL2	5,657	psf
WL2&LL3	5,894	psf

*Pressure due to vertical load, excluding concrete weight below proposed grade, on effective footing dimensions.

Load Case	I value at midpoint of layer							
	1	2	3	4	5	6	7	8
WL1&LL1	0.80	0.41	0.27	0.20	0.19	0.18	0.16	0.14
WL1&LL2	0.80	0.41	0.26	0.20	0.19	0.18	0.16	0.14
WL1&LL3	0.80	0.41	0.27	0.20	0.19	0.18	0.16	0.14
WL2&LL1	0.80	0.41	0.27	0.20	0.19	0.18	0.16	0.14
WL2&LL2	0.80	0.41	0.26	0.20	0.19	0.18	0.16	0.14
WL2&LL3	0.80	0.41	0.27	0.20	0.19	0.18	0.16	0.14

i: influence factor

change in pressure due to vertical loads**								
WL1&LL1	4,643	2,379	1,567	1,161	1,103	1,045	929	812
WL1&LL2	4,519	2,316	1,469	1,130	1,073	1,017	904	791
WL1&LL3	4,708	2,413	1,589	1,177	1,118	1,059	942	824
WL2&LL1	4,650	2,383	1,569	1,162	1,104	1,046	930	814
WL2&LL2	4,525	2,319	1,471	1,131	1,075	1,018	905	792
WL2&LL3	4,715	2,417	1,591	1,179	1,120	1,061	943	825

**Vertical loads excluding concrete weight below proposed grade.

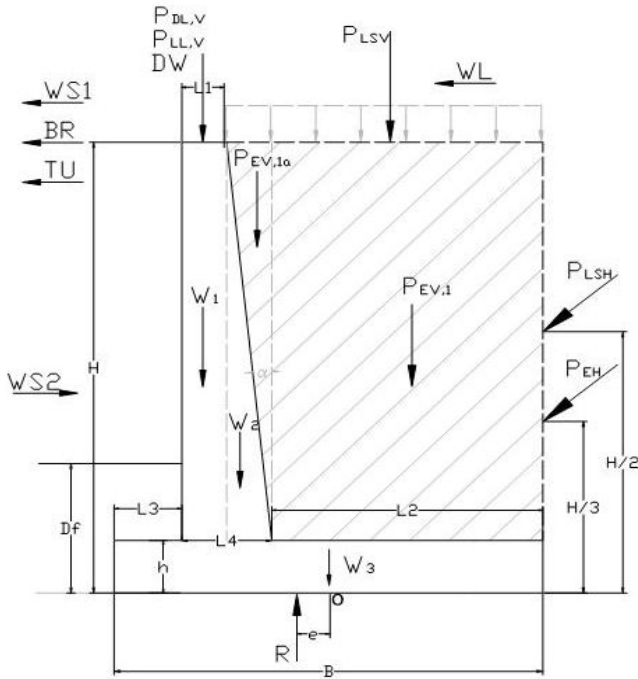
Yellow-highlighted is the critical case.

Rear Abutment

LRFD

BEARING RESISTANCE CALCULATION FOR CANTILEVER WALL

Ref: {AASHTO; LRFD BRIDGE DESIGN SPECIFICATIONS}



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	0	psf	Cohesion	Foundation soil
ϕ	=	34	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	34	deg.	Friction ang.	Foundation soil

Unfactored Loads and Parameters

TU	=	1,144.8	lb/ft	Temperature Load
WS1	=	248.3	lb/ft	Wind Loads on Superstructure
WS2	=	0.0	lb/ft	Wind Load on Substructure
WL	=	27.6	lb/ft	Wind on Live Load
BR	=	0.0	lb/ft	Braking Force
P_{LSV}/P_{LSH}	=	250	psf	Traffic Load
$P_{DL,V}$	=	34,790	lb/ft	Bridge Dead Load + Stem Wght (W1)
$P_{LL,V}$	=	5,590	lb/ft	Bridge Live Load
DW	=	2,560.0	lb/ft	Dead Load of Wearing Surf./Utilities
B	=	20.3	ft	Width of footing
L1	=	4.3	ft	Width of stem
L2	=	11.0	ft	Length of heel
L3	=	5.0	ft	Length of toe
L4	=	4.3	ft	Base of stem width
h	=	3.0	ft	Thickness of footing
D_f	=	4.0	ft	Embedment depth
Dw	=	4.0	ft	Groundwater depth
H-D	=	29.1	ft	Height Above Ground Surface
H	=	33.1	ft	Total height of wall
K_a	=	0.30		
B'	=	17.8	ft	
γ'	=	57.6	pcf	
α	=	0.0	degree	Back face batter angle
δ	=	20.0	degree	Embankment Interface Friction Ang.

