TECHNICAL MEMORANDUM

DATE November 6, 2024 **Project No.** 31404817.004

TO Laura Krusinski, PE
Maine Department of Transportation

CC Tim Aguilar, PE, MaineDOT; Steven Hodgdon, PE, VHB

FROM Melissa E. Landon, PhD, PE **EMAIL melissa.landon@wsp.com**

RE: GEOTECHNICAL EVALUATION OF EXISTING BRIDGE PILES – REV02
HOGAN ROAD BRIDGE REPLACEMENT, BANGOR, MAINE
WIN 018595.10

Dear Laura,

WSP USA, Inc. (WSP) is pleased to submit this revised Technical Memorandum (TM) summarizing the results of our updated evaluation of the existing foundations for Hogan Road Bridge #5823 that is proposed for renovation for the Hogan Road Diverging Diamond Interchange in Bangor, Maine. This TM supersedes our August 2024 TM[1] and provides updated information based on MaineDOT and VHB requested changes. These changes include the following:

- Use of an axial resistance factor of ϕ_c = 0.7 in the analyses for combined axial and flexural resistance in the upper portion of the piles per AASHTO[2] LRFD Section 6.5.4.2;
- Use of an axial resistance factor of ϕ_c = 0.5 for H-piles in pure axial compression in the analyses per AASHTO[2] LRFD Section 6.5.4.2 assuming severe driving conditions and the use of pile tips;
- Use of a dynamic resistance factor of ϕ_{dyn} = 0.5 (given in AASHTO[2] LRFD Table 10.5.5.2.3-1 for situations with field confirmation of hammer performance and a wave equation analysis) to back calculate the geotechnical resistance for the pile bearing on rock at the request of MaineDOT assuming pile QC consisting of field monitoring of piles during driving and a WEAP at the time of driving;
- Use of pile section loss of 0.05-inches for 50 years of additional service life;
- Relocation of the Abutment 1 and Abutment 2 resultant load application location 0.71 feet towards the front of each abutment;

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[1] WSP USA, Inc. (WSP), Technical Memorandum: Geotechnical Evaluation of Existing Bridge Piles – Rev01 Draft, Hogan Road Bridge Replacement, Bangor, Maine, WIN 018595.10, August 13, 2024, 180 pp. Submitted to Laura Krusinski, PE, Maine Department of Transportation.

[2] American Association of State Highway and Transportation Officials (AASHTO). LRFD Bridge Design Specifications. 9th Edition, dated 2020.

- Results based on the application of service and strength loading on Abutment 1 and Abutment 2 in concert with the proposed rehabilitation scenario; and
- Results based on the application of service and strength loading on the existing and proposed rehabilitation configuration of Pier 5.

This TM includes updated geotechnical and structural resistances for the foundations based on WSP's evaluation of the soil stratigraphy, geotechnical material properties, and structural modeling at the abutments and piers, and recommendations based on our findings. WSP's pile evaluation includes an evaluation of future corrosion section loss for the existing piles and a discussion of the estimated historical downdrag loading on the piles from embankment construction at the existing abutments. The following sections describe our analyses and findings in more detail.

Bridge Foundation Loading

VHB provided WSP with bridge foundation for the abutments and piers. For the two abutment substructures that are pile supported, VHB provided loads at the bottom of the pile caps at the pile group centroid locations, relocated 0.71 feet towards the front of each abutment. For the pier substructure that is pile supported, VHB provided unfactored loads at each bridge girder bearing location for the existing and proposed pier geometries. For substructures on spread footings (four piers), VHB provided loads at the bottom of the columns. Foundation loading includes consideration for the following:

- Semi-integral bridge abutments with replacement of existing backfill with engineered backfill behind the abutments, an at-grade approach slab with a sleeper slab, a modular joint at the end of the approach slab, and an increase in grade of 3 feet.
- Lateral loads at the abutments are based on over excavation and replacement of exiting fill soils behind the full abutment height with lightweight foamed glass aggregate (FGA) as the engineered backfill under at rest (K_0) lateral earth pressure conditions rather than active (K_a) lateral earth pressure conditions. The assumed FGA unit weight is γ = 20 pcf and the assumed friction angle is ϕ= 45º.
- Minimum and maximum load factors from AASHTO[2] LRFD Table 3.4.1-1 enveloped together for dead loads live loads and earth pressures at the abutments. The envelopes were summed with factored loading to form Strength I, Strength III, Strength V, and Service I load combinations, and all strength combinations were enveloped together. Temperature changes factored into load combinations.
- Unfactored pier loads were applied at each bridge girder bearing at the top of the pier cap. The loads were then factored in Strength I, Strength II, Strength III, Strength IV, Strength V, and Service I load combinations using maximum and minimum load factors in FB-MultiPier.

Foundation loads are provided in Attachment 1. Note that contemporary abutment and pier numbering is used, where contemporary Abutment 1 is in the north (historical Abutment 2), pier numbering increases from north to south, and contemporary Abutment 2 is in the south (historical Abutment 1). The historical abutment and pier numbering increases from south to north. The southern-most pier, Pier 5, and abutments are pile supported, while Pier 1 through Pier 4 are supported by spread footings on rock.

Pier Spread Footing Foundations Analyses

WSP evaluated the strength and service limit state bearing resistance for the pier footings on bedrock at the existing bridge. Calculations and input material properties were based on dimension of the piers and foundation elevations from the record plans[3], bedrock elevation from WSP's stratigraphic interpretation based on borings completed, AASHTO[2], the MaineDOT Bridge Design Guide (BDG)[4], Wyllie (1999)[5], rock core quality and laboratory testing results to be provided in WSP's geotechnical design report at a later date, and WSP's empirical correlations to rock properties. A summary of evaluation of pier spread footing bearing resistance on rock is provided in Table 1. Refer to the full methodology of the analyses, material properties, and calculations in Attachment 2.

Table 1: Summary of Pier Properties and Bearing Resistance

Pier No[1]	Footing Dimensions	Bedrock Information[2]	Factored Bearing Resistance Strength[3]	Factored Bearing Resistance Service[4]
4	8 feet x 8 feet	10.8 feet bgs Sloping at 6.4°	75 ksf	70 ksf
3	10 feet x 10 feet	8.2 feet bgs Sloping at 8.1°	97 ksf	70 ksf
2	8 feet x 8 feet	8.7 feet bgs Sloping at 3.3°	81 ksf	70 ksf
1	8 feet x 8 feet	6.5 feet bgs Sloping at 7.1°	81 ksf	70 ksf

Notes: [1]Pier numbering is contemporary based on increasing numbering with increasing Station; thus, Pier 1 is in the north and Pier 4 is in the south. [2]Bedrock depth below ground surface (bgs) is from the center of the footing and the slope is calculated over the footing width parallel to the centerline of the bridge, where bedrock is shallower in the north. [3]Strength limit state resistance factor is 0.45 per AASHTO[2] LRFD Table 10.5.5.2.2-1. [4]Service limit state resistance factor is 1.0 per AASHTO LRFD Section 10.5.5.1.

Where foundations are cast-in-place concrete bearing on bedrock, the bedrock subgrade is prepared under dry conditions and can be visually inspected, and the slope of the bedrock is not steeper than 4H:1V (~14.6°) in any direction, a sliding coefficient (tan δ) of 0.70 is recommended per AASHTO[2] LRFD Table C3.11.5.3-1. We recommend, and used, a sliding coefficient of 0.70 per AASHTO for our evaluations. Per AASHTO[2] LRFD Figure C11.5.5-2, the following load factors were used to evaluate eccentricity of the pier foundations on spread footings: 0.9 for the dead load and 1.0 for the overburden soil above the footing. Bearing resistance of the pier footings

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[3] Maine Department of Transportation Bridge Program, DRAFT February 3,2023, Preliminary Design Report, Hogan Road/I95 Bridge #5823 over Interstate 95, Exit 187 Diverging Diamond Interchange, Bangor, Maine, Federal Project #1859510, WIN 018595.10.

[4] Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.

[5] Wyllie, D.C. (1999). Foundations on Rock, 2nd Ed. E&FN Spon, NY

bearing on bedrock was calculated using a resistance factor of ϕ_{stat} = 0.45, and sliding resistance was calculated using a sliding resistance factor ϕ_{stat} = 0.9 in accordance with the MaineDOT BDG[4].

WSP performed a check of bearing, sliding, and overturning resistance using AASHTO[2] for Pier 4. This pier was selected because it has the smaller foundation dimensions, larger loading as provided by VHB, smallest strength limit state bearing resistance, and average bedrock slope of the four piers. The Pier 4 spread footing eccentricity, overturning resistance, bearing resistance, and sliding resistance are satisfactory for the loading provided by VHB. We assume that the eccentricity, overturning resistance, bearing resistance, and sliding resistance are also satisfactory for Pier 1, Pier 2, and Pier 3 loading and geometry.

Pile Foundation Loading Analyses

WSP evaluated the available structural and geotechnical resistance for the HP 10x42 piles at Abutment 1, Abutment 2, and Pier 5.

- Based on the record plans[3], these structures are founded on piles driven to bedrock. WSP determined the range of pile lengths at each structure based on the estimated driven lengths provided on the record plans[3] and our interpretation of the changing bedrock elevation from the 100-series, 200-series, and historical borings. Abutment 1 pile lengths range from 13.4 feet to 24.1 feet, Abutment 2 pile lengths range from 42 feet to 45.9 feet, and Pier 5 pile lengths range from 25.9 feet to 29.5 feet. Based on the poor RQD in the upper portions of the sampled bedrock, the piles are assumed to be embedded 1-foot into bedrock.
- The abutment structures are supported by HP 10x42 piles and include a row of vertical piles at the rear of the abutment and a row of 3H:12V battered piles at the front of the abutment, angled forward of the abutment. Pile spacings are provided in the record plans[3]. The pier structure is supported by two rows of HP 10x42 piles at a 3H:12V batter angled away from the pier in the northwest and southeast directions.
- We evaluated the piles at Abutment 1, Abutment 2, and Pier 5 for the proposed rehabilitation condition using the estimated steel section loss of 0.05 inches per side of the piles from present through the anticipated 50-year additional design life of the three structures. This is to account for potential corrosion from roadway deicing salts in soils that may accumulate with time. WSP's corrosion evaluation is discussed in more detail below.
- We evaluated the existing condition at Pier 5 without any section loss. The intent of this evaluation is to provide pile resistance at the time of construction and at present based on measured section dimensions near the top of select piles at Pier 5. The existing bridge pile section dimension measurements are discussed in more detail below.
- WSP performed an evaluation of settlement-induced downdrag loading at both Abutment 1 and Abutment 2 at three locations: for the pile with the maximum axial load, and for the pile locations with the minimum and maximum settlement-induced downdrag loading. We have included the estimated downdrag loading at the maximum axially loaded pile location in our Abutment 1 and Abutment 2 pile evaluations that will be discussed subsequently. This downdrag loading is estimated to have accumulated between construction of the bridge to present. WSP's evaluation of settlement-induced downdrag loading is discussed below.

Corrosion Loss of Section for Piles

WSP previously submitted a TM[6] discussing corrosion potential of site soils based on results from a suite of corrosivity tests recommended by AASHTO[2] and estimates of loss of section determined using AASHTO[2,7], FHWA[8,9], and NCHRP[10] guidance for pile foundations. We have updated the previous Corrosion Potential TM to include a discussion of the methods and results of WSP's and MaineDOT's measurements of exposed pile sections within test pits at Abutment 1, Abutment 2, and Pier 5, which were performed to assess the current condition of the piles. A draft of this updated Corrosion Potential TM is included as Attachment 3. Corrosion section loss for the steel piles is discussed below.

WSP evaluated the corrosion potential of site soils based on the results of laboratory testing, which indicate that of the laboratory measured electrical resistivity, pH, sulfate, and chloride values, only electrical resistivity values are below the 2000 ohm-cm threshold recommended by AASTHO[2], below which corrosive conditions may develop. The lower resistivity values were mainly encountered in near surface and shallow depth samples (less than 5 feet) at locations adjacent to I-95 northbound and southbound that would most be impacted by road salt and road salt runoff into the drainage ditches beside the highway. The highest measured resistivity values are located within the shallow subsurface soils on the Hogan Road western embankment at Abutment 1. Based on the low electrical resistivity, AASHTO[7] indicates there is a possibility for severe macrocell corrosion in the existing soils at the water table and a low possibility of uniform corrosion, which can be identified by excavation of the soils around the existing foundations to visually inspect the abutments and pier piles.

In July 2024, WSP and MaineDOT collaboratively performed a visual inspection of, and made dimensional measurements on, several piles at the existing bridge to identify corrosion that may have occurred since the piles were installed. This entailed excavating test pits to examine one pile at Abutment 1 and two piles each at Abutment 2 and Pier 5. We observed that Pier 5 piles were installed in the native silt and clay soils and showed no visual evidence of rust. We observed that the piles at Abutment 1 and Abutment 2 were installed in the embankment fill layer and showed small areas of rust, which was removed with a steel brush prior to making measurements. The measured dimensions of the five selected piles were not less than the nominal dimensions for HP 10x42 piles. Based on the measured section dimensions and limited rust, we conclude that section loss has not occurred between installation and the present.

Laboratory test data of soils from nearby the existing piers indicate there is the potential for macrocell corrosion of steel at the site. MaineDOT typically recommends using the greater of either 1/16th of an inch section loss or calculated section loss based on FHWA and NCHRP methods for potential corrosion for all new bridge pile foundations. in an email[11] dated July 26, 2024, WSP was given the direction by MaineDOT's Laura Krusinski and

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[6] WSP USA, Inc. (WSP), Technical Memorandum: Geotechnical Soil Corrosivity Findings, Hogan Road Bridge Replacement, Bangor, Maine, WIN 018595.10, April 5, 2024, 13 pp. Submitted to Laura Krusinski, PE, Maine Department of Transportation

[7] AASHTO. Standard Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications. Specification R27-01 (2023). Technically Revised: 2001, Reviewed but Not Updated: 2023

[8] Federal Highway Administration (FHWA). GEC 012: Design and Construction of Driven Pile Foundations – Volume I. Publication No. FHWA-NHI-16-009. Dated July 2016

[9] FHWA. Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes. Publication No. FHWA-NHI-09-087. Dated November 2009

[10] National Cooperative Highway Research Program (NCHRP). Report 675: LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal-Reinforced Systems. 2011

[11] Krusinski, Laura "RE: Bangor Hogan Road - 1983 Bridge Updated Geotechnical Pile Resistance – Corrosion" Received by Melissa Landon, July 26, 2024

Tim Aguilar to consider section loss for the existing bridge for the next 50 years of its design life for the lesser of either 1/16th of an inch or WSP's calculated section loss over 50 years from the updated Corrosion Potential TM (Attachment 3). WSP estimated the loss of steel section based on the FHWA[9] non-linear method to account for potential section loss for the additional 50 year design life for the bridge to be 0.05 inches. We recommend a steel section loss of 0.05 inches from each face of the pile over 50 years be used to estimate the structural resistance of the existing HP 10x42 piles for the bridge renovation.

Loading on Piles

WSP used the Bridge Software Institute FB-MultiPier[12] software to evaluate the response of Abutment 1, Abutment 2, and Pier 5 HP 10x42 piles to the loading provided by VHB. One model was developed for each substructure element to incorporate the geometry and stiffness of each abutment or the pier, the piles in the positions identified in the record plans[3], and the range of pile lengths from WSP's interpreted bedrock surface and estimated installed pile lengths from the record plans[3]. FB-MultiPier uses finite element analysis methods to evaluate foundation systems consisting of a cap supported on piles founded in soil and rock at Abutment 1 and 2, and a pier consisting of a pier cap, columns and pile cap supported on piles founded in soil and rock for Pier 5. Linear or nonlinear structural finite element analysis is coupled with nonlinear static soil models to determine axial, lateral, and torsional response and loading on each pile.

Due to the limitations of the FB-MultiPier software, the piles could only be oriented with the strong axis parallel or perpendicular to the abutment or pier axes. Thus, piles that are oriented with the strong axis parallel to the angled face of the wingwall in the existing bridge were reoriented in the FB-MultiPier model with the weak axis parallel to the abutment face. The loading magnitude, direction, and location provided by VHB were applied to the pile group for each model. WSP selected soil and rock input parameters based on soil models defined by FB-MultiPier (p-y, t-z, q-z, and t-θ) correlations to N_{60} blow counts determined from the field investigations and correlations to soil and rock properties identified in the FB-MultiPier user manual. Refer to the full methodology of the analyses, material properties, and calculations in Attachment 4.

To provide a more representative distribution of forces to the piles, WSP increased the stiffness of the pile caps by increasing the elastic modulus of the concrete by factor of 10. This was intended to account for the inability to include steel reinforcement in the pile cap when using the FB-MultiPier abutment and pier models and addresses the simplification inherent with applying the loads at the centroid of the abutments.

Table 2 and Table 3 present the results of the range of factored axial loads applied to all of the piles in the abutments and Pier 5 based on the loading provided by VHB. Furthermore, these tables provide the maximum axial load in compression, moment, shear, and deflection estimated from the FB-MultiPier models along with the corresponding pile numbers. Figure 1, Figure 2, and Figure 3 show the pile locations and numbers for Abutment 1, Abutment 2, and Pier 5, respectively.

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[12] Florida Bridge Software Institute. FB-MultiPier, Version 6.0.0

Table 2: Summary of the Pile Loading at Abutment 1 and Abutment 2 from the FB-MultiPier Analysis

Location	Range of Pile Axial Force, P_u (kips)	Range of Pile lengths[1] (feet)	Maximum[2] P_u (kips)	Maximum[2] Bending Moment M_x (kip-ft)	Maximum[2] Bending Moment M_y (kip-ft)	Maximum[2] Shear Force, V_u (kips)	Maximum[2] Lateral Deflection (strong axis) (in)
Rehab[3] Abutment 1 *(section loss)* *Strength*	39.7 to 117.3	13 to 25	117.3 (Pile 16)	11.6 (Pile 2)	93.2 (Pile 18)	25.1 (Pile 18)	0.35 (Pile 18)
Rehab[3] Abutment 1 *(section loss)* *Service*	39.4 to 115.2	13 to 25	115.2 (Pile 16)	11.1 (Pile 2)	90.3 (Pile 18)	24.0 (Pile 18)	0.31 (Pile 18)
Rehab[3] Abutment 2 *(section loss)* *Strength*	38.4 to 117.0	42 to 46	117.0 (Pile 6)	12.2 (Pile 17)	101.7 (Pile 2)	26.8 (Pile 2)	0.33 (Pile 1)
Rehab[3] Abutment 2 *(section loss)* *Service*	36.1 to 114.4	42 to 46	114.4 (Pile 6)	3.6 (Pile 17)	98.2 (Pile 2)	25.6 (Pile 2)	0.30 (Pile 1)

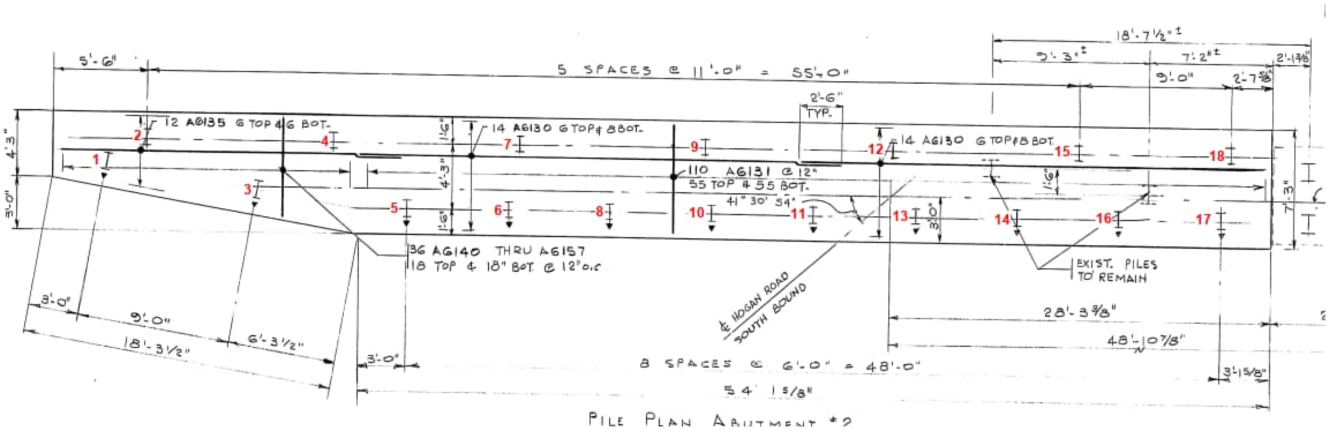
Notes: [1]Provided in record plans and adjusted for batter piles to indicate driven to rock. [2]Maximum values determined from the model and the corresponding pile number from the FB-MultiPier model as given in Figure 1, and Figure 2. [3]Rehab (i.e., rehabilitation) indicates the loading and geometry associated with the proposed rehabilitation condition, not the existing loading and geometry.

Table 3: Summary of the Pile Loading at Pier 5 from the FB-MultiPier Analysis

Location	Range of Pile Axial Force, P_u (kips)	Range of Pile lengths[1] (feet)	Maximum[2] P_u (kips)	Maximum[2] Bending Moment M_x (kip-ft)	Maximum[2] Bending Moment M_y (kip-ft)	Maximum[2] Shear Force, V_u (kips)	Maximum[2] Lateral Deflection (strong axis) (in)
Existing Pier 5 *(intact section)* *Strength* *(Case 1)*	103.1 to 134.0	25 to 30	134.0 (Pile 14)	9.1 (Pile 8)	1.0 (Pile 7)	4.3 (Pile 8)	0.01 (Pile 7)
Existing Pier 5 *(intact section)* *Service* *(Case 1)*	53.3 to 112.3	25 to 30	112.3 (Pile 7)	16.6 (Pile 7)	5.3 (Pile 7)	5.0 (Pile 7)	0.02 (Pile 8)
Existing Pier 5 *(intact section)* *Strength* *(Case 2)*	74.3 to 163.8	25 to 30	163.8 (Pile 12)	31.1 (Pile 11)	11.7 (Pile 11)	6.7 (Pile 8)	0.04 (Pile 7)

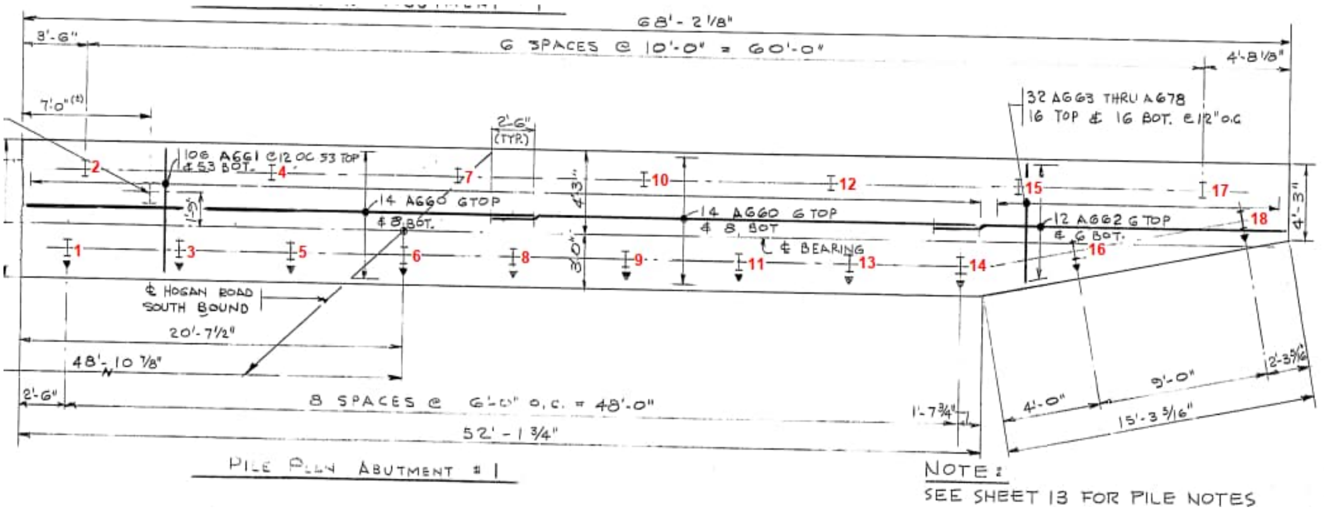
Location	Range of Pile Axial Force, P_u (kips)	Range of Pile lengths[1] (feet)	Maximum[2] P_u (kips)	Maximum[2] Bending Moment M_x (kip-ft)	Maximum[2] Bending Moment M_y (kip-ft)	Maximum[2] Shear Force, V_u (kips)	Maximum[2] Lateral Deflection (strong axis) (in)
Existing Pier 5 *(intact section)* *Service* *(Case 2)*	18.6 to 142.9	25 to 30	142.9 (Pile 8)	33.7 (Pile 14)	16.1 (Pile 11)	7.5 (Pile 5)	0.04 (Pile 1)
Rehab[3] Pier 5 *(section loss)* *Strength* *(Case 1)*	101.9 to 146.6	25 to 30	146.6 (Pile 14)	15.8 (Pile 12)	4.1 (Pile 8)	4.3 (Pile 12)	0.02 (Pile 7)
Rehab[3] Pier 5 *(section loss)* *Service* *(Case 1)*	73.5 to 107.9	25 to 30	107.9 (Pile 12)	13.6 (Pile 12)	3.8 (Pile 8)	3.9 (Pile 12)	0.01 (Pile 1)
Rehab[3] Pier 5 *(section loss)* *Strength* *(Case 2)*	90.8 to 156.1	25 to 30	156.1 (Pile 14)	15.6 (Pile 12)	3.6 (Pile 14)	4.3 (Pile 11)	0.02 (Pile 7)
Rehab[3] Pier 5 *(section loss)* *Service* *(Case 2)*	62.7 to 116.3	25 to 30	116.3 (Pile 14)	13.6 (Pile 12)	3.6 (Pile 14)	3.9 (Pile 12)	0.01 (Pile 7)

Notes: [1] Provided in record plans and adjusted for batter piles to indicate driven to rock. [2] Maximum values determined from the model and the corresponding pile number from the FB-MultiPier model as given in Figure 3. [3] Rehab (i.e., rehabilitation) indicates the loading and geometry associated with the proposed rehabilitation condition, not the existing loading and geometry.



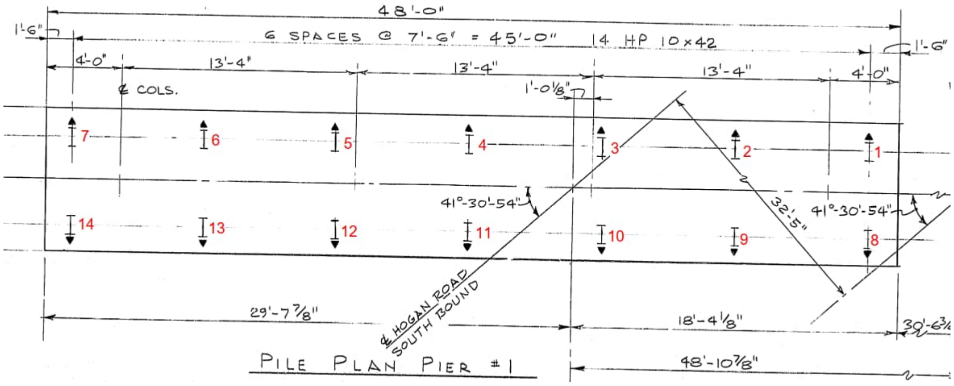
FB-MultiPier model of abutment and piles corresponding with record plan piles (below)

Figure 1: Abutment 1 Pile Locations from the Record Plans (upper) and the FB-MultiPier Model (lower).



FB-MultiPier model of abutment and piles corresponding with record plan piles (below)

Figure 2: Abutment 2 Pile Locations from the Record Plans (upper) and the FB-MultiPier Model (lower).



FB-MultiPier model of pier and piles corresponding with record plan piles (below)

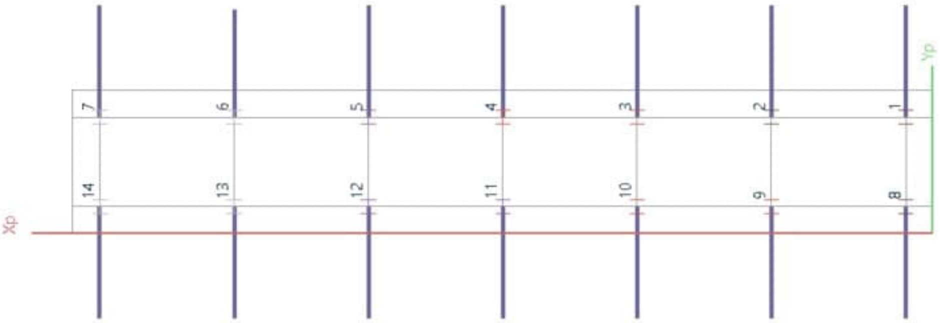


Figure 3: Pier 5 Pile Locations from the Record Plans (upper) and the FB-MultiPier Model (lower).

Additional Downdrag Loading on Abutment Piles

We expect that any downdrag loading that the existing abutment piles may be subject to is related to two possible phases of settlement:

- Settlement from the embankment widening estimated to have occurred between the construction of the 1983 bridge and present, which will be discussed subsequently. This downdrag loading will already have affected the piles and would have additionally added bending stresses to the battered piles.
- Settlement from the embankment widening to accommodate the proposed new bridge between present and 50 years into the future.

WSP estimated settlement that may have occurred between construction of the 1983 embankments and present at the existing bridge abutments using Rocscience's three-dimensional (3D) settlement analysis software *Settle3*[13]. We did not evaluate settlement at Pier 5, as any fills added at the pier location were minimal at the elevation of I-95. The *Settle3* model is based on WSP's interpretation of the original ground surface prior to the embankment construction for the 1983 bridge interpreted from the record plans[3] with assumptions of typical slope angles, offsets from the existing embankment, and a comparison of the pre-1983 and present grade at the

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[13] Rocscience, Inc. Settle3 software package, version 5.015, build date April 25, 2022.

approach embankments. We assume that the fills used to bring the ground surface from the pre-1983 embankment surfaces up to the existing present ground surface would have been granular borrow and this loading would have resulted in settlement at the two bridge abutments. Using the modern stationing provided in the PDR, we evaluated the 3D settlement for Abutment 1 at Station (Sta.) 109+52 based on a model developed between Sta. 107+42 and Sta. 109+68 and settlement for Abutment 2 at Sta. 113+91 based on a model developed between Sta. 113+68 and Sta. 116+50. The ground model was developed using WSP's interpreted subsurface layering, bedrock depth, and engineering parameters from our 100-series and 200-series boring program. We modeled cohesionless soil layers (i.e., proposed fills, existing fills, and glacial till) with immediate settlement only and estimated material properties for these layers from the SPT N_{60} values reported in the boring logs. We modeled the cohesive silty clay layer with immediate settlement, primary consolidation settlement, and secondary compression settlement. Material properties for the silty clay layer were based on consolidation laboratory test results from undisturbed thin-wall Shelby tube samples from the borings. Refer to the full methodology of the analyses, material properties, and calculations in Attachment 5.

Per AASHTO[2] LRFD Article 3.11.8, downdrag can be assumed to fully develop for the length of pile with settlement equal to or greater than 0.4 inches. The settlement analyses at Abutment 1 and Abutment 2 indicate downdrag loads would have been imposed due to settlement of the surrounding soil. While settlements at the abutment pile locations are expected to vary along the length of the abutment due to variation in loading and soil stratigraphy, WSP calculated a downdrag loading for the pile at each abutment with the maximum axial loading for the intact section of the HP 10x42 piles.

We used the software package APile[14] to calculate shaft resistance contributing to downdrag loads along the length of the pile. Shaft resistance was modeled using the FHWA method for computation of unit load transfers and axial pile capacity. Specifically, the Nordlund method was used for unit shaft resistance in cohesionless soils, and the alpha method was used for unit shaft resistance in cohesive soils. WSP calculated downdrag loads in accordance with the methods described in AASHTO LRFD Article 10.7.3.7. In accordance with AASHTO LRFD Tables 3.4.1-1 and 3.4.1-2, Strength I downdrag load factors of 1.05 and 1.40 were used for the cohesionless and cohesive soils, respectively. The accumulated downdrag load on the pile was calculated to a depth where settlement in the soil layer is less than 0.4 inches relative to the pile. Refer to the full methodology of the analyses, material properties, and calculations in Attachment 5.

Table 4 shows the total settlement estimated at each abutment due to embankment loading. The greatest embankment fills at the western portion of the project site are expected result in the greatest loading and settlements. Table 4 also provides total settlement and corresponding downdrag load adjacent to the following piles within each abutment: 1) pile with the most predicted settlement; 2) pile with the largest axial load from the FB-MultiPier analysis, and 3) pile with the least predicted settlement. This information is provided to bound the likely downdrag loads on the piles and the effect these loads may have on total pile loading. It does not, however, capture all possible settlements, structural loading, and downdrag loading scenarios.

Table 4 illustrates that additional loads may be present from settlement induced downdrag. As such, there may be several piles where the applied loading that includes downdrag loading exceeds the factored geotechnical and/ or

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[14] Ensoft, Inc. APILE Version 2019.9.11 - Offshore. Release date October 15, 2021.

structural axial resistance that will be discussed subsequently. Additionally, settlement is likely to have caused additional bending in the battered piles, not just axial loading due to downdrag.

At the time MaineDOT directs us to pursue alternatives to retrofitting the bridge existing abutments to accommodate the additional 50-year design life and AASHTO[2] LRFD requirements, we can provide a design to accommodate existing and potential new additional downdrag loading from the proposed embankment widening.

Table 4: Summary of Estimated Total Settlement from 1983 to Present and Associated Downdrag Loading.

Location	Pile Number	Maximum Total Settlement[1] (in)	Total Settlement at Pile[1] (in)	Length of Pile[2] (feet)	Total Downdrag Load for Pile[3] (kips)	Strength I Factored Axial Load for Pile[4] (kips)
Abutment 1	7	1.2	1.2	19.4	66.0	136.9
	16 [5]		0.6	13.4	40.8	158.6
	17		0.4	13.4	9.9	123.1
Abutment 2	2	1.5	0.4	41.5	13.4	95.9
	6 [5]		0.7	42.2	70.4	188.1
	18		1.5	45.3	89.7	136.9

Notes: [1]From Settle3D model estimating settlement between construction of the 1983 embankments and present. [2]From Record Plans. [3]Based on resistance factors for Strength I for subsurface soils. [4]Sum of the factored Strength I axial load on the pile from the FB-MultiPier analyses including downdrag and pile weight. [5]Pile with maximum load from Table 2. [6]Values in red exceed the factored geotechnical and/or structural axial resistance indicated below.

Pile Resistance

Structural Resistance

WSP analyzed the nominal structural and geotechnical pile resistance of the piles following the design procedures outlined in AASHTO[2] and the MaineDOT BDG[4]. In accordance with the MaineDOT BDG[4], the maximum factored axial design load applied to the existing piles is controlled by the lesser of the factored structural pile resistance and the factored geotechnical pile resistance. The factored structural pile resistance in axial compression, P_r, was calculated for the piles with a resistance factor of $\phi_c = 0.50$ for piles subject to damage due to severe driving where a pile tip is required when piles are driven to bedrock (AASHTO[2] LRFD Section 6.5.4.2). The combined axial and flexural structural pile resistance of the piles was calculated using a resistance factor of $\phi_c = 0.70$ for undamaged H-piles (AASHTO[2] LRFD Section 6.5.4.2).

Geotechnical Axial Resistance, Static Analysis

WSP analyzed the factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis. The static analysis consisted of calculating the shaft resistance along the length of the piles and tip resistance at the base of the piles using the Nordlund method for cohesionless soils, the Alpha method for cohesive soils, and the Canadian Geotechnical Society method for toe and shaft resistance in rock. The factored geotechnical resistance of piles in compression was calculated using a geotechnical resistance factor $\phi_{stat} = 0.45$ for cohesionless soils, $\phi_{stat} = 0.35$ for cohesive soils, and $\phi_{stat} = 0.45$ for rock.

The static analysis was performed predominantly for piles assumed to have the section loss of 0.07 inches from each pile face aligned with our initial Corrosion Potential[6] TM prior to pile section measurements. The factored geotechnical axial resistances varied between 25 kips and 61 kips for the piles with 0.07 inches of section loss for Abutment 1, Abutment 2, and Pier 5. The factored geotechnical axial resistance for the Abutment 1 intact section was 29 kips. The results are discussed in WSP's May 2024 TM[15] related to the geotechnical evaluation of the existing bridge piles. The low factored geotechnical axial resistance in the static analysis can be attributed to the following:

- Very low Rock Quality Designation (RQD) observed in the recovered rock core samples from the 100- and 200-series borings (range of 0% to 76%, average of 25%, and median of 25%). The Kulhawy and Goodman method for hard rock outlined in Section 7.2.1.4.2 of FHWA GEC 12[8] estimates the nominal toe resistance (R_p) for hard rock as $0.33q_u$ when RQD ≤ 70%, whereas R_p is estimated as $0.80q_u$ when RQD = 100%.
- Low laboratory-measured unconfined compressive strength (q_u) from the most intact rock core (908 ksf to 3,107 ksf)
- Small toe area of the HP 10x42 piles.

Because the factored geotechnical axial resistance from the static analysis was very low compared to the 55.5 ton maximum pile loading provided on the record plans[3], MaineDOT requested WSP use wave equation analysis to determine resistance. This is discussed below.

Geotechnical Axial Resistance, Wave Equation Analysis

WSP performed at wave equation analysis to estimate the nominal resistance of HP 10x42 driven to rock at Abutment 1, Abutment 2, and Pier 5. Based on discussions with Maine DOT and communications with local pile driving subcontractors, the American Pile Driving Equipment APE D12-42 single acting diesel pile driving hammer was a common hammer used in bridge construction in the 1980's and we used this in our evaluations. Additionally, the record plans[3] indicate that all work was to comply with MaineDOT's Standard Specifications from 1968. The 1968[16] Standard Specifications Section 501 Foundations Piles provides a driving criterion of 10 blows not exceeding 1 inch of penetration.

Drivability was evaluated with the wave equation analysis program GRLWEAP[17] by Pile Dynamics, Inc. by comparing estimated driving stresses required to drive the piles to meet the driving criterion of 10 blows not exceeding 1 inch of penetration. The analysis evaluated resistance assuming the pile was driven to rock per direction the record plans[3] and a resistance distribution of 10% in the shaft and 90% at the toe for the shortest pile length at Abutment 1, Abutment 2, and Pier 5 established considering the driven pile lengths and bedrock elevation. We evaluated driving stresses for intact HP 10x42 piles with a yield strength of 36 ksi at both 90% and 100% of the steel yield strength where the driving pressures for the APE D12-42 hammer were varied to achieve

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[15] WSP USA, Inc. (WSP), Technical Memorandum: Geotechnical Evaluation of Existing Bridge Piles, Hogan Road Bridge Replacement, Bangor, Maine, WIN 018595.10, March 23, 2024, 144 pp. Submitted to Laura Krusinski, PE, Maine Department of Transportation

[16] Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1968

[17] Pile Dynamics, Inc. GRLWEAP Software Package Version 14

the driving criterion of 10 blows for 1 inch of penetration. We selected 90% of the pile yield strength as the limiting pile stress from the current MaineDOT Standard Specifications and 100% of the pile yield stress at the direction of MaineDOT to account for likely driving methods at the time the bridge was constructed.

The record plans do not provide pile driving records, nor are construction diaries or reports for the bridge that include pile driving records, load test results, or WEAP evaluation in MaineDOT's archives. We used a resistance factor of $\phi_{dyn} = 0.50$ associated with piles conditions where a wave equation analysis is available, dynamic measurements or load tests are not available, and field confirmation of hammer performance is available (AASHTO LRFD Table 10.5.5.2.3-1) to calculate the factored geotechnical resistance at MaineDOT's request. Per discussions with MaineDOT, WSP used the wave equation analysis factored geotechnical resistances for Abutment 1, Abutment 2, and Pier 5 at driving stresses of 100% of the steel yield strength in the pile evaluations. The resistance factor of $\phi_{dyn} = 0.50$ was applied to the nominal geotechnical resistance obtained from the GRLWEAP wave equation analysis.

Table 5 provides the factored structural axial resistance, factored geotechnical axial resistance, and the combined axial and flexural interaction ratio for abutment piles for the largest applied axial loading from FB-MultiPier from Table 2 with section loss as well as for these piles with additional downdrag loading provided in Table 4. Table 6 provides the factored structural axial resistance, factored geotechnical axial resistance, and the combined axial and flexural interaction ratio for Pier 5 piles for the largest applied axial loading from FB-MultiPier from Table 3.

Table 5 and Table 6 indicate values in red where the axial load applied to the piles without and with estimated downdrag loading from the original bridge embankment construction exceed the factored structural and/or geotechnical resistance, and where the combined axial and flexural interaction exceeds the AASHTO[2] LRFD limiting value of 1.0.

Table 5: Strength Limit State Factored Structural and Geotechnical Axial Pile Resistance for Abutment 1 and Abutment 2.

Location	Factored Structural Axial Resistance[1] ϕ_c (kips)	Factored Geotechnical Axial Resistance[2] Φ_{dyn} (kips)	Combined Axial and Flexural interaction[3]
Abutment 1 *(incl. section loss)*	160	144	1.3
Abutment 1 *(incl. section loss and downdrag[4])*	160	144	1.5
Abutment 2 *(incl. section loss)*	155	164	1.4
Abutment 2 *(incl. section loss and downdrag[4])*	155	164	1.7

Note: [1]Factored structural resistance in HP10x42 piles. [2]Factored Geotechnical Resistance based on wave equation analysis with $\Phi_{dyn} = 0.50$. [3]Combined axial and flexural interaction, values in red exceed the limiting value of 1.0. [4]Downdrag estimated from construction of the 1983 bridge embankment and presented in Table 4. [5]Values in red are less than the maximum factored axial load applied to one or more piles in the structure as determined from the FB-MultiPier analysis.

Table 6: Strength Limit State Factored Structural and Geotechnical Axial Pile Resistance for Pier 5.

Location	Factored Structural Axial Resistance[1] ϕ_c (kips)	Factored Geotechnical Axial Resistance[1,2] Φ_{dyn} (kips)	Combined Axial and Flexural interaction[3]
Rehab Pier 5 *(incl. section loss)* *Strength Case 1*	169	155	0.8
Rehab Pier 5 *(incl. section loss)* Strength Case 2	169	155	0.9
Existing Pier 5 *(intact section)* Service Case 1	222	155	0.5
Existing Pier 5 *(intact section)* Service Case 2	220	155	0.9

Note: [1]Factored structural resistance in the intact or section loss HP10x42 piles.[2]Factored Geotechnical Resistance based on wave equation analysis [3] Combined axial and flexural interaction, values in red exceed the limiting value of 1. [4]Values in red are less than the maximum factored axial load applied to one or more piles in the structure as determined from the FB-MultiPier analysis.

Table 7 provides a summary of axial load for each pile in Abutment 1 and Abutment 2 from the FB-MultiPier analysis for the proposed bridge rehabilitation loading. Table 8 and Table 9 provide a summary of axial load for each pile in Pier 5 from the FB-MultiPier analysis for the existing loading and pier geometry and proposed pier rehabilitation loading and geometry, respectively. The axial loads in red in these tables identify the piles where the axial load exceeds the factored geotechnical axial resistance from Table 5 and Table 6. We present this information to show how many piles have loads that exceed the factored axial resistances and identify where those piles are concentrated to identify where mitigation may be required. In summary:

- The FB-MultiPier loading from the proposed rehabilitation scenario does not exceed the factored geotechnical resistance for Abutment 1 and Abutment 2. This is the case when existing and potential future downdrag loading is not considered. As previously discussed, there is potential additional downdrag loading associated with some piles within the abutments from the original construction, and there may be additional downdrag loading for piles nearest to the proposed new bridge embankment.
- For existing geometry and loading at Pier 5, the FB-MultiPier loading, 7 of the 14 piles (50%) have loading that exceeds the factored geotechnical resistance by 0.6 kips to 8.8 kips.
- For proposed rehabilitation geometry and loading at Pier 5, the FB-MultiPier loading, 1 of the 14 piles has loading that exceeds the factored geotechnical resistance by 1.1 kips.