Max Factored Uniform Bearing Pressure

From Table 2 on following page,

$$q_{uni} = \underline{\underline{7,784 \text{ psf}}}$$

Factored Unit Bearing Resistance, q_r (Undrained)

$$q_{ULT} = cN_c + \gamma D_f N_q C_{wq} + 0.5 \gamma B' N_\gamma C_{w\gamma}$$

$$q_{ULT} = \text{N/A psf}$$

$$q_R = \phi q_{ULT}$$

$$^1 \phi = 0.55$$

$$q_r = \text{N/A psf}$$

Undrained Bearing resistance is N/A

Factored Unit Bearing Resistance, q_r (Drained)

$$q_{ULT} = cN_c + \gamma D_f N_q C_{wq} + 0.5 \gamma B' N_\gamma C_{w\gamma}$$

$$q_{ULT} = 36,044 \text{ psf}$$

$$q_R = \phi q_{ULT}$$

$$^1 \phi = 0.55$$

$$q_r = 19,824 \text{ psf}$$

Drained Bearing resistance is OK

Bearing Capacity Factors for Equations

	Undrained	Drained (AASHTO Table 10.6.3.1.2a-1)
N_c	42.20	42.20
N_q	29.40	29.40
N_γ	41.10	41.10

AASHTO Table 10.6.3.1.2a-2

$$C_{wq} = 1.0 \quad C_{w\gamma} = 0.5$$

¹ Resistance factors from AASHTO LRFD, Table 11.5.7-1

Client ms consultants/ ODOT D-6
 Project FRA-70-14.48
 Item Stability Calculations
 Bridge 1716C Rear Abutment

JOB NUMBER 1021-1005.01
 SHEET NO. 2 of 2
 COMP. BY EWT Date 11/14/2013
 CHECKED BY TAH Date 11/14/2013

LRFD

STABILITY CALCULATIONS FOR CANTILEVER WALL
 Ref: (AASHTO; LRFD BRIDGE DESIGN SPECIFICATIONS)

Summary of Unfactored Loads

Item	Force (lb/ft)	Arm (ft)	M (ft-lb/ft)
P _{LSV}	2,750	4.63	12,719
P _{LSH,b}	2,312	-16.55	-38,262
P _{LSH,v}	841	10.125	8,520
P _{EV,I}	39,732	4.63	183,761
P _{EV,la}	0	-0.88	0
P _{LL,V}	5,590	-3.00	-16,770
P _{DL,V}	34,790	-3.00	-104,370
P _{EH,b}	18,366	-11.03	-202,635
P _{EH,v}	6,685	10.13	67,681
W1	0	-3.00	0
W2	0	-0.88	0
W3	9,113	0.00	0
WS1	248	-35.20	-8,740
WS2	0	13.10	0
WL	28	-39.10	-1,079
TU	1145	-26.20	-29,994
DW	2,560	-3.00	-7,680
BR	0	-33.10	0

Load Factors and Load Combinations

Group	EV	EH	LS	DC	TU	WS	DW	BR	WL
Strength I-a	1.00	1.50	1.75	0.90	0.50	0.00	0.65	1.75	0.00
Strength I-b	1.35	1.50	1.75	1.25	0.50	0.00	1.50	1.75	0.00
Service I	1.00	1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00
Strength IV	1.35	1.50	0.00	1.50	0.50	0.00	1.50	0.00	0.00

Vertical Factored Loads (lbs/ft width)			Vertical Factored Moments, M _v (ft-lbs/ft)		
Group	P _{LSH,v}	P _{EH,v}	Group	P _{LSH,v}	P _{EH,v}
Strength I-a	1,473	10,027	Strength I-a	14,910	101,522
Strength I-b	1,473	10,027	Strength I-b	14,910	101,522
Service I	841	6,685	Service I	8,520	67,681
Strength IV	0	10,027	Strength IV	0	101,522

Vertical Factored Loads (lbs per ft width)

Group	P _{EV,I}	P _{EV,la}	P _{LL,V} , P _{LSV}	W1 (DC1)	W2 (DC2)	W3 (DC3)	P _{DL,V}	DW	Total
Strength I-a	39,732	0	14,595	0	0	8,201	31,311	1,664	107,003
Strength I-b	53,638	0	14,595	0	0	11,391	43,488	3,840	138,451
Service I	39,732	0	8,340	0	0	9,113	34,790	2,560	102,061
Strength IV	53,638	0	0	0	0	13,669	52,185	3,840	133,359

Horizontal Factored Loads (lbs per ft width)

Group	P _{EH,b}	P _{LSH,b}	TU	WS1	WS2	BR	WL	Total
Strength I-a	27,549	4,046	572	0	0	0	0	32,167
Strength I-b	27,549	4,046	572	0	0	0	0	32,167
Service I	18,366	2,312	1,145	74	0	0	28	21,925
Strength IV	27,549	0	572	0	0	0	0	28,121

Vertical Factored Moments, M_v (ft-lbs/ft)

Group	P _{EV,I}	P _{EV,la}	P _{LL,V} , P _{LSV}	W1 (DC1)	W2 (DC2)	W3 (DC3)	P _{DL,V}	DW	Total
Strength I-a	183,761	0	-7,090	0	0	0	-93,933	-4,992	194,178
Strength I-b	248,077	0	-7,090	0	0	0	-130,463	-11,520	215,436
Service I	183,761	0	-4,051	0	0	0	-104,370	-7,680	143,860
Strength IV	248,077	0	0	0	0	0	-156,555	-11,520	181,524

Horizontal Factored Moments, M_h (ft-lbs/ft)