Table 7: Comparison of Pile Axial Loads to Factored Geotechnical Resistance at Abutment 1 and Abutment 2: Proposed Rehabilitation Scenario.

Pile	Abutment 1[1] (incl. section loss) Strength		Abutment 1[1] (incl. section loss) Service		Abutment 2[2] (incl. section loss) Strength		Abutment 2[2] (incl. section loss) Service	
	Length (feet)	Axial Load (kips)	Length (feet)	Axial Load (kips)	Length (feet)	Axial Load (kips)	Length (feet)	Axial Load (kips)
1	24.1	42.5	24.1	42.5	42.0	111.2	42.0	108.8
2	24.1	39.7	24.1	39.4	42.0	82.2	42.0	76.7
3	24.1	81.1	24.1	79.4	42.0	113.4	42.0	111.0
4	21.6	56.3	21.6	54.5	42.7	81.7	42.7	76.5
5	21.6	110.1	21.6	107.6	42.7	115.2	42.7	112.7
6	21.6	111.5	21.6	109.0	42.7	117.0	42.7	114.4
7	19.5	70.3	19.5	66.9	43.5	80.4	43.5	75.7
8	19.5	110.7	19.5	108.2	43.5	113.2	43.5	110.7
9	17.6	81.0	17.6	76.7	43.5	113.2	43.5	110.8
10	19.5	110.9	19.5	108.5	44.2	72.1	44.2	68.1
11	17.6	109.8	17.6	107.6	44.2	111.6	44.2	109.6
12	17.6	83.3	17.6	78.1	44.2	62.8	44.2	59.3
13	15.4	107.3	15.4	105.2	44.9	109.5	44.9	107.7
14	15.4	103.9	15.4	102.1	44.9	107.0	44.9	105.3
15	15.4	81.0	15.4	75.3	44.9	51.4	44.9	48.4
16	13.4	117.3	13.4	115.2	45.9	81.8	45.9	80.7
17	13.4	113.0	13.4	111.0	45.9	38.4	45.9	36.2
18	13.4	79.6	13.4	73.4	45.9	46.2	45.9	46.2

Notes: [1]Refer to Figure 1 for Abutment 1 pile locations. [2]Refer to Figure 2 for Abutment 2. [3]Values in red exceed the factored geotechnical axial resistance reported in Table 5, and do not include estimated downdrag loading.

Table 8: Comparison of Pile Axial Loads to Factored Geotechnical Resistance at Pier 5: Existing Scenario

Pile	Pier 5[1] (intact section) Case 1 Strength		Pier 5[1] (intact section) Case 1 Service		Pier 5[1] (intact section) Case 2 Strength		Pier 5[1] (intact section) Case 2 Service	
	Length (feet)	Axial Load (kips)	Length (feet)	Axial Load (kips)	Length (feet)	Axial Load (kips)	Length (feet)	Axial Load (kips)
1	25.9	103.1	25.9	90.3	25.9	77.3	25.9	43.6
2	25.9	108.6	25.9	94.7	25.9	78.2	25.9	40.5
3	27.1	113.6	27.1	98.5	27.1	79.0	27.1	37.6
4	27.1	117.2	27.1	101.4	27.1	78.8	27.1	34.2
5	28.2	124.3	28.2	107.8	28.2	80.7	28.2	31.6

	Pier 5[1] (intact section) Case 1 Strength		Pier 5[1] (intact section) Case 1 Service		Pier 5[1] (intact section) Case 2 Strength		Pier 5[1] (intact section) Case 2 Service	
Pile	Length (feet)	Axial Load (kips)	Length (feet)	Axial Load (kips)	Length (feet)	Axial Load (kips)	Length (feet)	Axial Load (kips)
6	29.5	126.5	29.5	110.0	29.5	77.9	29.5	25.9
7	29.5	128.4	29.5	112.3	29.5	74.3	29.5	18.6
8	25.9	104.5	25.9	53.3	25.9	155.6	25.9	142.9
9	27.1	110.7	27.1	59.2	27.1	157.6	27.1	141.5
10	27.1	116.5	27.1	64.8	27.1	158.8	27.1	139.7
11	28.2	124.6	28.2	71.8	28.2	163.7	28.2	141.4
12	28.2	128.6	28.2	76.1	28.2	163.8	28.2	139.0
13	29.5	131.5	29.5	79.5	29.5	163.4	29.5	135.8
14	29.5	134.0	29.5	82.9	29.5	161.7	29.5	131.6

Notes: [1]Refer to Figure 3 for Pier 5 pile locations. [2]Values in red exceed the factored geotechnical axial resistance reported in Table 6.

Table 9: Comparison of Pile Axial Loads to Factored Geotechnical Resistance at Pier 5: Proposed Rehabilitation Scenario

	Pier 5[1] (incl. section loss) Case 1 Strength		Pier 5[1] (incl. section loss) Case 1 Service		Pier 5[1] (incl. section loss) Case 2 Strength		Pier 5[1] (incl. section loss) Case 2 Service	
Pile	Length (feet)	Axial Load (kips)	Length (feet)	Axial Load (kips)	Length (feet)	Axial Load (kips)	Length (feet)	Axial Load (kips)
1	25.9	101.9	25.9	73.5	25.9	90.8	25.9	62.7
2	25.9	104.9	25.9	74.1	25.9	97.9	25.9	67.2
3	27.1	107.6	27.1	74.5	27.1	104.3	27.1	71.2
4	27.1	109.8	27.1	74.7	27.1	109.9	27.1	74.7
5	28.2	115.3	28.2	77.9	28.2	118.8	28.2	81.3
6	29.5	116.2	29.5	77.2	29.5	123.2	29.5	84.1
7	29.5	117.1	29.5	76.6	29.5	127.8	29.5	87.3
8	25.9	132.0	25.9	103.4	25.9	122.5	25.9	93.9
9	27.1	134.6	27.1	103.6	27.1	128.4	27.1	97.5
10	27.1	137.2	27.1	104.2	27.1	134.3	27.1	101.2
11	28.2	143.0	28.2	107.7	28.2	143.1	28.2	107.7
12	28.2	144.8	28.2	107.9	28.2	147.9	28.2	110.9
13	29.5	145.8	29.5	107.3	29.5	152.0	29.5	113.4
14	29.5	146.6	29.5	106.8	29.5	156.1	29.5	116.3

Notes: [1]Refer to Figure 3 for Pier 5 pile locations. [2]Values in red exceed the factored geotechnical axial resistance reported in Table 6.

Evaluation of Pile Moment, Shear, and Displacement

WSP evaluated the maximum moments in the strong and weak axes, shear in the strong axis, and displacement in the strong axis for the piles as reported in Table 2 and Table 3. The factored flexural resistance $M_{r,}$ in the strong (x) and weak (y) axis for Pier 5, and strong (y) and weak (x) axis for Abutment 1 and Abutment 2 was calculated for the piles with a resistance factor of ϕ_f = 1.00 per AASHTO[2] LRFD Section 6.5.4.2.

- The factored flexural resistance for piles with an intact section is 141.0 kip-feet in the strong axis, and 63.2 kip-feet in the weak axis.
- The factored flexural resistance for piles with section loss is 104.2 kip-feet in the strong axis, and 42.4 kip-feet in the weak axis.

The factored shear resistance $V_{r,}$ is calculated for the maximum shear force in either the weak or strong axis of the piles with a resistance factor of ϕ_v = 1.00 per AASHTO[2] LRFD Section 6.5.4.2.

- The factored shear resistance for the piles with an intact section is 77 kips.
- The factored shear resistance for piles with section loss is 59 kips.

Results are summarized below for the pile or piles with the greatest axial load under the given loading and geometry conditions for each substructure element.

- Abutment 1 Pile 16 under proposed rehabilitation loading and geometry with pile section loss:
  - The deflection in the strong axis is 0.30 inches for piles with section loss for Service I. The pile achieves fixity with no lateral deflection at the pile tip.
  - The combined interactions of axial compression load and weak- and strong-axis flexural moments calculated <u>exceed</u> the combined axial and flexural limiting value of 1.0.
  - The shear calculated in the strong axis does not exceed the shear resistance.
- Abutment 2 Pile 6 under proposed rehabilitation loading and geometry with pile section loss:
  - The deflection in the strong axis is 0.30 inches for piles with section loss for Service I. The achieves fixity, with no lateral deflection at the pile tip.
  - The combined interactions of axial compression load and weak- and strong-axis flexural moments calculated <u>exceed</u> the combined axial and flexural limiting value of 1.0.
  - The shear calculated in the strong axis does not exceed the shear resistance.
- Pier 5 for Piles 7, 8, 12, and 14 under existing loading and geometry with an intact section:
  - The deflection in the strong axis for Service I is 0.02 inches for Pile 7 under Load Case 1 and 0.04 inches for Pile 8 under Load Case 2. The piles achieve fixity with no lateral deflection at the pile tip.
  - The combined interactions of axial compression load and weak- and strong-axis flexural moments calculated do not exceed the combined axial and flexural limiting value of 1.0.
  - The shear values calculated in the strong axis do not exceed the shear resistance.

- Pier 5 for Piles 12 and 14 under proposed rehabilitation loading and geometry with pile section loss:
    - The deflection in the strong axis for Service I is 0.01 inches for Pile 11 under Load Case 1 and 0.01 inches for Pile 14 under Load Case 2. The pile achieves fixity with no lateral deflection at the pile tip.
    - The combined interactions of axial compression load and weak- and strong-axis flexural moments calculated do not exceed the combined axial and flexural limiting value of 1.0.
    - The shear values calculated in the strong axis do not exceed the shear resistance.

Summary of Pile Analysis and Findings

WSP's evaluation included several loading and geometry scenarios for Abutment 1, Abutment 2, and Pier 5 provided by VHB applied to the HP 10x42 pile foundations using multiple FB-MultiPier models incorporating the loading, pile cap, pier, and subsurface materials. Resultant axial loading for piles at Abutment 1, Abutment 2, and Pier 5 were compared to the wave equation analysis factored geotechnical axial resistance and the AASHTO LRFD calculated factored structural axial resistances for the following scenarios:

- Proposed rehabilitation Strength and Service loading and geometry at Abutment 1, Abutment 2, where WSP considered the controlling case, where the effects of downdrag are included for three piles.
- Proposed rehabilitation Strength I and Service I (Case 1 and Case 2) loading and geometry at Pier 5.
- Existing Strength I and Service I (Case 1 and Case 2) loading and geometry at Pier 5.

Comparison of the Strength I loading to factored geotechnical resistance showed that several Pier 5 piles have factored loads that exceed the factored geotechnical resistance in the existing condition and one Pier 5 pile will have a factored load that minimally exceeds the factored geotechnical resistance in the proposed condition, i.e. with a demand to capacity ratio of 1.01.

Additionally, the combined interaction of axial compression loads and weak- and strong-axis flexural moments calculated for Abutment 1 and Abutment 2 most heavily loaded piles with section loss at exceed the combined axial and flexural resistance of the piles. These conditions will need to be mitigated for each abutment during renovation of the existing bridge.

Lastly, downdrag loading associated with the historical embankment settlement (1983 to present) and future embankment settlement will need to be considered in additional pile evaluations and mitigation efforts for the renovation of the existing bridge. This includes estimating the settlement-induced bending moments in the battered piles from existing embankment settlements.

Recommendations

WSP's evaluation of the Abutment 1, Abutment 2, and Pier 5 pile foundations at the existing Hogan Road bridge is based on vertical loading that accounts for the increase in the abutment height and lateral loading mitigation measures that include the use of lightweight FGA behind the abutments and changes to the bearing system as proposed by VHB.

We recommend that MaineDOT and VHB consider the following options for design and remediation of the existing abutment piles. The remediation should include mechanisms for reducing the moments in the existing piles and providing additional axial and flexural capacity to the system, including accommodating settlement-induced downdrag loading experienced by the piles between installation and present day and future yet to be determined embankment settlement-induced downdrag loading at the abutments. To do this, we recommend the following options that are consistent with VHB's 1983 Bridge Alternatives Summary Memorandum[18]:

- Maintain the abutments and piers while supplementing the abutment substructures using a micropile retrofit of the existing foundation footprint.
- Replace the abutments in their current locations.

Despite one pile having a factored load minimally exceeding the factored geotechnical resistance at Pier 5 for the proposed rehabilitation geometry at the Case 2 Strength load condition, i.e., a demand to capacity ratio of 1.01, we believe the existing Pier 5 pile foundation system is adequate for the proposed loading scenario.

CLOSING

WSP looks forward to your review of these analyses and discussion with MaineDOT on how to advance the project based on these evaluations and findings.

WSP USA, Inc.

Melissa E. Landon, PhD, PE	Christopher C. Benda, PE
Lead Consultant, Geotechnical Engineering	*Vice President, Geotechnical Engineering*

DEB/MEL/CCB

Distribution: Jeffrey D. Lloyd, PE, WSP

Attachments: 1: Foundation Loading
2: Pier Foundation Analyses
3: Corrosion Technical Memorandum: Updated
4: Abutment and Pier 5 Pile Foundation Analyses
5: Abutment Pile Foundation Downdrag and Settlement Estimates

https://wsponlinenam.sharepoint.com/sites/gld-157336/project files/6 deliverables/memo - existing bridge piles/rev02 - win 018595.10 hogan road bridge memo - existing bridge piles.docx

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[18] VHB, Memorandum: Hogan Rd Bridges over I-95 at Exist 187 – Bangor (WIN 018595.10) 1983 Bridge – Construction Estimate and Alternatives Summary, June 26, 2024, 3 pp. Submitted to Laurie Rowe, Project Manager, Maine Department of Transportation.

Attachment 1

Computations

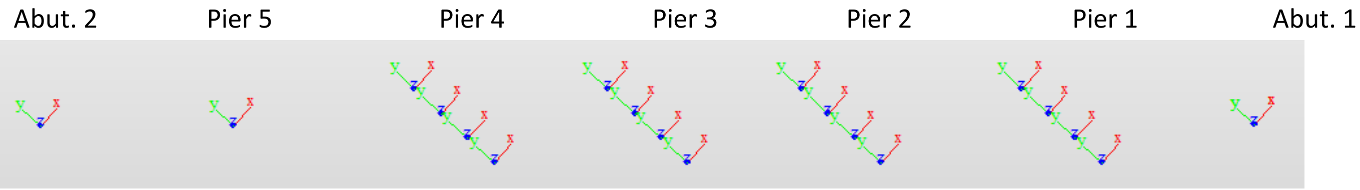
Project: Bangor DDI
Project #: 55758.00
Location: Bangor, ME
Calculated by: TSR
Date: 4/5/2024
Checked by: YP
Date: 4/5/2024
Title: 1983 Bridge Foundation Loads

Notes:

Foundation loads for pile foundations are provided at the bottom of pile caps. Foundation loads for spread footings are provided at bottom of columns. Local axis' are provided for each foundation location in the figure below. See figures on the following page for pile group centroid locations.

The minimum and maximum load factors from LRFD Table 3.4.1-1 are enveloped together for DC, DW, EH and EV. The envelopes are then summed with factored LL, BR and FR to form Strength I, Strength III, Strength V and Service I load combinations. Temperature rise and temperature fall are included in the enveloped factored load combinations. Strength II doesn't apply to this structure and Strength IV will not govern by inspection therefore these combinations were excluded. All Strength combinations are enveloped together.

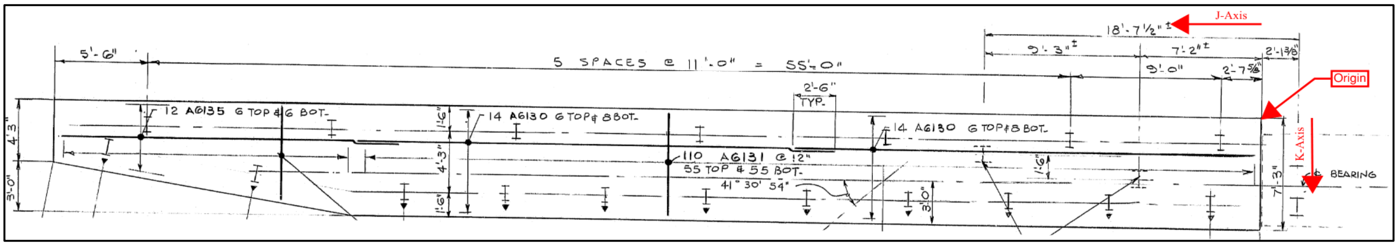
Lightweight fill of density 20 pcf was used for abutment loading and was assumed to extend the full depth to top of footing.



Node Local Axis

Computations

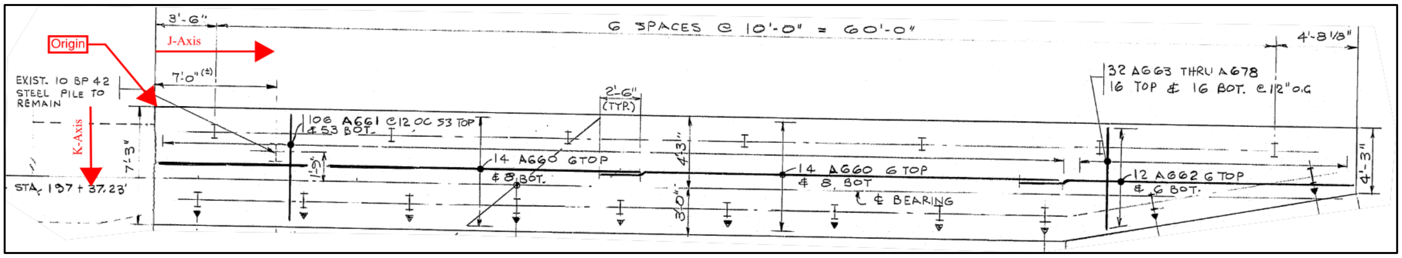
Project:	Bangor DDI	**Project #:**	55758.00
Location:	Bangor, ME		
Calculated by:	TSR	**Date:**	4/5/2024
Checked by:	YP	**Date:**	4/5/2024
Title:	1983 Bridge Foundation Loads		



Pile Group Centroid Coordinates (Old Abut. 2 | New Abut. 1)

J	**K**
33.74 ft	4.61 ft

updated on 10/10/24,
per email from VHB



Pile Group Centroid Coordinates (Old Abut. 1 | New Abut. 2)

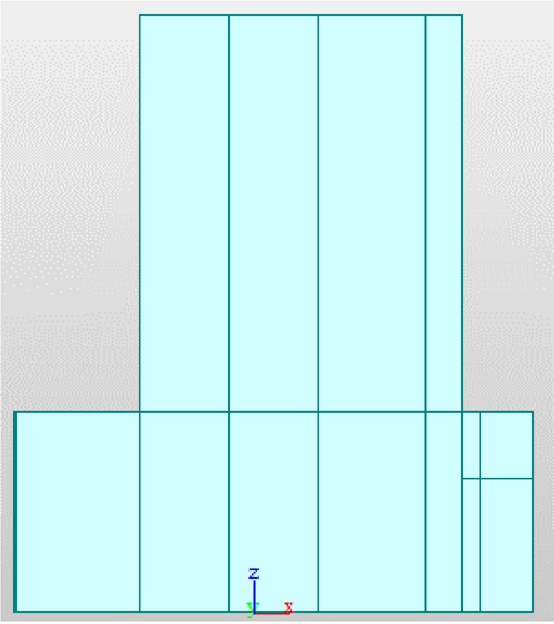
J	**K**
32.94 ft	4.62 ft

updated on 10/10/24,
per email from VHB

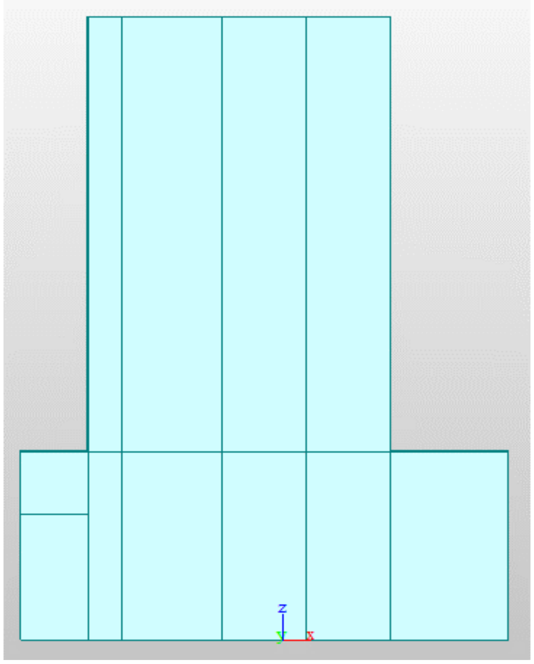
Note: Pile group centroid for pier 5 (not shown) is at the geometric center of the rectangular pile cap.

Computations

Project:	**Bangor DDI**	**Project #:**	**55758.00**
Location:	**Bangor, ME**		
Calculated by:	**TSR**	**Date:**	**4/5/2024**
Checked by:	**YP**	**Date:**	**4/5/2024**
Title:	**1983 Bridge Foundation Loads**		

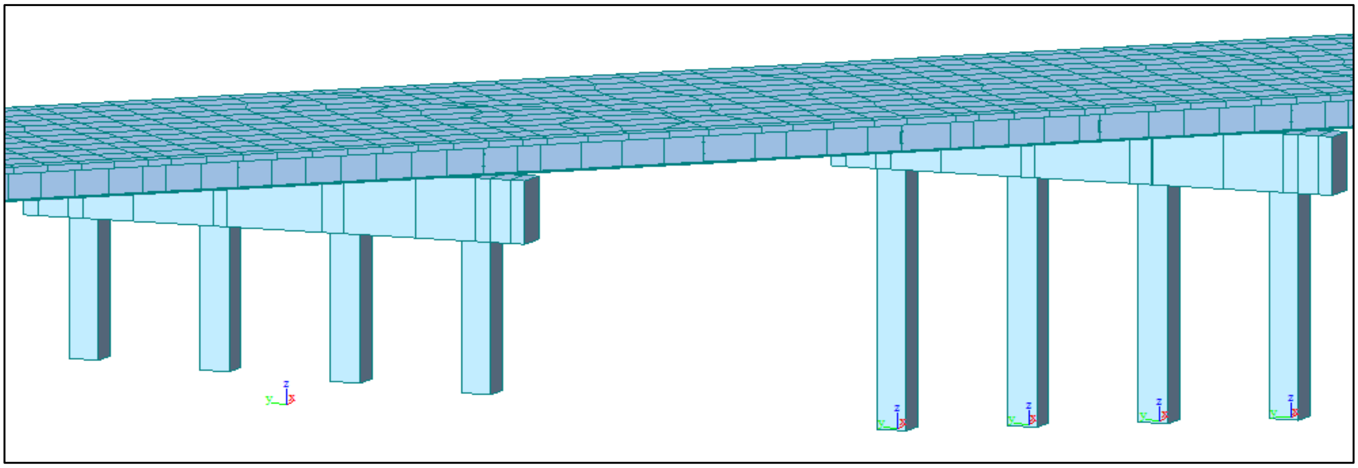


Abutment 1 - Typical Section



Abutment 2 - Typical Section

Note: Sections inlcude adjacent wingwall stem. Moments acting about the longitudinal axis of abutment 1 and abutment 2 are opposite in signs (+/-) due to software restraints; therefore, positive moment at abutment 2 is equivalent to negative moment at abutment 1.



Pier 5 and Pier 4 Nodal Axis

Note: Pier 1 through Pier 3 are similar to Pier 4. The single support node beneath Pier 5 is rigidly connected to the bottom of all four columns and offset 3 ft below bottom of columns.

Computations

Project: Bangor DDI
Project #: 55758.00
Location: Bangor, ME
Calculated by: TSR
Date: 4/5/2024
Checked by: YP
Date: 4/5/2024
Title: 1983 Bridge Foundation Loads

Resultant Force Effects

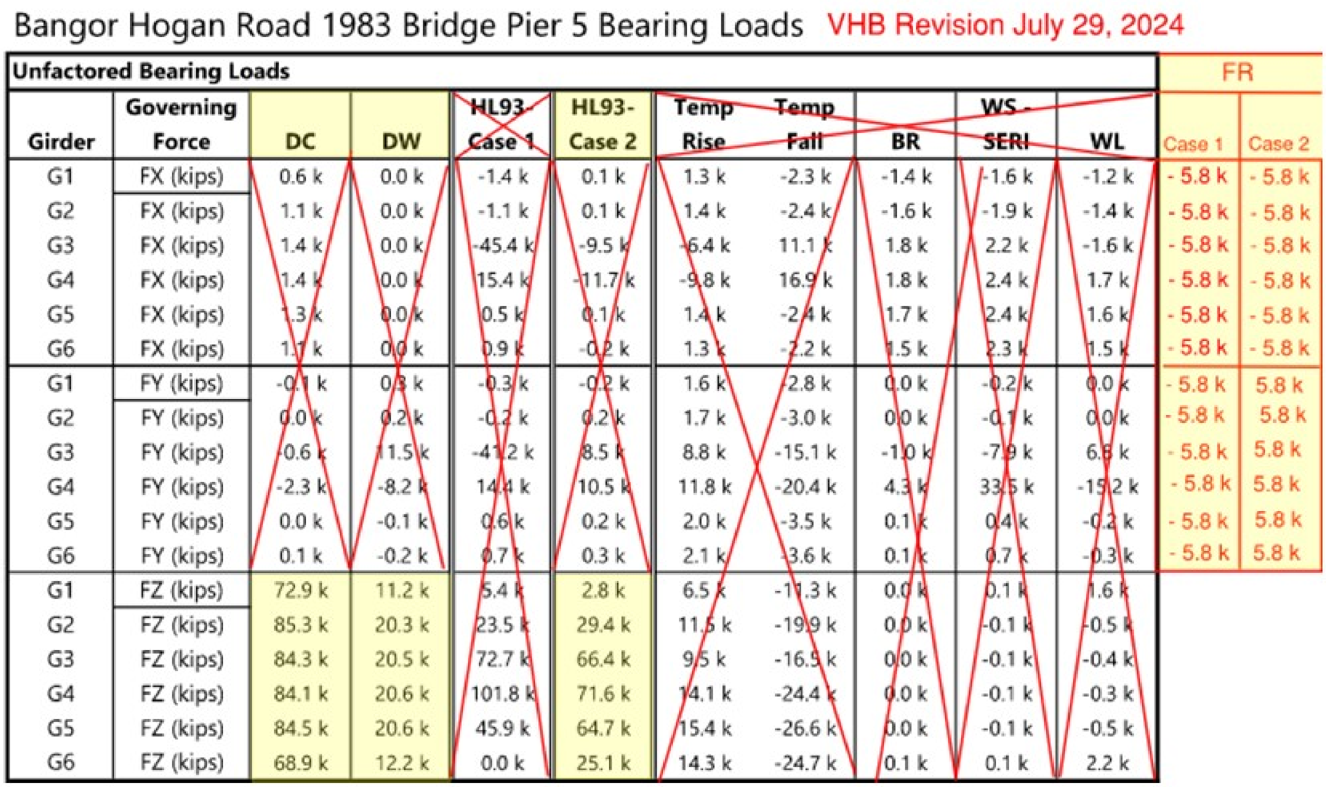
Limit State	Fx (kips)	Fy (kips)	Fz (kips)	Mx (ft·kips)	My (ft·kips)		
Strength	-90	50	-1390	-5510	-1221	**Abut. 1**	**Bottom of Pile Cap**
Service	-80	50	-1340	-5150	-1221		
Strength	50	40	-590	340	660	**Piers 1, 2 & 4**	**Base of Column**
Service	40	40	-570	260	510		
Strength	30	30	-550	240	370	**Pier 3**	**Base of Column**
Service	20	30	-534	190	310		
~~Strength~~	~~80~~	~~130~~	~~-1810~~	~~6370~~	~~1860~~	~~**Pier 5**~~	~~**Bottom of Pile Cap**~~
~~Service~~	~~60~~	~~110~~	~~-1747~~	~~4800~~	~~1760~~		
Strength	-90	-50	-1410	5510	-1330	**Abut. 2**	**Bottom of Pile Cap**
Service	-80	-10	-1360	5150	-1330		

Received form VHB on July 22, 2024

Bangor Hogan Road 1983 Bridge Pier 5 Bearing Loads

Unfactored Bearing Loads

Girder	Governing Force	DC	DW	HL93-Case 1	HL93-Case 2	Temp Rise	Temp Fall	BR	WS - SERI	WL
G1	FX (kips)	0.6 k	0.0 k	-1.4 k	0.1 k	1.3 k	-2.3 k	-1.4 k	-1.6 k	-1.2 k
G2	FX (kips)	1.1 k	0.0 k	-1.1 k	0.1 k	1.4 k	-2.4 k	-1.6 k	-1.9 k	-1.4 k
G3	FX (kips)	1.4 k	0.0 k	-45.4 k	-9.5 k	-6.4 k	11.1 k	1.8 k	2.2 k	-1.6 k
G4	FX (kips)	1.4 k	0.0 k	15.4 k	-11.7 k	-9.8 k	16.9 k	1.8 k	2.4 k	1.7 k
G5	FX (kips)	1.3 k	0.0 k	0.5 k	0.1 k	1.4 k	-2.4 k	1.7 k	2.4 k	1.6 k
G6	FX (kips)	1.1 k	0.0 k	0.9 k	-0.2 k	1.3 k	-2.2 k	1.5 k	2.3 k	1.5 k
G1	FY (kips)	-0.1 k	0.3 k	-0.3 k	-0.2 k	1.6 k	-2.8 k	0.0 k	-0.2 k	0.0 k
G2	FY (kips)	0.0 k	0.2 k	-0.2 k	0.2 k	1.7 k	-3.0 k	0.0 k	-0.1 k	0.0 k
G3	FY (kips)	-0.6 k	11.5 k	-41.2 k	8.5 k	8.8 k	-15.1 k	-1.0 k	-7.9 k	6.8 k
G4	FY (kips)	-2.3 k	-8.2 k	14.4 k	10.5 k	11.8 k	-20.4 k	4.3 k	33.5 k	-15.2 k
G5	FY (kips)	0.0 k	-0.1 k	0.6 k	0.2 k	2.0 k	-3.5 k	0.1 k	0.4 k	-0.2 k
G6	FY (kips)	0.1 k	-0.2 k	0.7 k	0.3 k	2.1 k	-3.6 k	0.1 k	0.7 k	-0.3 k
G1	FZ (kips)	72.9 k	11.2 k	5.4 k	2.8 k	6.5 k	-11.3 k	0.0 k	0.1 k	1.6 k
G2	FZ (kips)	85.3 k	20.3 k	23.5 k	29.4 k	11.5 k	-19.9 k	0.0 k	-0.1 k	-0.5 k
G3	FZ (kips)	84.3 k	20.5 k	72.7 k	66.4 k	9.5 k	-16.5 k	0.0 k	-0.1 k	-0.4 k
G4	FZ (kips)	84.1 k	20.6 k	101.8 k	71.6 k	14.1 k	-24.4 k	0.0 k	-0.1 k	-0.3 k
G5	FZ (kips)	84.5 k	20.6 k	45.9 k	64.7 k	15.4 k	-26.6 k	0.0 k	-0.1 k	-0.5 k
G6	FZ (kips)	68.9 k	12.2 k	0.0 k	25.1 k	14.3 k	-24.7 k	0.1 k	0.1 k	2.2 k



Bangor Hogan Road 1983 Bridge Pier 5 Bearing Loads

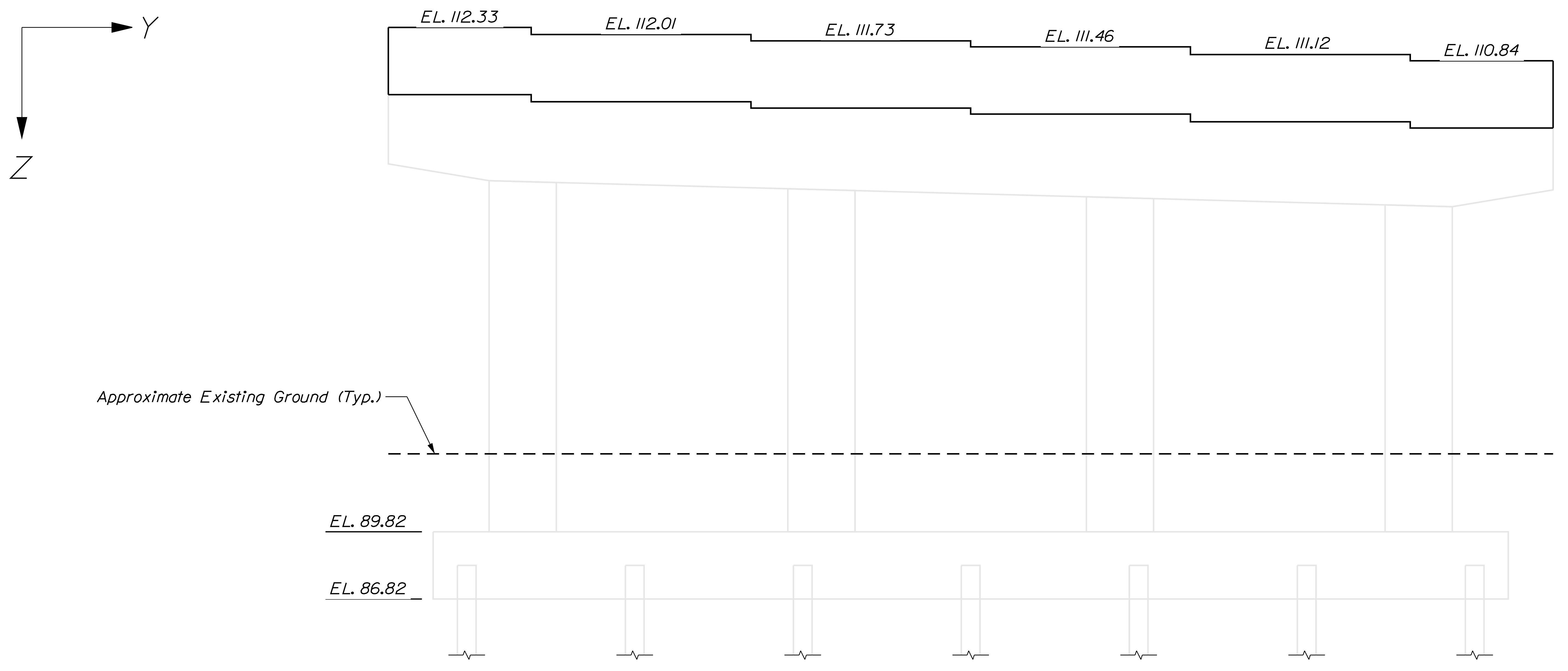
VHB Revision July 29, 2024

Unfactored Bearing Loads

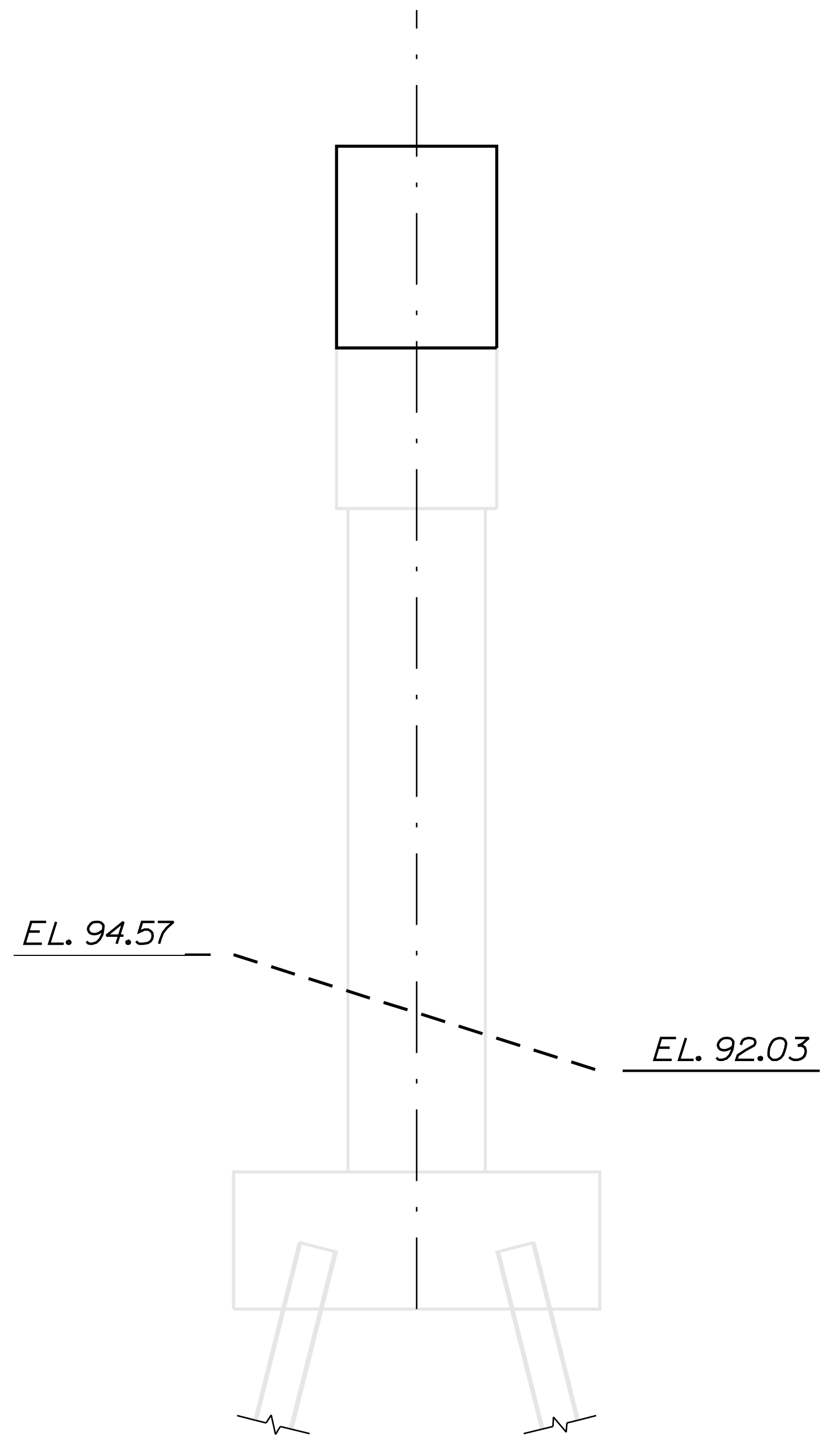
											FR	
Girder	Governing Force	DC	DW	~~HL93-Case 1~~	HL93-Case 2	~~Temp Rise~~	~~Temp Fall~~	~~BR~~	~~WS - SERI~~	~~WL~~	Case 1	Case 2
G1	FX (kips)	~~0.6 k~~	~~0.0 k~~	~~-1.4 k~~	~~0.1 k~~	~~1.3 k~~	~~-2.3 k~~	~~-1.4 k~~	~~-1.6 k~~	~~-1.2 k~~	- 5.8 k	- 5.8 k
G2	FX (kips)	~~1.1 k~~	~~0.0 k~~	~~-1.1 k~~	~~0.1 k~~	~~1.4 k~~	~~-2.4 k~~	~~-1.6 k~~	~~-1.9 k~~	~~-1.4 k~~	- 5.8 k	- 5.8 k
G3	FX (kips)	~~1.4 k~~	~~0.0 k~~	~~-45.4 k~~	~~-9.5 k~~	~~-6.4 k~~	~~11.1 k~~	~~1.8 k~~	~~2.2 k~~	~~-1.6 k~~	- 5.8 k	- 5.8 k
G4	FX (kips)	~~1.4 k~~	~~0.0 k~~	~~15.4 k~~	~~-11.7 k~~	~~-9.8 k~~	~~16.9 k~~	~~1.8 k~~	~~2.4 k~~	~~1.7 k~~	- 5.8 k	- 5.8 k
G5	FX (kips)	~~1.3 k~~	~~0.0 k~~	~~0.5 k~~	~~0.1 k~~	~~1.4 k~~	~~-2.4 k~~	~~1.7 k~~	~~2.4 k~~	~~1.6 k~~	- 5.8 k	- 5.8 k
G6	FX (kips)	~~1.1 k~~	~~0.0 k~~	~~0.9 k~~	~~-0.2 k~~	~~1.3 k~~	~~-2.2 k~~	~~1.5 k~~	~~2.3 k~~	~~1.5 k~~	- 5.8 k	- 5.8 k
G1	FY (kips)	~~-0.1 k~~	~~0.3 k~~	~~-0.3 k~~	~~-0.2 k~~	~~1.6 k~~	~~-2.8 k~~	~~0.0 k~~	~~-0.2 k~~	~~0.0 k~~	- 5.8 k	5.8 k
G2	FY (kips)	~~0.0 k~~	~~0.2 k~~	~~-0.2 k~~	~~0.2 k~~	~~1.7 k~~	~~-3.0 k~~	~~0.0 k~~	~~-0.1 k~~	~~0.0 k~~	- 5.8 k	5.8 k
G3	FY (kips)	~~-0.6 k~~	~~11.5 k~~	~~-41.2 k~~	~~8.5 k~~	~~8.8 k~~	~~-15.1 k~~	~~-1.0 k~~	~~-7.9 k~~	~~6.8 k~~	- 5.8 k	5.8 k
G4	FY (kips)	~~-2.3 k~~	~~-8.2 k~~	~~14.4 k~~	~~10.5 k~~	~~11.8 k~~	~~-20.4 k~~	~~4.3 k~~	~~33.5 k~~	~~-15.2 k~~	- 5.8 k	5.8 k
G5	FY (kips)	~~0.0 k~~	~~-0.1 k~~	~~0.6 k~~	~~0.2 k~~	~~2.0 k~~	~~-3.5 k~~	~~0.1 k~~	~~0.4 k~~	~~-0.2 k~~	- 5.8 k	5.8 k
G6	FY (kips)	~~0.1 k~~	~~-0.2 k~~	~~0.7 k~~	~~0.3 k~~	~~2.1 k~~	~~-3.6 k~~	~~0.1 k~~	~~0.7 k~~	~~-0.3 k~~	- 5.8 k	5.8 k
G1	FZ (kips)	72.9 k	11.2 k	~~5.4 k~~	2.8 k	~~6.5 k~~	~~-11.3 k~~	~~0.0 k~~	~~0.1 k~~	~~1.6 k~~		
G2	FZ (kips)	85.3 k	20.3 k	~~23.5 k~~	29.4 k	~~11.5 k~~	~~-19.9 k~~	~~0.0 k~~	~~-0.1 k~~	~~-0.5 k~~		
G3	FZ (kips)	84.3 k	20.5 k	~~72.7 k~~	66.4 k	~~9.5 k~~	~~-16.5 k~~	~~0.0 k~~	~~-0.1 k~~	~~-0.4 k~~		
G4	FZ (kips)	84.1 k	20.6 k	~~101.8 k~~	71.6 k	~~14.1 k~~	~~-24.4 k~~	~~0.0 k~~	~~-0.1 k~~	~~-0.3 k~~		
G5	FZ (kips)	84.5 k	20.6 k	~~45.9 k~~	64.7 k	~~15.4 k~~	~~-26.6 k~~	~~0.0 k~~	~~-0.1 k~~	~~-0.5 k~~		
G6	FZ (kips)	68.9 k	12.2 k	~~0.0 k~~	25.1 k	~~14.3 k~~	~~-24.7 k~~	~~0.1 k~~	~~0.1 k~~	~~2.2 k~~		

Filename: ...\031_EB_Existing_Pier_5.dgn Division: HIGHWAY Username: jharris Date:7/22/2024

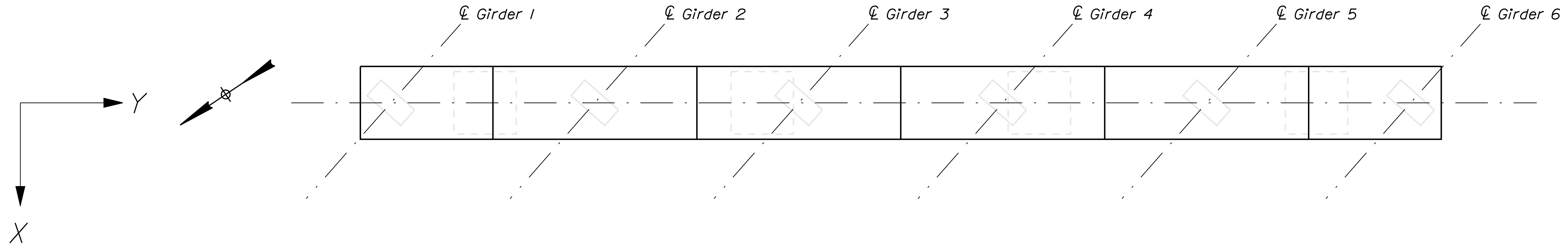
NOTE:
INFORMATION ON THIS SHEET IS FOR
PIER 5 MODELING PURPOSES ONLY.