Group	P _{EH,b}	P _{LSH,b}	TU	WS1	WS2	BR	WL	Total
Strength I-a	-303,953	-66,958	-14,997	0	0	0	0	-385,908
Strength I-b	-303,953	-66,958	-14,997	0	0	0	0	-385,908
Service I	-202,635	-38,262	-29,994	-2,622	0	0	-7,680	-281,193
Strength IV	-303,953	0	-14,997	0	0	0	0	-318,950

Check Overturning (Eccentricity)³

Table 1

Group	V _{TOT} (lb/ft)	M _v (ft-lbs/ft)	M _h (ft-lbs/ft)	e (ft)	B/3 (ft)
Strength I-a	100,718	157,010	-385,908	2.27	6.75
*Strength IV	133,359	181,524	-318,950	1.03	6.75

* Strength IV does not apply (Superstructure DL/LL<.7.0)

$e_{all\ cases}$ is less than Cal. $e_{max} = 2.27$ B/3= 6.75 $e_{max} = B/3$ Overturning is **OK**
 Check if Cal. $e_{max} \leq B/3 =$ **YES Inside middle 2/3**

Check Sliding⁴

Factored resistance against failure by sliding $R_R = \phi R_n = \phi : R_t + \phi_{ep} R_{ep}$
 Because of the potential for loss of soil in front of wall, assume: $\phi_{ep} R_{ep} = 0$

Check sliding using Strength I-a loading

Cohesionless Soil:

$R_t = V \tan \phi_r$
 $\tan \phi_r = 0.67$ $R_t = 61,337$ lbs/ft

Cohesive Soil: $R_t =$ area under q_s diagram (pg3)

$R_t =$ N/A lbs/ft

$\phi_r = 1$ $R_R = 61,337$ lbs/ft

For $R_R = 61,337$ lbs/ft Active Driving Horizontal Force, $H_{TOTAL} = 32,167$ lbs/ft Sliding is **OK**

Check Bearing Pressure

Table 2

Group	V _{TOT} (lb/ft)	M _v (ft-lbs/ft)	M _h (ft-lbs/ft)	e (ft)	B'	q _{uniform} (psf)	e (ft)
Strength I-b	138,451	215,436	-385,908	1.23	17.79	7,784	1.23
Service I	102,061	143,860	-281,193	1.35	17.56	5,813	1.35
*Strength IV	133,359	181,524	-318,950	1.03	18.19	7,332	1.03

Max q_{uniform} (psf) = 7,784

¹ Resistance factors from AASHTO LRFD, Table 11.5.7-1

² Load factors and load combinations from AASHTO LRFD, Table 3.4.1-1, Table 3.4.1-2 & Section C3.4.1 pg 3-11 & 3-12

³ Vertical Traffic Loads (P_{LSV} & P_{LSH,v}) & horizontal wind loads (WS) are not included in overturning analysis

⁴ Live Vertical Loads (P_{LL,V}, P_{LSV}, & P_{LSH,v}) & horizontal wind loads (WS) are not included in sliding analysis

* Strength IV does not apply (Superstructure DL/LL<.7.0)

CLIENT ODOT D-6 / ms consultants
PROJECT E- Interchange
SUBJECT Bridge 1716
Settlement - Rear Abutment

PROJECT NO. 1021-1005.01
SHEET NO. 1 OF 1
COMP. BY gwt DATE 6-12-13
CHECKED BY BTA DATE 6-12-13

Loads on rear abutment:

$$LL = 5.59 \text{ k/ft}$$

$$DL = 34.79 \text{ k/ft (including stem wt)}$$

$$DW = 2.56 \text{ k/ft}$$

$$\begin{aligned} \text{Total stem wt} &= 19189 \\ &\text{lb/ft} \\ &= 19.2 \text{ k/ft} \end{aligned}$$

For settlement analysis, include concrete weight between top of wall and existing ground surface only

- existing ground surface @ 767.0'
- top of wall @ 783.6'
- stem thickness = 4.25'

$$\gamma_{\text{conc}} = 150 \text{ pcf}$$

$$\begin{aligned} \text{Total vert loads} &= 5.59 + 34.79 + 2.56 \\ &\quad - 19.2 + [(783.6 - 767) \times 4.25 \times 0.15] \\ &= 34.3 \text{ k/ft} \end{aligned}$$

$$B' = 17.56' \text{ (Service Case)}$$

$$\begin{aligned} \text{Input pressure} &= \frac{34.3 \times 1000}{17.56} \\ &= \underline{\underline{1953}} \text{ psf} \end{aligned}$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-71-1716- Rear Abutment
 Boring Overburden

JOB NUMBER 1021-1005.01
 SHEET NO. 1 OF 5
 COMP. BY EWT DATE 06/08/13
 CHECKED BY BTA DATE 06/08/13

Based on Boring B-001-M-59	Existing Fndn Load"	psf	Bearing Depth	750.5 ft	Water Elev.	750.5 ft
	Layer	Zone (ft)	Mid-Pt Elev (ft)	γ_m (pcf)	(Assumed) Hc (ft)	
	A	784.1 to 750.5	-	120	-	
	B	750.5 to 749.1	749.8	120	1.4	
	C	749.1 to 746.6	747.85	120	2.5	
	D	746.6 to 741.6	744.1	120	5	
	E	741.6 to 731.6	736.6	120	10	
	F	731.6 to 718.1	724.85	120	13.5	