PROPOSED PIER ELEVATION



PROPOSED PIER SECTION



PROPOSED PIER CAP PLAN

EXISTING FOOTING PLAN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

PROJ. MANAGER	R. MOULTON	BY	DATE
DESIGN-DETAILED			7/12/24
CHECKED-REVIEWED	YP	SMH	7/12/24
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD BRIDGE
I-95

BANGOR PENOBSCOT

EASTBOUND BRIDGE PIER 5
REHAB DETAILS

SHEET NUMBER
B1
OF 1

Attachment 2

CALCULATIONS

Date: 5/9/2024
Made by: ATM
Project No.: 31404817.004
Checked by: DEB
Subject: Bearing resistance of existing footings - Historical Pier 2
Reviewed by: CCB
Project Short Title: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME

OBJECTIVE

Determine nominal and factored bearing resistance at Historical Pier 2 (Contemporary Pier 4) of the existing 1983 bridge at the footing with smallest embedment depth which is the most northeast footing. Verify eccentricity, overturning, bearing, and sliding resistance based on the loads provided by VHB.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Carrol E. Taylor & Associates Consulting Engineers for Maine Department of Transportation, Hogan Road Bridge over Interstate 95 in the City of Bangor Penobscot County, As Built Drawings, 1983
4. Wyllie, D.C. 1999. Foundations on Rock, 2nd ed. E&FN Spon, NY.
5. Maine Department of Transportation, Preliminary Design Report, Hogan Road/I95 Bridge #5823 over Interstate 95 Exist 187 Diverging Diamond Interchange, Bangor, Maine Federal Project # 1859510, WIN 018595.10, February 3, 2023.
6. WSP summary of rock core quality (Table 3, Geotechnical Design Report).
7. GeoTesting Express laboratory testing results, dated November 16, 2023 (Appendix C, Geotechnical Design Report).
8. WSP Interpretive subsurface profile D-D', Sheet X Geotechnical Design Report.
9. Maine Department of Transportation Materials and Research Division, Soils Report 80-04, Bangor - Penobscot County, M149-0(2), Hogan Road Bridge, January 1980
10. Loading provided by VHB in Computation package titled 1983 Bridge Foundation Loads, dated 4/5/2024

ASSUMPTIONS

1. Assume the rock mass rating (RMR) of the bedrock the footing is founded on is equal to the RMR of the top run of the nearest recent boring (BB-BHR-102) (Ref. 6).
2. Assume backfill around the footing was placed at 1H:1V slope from the bottom of the footing up to the ground surface.
3. Assume the unconfined compressive strength of the bedrock the footing is founded on is equal to the minimum UCS from the lab test data (Ref. 7).
4. Assume the footing concrete compressive strength is equal to 5 ksi.
5. The slope of the bedrock surface is assumed to be linear between the two outer most pier footings.
6. The following loading conditions were provided by VHB at the top of footing:

					Resultant Forces	
Limit State	**Fx (kips)**	**Fy (kips)**	**Fz (kips)**	**Mx (ft-kips)**	**My (ft-kips)**	**Fxy (kips)**	**Mxy (ft-kips)**
Strength	50	40	-590	340	660	64.0	742.4
Service	40	40	-570	260	510	56.6	572.5

CALCULATION

A. Determine the bearing resistance at the strength limit state.

As per AASHTO LRFD (Ref. 1) Article 10.6.3.2.2, the nominal bearing resistance of rock should be determined using empirical correlation with the geomechanics RMR system. Since AASHTO LRFD does not directly address bearing resistance on bedrock, this analysis will use Wyllie (1999) Foundations on Rock (Ref. 4) to calculate the unfactored bearing resistance based on correlation to the average RMR value determined for the pier.

Since the footing is founded on sloping bedrock surface (Ref. 5), the procedure in Ref. 4 Section 5.2.5 will be used.

1. Use the average RMR value determined for the bedrock at the pier location to calculate the rock mass friction angle and cohesion.

Rock mass friction angle, ϕ'_i:

$$\phi'_i = \arctan\left[\frac{1}{(4h\cos^2\theta - 1)^{1/2}}\right] \qquad \text{(Ref. 4, Eqn 3.16)}$$

$$h = 1 + \frac{16(m\sigma' + s\sigma_{u(r)})}{3m^2\sigma_{u(r)}} \qquad \text{(Ref. 4, Eqn 3.17)}$$

$$\theta = \frac{1}{3}\left\{90 + \arctan\left[\frac{1}{(h^3 - 1)^{1/2}}\right]\right\} \qquad \text{(Ref. 4, Eqn 3.18)}$$

where:

σ' = vertical effective normal stress on bedrock

γ_{fill} = 125 pcf (Ref. 2, Table 3-3)
φ'_{fill} = 32 degrees (Ref. 2, Table 3-3)
K_{pfill} = 2.14
fill thickness at pier location = 10.8 ft (Ref. 8)
$\sigma' = \gamma_{fill}$ x fill thickness = 1.35 ksf
Area of Column = 9.0 ft^2
Area of Footing = 64.0 ft^2
Weight of Soil on Footing = 74.3 kips
$\gamma_{concrete}$= 150.0 pcf
Weight of Footing = 28.8 kips

RMR = rock mass rating	47	(Ref. 6, Run 2 of BB-BHR-102)
rock type:	Metawacke	(Ref. 6)
m = constant, dependent on rock type and RMR	0.686	(Ref. 4, Table 3.7)
s = constant, dependent on rock type and RMR	0.00050	(Ref. 4, Table 3.7)
$\sigma_{u(r)}$ = unconfined compressive strength of intact rock	908	ksf (Ref. 7, minimum UCS from lab test data)

h = 1.02
θ = 55.7 degrees
ϕ'_i = 61.6 degrees

Rock mass cohesion, c_i:

$$c_i = \tau - \sigma' \tan\phi'_i \qquad \text{(Ref. 4, Eqn 3.19)}$$

$$\tau = (\cot\phi'_i - \cos\phi'_i)\frac{m\sigma_{u(r)}}{8} \qquad \text{(Ref. 4, Eqn 3.15)}$$

τ = 5.1 ksf
c_i = 2.6 ksf

2. Calculate the nominal bearing resistance.

$$q_n = C_{f1}cN_{cq} + (C_{f2}B\gamma_r/2)N_{\gamma q} \qquad \text{(Ref. 4, Eqn 5.10)}$$

$$N_o = \frac{\gamma_r H}{c} \qquad \text{(Ref. 4, Eqn 5.11)}$$

where:

t_f = footing thickness	3	ft (Ref. 3, Sheet 14 of 53, concrete footing dimension)
L = footing length	8	ft (Ref. 3, Sheet 14 of 53, concrete footing dimension)
B = footing width	8	ft (Ref. 3, Sheet 14 of 53, concrete footing dimension)
D = footing embedment depth	10.8	ft (Ref. 8)
D / B =	1.35	
L / B =	1.0	
C_{f1} = foundation shape correction factor	1.25	(Ref. 4, Table 5.4, using Square footing)
C_{f2} = foundation shape correction factor	0.85	(Ref. 4, Table 5.4, using Square footing)
c = rock mass cohesion	2,560	psf (Step 1)
γ_r = rock density	171	pcf (Ref. 7, average from lab test data)
Interpreted bedrock elevation at southwest footing =	77.3	ft (Ref. 9)
Interpreted bedrock elevation at northeast footing =	81.8	ft (Ref. 9)
H = slope height	4.5	ft (difference in bedrock elev. at abutments)
X = distance between outside footings (CTC)	40.0	ft (Ref. 3 Sheet 14 of 53)
β = slope angle	6.4	degrees
N_o = stability number	0.30	(Ref. 4, Eqn 5.11)
N_{cq} = bearing capacity factor	6.8	(Ref. 4, Figure 5.5)
$N_{\gamma q}$ = bearing capacity factor	250	(Ref. 4, Figure 5.5, using ϕ_i from Step 1)

q_n = 167,110 psf
= 167 ksf

As per AASHTO LRFD (Ref. 1) Article 10.6.2.5.2, if the recommended value of bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the bearing resistance shall be taken as the lesser of those values. The nominal resistance of concrete shall be taken as $0.3f'_c$.

f'_c = 5000 psi = 720 ksf (assumed)
$q_{n,concrete} = 0.3f'_c$ = 216 ksf

$q_{n,concrete}$		$q_{n,calculated}$		$\sigma_{u(r)}$	
216	>	167	<	908	ksf

Thus, use q_n = 167 ksf

3. Calculate the factored bearing resistance

$$q_r = \phi_b q_n$$

where:

ϕ_b = bearing resistance factor 0.45 (Ref. 1, Table 10.5.5.2.2-1, "Footings on rock")

q_r = 75 ksf

CALCULATIONS

Date:	5/9/2024	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Bearing resistance of existing footings - Historical Pier 2	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

B. Determine the bearing resistance at the service limit state.

Use AASHTO LRFD (Ref. 1) Table C10.6.2.5.1-1 to determine the presumptive bearing resistance at the service limit state.

Type of Bearing Material: Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)

Bearing Resistance Recommended Value of Use = 70 ksf

Note: This bearing resistance is settlement limited (1.0 inch as per AASHTO LRFD Section 10.6.2.5.1) and applies only at the service limit state.

Resistance factor for the service limit state: 1.0 (Ref. 1, Section 10.5.5.1)

Factored bearing resistance = 70 ksf

C. Check Eccentricity and Overturning Based on Moments about the X and Y directions

Use AASHTO LRFD (Ref. 1) Figure C11.5.5-2 to determine the applicable load factors for eccentricity per Ref, 2 section 5.3.9.

For Strength, the following load factors are applied:

Load Factor for Concrete = 0.9
Load Factor for Overburden Soil = 1.0
Total Factored Vertical Load (V) = 690.2 kips

$$CDR_e = \text{OK if } e < \frac{9}{10} \text{ B or L for footings on rock}$$ (Ref. 2, section 5.3.9)

Load Group	V	Mx	My	e_B	e_L	Eccentricity CDR$_e$ (L)	Eccentricity CDR$_e$ (L)
	(kips)	(kip-ft)	(kip-ft)	(ft)	(ft)		
Strength	690.2	340.0	660.0	0.49	0.96	OK	OK

D. Determine CDR Against Bearing Failure

Load Factor maximum (soil) $\gamma_{P,\ soil}$ = 1.5 (Ref. 1 Table 3.4.1-2)
Load Factor Maximum (concrete) $\gamma_{P,\ concrete}$ = 1.25 (Ref. 1 Table 3.4.1-2)
Resistance Factor for Footings on rock (strength) φ_{tb} = 0.45 (Ref. 2 Table 5-2)
Resistance Factor, Service Limit State φ_{tb} = 1.0 (Ref. 1 Section 10.5.5.1)

$B' = B - 2e_B$ (Ref. 2 Section 5.3.5.2)
$L' = L - 2e_L$ (Ref. 2 Section 5.3.5.2)

For footings bearing on rock:

$$q_{app} = \frac{V}{BL}\left(1 + \frac{6e}{B}\right)$$ (Ref. 1 Eqn. 11.6.3.2-2)

CDR_b = OK if $q_{app} < q_r$

Load Group	q_r	V	B'	L'	A'	e	q_{app}	Bearing CDR$_b$
	(ksf)	(kip)	(ft)	(ft)	(ft^2)	(ft)	(ksf)	
Strength	75	737.4	7.0	6.1	42.7	1.0	20.2	OK
Service	70	673.1	7.0	6.1	42.7	0.9	17.2	OK

D. Determine CDR Against Sliding

Load Factor for Sliding Minimum (concrete)	Y_T = 0.9	(Ref. 1 Table 3.4.1-2)
Load Factor for Sliding Minimum (soil)	Y_T = 0.75	(Ref. 1 Table 3.4.1-2)
Resistance Factor for Shear Cast-in-place concrete on rock	φ_s = 0.9	(Ref. 2 Table 5-3)
Resistance Factor for Passive Soil	φ_{ep} = 0.5	(Ref. 2 Section 5.3.8)
Interface Friction Angle for Concrete on clean sound bedrock	φ = 35	deg (Ref. 1, Table C3.11.5.3-1)
Resistance Factor, Service Limit State	φ_{tb} = 1.0	(Ref. 1 Section 10.5.5.1)

The factored resistance against sliding failure:

$$R_r = \varphi R_n = \varphi_s R_f + \varphi_{ep} R_{ep}$$ (Ref. 2, Section 5.3.8)

Rn = nominal sliding resistance
Rf = nominal sliding resistance between soil and foundation
Rep = nominal passive resistance of the soil available throughout the design life of the structure

$$Rf = V \times \tan\varphi$$ (Ref. 2, Section 5.3.8)

CDRs = OK if Shear, fxy < Rr

Load Group	V	R_f	$\varphi_s R_f$	R_{ep}	$\varphi_{ep} R_{ep}$	R_r	Shear, f_{xy}	Sliding CDR_s
	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	
Strength	671.6	470.3	423.2	1.2	0.6	423.8	139.6	OK
Service	673.1	471.3	424.1	1.2	0.6	424.7	129.9	OK

CONCLUSIONS

For the existing spread footings on bedrock at the historical Pier 2 of the1983 bridge, the recommended nominal bearing resistance is 167 ksf for strength and 70 ksf for serviceability. A resistance factor of 0.45 is recommended for use at the strength limit state and a resistance factor of 1.0 is recommended for use at the service limit state. This results in a factored bearing resistance of 75 ksf at the strength limit state and 70 ksf at the service limit state. The existing pier spread footing eccentricity, overturning resistance, bearing resistance, and sliding resistance are satisfactory for the proposed loads provided by VHB.

Date:	4/2/2024	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Bearing resistance of existing footings - Historical Pier 3	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

OBJECTIVE

Determine nominal and factored bearing resistance at Historical Pier 3 (Contemporary Pier 3) of the existing 1983 bridge at the footing with smallest embedment depth which is the northeast most footing.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Carrol E. Taylor & Associates Consulting Engineers for Maine Department of Transportation, Hogan Road Bridge over Interstate 95 in the City of Bangor Penobscot County, As Built Drawings, 1983
4. Wyllie, D.C. 1999. Foundations on Rock, 2nd ed. E&FN Spon, NY.
5. Maine Department of Transportation, Preliminary Design Report, Hogan Road/I95 Bridge #5823 over Interstate 95 Exist 187 Diverging Diamond Interchange, Bangor, Maine Federal Project # 1859510, WIN 018595.10, February 3, 2023.
6. WSP summary of rock core quality (Table 3, Geotechnical Design Report).
7. GeoTesting Express laboratory testing results, dated November 16, 2023 (Appendix C, Geotechnical Design Report).
8. WSP Interpretive subsurface profile D-D', Sheet X Geotechnical Design Report.
9. Maine Department of Transportation Materials and Research Division, Soils Report 80-04, Bangor - Penobscot County, M149-0(2), Hogan Road Bridge, January 1980

ASSUMPTIONS

1. Assume the rock mass rating (RMR) of the bedrock the footing is founded on is equal to the RMR of the top run of the nearest recent boring (BRP-BHR-202) (Ref. 6).
2. Assume backfill around the footing was placed at 1H:1V slope from the bottom of the footing.
3. Assume the unconfined compressive strength of the bedrock the footing is founded on is equal to the minimum UCS from the lab test data (Ref. 7).
4. Assume the footing concrete compressive strength is equal to 5 ksi.
5. The slope of the bedrock surface is assumed to be linear between the two outer most pier footings.

CALCULATION

A. Determine the bearing resistance at the strength limit state.

As per AASHTO LRFD (Ref. 1) Article 10.6.3.2.2, the nominal bearing resistance of rock should be determined using empirical correlation with the geomechanics RMR system. Since AASHTO LRFD does not directly address bearing resistance on bedrock, this analysis will use Wyllie (1999) Foundations on Rock (Ref. 4) to calculate the unfactored bearing resistance based on correlation to the average RMR value determined for the pier.

Since the footing is founded on sloping bedrock surface (Ref. 5), the procedure in Ref. 4 Section 5.2.5 will be used.

CALCULATIONS

Date: 4/2/2024 **Made by:** ATM
Project No.: 31404817.004 **Checked by:** DEB
Subject: Bearing resistance of existing footings - Historical Pier 3 **Reviewed by:** CCB
Project Short Title: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME

1. Use the average RMR value determined for the bedrock at the pier location to calculate the rock mass friction angle and cohesion.

Rock mass friction angle, ϕ'_i:

$$\phi'_i = \arctan\left[\frac{1}{(4h\cos^2\theta - 1)^{1/2}}\right]$$ (Ref. 4, Eqn 3.16)

$$h = 1 + \frac{16\left(m\sigma' + s\sigma_{u(r)}\right)}{3m^2\sigma_{u(r)}}$$ (Ref. 4, Eqn 3.17)

$$\theta = \frac{1}{3}\left\{90 + \arctan\left[\frac{1}{(h^3 - 1)^{1/2}}\right]\right\}$$ (Ref. 4, Eqn 3.18)

where:

σ' = vertical effective normal stress on bedrock

γ_{fill} = 125 pcf (Ref. 2, Table 3-3)
fill thickness at pier location = 8.2 ft (Ref. 8)
$\sigma' = \gamma_{fill}$ x fill thickness = 1.03 ksf

RMR = rock mass rating	57		(Ref. 6, Run 1 of BRP-BHR-208)
rock type:	Metawacke		(Ref. 6)
m = constant, dependent on rock type and RMR	1.445		(Ref. 4, Table 3.7)
s = constant, dependent on rock type and RMR	0.00185		(Ref. 4, Table 3.7)
$\sigma_{u(r)}$ = unconfined compressive strength of intact rock	908	ksf	(Ref. 7, minimum UCS from lab test data)

h = 1.01
θ = 56.9 degrees
ϕ'_i = 65.7 degrees

Rock mass cohesion, c_i:

$$c_i = \tau - \sigma' \tan\phi'_i$$ (Ref. 4, Eqn 3.19)

$$\tau = (\cot\phi'_i - \cos\phi'_i)\frac{m\sigma_{u(r)}}{8}$$ (Ref. 4, Eqn 3.15)

τ = 6.6 ksf
c_i = 4.3 ksf

2. Calculate the nominal bearing resistance.

$$q_n = C_{f1}cN_{cq} + (C_{f2}B\gamma_r/2)N_{\gamma q}$$ (Ref. 4, Eqn 5.10)

$$N_o = \frac{\gamma_r H}{c}$$ (Ref. 4, Eqn 5.11)

where:

L = footing length	10	ft (Ref. 3, Sheet 15 of 53, concrete footing dimension)
B = footing width	10	ft (Ref. 3, Sheet 15 of 53, concrete footing dimension)
D = footing embedment depth	8.2	ft (Ref. 8)
D / B =	0.82	
L / B =	1.0	
C_{f1} = foundation shape correction factor	1.25	(Ref. 4, Table 5.4, using Square Footing)
C_{f2} = foundation shape correction factor	0.85	(Ref. 4, Table 5.4, using Square Footing)
c = rock mass cohesion	4,288	psf (Step 1)
γ_r = rock density	171	pcf (Ref. 7, average from lab test data)
Interpreted bedrock elevation at southwest footing =	80.5	ft (Ref. 9)
Interpreted bedrock elevation at northeast footing =	86.2	ft (Ref. 9)
H = slope height	5.7	ft (difference in bedrock elev. at abutments)
X = distance between outside footings (CTC)	40.0	ft (Ref. 3 Sheet 14)
β = slope angle	8.1	degrees
N_o = stability number	0.23	(Ref. 4, Eqn 5.11)
N_{cq} = bearing capacity factor	6.7	(Ref. 4, Figure 5.5)
$N_{\gamma q}$ = bearing capacity factor	250	(Ref. 4, Figure 5.5, using ϕ_i from Step 1)

q_n = 217,601 psf

= 218 ksf

As per AASHTO LRFD (Ref. 1) Article 10.6.2.5.2, if the recommended value of bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the bearing resistance shall be taken as the lesser of those values. The nominal resistance of concrete shall be taken as $0.3f'_c$.

f'_c = 5000 psi = 720 ksf (assumed)

$q_{n,concrete} = 0.3f'_c$ = 216 ksf

$q_{n,concrete}$		$q_{n,calculated}$		$\sigma_{u(r)}$	
216	<	218	<	908	ksf

Thus, use q_n = 216 ksf

3. Calculate the factored bearing resistance

$$q_r = \phi_b q_n$$

where:

ϕ_b = bearing resistance factor 0.45 (Ref. 1, Table 10.5.5.2.2-1, "Footings on rock")

q_r = 97 ksf

Date:	4/2/2024	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Bearing resistance of existing footings - Historical Pier 3	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

B. Determine the bearing resistance at the service limit state.

Use AASHTO LRFD (Ref. 1) Table C10.6.2.5.1-1 to determine the presumptive bearing resistance at the service limit state.