Calculate Existing Load

Point 1

$$\begin{aligned} \sigma'_{o1} &= 4032 \text{ psf} + (750.5' - 749.8')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4032 \text{ psf} + (0.7')(57.6 \text{ pcf}) \\ &= 4072 \text{ psf} \end{aligned}$$

Point 2

$$\begin{aligned} \sigma'_{o2} &= 4072 \text{ psf} + (749.8' - 747.85')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4072 \text{ psf} + (1.9')(57.6 \text{ pcf}) \\ &= 4184 \text{ psf} \end{aligned}$$

Point 3

$$\begin{aligned} \sigma'_{o3} &= 4184 \text{ psf} + (747.85' - 744.1')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4184 \text{ psf} + (3.8')(57.6 \text{ pcf}) \\ &= 4400 \text{ psf} \end{aligned}$$

Point 4

$$\begin{aligned} \sigma'_{o4} &= 4400 \text{ psf} + (744.1' - 736.6')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4400 \text{ psf} + (7.5')(57.6 \text{ pcf}) \\ &= 4832 \text{ psf} \end{aligned}$$

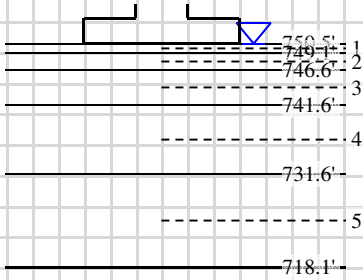
Point 5

$$\begin{aligned} &= 4832 \text{ psf} + (736.6' - 724.85')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 4832 \text{ psf} + (11.8')(57.6 \text{ pcf}) \\ &= 5509 \text{ psf} \end{aligned}$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-71-1716- Rear Abutment
 Existing Overburden

JOB NUMBER 1021-1005.01
 SHEET NO. 2 OF 5
 COMP. BY EWT DATE 06/08/13
 CHECKED BY BTA DATE 06/08/13

Based on Boring B-001-M-59	Existing Fndn Load"		Bearing Depth	750.5 ft	Water Elev.	750.5 ft
	Layer	Zone (ft)	Mid-Pt Elev (ft)	γ_m (pcf)	(Assumed) Hc (ft)	
-----767.0-----	A	767 to 750.5	-	120	-	
	B	750.5 to 749.1	749.8	120	1.4	
	C	749.1 to 746.6	747.85	120	2.5	
	D	746.6 to 741.6	744.1	120	5	
	E	741.6 to 731.6	736.6	120	10	
	F	731.6 to 718.1	724.85	120	13.5	
-----731.6-----						
-----718.1-----						

Calculate Existing Load

Point 1

$$\begin{aligned} \sigma'_{o1} &= 1980 \text{ psf} + (750.5' - 749.8')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 1980 \text{ psf} + (0.7')(57.6 \text{ pcf}) \\ &= 2020 \text{ psf} \end{aligned}$$

Point 2

$$\begin{aligned} \sigma'_{o2} &= 2020 \text{ psf} + (749.8' - 747.85')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2020 \text{ psf} + (1.9')(57.6 \text{ pcf}) \\ &= 2132 \text{ psf} \end{aligned}$$

Point 3

$$\begin{aligned} \sigma'_{o3} &= 2132 \text{ psf} + (747.85' - 744.1')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2132 \text{ psf} + (3.8')(57.6 \text{ pcf}) \\ &= 2348 \text{ psf} \end{aligned}$$

Point 4

$$\begin{aligned} \sigma'_{o4} &= 2348 \text{ psf} + (744.1' - 736.6')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2348 \text{ psf} + (7.5')(57.6 \text{ pcf}) \\ &= 2780 \text{ psf} \end{aligned}$$

Point 5

$$\begin{aligned} &= 2780 \text{ psf} + (736.6' - 724.85')(120 \text{ pcf} - 62.4 \text{ pcf}) \\ &= 2780 \text{ psf} + (11.8')(57.6 \text{ pcf}) \\ &= 3457 \text{ psf} \end{aligned}$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-71-1716- Rear Abutment
 Proposed Overburden

JOB NUMBER 1021-1005.01
 SHEET NO. 3 OF 5
 COMP. BY EWT DATE 06/08/13
 CHECKED BY BTA DATE 06/08/13

Based on Boring B-001-M-59	Foundation Load ^a	1953 psf	Bearing Depth	750.5 ft	Water Elev.	750.5 ft	
	Layer	Zone (ft)		Mid-Pt Elev (ft)	γ_m (pcf)	Hc (ft)	i*
767.0'	A	767.0	to 750.5	-	120	-	
	B	750.5	to 749.1	749.8	120	1.4	1.00
	C	749.1	to 746.6	747.85	120	2.5	1.00
	D	746.6	to 741.6	744.1	120	5	0.88
	E	741.6	to 731.6	736.6	120	10	0.63
	F	731.6	to 718.1	724.85	120	13.5	0.40

Calculate Proposed Load

^a - *fdn load at center of footing*

* - *see settlement calcs for determination of i values*

Point 1

$$\begin{aligned} \sigma'_{o1} &= 1980 \text{ psf} + (750.5' - 749.8')(120 \text{ pcf} - 62.4 \text{ pcf}) + (1953 \text{ psf})(1) \\ &= 1980 \text{ psf} + (0.7')(57.6 \text{ pcf}) + (1953 \text{ psf}) \\ &= 3973 \text{ psf} \end{aligned}$$

Point 2

$$\begin{aligned} \sigma'_{o2} &= 2020 \text{ psf} + (749.8' - 747.85')(120 \text{ pcf} - 62.4 \text{ pcf}) + (1953 \text{ psf})(1) \\ &= 2020 \text{ psf} + (1.9')(57.6 \text{ pcf}) + (1953 \text{ psf}) \\ &= 4085 \text{ psf} \end{aligned}$$

Point 3

$$\begin{aligned} \sigma'_{o3} &= 2132 \text{ psf} + (747.85' - 744.1')(120 \text{ pcf} - 62.4 \text{ pcf}) + (1953 \text{ psf})(0.88) \\ &= 2132 \text{ psf} + (3.8')(57.6 \text{ pcf}) + (1719 \text{ psf}) \\ &= 4066.64 \text{ psf} \end{aligned}$$

Point 4

$$\begin{aligned} \sigma'_{o4} &= 2348 \text{ psf} + (744.1' - 736.6')(120 \text{ pcf} - 62.4 \text{ pcf}) + (1953 \text{ psf})(0.63) \\ &= 2348 \text{ psf} + (7.5')(57.6 \text{ pcf}) + (1230 \text{ psf}) \\ &= 4010.39 \text{ psf} \end{aligned}$$

Point 5

$$\begin{aligned} &= 2780 \text{ psf} + (736.6' - 724.85')(120 \text{ pcf} - 62.4 \text{ pcf}) + (1953 \text{ psf})(0.4) \\ &= 2780 \text{ psf} + (11.8')(57.6 \text{ pcf}) + (781 \text{ psf}) \\ &= 4238.2 \text{ psf} \end{aligned}$$



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-71-1716- Rear Abutment
 Settlement

JOB NUMBER 1021-1005.01
 SHEET NO. 4 OF 5
 COMP. BY EWT DATE 06/08/13
 CHECKED BY BTA DATE 06/08/13

Proposed Loading										
Assume loading =		1953 psf								
Assume width =		17.6 ft								
0										
Point 1	(749.8')	Depth=	(750.5' - 749.8')	= 0.7'	→	$\frac{0.7'}{17.6'}$	=	0.04B	→ i = 1.00	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{4.1}\right) = 0.76$	$N_{60} = 49$	$N_{160} = 49 * 0.76 = 37.4$					
		$C' = 137$	Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand and gravel"					
$\Delta H = 1.4' \frac{1}{137} \log\left(\frac{4.0 \text{ ksf}}{2.0 \text{ ksf}}\right) = 0.003 \text{ ft} = 0.04 \text{ in}$		$\Sigma H = 0.04 \text{ in}$								
0										
Point 2	(747.9')	Depth=	(750.5' - 747.9')	= 2.6'	→	$\frac{2.6'}{17.6'}$	=	0.15B	→ i = 1.00	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{4.2}\right) = 0.75$	$N_{60} = 63$	$N_{160} = 63 * 0.75 = 47.5$					
		$C' = 155$	Fig 10.6.2.4.2-1 (10-52)		"Well graded silty sand and gravel"					
$\Delta H = 2.5' \frac{1}{155} \log\left(\frac{4.1 \text{ ksf}}{2.1 \text{ ksf}}\right) = 0.005 \text{ ft} = 0.05 \text{ in}$		$\Sigma H = 0.09 \text{ in}$								
0										
Point 3	(744.1')	Depth=	(750.5' - 744.1')	= 6.4'	→	$\frac{6.4'}{17.6'}$	=	0.36B	→ i = 0.88	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{4.4}\right) = 0.74$	$N_{60} = 50$	$N_{160} = 50 * 0.74 = 36.9$					
		$C' = 105$	Fig 10.6.2.4.2-1 (10-52)		"Well graded fine to coarse sand"					
$\Delta H = 5.0' \frac{1}{105} \log\left(\frac{4.1 \text{ ksf}}{2.3 \text{ ksf}}\right) = 0.011 \text{ ft} = 0.14 \text{ in}$		$\Sigma H = 0.23 \text{ in}$								
0										
Point 4	(736.6')	Depth=	(750.5' - 736.6')	= 13.9'	→	$\frac{13.9'}{17.6'}$	=	0.79B	→ i = 0.63	Fig 10.6.2.4.1-1a (10-49)
Granular	→	$\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$	$C_N = 0.77 \log\left(\frac{40}{4.8}\right) = 0.71$	$N_{60} = 63$	$N_{160} = 63 * 0.71 = 44.5$					
		$C' = 125$	Fig 10.6.2.4.2-1 (10-52)		"Well graded fine to coarse sand"					
$\Delta H = 10.0' \frac{1}{125} \log\left(\frac{4.0 \text{ ksf}}{2.8 \text{ ksf}}\right) = 0.013 \text{ ft} = 0.15 \text{ in}$		$\Sigma H = 0.38 \text{ in}$								