Type of Bearing Material: Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)

Bearing Resistance Recommended Value of Use = 70 ksf

Note: This bearing resistance is settlement limited (1.0 inch as per AASHTO LRFD Section 10.6.2.5.1) and applies only at the service limit state.

Resistance factor for the service limit state: 1.0 (Ref. 1, Section 10.5.5.1)

Factored bearing resistance = 70 ksf

CONCLUSIONS

For the existing spread footings on bedrock at the Pier 3 of the1983 bridge, the recommended nominal bearing resistance is 218 ksf for strength and 70 ksf for serviceability. A resistance factor of 0.45 is recommended for use at the strength limit state and a resistance factor of 1.0 is recommended for use at the service limit state. This results in a factored bearing resistance of 97 ksf at the strength limit state and 70 ksf at the service limit state.

CALCULATIONS

Date:	3/20/2024	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Bearing resistance of existing footings - Historical Pier 4	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

OBJECTIVE

Determine nominal and factored bearing resistance at Historical Pier 4 (Contemporary Pier 2) of the existing 1983 bridge at the footing with smallest embedment depth which is the northeast most footing.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Carrol E. Taylor & Associates Consulting Engineers for Maine Department of Transportation, Hogan Road Bridge over Interstate 95 in the City of Bangor Penobscot County, As Built Drawings, 1983
4. Wyllie, D.C. 1999. Foundations on Rock, 2nd ed. E&FN Spon, NY.
5. Maine Department of Transportation, Preliminary Design Report, Hogan Road/I95 Bridge #5823 over Interstate 95 Exist 187 Diverging Diamond Interchange, Bangor, Maine Federal Project # 1859510, WIN 018595.10, February 3, 2023.
6. WSP summary of rock core quality (Table 3, Geotechnical Design Report).
7. GeoTesting Express laboratory testing results, dated November 16, 2023 (Appendix C, Geotechnical Design Report).
8. WSP Interpretive subsurface profile D-D', Sheet X Geotechnical Design Report.
9. Maine Department of Transportation Materials and Research Division, Soils Report 80-04, Bangor - Penobscot County, M149-0(2), Hogan Road Bridge, January 1980

ASSUMPTIONS

1. Assume the rock mass rating (RMR) of the bedrock the footing is founded on is equal to the RMR of the top run of the nearest recent boring (BB-BHR-202) (Ref. 6).
2. Assume backfill around the footing was placed at 1H:1V slope from the bottom of the footing.
3. Assume the unconfined compressive strength of the bedrock the footing is founded on is equal to the minimum UCS from the lab test data (Ref. 7).
4. Assume the footing concrete compressive strength is equal to 5 ksi.
5. The slope of the bedrock surface is assumed to be linear between the two outer most pier footings.

CALCULATION

A. Determine the bearing resistance at the strength limit state.

As per AASHTO LRFD (Ref. 1) Article 10.6.3.2.2, the nominal bearing resistance of rock should be determined using empirical correlation with the geomechanics RMR system. Since AASHTO LRFD does not directly address bearing resistance on bedrock, this analysis will use Wyllie (1999) Foundations on Rock (Ref. 4) to calculate the unfactored bearing resistance based on correlation to the average RMR value determined for the pier.

Since the footing is founded on sloping bedrock surface (Ref. 5), the procedure in Ref. 4 Section 5.2.5 will be used.

1. Use the average RMR value determined for the bedrock at the pier location to calculate the rock mass friction angle and cohesion.

Rock mass friction angle, ϕ'_i:

$$\phi'_i = \arctan\left[\frac{1}{(4h\cos^2\theta - 1)^{1/2}}\right] \quad \text{(Ref. 4, Eqn 3.16)}$$

$$h = 1 + \frac{16\left(m\sigma' + s\sigma_{u(r)}\right)}{3m^2\sigma_{u(r)}} \quad \text{(Ref. 4, Eqn 3.17)}$$

$$\theta = \frac{1}{3}\left\{90 + \arctan\left[\frac{1}{(h^3 - 1)^{1/2}}\right]\right\} \quad \text{(Ref. 4, Eqn 3.18)}$$

where:

σ' = vertical effective normal stress on bedrock

γ_{fill} = 125 pcf (Ref. 2, Table 3-3)

fill thickness at pier location = 8.7 ft (Ref. 8)

σ' = γ_{fill} x fill thickness = 1.09 ksf

RMR = rock mass rating 55 (Ref. 6, Run 1 of BB-BHR-202)

rock type: Metawacke (Ref. 6)

m = constant, dependent on rock type and RMR 1.293 (Ref. 4, Table 3.7)

s = constant, dependent on rock type and RMR 0.00158 (Ref. 4, Table 3.7)

$\sigma_{u(r)}$ = unconfined compressive strength of intact rock 908 ksf (Ref. 7, minimum UCS from lab test data)

h = 1.01

θ = 56.7 degrees

ϕ'_i = 65.0 degrees

Rock mass cohesion, c_i:

$$c_i = \tau - \sigma' \tan\phi'_i \quad \text{(Ref. 4, Eqn 3.19)}$$

$$\tau = (\cot\phi'_i - \cos\phi'_i)\frac{m\sigma_{u(r)}}{8} \quad \text{(Ref. 4, Eqn 3.15)}$$

τ = 6.4 ksf

c_i = 4.1 ksf

2. Calculate the nominal bearing resistance.

$$q_n = C_{f1}cN_{cq} + (C_{f2}B\gamma_r/2)N_{\gamma q} \quad \text{(Ref. 4, Eqn 5.10)}$$

$$N_o = \frac{\gamma_r H}{c} \quad \text{(Ref. 4, Eqn 5.11)}$$

where:

L = footing length	8	ft (Ref. 3, Sheet 16 of 53, concrete footing dimension)
B = footing width	8	ft (Ref. 3, Sheet 16 of 53, concrete footing dimension)
D = footing embedment depth	8.7	ft (Ref. 8)
D / B =	1.0875	
L / B =	1.0	
C_{f1} = foundation shape correction factor	1.25	(Ref. 4, Table 5.4, using Square Footing)
C_{f2} = foundation shape correction factor	0.85	(Ref. 4, Table 5.4, using Square Footing)
c = rock mass cohesion	4,055	psf (Step 1)
γ_r = rock density	171	pcf (Ref. 7, average from lab test data)
Interpreted bedrock elevation at southwest footing =	82.3	ft (Ref. 9)
Interpreted bedrock elevation at northeast footing =	84.6	ft (Ref. 9)
H = slope height	2.3	ft (difference in bedrock elev. at abutments)
X = distance between outside footings (CTC)	40.0	ft (Ref. 3 Sheet 14)
β = slope angle	3.3	degrees
N_o = stability number	0.10	(Ref. 4, Eqn 5.11)
N_{cq} = bearing capacity factor	6.9	(Ref. 4, Figure 5.5)
$N_{\gamma q}$ = bearing capacity factor	250	(Ref. 4, Figure 5.5, using ϕ_i from Step 1)

q_n = 180,321 psf
= 180 ksf

As per AASHTO LRFD (Ref. 1) Article 10.6.2.5.2, if the recommended value of bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the bearing resistance shall be taken as the lesser of those values. The nominal resistance of concrete shall be taken as $0.3f'_c$.

f'_c = 5000 psi = 720 ksf (assumed)
$q_{n,concrete} = 0.3f'_c$ = 216 ksf

$q_{n,concrete}$		$q_{n,calculated}$		$\sigma_{u(r)}$	
216	>	180	<	908	ksf

Thus, use q_n = 180 ksf

3. Calculate the factored bearing resistance

$$q_r = \phi_b q_n$$

where:

ϕ_b = bearing resistance factor 0.45 (Ref. 1, Table 10.5.5.2.2-1, "Footings on rock")

q_r = 81 ksf

B. Determine the bearing resistance at the service limit state.

Use AASHTO LRFD (Ref. 1) Table C10.6.2.5.1-1 to determine the presumptive bearing resistance at the service limit state.

Type of Bearing Material: Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)

Bearing Resistance Recommended Value of Use = 70 ksf

Note: This bearing resistance is settlement limited (1.0 inch as per AASHTO LRFD Section 10.6.2.5.1) and applies only at the service limit state.

Resistance factor for the service limit state: 1.0 (Ref. 1, Section 10.5.5.1)

Factored bearing resistance = 70 ksf

CONCLUSIONS

For the existing spread footings on bedrock at the Pier 4 of the 1983 bridge, the recommended nominal bearing resistance is 180 ksf for strength and 70 ksf for serviceability. A resistance factor of 0.45 is recommended for use at the strength limit state and a resistance factor of 1.0 is recommended for use at the service limit state. This results in a factored bearing resistance of 81 ksf at the strength limit state and 70 ksf at the service limit state.

CALCULATIONS

Date:	4/1/2024	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Bearing resistance of existing footings - Historical Pier 5	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

OBJECTIVE

Determine nominal and factored bearing resistance at Historical Pier 5 (Contemporary Pier 1) of the existing 1983 bridge at the southwest most footing which is founded on concrete fill.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Carrol E. Taylor & Associates Consulting Engineers for Maine Department of Transportation, Hogan Road Bridge over Interstate 95 in the City of Bangor Penobscot County, As Built Drawings, 1983
4. Wyllie, D.C. 1999. Foundations on Rock, 2nd ed. E&FN Spon, NY.
5. Maine Department of Transportation, Preliminary Design Report, Hogan Road/I95 Bridge #5823 over Interstate 95 Exist 187 Diverging Diamond Interchange, Bangor, Maine Federal Project # 1859510, WIN 018595.10, February 3, 2023.
6. WSP summary of rock core quality (Table 3, Geotechnical Design Report).
7. GeoTesting Express laboratory testing results, dated November 16, 2023 (Appendix C, Geotechnical Design Report).
8. WSP Interpretive subsurface profile D-D', Sheet X Geotechnical Design Report.
9. Maine Department of Transportation Materials and Research Division, Soils Report 80-04, Bangor - Penobscot County, M149-0(2), Hogan Road Bridge, January 1980

ASSUMPTIONS

1. Assume approximately 4 feet of bedrock was removed prior to the rock mass rating (RMR) of the bedrock the footing is founded on is equal to the RMR of the top run of the nearest recent boring (BB-BHR-101) (Ref. 6).
2. Assume backfill around the footing was placed at 1H:1V slope from the bottom of the footing.
3. Assume the unconfined compressive strength of the bedrock the footing is founded on is equal to the minimum UCS from the lab test data (Ref. 7).
4. Assume the footing concrete compressive strength is equal to 5 ksi.
5. The slope of the bedrock surface is assumed to be linear between the two outer most pier footings.
6. Assume the concrete fill placed below the pier footings (Ref. 5) was the same concrete mix as the footings themselves and has equal strength.

CALCULATION

A. Determine the bearing resistance at the strength limit state.

As per AASHTO LRFD (Ref. 1) Article 10.6.3.2.2, the nominal bearing resistance of rock should be determined using empirical correlation with the geomechanics RMR system. Since AASHTO LRFD does not directly address bearing resistance on bedrock, this analysis will use Wyllie (1999) Foundations on Rock (Ref. 4) to calculate the unfactored bearing resistance based on correlation to the average RMR value determined for the pier.

Since the footing is founded on sloping bedrock surface (Ref. 5), the procedure in Ref. 4 Section 5.2.5 will be used.

1. Use the average RMR value determined for the bedrock at the pier location to calculate the rock mass friction angle and cohesion.

Rock mass friction angle, ϕ'_i:

$$\phi'_i = \arctan\left[\frac{1}{(4h\cos^2\theta - 1)^{1/2}}\right] \qquad \text{(Ref. 4, Eqn 3.16)}$$

$$h = 1 + \frac{16\left(m\sigma' + s\sigma_{u(r)}\right)}{3m^2\sigma_{u(r)}} \qquad \text{(Ref. 4, Eqn 3.17)}$$

$$\theta = \frac{1}{3}\left\{90 + \arctan\left[\frac{1}{(h^3 - 1)^{1/2}}\right]\right\} \qquad \text{(Ref. 4, Eqn 3.18)}$$

where:

σ' = vertical effective normal stress on bedrock

γ_{fill} =	125	pcf	(Ref. 2, Table 3-3)
fill thickness at pier location =	6.5	ft	(Ref. 8)
$\sigma' = \gamma_{fill}$ x fill thickness =	0.81	ksf	
RMR = rock mass rating	60		(Ref. 6, Run 1 of BB-BHR-101)
rock type:	Metawacke		(Ref. 6)
m = constant, dependent on rock type and RMR	1.672		(Ref. 4, Table 3.7)
s = constant, dependent on rock type and RMR	0.00225		(Ref. 4, Table 3.7)
$\sigma_{u(r)}$ = unconfined compressive strength of intact rock	908	ksf	(Ref. 7, minimum UCS from lab test data)

h = 1.01

θ = 57.2 degrees

ϕ'_i = 66.9 degrees

Rock mass cohesion, c_i:

$$c_i = \tau - \sigma' \tan\phi'_i \qquad \text{(Ref. 4, Eqn 3.19)}$$

$$\tau = (\cot\phi'_i - \cos\phi'_i)\frac{m\sigma_{u(r)}}{8} \qquad \text{(Ref. 4, Eqn 3.15)}$$

τ = 6.5 ksf

c_i = 4.6 ksf

2. Calculate the nominal bearing resistance.

$$q_n = C_{f1}cN_{cq} + (C_{f2}B\gamma_r/2)N_{\gamma q} \qquad \text{(Ref. 4, Eqn 5.10)}$$

$$N_o = \frac{\gamma_r H}{c} \qquad \text{(Ref. 4, Eqn 5.11)}$$

where:

L = footing length	8	ft (Ref. 3, Sheet 17 of 53, concrete footing dimension)
B = footing width	8	ft (Ref. 3, Sheet 17 of 53, concrete footing dimension)
D = footing embedment depth	6.5	ft (Ref. 8)
D / B =	0.8125	
L / B =	1.0	
C_{f1} = foundation shape correction factor	1.25	(Ref. 4, Table 5.4, using Square Footing)
C_{f2} = foundation shape correction factor	0.85	(Ref. 4, Table 5.4, using Square Footing)
c = rock mass cohesion	4,557	psf (Step 1)
γ_r = rock density	171	pcf (Ref. 7, average from lab test data)
Interpreted bedrock elevation at southwest footing =	87.0	ft (Ref. 9)
Interpreted bedrock elevation at northeast footing =	92.0	ft (Ref. 9)
H = slope height	5.0	ft (difference in bedrock elev. at abutments)
X = distance between outside footings (CTC)	40.0	ft (Ref. 3 Sheet 14)
β = slope angle	7.1	degrees
N_o = stability number	0.19	(Ref. 4, Eqn 5.11)
N_{cq} = bearing capacity factor	6.2	(Ref. 4, Figure 5.5)
$N_{\gamma q}$ = bearing capacity factor	250	(Ref. 4, Figure 5.5, using ϕ_i from Step 1)

q_n = 180,663 psf 176692
= 181 ksf

As per AASHTO LRFD (Ref. 1) Article 10.6.2.5.2, if the recommended value of bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the bearing resistance shall be taken as the lesser of those values. The nominal resistance of concrete shall be taken as $0.3f'_c$.

f'_c = 5000 psi = 720 ksf (assumed)
$q_{n,concrete} = 0.3f'_c$ = 216 ksf

$q_{n,concrete}$		$q_{n,calculated}$		$\sigma_{u(r)}$	
216	>	181	<	908	ksf

Thus, use q_n = 181 ksf

3. Calculate the factored bearing resistance

$$q_r = \phi_b q_n$$

where:
ϕ_b = bearing resistance factor 0.45 (Ref. 1, Table 10.5.5.2.2-1, "Footings on rock")

q_r = 81 ksf

Date:	4/1/2024	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Bearing resistance of existing footings - Historical Pier 5	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

B. Determine the bearing resistance at the service limit state.

Use AASHTO LRFD (Ref. 1) Table C10.6.2.5.1-1 to determine the presumptive bearing resistance at the service limit state.

Type of Bearing Material: Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)

Bearing Resistance Recommended Value of Use = 70 ksf

Note: This bearing resistance is settlement limited (1.0 inch as per AASHTO LRFD Section 10.6.2.5.1) and applies only at the service limit state.

Resistance factor for the service limit state: 1.0 (Ref. 1, Section 10.5.5.1)

Factored bearing resistance = 70 ksf

CONCLUSIONS

For the existing spread footings on bedrock at the Pier 5 of the 1983 bridge, the recommended nominal bearing resistance is 181 ksf for strength and 70 ksf for serviceability. A resistance factor of 0.45 is recommended for use at the strength limit state and a resistance factor of 1.0 is recommended for use at the service limit state. This results in a factored bearing resistance of 81 ksf at the strength limit state and 70 ksf at the service limit state.

Attachment 3

TECHNICAL MEMORANDUM

DATE August 13, 2024

Project No. 31404817.004

TO Laura Krusinski, PE
Bridge Division
Maine Department of Transportation

CC

FROM Melissa E. Landon, PhD, PE

EMAIL melissa.landon@wsp.com

RE: GEOTECHNICAL SOIL CORROSIVITY FINDINGS – REV01
HOGAN ROAD BRIDGE REPLACEMENT, BANGOR, MAINE
WIN 018595.10

This updated Technical Memorandum summarizes the results of WSP USA, Inc.'s (WSP) testing and evaluation of the corrosion potential of in situ soils to support the design of proposed Hogan Road eastbound replacement bridge and evaluation of the existing condition of in-place Hogan Road westbound Bridge #5823 pile foundations relevant to this bridge's proposed bridge renovation in support of the geotechnical design for the Hogan Road Diverging Diamond Interchange bridges in Bangor, Maine.

WSP developed draft interpretive stratigraphic profiles (ISPs) along the centerline of the proposed new bridge and existing bridge as shown in Attachment 1 based on WSP's 2022 100-series borings, WSP's 2023 200-series and 300-series borings, and the 1979 historical borings used for design of the 1983 bridge that will be renovated. Boring locations are additionally shown in Attachment 1. We additionally included the depth and location of the 1983 bridge abutment and pier foundations from the historical plans and the depth and location of the proposed new bridge abutments and piers from the PDR plan set.

The draft ISP at the 1983 bridge shows that the bottom of the historical abutments encasing the steel H-piles are located in embankment fills above the water table, while the bottom of the historical Pier 1 is founded on the in situ silt and clay below the water table, likely with a thin layer of fill between the concrete and in situ soil. The piles for the existing abutments extend through the embankment fills into a layer of silty clay both above and below the interpreted water table.

The draft ISP for the proposed new bridge shows that the piles for the proposed abutments will be located in the existing fill and silty clay above and below the water table interpreted from the borings. For the northern piers on either side of I-95 southbound, the pier pile caps and piles will be founded in silty clay below the water table. For the southern piers on either side of I-95 northbound, the pier pile caps and piles will be founded in silty clay above the water table and piles will extend below the water table.

Corrosivity Testing

WSP completed corrosivity testing of 19 soil samples from 10 borings across the site: BB-BHR-101, -102, -103, -202, -203, -204, -205, -301, -302, and -303. This included the following tests recommended by AASHTO[1]:

- Chloride testing for 100-series borings in accordance with ASTM D512-12, Standard Test Methods for Chloride Ion in Water Method B and 200- and 300-series borings in accordance with AASHTO T 291-Chloride Method B.
- Sulfate testing for 100-series boring in accordance with ASTM D516-16, Standard Test Method for Sulfate Ion in Water and 200- and 300-series borings in accordance with AASHTO T290-Sulfates (Soluble).
- pH testing for 100-series borings in accordance with ASTM D4972, Standard Test Method for pH of Soils and 200- and 300-series borings in accordance with AASHTO T 289 for pH of Soils.
- Soil Resistivity testing was performed in accordance with ASTM G57, Standard Test Method for Measurement of Soil Resistivity Using the Wenner Four-Electrode Method for all borings.

Results of these laboratory tests are shown in Attachment 2. Attachment 2 additionally includes our evaluation of the corrosion potential with respect to the soil and groundwater conditions and our estimated steel section loss from corrosive soil conditions for the existing bridge piles with respect to relevant AASHTO[1,2], FHWA[3,4], and NCHRP[5] guidance for pile foundations. Results indicate that of the measured resistivity, pH, sulfate, and chloride values, only resistivity values are below the 2000 ohm-cm threshold in AASTHO[1] below which corrosive conditions may develop. These lower resistivity values were mainly encountered in near surface and shallow depths (less than 5 feet) at locations adjacent to I-95 northbound and southbound that would most be impacted by road salt and road salt runoff into the drainage ditches beside the highway. The highest measured resistivity values are located within the shallow subsurface soils on the Hogan Road western embankment for the north abutment.

As a result of the low resistivity values, AASHTO[2] indicates there is a possibility for severe macrocell corrosion in the existing soils at the water table and a low possibility of uniform corrosion. AASHTO[2] recommends using a corrosion probe to monitor macrocell corrosion. The presence of existing macrocell corrosion can be identified by excavation of the soils around the existing foundations to visually inspect the abutments, piers, and piles.

Assuming the measured soil corrosivity parameters are representative for similar areas (e.g., adjacent to I-95 or within the shallow subsurface), the corrosion potential with low resistivity conditions at or near the water table may create an environment where corrosion of steel piles is possible. We interpret corrosion potential is relevant to the piles at the abutments and Pier 1 near WSP's interpreted water table as road salt at the historical highway and

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[1] American Association of State Highway and Transportation Officials (AASHTO). LRFD Bridge Design Specifications. 9th Edition, dated 2020.

[2] AASHTO. Standard Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications. Specification R27-01 (2023). Technically Revised: 2001, Reviewed but Not Updated: 2023

[3] Federal Highway Administration (FHWA). GEC 012: Design and Construction of Driven Pile Foundations – Volume I. Publication No. FHWA-NHI-16-009. Dated July 2016

[4] FHWA. Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes. Publication No. FHWA-NHI-09-087. Dated November 2009

[5] National Cooperative Highway Research Program (NCHRP). Report 675: LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal-Reinforced Systems. 2011

highway drainage level may have penetrated in situ soils between the time of the construction of I-95 through the construction of the 1983 bridge. We additionally interpret corrosion potential is relevant to the piles for each of the pier piles adjacent to I-95 northbound and southbound, and additionally to piles at the abutments at a similar elevation to the piers.