CLIENT ODOT District 6
 PROJECT FRA-70-14.48
 SUBJECT FRA-71-1716- Rear Abutment
 Settlement

JOB NUMBER 1021-1005.01
 SHEET NO. 5 OF 5
 COMP. BY EWT DATE 06/08/13
 CHECKED BY BTA DATE 06/08/13

Proposed Loading

Assume loading = 1953 psf

Assume width = 17.6 ft

Point 5 (724.9') Depth = (750.5' - 724.9') = 25.7' → $\frac{25.7'}{17.6'} = 1.46B$ → $i = 0.40$ *Fig 10.6.2.4.1-1a (10-49)*

Granular → $\Delta H = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$ $C_N = 0.77 \log\left(\frac{40}{5.5}\right) = 0.66$ $N_{60} = 63$ $N_{160} = 63 * 0.66 = 41.7$

$C' = 120$ *Fig 10.6.2.4.2-1 (10-52)* "Well graded fine to coarse sand"

$$\Delta H = 13.5' \frac{1}{120} \log\left(\frac{4.2 \text{ ksf}}{3.5 \text{ ksf}}\right) = 0.010 \text{ ft} = 0.12 \text{ in}$$

$\Sigma H = 0.50 \text{ in}$

Total Settlement = 0.4" (new fill) + 0.5" (loads) = 0.9 inches



SUBJECT

Client ODOT District 6

JOB NUMBER

1021-1005.01

Project East Interchange

SHEET NO.

1 OF 1

Item Bridge 1716

COMP. BY

EWT DATE 06/08/13

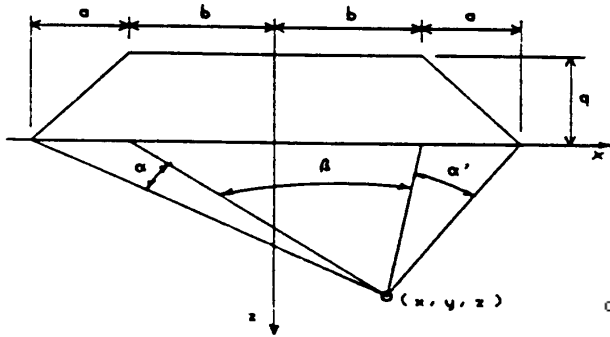
Rear Abutment - New Fill B-001-M-59

CHECKED BY

BTA DATE 06/08/13

SETTLEMENT ANALYSIS - EMBANKMENT

Embankment Informaiton:



Groundwater Table: D= 16.5 ft (Assumed at Elev 750.5)

Embankment Height: H= 16.6 ft

Fill Unit Weight: $\gamma_{emb} = 120$ pcf $q = 1,992$ psf

Width of Slope: a = 0.1

Top half-width of Emb: b = 16 **Estimated**

Distance from CL: x = 16

Output Range: z = 0 to 49 ft

*See Data output Attached

$$\sigma_{v'}(z) := \left(\frac{q}{\pi a} \right) (a \cdot (\alpha(z) + \beta(z) + \alpha'(z)) + b \cdot (\alpha(z) + \alpha'(z)) + x \cdot (\alpha(z) - \alpha'(z)))$$

$$\beta(z) := \text{atan} \left[\frac{(b-x)}{z} \right] + \text{atan} \left[\frac{(b+x)}{z} \right]$$

$$\alpha'(z) := \text{atan} \left[\frac{(a+b-x)}{z} \right] - \text{atan} \left[\frac{(b-x)}{z} \right]$$

$$\alpha(z) := \text{atan} \left[\frac{(a+b+x)}{z} \right] - \text{atan} \left[\frac{(b+x)}{z} \right]$$

Reference: US Army Corps of Engineers EM 1110-1-1904 "Settlement Analysis", Table C-1

Cohesionless

Soil Properites:

Settlement is calculated at mid-point of layer

No.	Bot. of Layer	Soil Type	γ_{soil} (pcf)	σ'_c (psf)	σ'_o (psf)	$\Delta\sigma_z$ (psf)	σ'_f (psf)	Cohesive Soils			
								C'	C_r	C_c	e_o
1	16.5 ft	Incompressible	120	5,000	990	998	1,988	0.0	0.00	0.00	0.000
2	17.9 ft	A--3a	120	5,000	2,020	952	2,972	137.0	0.00	0.00	0.000
3	20.4 ft	A-1-b	120	5,000	2,133	937	3,070	155.0	0.00	0.00	0.000
4	25.4 ft	A-3a	120	5,000	2,349	909	3,257	105.0	0.00	0.00	0.000
5	35.4 ft	A-3a	120	5,000	2,781	834	3,614	125.0	0.00	0.00	0.000
6	48.9 ft	A-3a	120	5,000	3,457	719	4,177	120.0	0.00	0.00	0.000
7											
8											
9											
10											

Reference: Geotechnical Engineering Principles and Practices; Coduto, 1999

Overconsolidated Soils - Case I ($\sigma'_o < \sigma'_c$) Eqn:11.24

$$(\delta_c)_{ult} = \sum \frac{C_r}{1+e_o} H \log \left(\frac{\sigma'_f}{\sigma'_o} \right)$$

Overconsolidated Soils - Case II ($\sigma'_o < \sigma'_c < \sigma'_f$) Eqn:11.25

$$(\delta_c)_{ult} = \sum \left[\frac{C_r}{1+e_o} H \log \left(\frac{\sigma'_c}{\sigma'_o} \right) + \frac{C_c}{1+e_o} H \log \left(\frac{\sigma'_f}{\sigma'_c} \right) \right]$$

Normally Consolidated Soils ($\sigma'_o = \sigma'_c$) Eqn: 11.23

$$(\delta_c)_{ult} = \sum \frac{C_c}{1+e_o} H \log \left(\frac{\sigma'_f}{\sigma'_o} \right)$$

Reference: FHWA NHI-00-045

Cohesionless Soils ($\sigma'_o = \sigma'_c$)

$$(\delta_c)_{ult} = \sum \frac{1}{C'} H \log \left(\frac{\sigma'_f}{\sigma'_o} \right)$$

No. Settlement:

Total Settlement

1	0.000	ft
2	0.002	ft
3	0.003	ft
4	0.007	ft
5	0.009	ft
6	0.009	ft
7		
8		
9		
10		

0.029 ft

0.4 in



SUBJECT

Client ODOT District 6

JOB NUMBER

Project East Interchange

SHEET NO. _____ OF _____

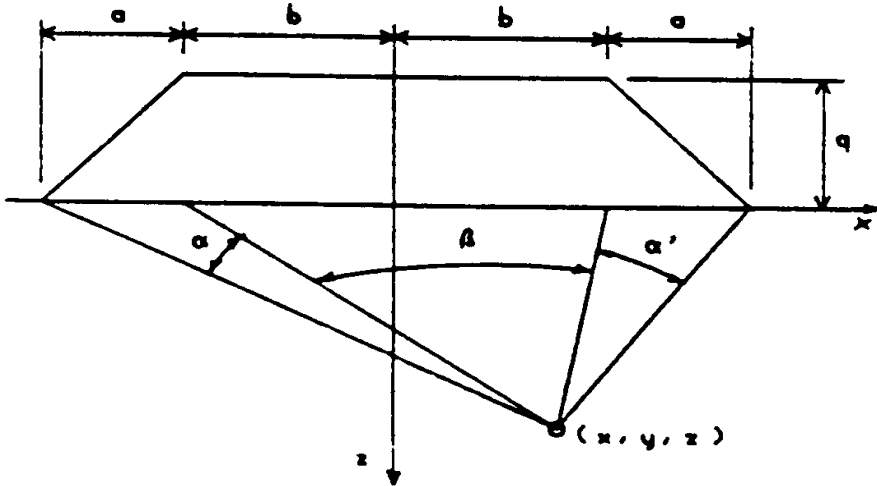
Item Bridge 1716

COMP. BY _____ DATE _____

Rear Abutment - New Fill B-001-M-59

CHECKED BY _____ DATE _____

INCREASE IN VERTICAL STRESS DUE TO EMBANKMENT LOADING

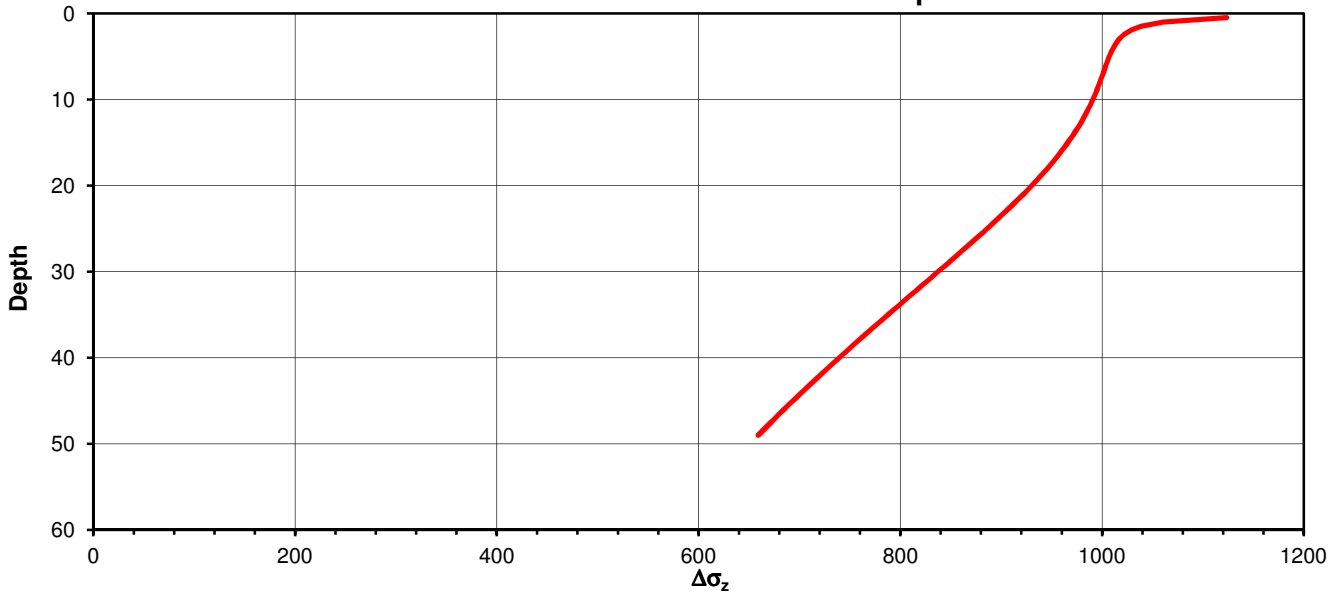


- q = 1992 load
- a = 0.1 width of slope
- b = 16 top half-width of embankment
- x = 16 distance from CL
- z = 0 to 49 depth range

$$\sigma_v(z) := \left(\frac{q}{\pi a}\right) (a(\alpha(z) + \beta(z) + \alpha'(z)) + b(\alpha(z) + \alpha'(z)) + x(\alpha(z) - \alpha'(z)))$$