Evaluation of Existing Pile Sections

In July 2024, WSP and MaineDOT collaboratively performed a visual inspection of, and made dimensional measurements on, several exposed piles at the existing bridge to identify corrosion that may have occurred since the piles were installed. This entailed excavating test pits to examine one pile at Abutment 1 and two piles at Abutment 2 and Pier 5.

- We observed that Pier 5 piles were installed in the native silt and clay soils and showed no visual evidence of rust. Prior to measuring the piles at Pier 5 the piles were scraped clean of silt and clay and then were cleaned with a wire brush.
- We observed that the piles at Abutment 1 and Abutment 2 were installed in embankment fill and showed small areas of rust. Prior to measuring the piles at Abutment 1 and Abutment 2, cobbles mixed with sand and gravel stuck between the flanges of the piles were dislodged using a breaker bar and rock hammer. The piles were then cleaned with a wire brush.

The flange thicknesses were measured using two calipers accurate to 1/1000th of an inch. Flange width and flange depth were measured by WSP using a ruler accurate to 1/32nd of an inch and by MaineDOT using a tape measure accurate to 1/100th of a foot. Two sets of measurements were completed at different depths on the upper exposed pile section for each pile. For each set of measurements WSP made an “original” measurement and “repeat” measurement of each dimension and MaineDOT made a third measurement of each dimension. Table 1 provides the average measurements for each pile examined. Attachment 2 provides a full summary of the measurements made and Attachment 3 provides photos taken of the piles during the July 2024 field inspection.

Table 1: Summary of Pile Measurements

Structure	Pile Location[1]	Flange Thickness, t_{f1} (inches)	Flange Thickness, t_{f2} (inches)	Flange Width, b_f (inches)	Depth, d (inches)	Web Thickness[2], t_w (inches)
Pier 5	Pile 7	0.475	0.490	10.447	9.757	-
	Pile 14	0.456	0.480	10.210	9.823	-
Abutment 1	Pile 1	0.435	0.447	10.202	9.770	0.437
Abutment 2	Pile 16	0.441	0.443	10.120	9.853	0.490
	Pile 18	0.428	0.454	10.120	9.719	-
HP 10x42 Intact Section		0.420	0.420	10.100	9.700	0.415

Notes: 1. Pile location number corresponds to the numbering provided in WSP’s Geotechnical Evaluation of Existing Bridge Piles – REV01 Technical Memorandum dated 8/13/2024. 2. - indicates measurements could not be made due to difficulty of maneuvering around the exposed pile.

The measured dimensions of the five selected piles were not less than the nominal dimensions for HP 10x42 piles. This indicates that the soil environment surrounding the measured piles is such that corrosion does not occur or results in a slower rate of corrosion. This is consistent with the laboratory data discussed above, where results indicate that of the measured resistivity, pH, sulfate, and chloride values, only resistivity values are below the 2000 ohm-cm threshold in AASTHO[1] below which corrosive conditions may develop.

Potential Corrosion Recommendations

Because of the presence of some rust on the three exposed abutment piles and measured resistivity that could lead to corrosion, WSP recommends corrosion section loss be part of the evaluation of the pile structural resistance for the bridge renovation. In an email[6] dated July 26, 2024, WSP was directed by MaineDOT's Laura Krusinski and Tim Aguilar to consider section loss for the existing bridge for the future 50-year design life for the lesser of either 1/16th of an inch or WSP's calculated section loss over 50 years.

WSP estimated the loss of steel section based on the FHWA[3] and NCHRP[5] linear methods and the FHWA[4] non-linear method to account for potential section loss during an additional 50 year design life for the bridge. The loss of section for these three methods ranged between 0.05 inches and 0.15 inches per steel face over the next 50 years, assuming no section loss from 1983 to present. WSP's recommendation is the use of the non-linear section loss method value of 0.05 inches of steel loss from each face of the pile over 50 years as described in Attachment 4, which may be used to decrease the pile section to estimate the capacity of the existing HP 10x42 piles. For the proposed bridge pile foundations, we recommend accounting for non-linear section loss for the pile design.

We look forward to your review of our evaluation and future discussion about the potential for corrosion of the existing bridge piles.

WSP USA, Inc.

Melissa E. Landon, PhD, PE
Lead Consultant, Geotechnical Engineering

Christopher C. Benda, PE
Vice President, Geotechnical Engineering

MEL/CCB

Distribution: Jeffrey D. Lloyd, PE, WSP

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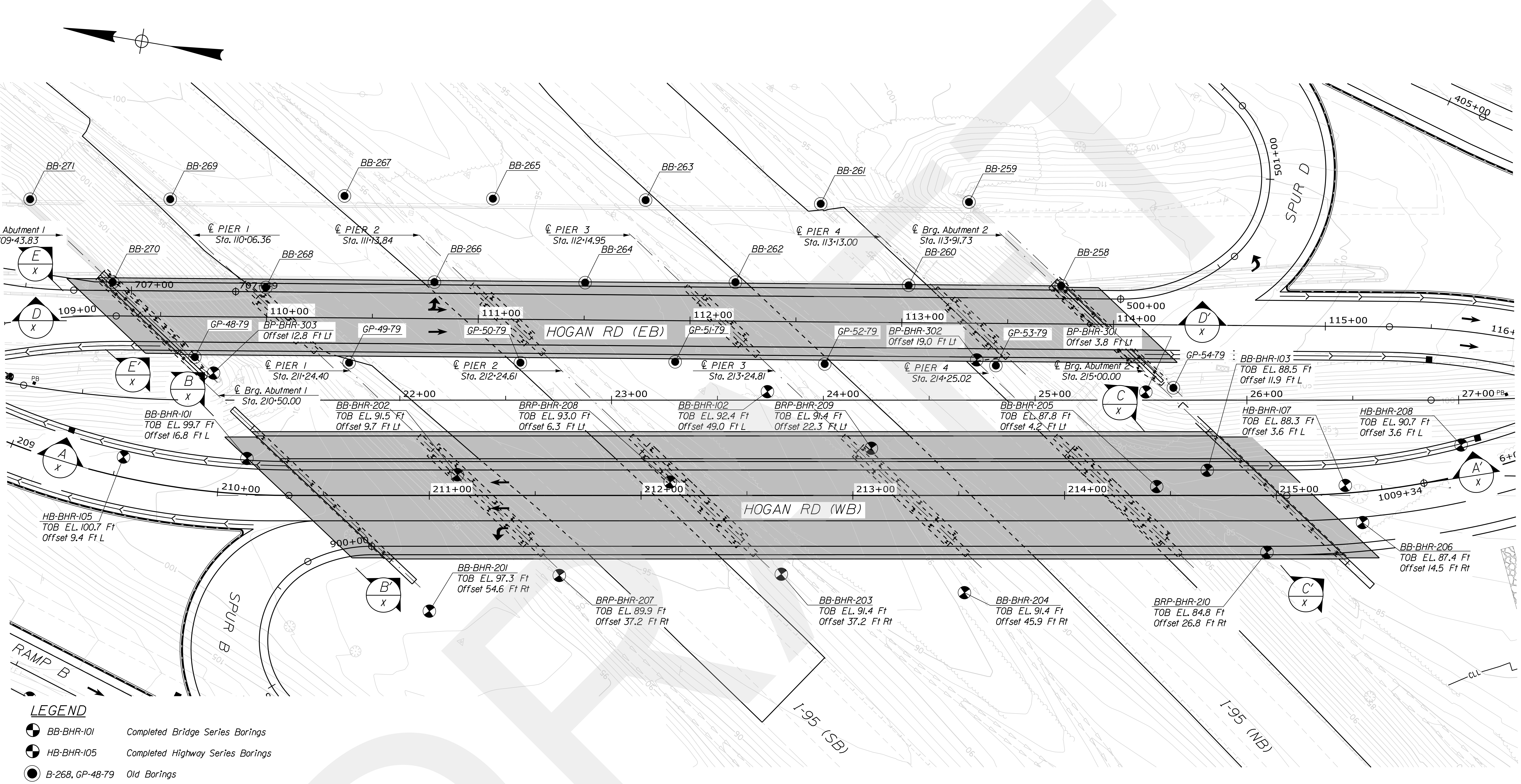
[6] Krusinski, Laura "RE: Bangor Hogan Road - 1983 Bridge Updated Geotechnical Pile Resistance – Corrosion" Received by Melissa Landon, July 26, 2024

Attachment: 1: Draft Interpretive Stratigraphic Profiles and Boring Location Plan
2: Pile Measurements Summary
3: Pile Photos from Field Investigation
4: Soil Corrosivity Evaluation for Bridge Foundations

https://wsponlinenam.sharepoint.com/sites/gld-157336/project files/6 deliverables/memo-corrosion/win 018595.10 hogan road bridge - soil corrosivity findings revised 20240813.docx

Filename: ...\MSTA\BorLocPlan_5Span_wb.dgn   Division: BRIDGE   Username: Date:2/5/2024



LEGEND

◐	BB-BHR-101	Completed Bridge Series Borings
◐	HB-BHR-105	Completed Highway Series Borings
◉	B-268, GP-48-79	Old Borings
	TOB EL. 99.7	Elevation of Top of Boring

NOTES:

1. As Drilled Boring Locations for the Shown Borings Derived from an Electronic File "Borings.dgn" Provided to WSP GOLDER by Maine DOT on May 24, 2022.
2. Basemap Elements Shown Derived from a group of Electronic Files "WIN 18595.10-Bangor" Provided to WSP GOLDER by VHB on May 26, 2022 and from Maine DOT Drawing Titled "Bridge_12-3-2022.dwg" Received on May 5, 2022.
3. Proposed Stationing Provided to WSP GOLDER by VHB on May 26, 2022 in an Electronic FIle "Alignments.dgn".
4. For Detailed Lithologic Descriptions see Boring Logs in Appendix A Bridge Borings (BB-Series) and Highway Borings (HB-Series).
5. For Complete Laboratory Data see Laboratory Reports Appendix C (BB-Series).
6. Groundwater Surface is Interpreted From Localized Surface Water Levels and Measurements Taken During the Subsurface Exploration Programs. For Details of the Subsurface Exploration Programs see Boring Logs in Appendix A Bridge Borings (BB-Series) and Highway Borings (HB-Series).
7. This Generalized Subsurface Profile is Intended to Convey Trends in Subsurface Conditions. The Boundaries Between Strata are Approximate and Idealized and Have Been Developed Based on Interpretations of Widely Spaced Explorations. Actual Soil and Rock Transitions may Vary and are Probably More Erratic. For More Specific Information, Refer to Boring Logs in Appendix A (BB-Series) and (HB-Series).
8. Proposed Abutment Details Interpreted From Electronic File Name "Bridge_12-3-2020" Provided to WSP GOLDER by MAINE DOT on May 5, 2022.

BORING LOCATION PLAN 5 SPAN ALTERNATE

25 0 25 50

1" = 25 Feet

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

BRIDGE NO. 5823
WIN
18595.10
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M. LANDON		
DESIGN-DETAILED	MEL	WEG	XXX
CHECKED-REVIEWED	CCB	XXX	XXX
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1	-		-
REVISIONS 2	-		-
REVISIONS 3	-		-
REVISIONS 4	-		-
FIELD CHANGES	-		-

SIGNATURE

P.E. NUMBER

DATE

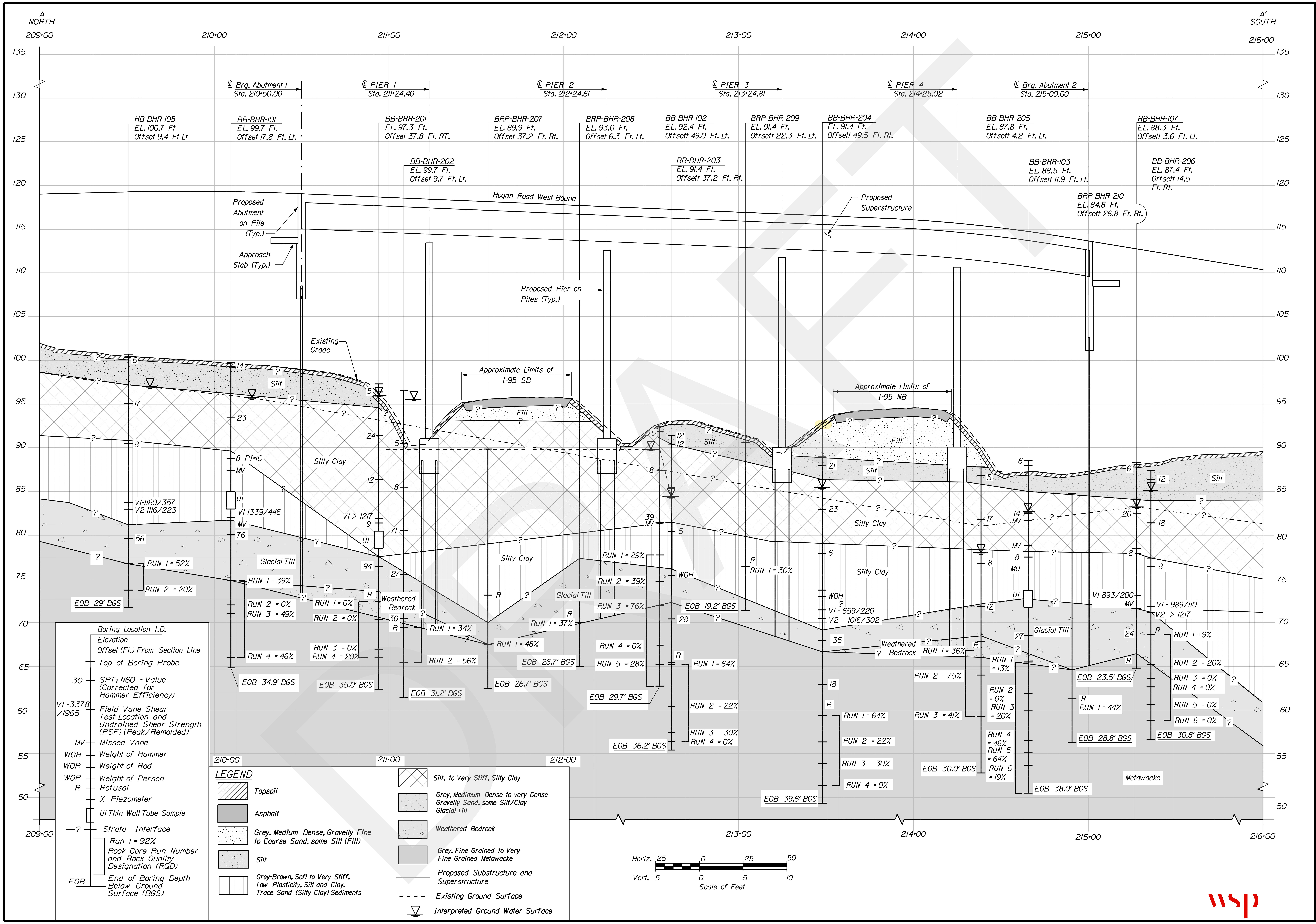
I-95 HOGAN ROAD BRIDGE
REPLACEMENT #5823 (EXIT 187)
BANGOR MAINE

BORING LOCATION PLAN

SHEET NUMBER

X

OF X



Boring Location I.D.
Elevation
Offset (Ft.) From Section Line
Top of Boring Probe
30 — SPT: N60 - Value (Corrected for Hammer Efficiency)
VI-3378/1965 — Field Vane Shear Test Location and Undrained Shear Strength (PSF)(Peak/Remolded)
MV — Missed Vane
WOH — Weight of Hammer
WOR — Weight of Rod
WOP — Weight of Person
R — Refusal
X Piezometer
UI Thin Wall Tube Sample
—? Strata Interface
Run 1 = 92% Rock Core Run Number and Rock Quality Designation (RQD)
EOB — End of Boring Depth Below Ground Surface (BGS)

LEGEND

Topsoil
Asphalt
Grey, Medium Dense, Gravelly Fine to Coarse Sand, some Silt (Fill)
Silt
Grey-Brown, Soft to Very Stiff, Low Plasticity, Silt and Clay, Trace Sand (Silty Clay) Sediments
Silt, to Very Stiff, Silty Clay
Grey, Medium Dense to very Dense Gravelly Sand, some Silt/Clay Glacial Till
Weathered Bedrock
Grey, Fine Grained to Very Fine Grained Metawacke
Proposed Substructure and Superstructure
Existing Ground Surface
Interpreted Ground Water Surface

wsp

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

BRIDGE No. 5823

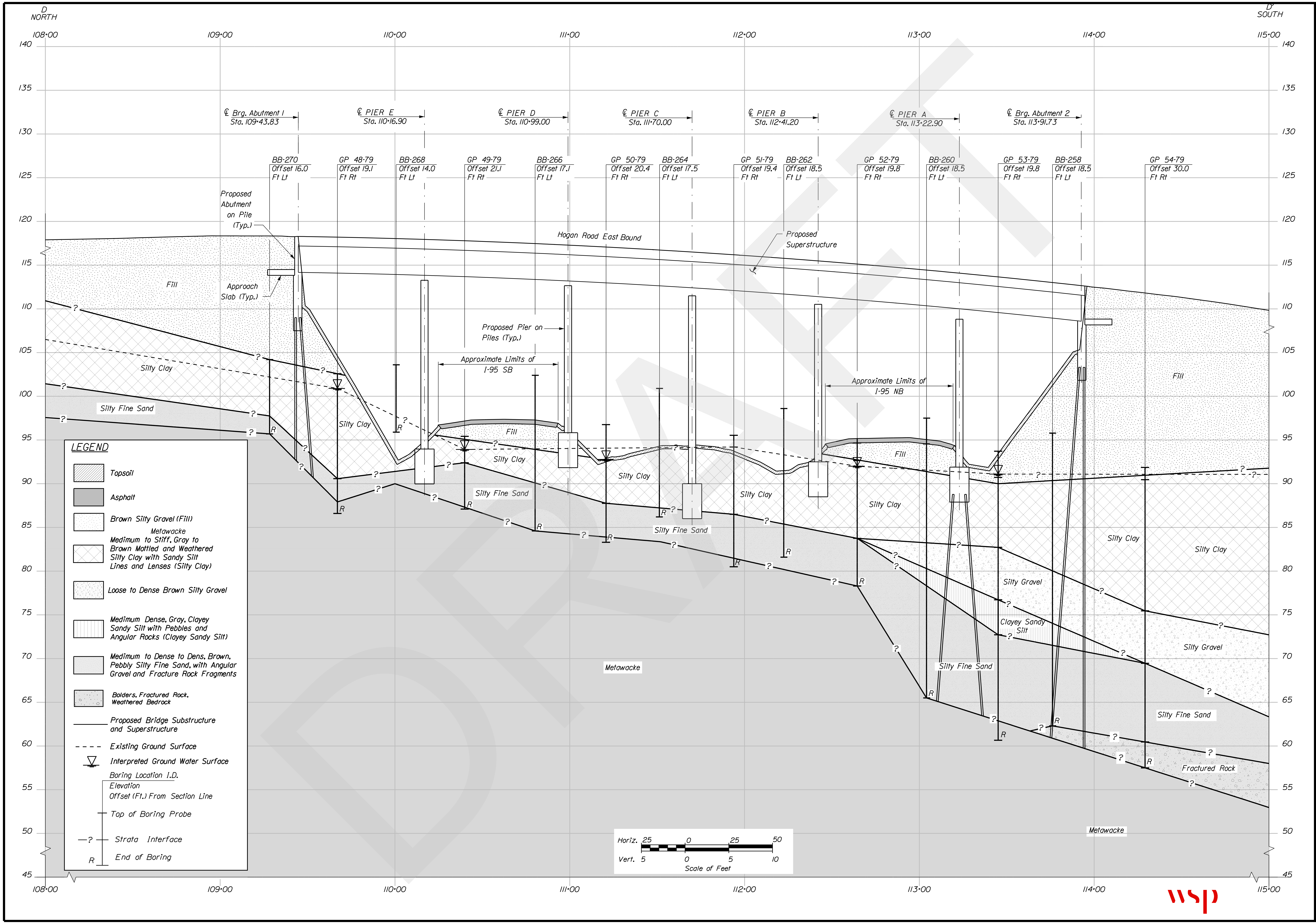
PROJ. MANAGER M. LANDON
DESIGN-DETAILED WEG NVW XXX
CHECKED-REVIEWED CCB XXX XXX
DESIGN2-DETAILED2
DESIGN3-DETAILED3
REVISIONS 1 XXX XXX
REVISIONS 2 XXX XXX
REVISIONS 3
REVISIONS 4
FIELD CHANGES
BY
DATE
SIGNATURE
P.E. NUMBER
DATE

I-95 HOGAN ROAD BRIDGE
REPLACEMENT #5823 (EXIT 187)
BANGOR MAINE

INTERPRETIVE SUBSURFACE PROFILE A-A'

SHEET NUMBER
X
OF X

Filename: ...\MSTA\GeoProfile_5Span_eb.dgn Division: BRIDGE Username: Date:2/6/2024



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

BRIDGE No. 5823

	BY	DATE
PROJ. MANAGER	M. LANDON	
DESIGN-DETAILED	WEG	XXX
CHECKED-REVIEWED	CCB	XXX
DESIGN2-DETAILED2	NVW	
DESIGN3-DETAILED3	XXX	
REVISIONS 1	XXX	XXX
REVISIONS 2	XXX	XXX
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

I-95 HOGAN ROAD BRIDGE
REPLACEMENT #5823 (EXIT 187)
BANGOR MAINE

INTERPRETIVE SUBSURFACE PROFILE D-D'

SHEET NUMBER

X

OF X

CALCULATIONS

Date: 7/29/2024
Project No.: 31404817.004
Subject: Field Inspection of Pier 5 Piles - Measurement Summary
Project Title: MaineDOT Hogan Road Bridge Replacement
Made by: ATM
Checked by: RJN
Reviewed by: CCB

Objective

Summarize the field measurements of the existing Pier 5 piles made by WSP and MaineDOT on Monday July 22, 2024.