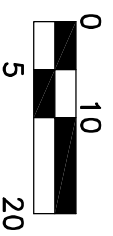
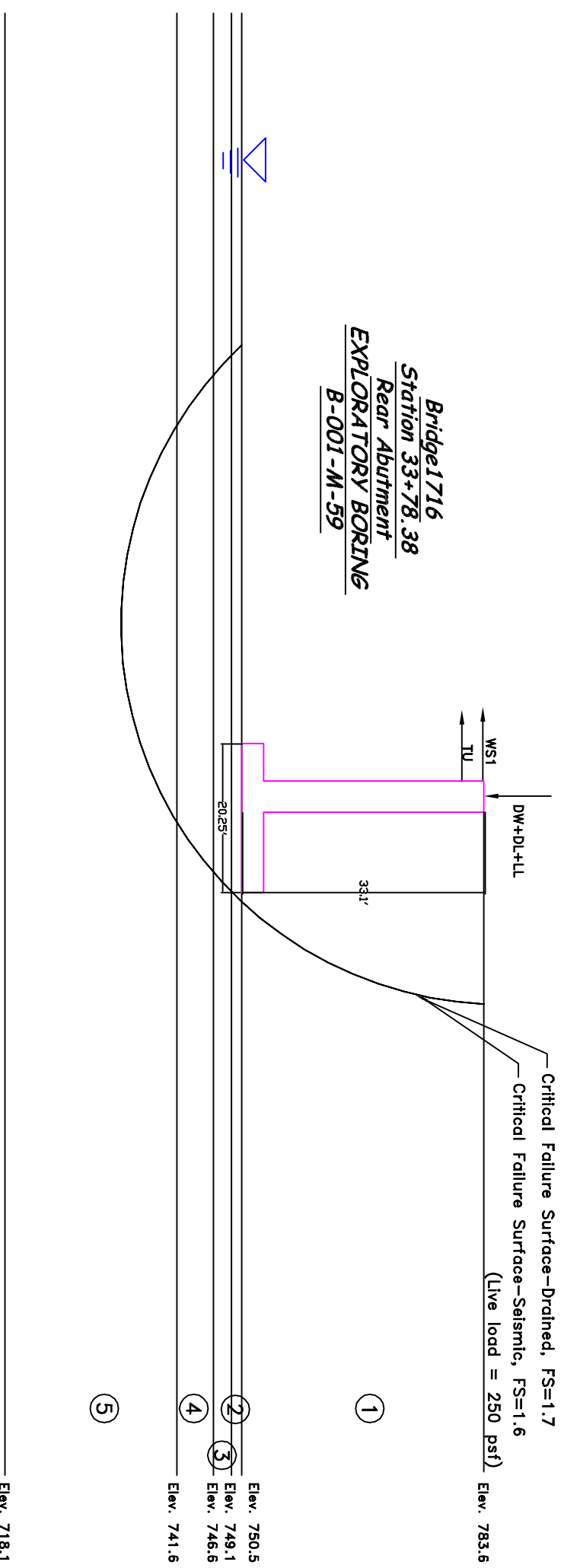
$$\beta(z) := \operatorname{atan}\left[\frac{(b-x)}{z}\right] + \operatorname{atan}\left[\frac{(b+x)}{z}\right]; \quad \alpha'(z) := \operatorname{atan}\left[\frac{(a+b-x)}{z}\right] - \operatorname{atan}\left[\frac{(b-x)}{z}\right] \quad \alpha(z) := \operatorname{atan}\left[\frac{(a+b+x)}{z}\right] - \operatorname{atan}\left[\frac{(b+x)}{z}\right]$$

Vertical Stress Increase Vs. Depth



Reference: US Army Corps of Engineers EM 1110-1-1904 "Settlement Analysis", Table C-1

Material	Soil Type	c (psf)	ϕ (deg)	c' (psf)	ϕ' (deg)	γ (pcf)
Material 1	Embankment Fill	0	30	0	30	120
Material 2	Dense A-3a	0	34	0	34	120
Material 3	Very Dense A-1-b	0	36	0	36	120
Material 2	Dense to V. Dense A-3a	0	34	0	34	120
Material 3	Very Dense A-3a	0	36	0	36	120



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GLOBAL STABILITY EVALUATION	
FRA-70-14.48	
FRA-70-1716 BRIDGE	
STATION 33+78.38	
Global Stability – Rear Abutment	
PROJECT NO. 1021-1005.01	CALC: EWT
	DATE 06/10/13