Method

The flange thicknesses were measured by WSP and MaineDOT with separate digital calipers accurate to 1/1000th of an inch. Flange width and depth were measured by WSP using a ruler accurate to 1/32nd of an inch and by MaineDOT using a tape measure accurate to 1/100th of a foot. Prior to measuring the piles, any silt and clay stuck to the piles was scarped off using a paint scraper and then the piles were cleaned with a wire brush. Figure 1 shown below, depicts the location of the piles measured at Pier 5. Figure 2 shows a typical H-Pile section with the labels for the dimensions that were measured in the field.

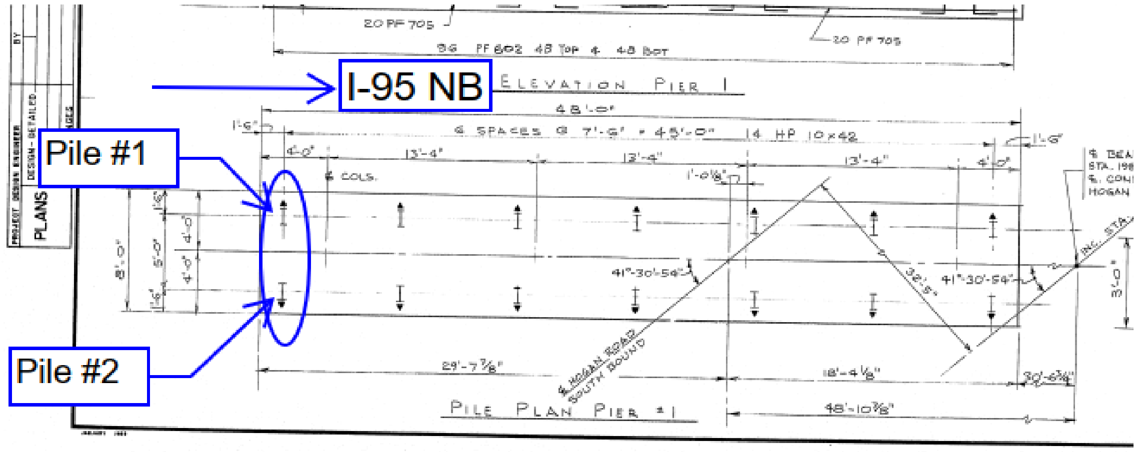


Figure 1

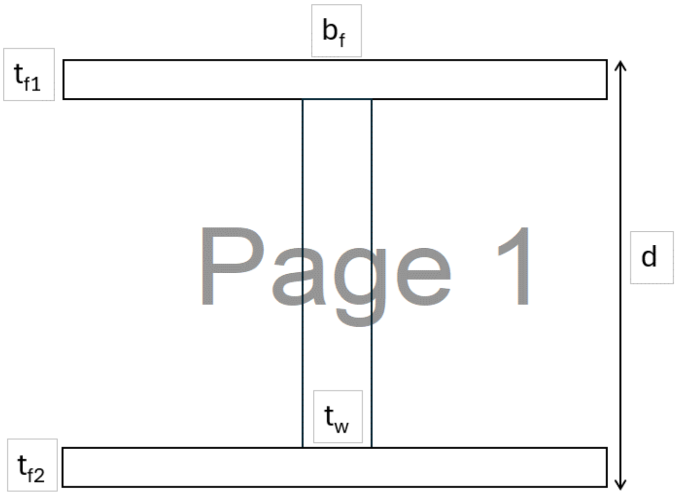


Figure 2

Observations

No rust was observed on either pile measured at Pier 5. The pile cap was installed on a silty clay layer. Groundwater was observed seeping in to the excavation approximately 1-foot below the pile cap.

Data Summary

Structure	Pile Location	Depth below pile cap	Measurement	Flange Thickness, t_{f1}	Flange Thickness, t_{f2}	Flange Width, b_f	Depth, d	Web Thickness, t_w
Pier 5	Pile #1	6	WSP - Original	0.498	0.518	10.270	9.799	-
			WSP - Repeat	0.480	0.508	10.415	9.625	-
			MaineDOT	0.485	0.473	10.740	9.840	-
		11	WSP - Original	0.485	0.478	10.404	9.813	-
			WSP - Repeat	0.451	0.484	10.594	9.688	-
			MaineDOT	0.448	0.478	10.260	9.780	-
	Pile #2	7	WSP - Original	0.462	0.475	10.125	9.875	-
			WSP - Repeat	0.456	0.477	10.063	9.875	-
			MaineDOT	0.445	0.454	10.320	9.780	-
		12	WSP - Original	0.456	0.491	10.125	9.813	-
			WSP - Repeat	0.446	0.495	10.188	9.813	-
			MaineDOT	0.473	0.488	10.440	9.780	-

Note: All measurements shown are in inches

Structure	Pile Location	Depth below pile cap	Average Measurement	Flange Thickness, t_{f1}	Flange Thickness, t_{f2}	Flange Width, b_f	Depth, d	Web Thickness, t_w
Pier 5	Pile #1	6	WSP	0.489	0.513	10.343	9.712	-
			WSP and MaineDOT	0.488	0.500	10.475	9.755	-
		11	WSP	0.468	0.481	10.499	9.750	-
			WSP and MaineDOT	0.461	0.480	10.419	9.760	-
		Combined	Avg. for Pile	0.475	0.490	10.447	9.757	
	Pile #2	7	WSP	0.459	0.476	10.094	9.875	-
			WSP and MaineDOT	0.454	0.469	10.169	9.843	-
		12	WSP	0.451	0.493	10.156	9.813	-
			WSP and MaineDOT	0.458	0.491	10.251	9.802	-
		Combined	Avg. for Pile	0.456	0.480	10.210	9.823	-
	Avg. for Structure			**0.465**	**0.485**	**10.329**	**9.790**	

Note: All measurements shown are in inches

Date: 7/30/2024
Project No.: 31404817.004
Subject: Field Inspection of Abutment 1 Piles - Measurement Summary
Project Title: MaineDOT Hogan Road Bridge Replacement

Made by: RJN
Checked by: ATM
Reviewed by: CCB

Objective

Summarize the field measurements of the existing Abutment 1 piles made by WSP and MaineDOT on Tuesday July 23, 2024.

Method

The flange thicknesses were measured by WSP and MaineDOT with separate digital calipers accurate to 1/1000th of an inch. Flange width and depth were measured by WSP using a ruler accurate to 1/32nd of an inch and by MaineDOT using a tape measure accurate to 1/100th of a foot. Prior to measuring the piles, any sands/gravels and cobbles stuck between the flanges of the pile were dislodged using a rock hammer and then the piles were cleaned with a wire brush. Figure 1 shown below, depicts the location of the pile measured at Abutment 1. Figure 2 shows a typical H-Pile section with the labels for the dimensions that were measured in the

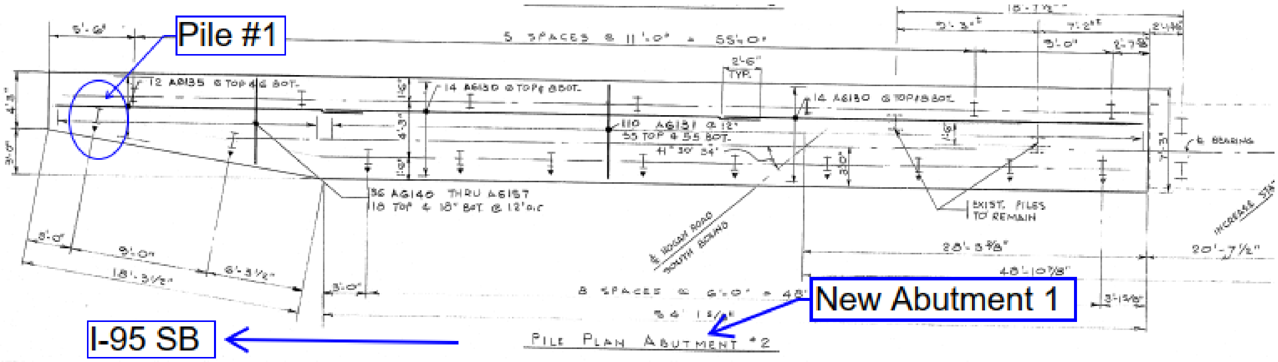


Figure 1

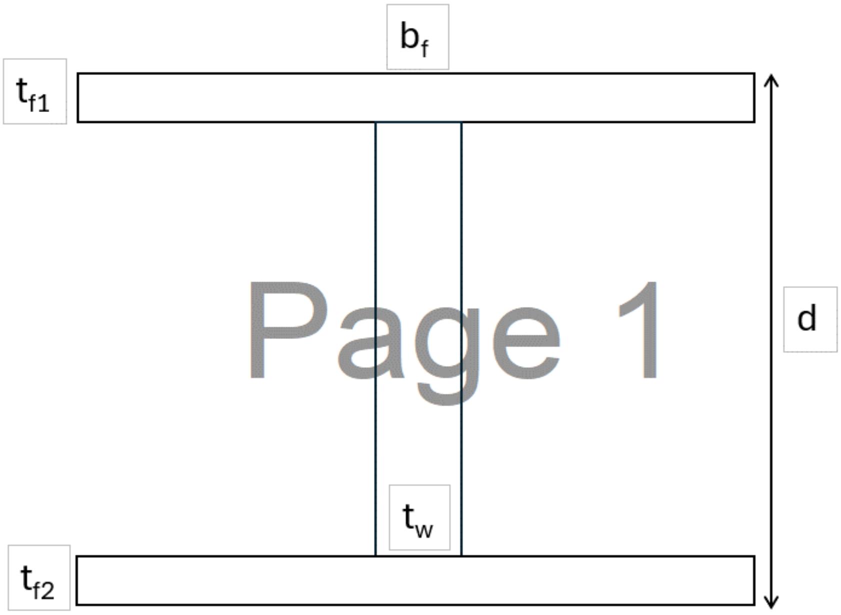


Figure 2

Observations

Small spots of rust were observed on the pile, otherwise the pile was in good condition. Pile cap installed in the fill layer. No Groundwater observed during excavation of the piles.

Data Summary

Structure	Pile Location	Depth below pile cap	Measurement	Flange Thickness, t_{f1}	Flange Thickness, t_{f2}	Flange Width, b_f	Depth, d	Web Thickness, t_w
Abutment 1	Pile #1	10	WSP - Original	0.429	0.430	10.188	9.750	0.421
			WSP - Repeat	0.433	0.437	10.188	9.750	-
			MaineDOT	0.475	0.475	10.200	9.840	-
		18	WSP - Original	0.427	0.433	10.188	9.750	0.452
			WSP - Repeat	0.424	0.431	10.188	9.750	-
			MaineDOT	0.420	0.475	10.260	9.780	-

Note: All measurements shown are in inches

Structure	Pile Location	Depth below pile cap	Average Measurement	Flange Thickness, t_{f1}	Flange Thickness, t_{f2}	Flange Width, b_f	Depth, d	Web Thickness, t_w
Abutment 1	Pile #1	10	WSP	0.431	0.434	10.188	9.750	0.421
			WSP and MaineDOT	0.446	0.447	10.192	9.780	-
		18	WSP	0.425	0.432	10.188	9.750	0.452
			WSP and MaineDOT	0.424	0.446	10.212	9.760	-
	Avg. for Structure			**0.435**	**0.447**	**10.202**	**9.770**	**0.437**

Note: All measurements shown are in inches

CALCULATIONS

Date: 7/30/2024
Project No.: 31404817.004
Subject: Field Inspection of Abutment 2 Piles - Measurement Summary
Project Title: MaineDOT Hogan Road Bridge Replacement

Made by: RJN
Checked by: ATM
Reviewed by: CCB

Objective

Summarize the field measurements of the existing Abutment 2 piles made by WSP and MaineDOT on Monday July 22, 2024.

Method

The flange thicknesses were measured by WSP and MaineDOT with separate digital calipers accurate to 1/1000th of an inch. Flange width and depth were measured by WSP using a ruler accurate to 1/32nd of an inch and by MaineDOT using a tape measure accurate to 1/100th of a foot. Prior to measuring the piles, any sands/gravels and cobbles stuck between the flanges of the pile were dislodged using a rock hammer and then the piles were cleaned with a wire brush. Figure 1 shown below, depicts the location of the piles measured at Abutment 2. Figure 2 shows a typical H-Pile section with the labels for the dimensions that were measured in the field.

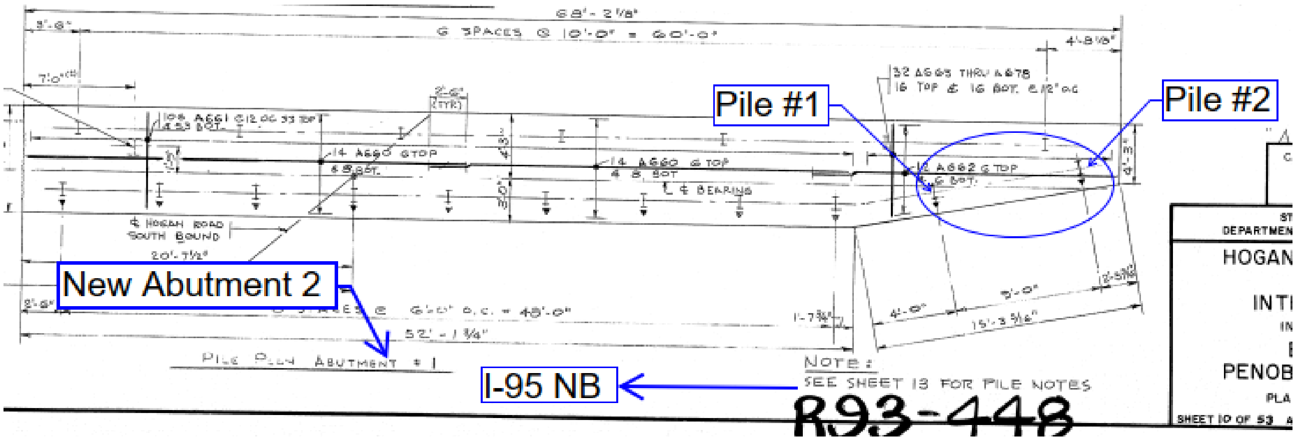


Figure 1

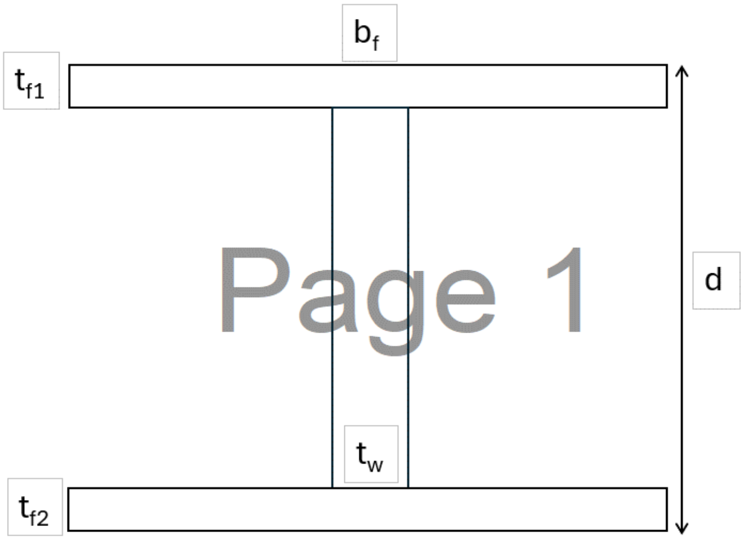


Figure 2

Observations

Small spots of rust were observed on the pile, otherwise the pile was in good condition. Pile cap installed in the fill layer. No Groundwater observed during excavation of the piles.

Data Summary

Structure	Pile Location	Depth below pile cap	Measurement	Flange Thickness, t_{f1}	Flange Thickness, t_{f2}	Flange Width, b_f	Depth, d	Web Thickness, t_w
Abutment 2	Pile #1	6	WSP - Original	0.457	0.450	10.125	9.813	0.49
			WSP - Repeat	0.468	0.436	10.063	9.875	-
			MaineDOT	0.462	0.455	10.140	9.840	-
		12	WSP - Original	0.421	0.446	10.125	9.875	-
			WSP - Repeat	0.418	0.441	10.125	9.875	-
			MaineDOT	0.419	0.428	10.140	9.840	-
	Pile #2	5	WSP - Original	0.443	0.450	10.063	9.688	-
			WSP - Repeat	0.415	0.473	10.125	9.625	-
			MaineDOT	0.436	0.456	10.140	9.720	-
		10	WSP - Original	0.420	0.455	10.125	9.750	-
			WSP - Repeat	0.423	0.453	10.125	9.813	-
			MaineDOT	0.430	0.436	10.140	9.720	-

Note: All measurements shown are in inches

Structure	Pile Location	Depth below pile cap	Average Measurement	Flange Thickness, t_{f1}	Flange Thickness, t_{f2}	Flange Width, b_f	Depth, d	Web Thickness, t_w
Abutment 2	Pile #1	6	WSP	0.463	0.443	10.094	9.844	0.49
			WSP and MaineDOT	0.462	0.447	10.109	9.843	-
		12	WSP	0.420	0.444	10.125	9.875	-
			WSP and MaineDOT	0.419	0.438	10.130	9.863	-
		Combined	Avg. for Pile	0.441	0.443	10.120	9.853	0.49
	Pile #2	5	WSP	0.429	0.462	10.094	9.656	-
			WSP and MaineDOT	0.431	0.460	10.109	9.678	-
		10	WSP	0.422	0.454	10.125	9.781	-
			WSP and MaineDOT	0.424	0.448	10.130	9.761	-
		Combined	Avg. for Pile	0.428	0.454	10.120	9.719	-
	Avg. for Structure			**0.434**	**0.448**	**10.120**	**9.786**	0.49

Note: All measurements shown are in inches

Abutment 1 Pile #1

Abutment 2 Pile #1

Abutment 2 Pile #2

Pier 5 Pile #1

Pier 5 Pile #2

CALCULATIONS

Date:	8/1/2024	**Made by:**	KAR
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Soil Corrosivity Evaluation for Bridge Foundations	**Reviewed by:**	CCB
Project Title:	MaineDOT Hogan Rd Bridge Phase II		

OBJECTIVE

Estimate the corrosion that the existing Hogan Road Bridge HP10x42 piles may experience over the desired remaining service life of 50 years.

METHOD

Use references from AASHTO, FHWA, and NCHRP to determine potential corrosion rates based on the results of pH, electrical resistivity, chloride, and sulfate laboratory testing on soil samples obtained during the subsurface investigation at site.

REFERENCES

1. GeoTesting Express. Laboratory test results for bridge and highway 100- and 200-series borings and 300-series hand probes. Received by WSP on May 26, 2022, May 31, 2022, January 3, 2024, and March 5, 2024. Summarized in Table 4 and included in Appendix C of the WSP Geotechnical Design Report.

2. American Association of State Highway and Transportation Officials (AASHTO). LRFD Bridge Design Specifications. 9th Edition, dated 2020.

3. AASHTO. Standard Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications. Specification R 27-01 (2023). Technically Revised: 2001, Reviewed but Not Updated: 2023

4. Federal Highway Administration (FHWA). GEC 012: Design and Construction of Driven Pile Foundations – Volume I. Publication No. FHWA-NHI-16-009. Dated July 2016.

5. National Cooperative Highway Research Program (NCHRP). Report 675: LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal-Reinforced Systems. 2011.

6. FHWA. Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes. Publication No. FHWA-NHI-09-087. Dated November 2009.

7. Carroll E. Taylor & Associates for MaineDOT. As-Built Plans: Hogan Road Bridge over Interstate 95 in the city of Bangor, Penobscot County. Dated 1983.

8. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx

ASSUMPTIONS

1. Based on field measurements and observations made in the field by WSP and MaineDOT field engineers on July 22nd and July 23rd, 2024, assume no section loss due to corrosion has occurred from pile installation to present.

ATTACHMENTS

1. Summary table of corrosivity laboratory test results for bridge and highway 100- and 200-series borings and 300-series hand probes

2. Draft boring location plan for bridge and highway 100- and 200-series borings and 300-series hand probes in the vicinity of the existing and proposed bridges

Date:	8/1/2024	**Made by:**	KAR
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Soil Corrosivity Evaluation for Bridge Foundations	**Reviewed by:**	CCB
Project Title:	MaineDOT Hogan Rd Bridge Phase II		

CALCULATION

A. Use the corrosivity laboratory test results for the soil at site (Ref. 1, also included as Attachment 1) to determine the corrosion potential at the existing piles.

Per AASHTO LRFD Article 10.7.5 (Ref. 2), the following soil or water conditions should be considered as indicative of a potential pile deterioration or corrosion situation:

- Resistivity less than 2,000 ohm-cm
- pH less than 5.5
- pH between 5.5 and 8.5 in soils with high organic content
- Sulfate concentration greater than 1,000 ppm (soil) or 500 ppm (water)
- Chloride content greater than 500 ppm

The corrosivity laboratory test results for the soil at site (Ref. 1) are compared to these conditions:

Condition	Range from lab test results	Median from lab test results	Meets condition?
Resistivity less than 2,000 ohm-cm	1,038 ohm-cm to 7,666 ohm-cm	2,212 ohm-cm	Yes*
pH less than 5.5	6.4 to 8.2	7.5	No
pH between 5.5 and 8.5 in soils with high organic content	0.5% to 0.7% organic content	0.6% organic content	No
Sulfate concentration greater than 1,000 ppm (soil) or 500 ppm (water)	<10 ppm to 30 ppm	10 ppm	No
Chloride content greater than 500 ppm	<10 ppm to 120 ppm	48 ppm	No

*While the median value is above the 2,000 ohm-cm threshold, a sufficient number of test results are below 2,000 ohm-cm to warrant further consideration.

Since the lab test results only indicate a marginal potential for pile deterioration due to corrosion, the AASHTO Standard Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications (Ref. 3) is used to further assess corrosion potential.

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B. Use the flowcharts from the AASHTO Standard Practice (Ref. 3) to determine the potential corrosion situation at the existing piles.

Step 1: use AASHTO flowchart Figure 3 (Ref. 3) and the corrosivity laboratory test results for the soil at site (Ref. 1) to determine the possibility of macrocell and uniform corrosion of the existing piles.

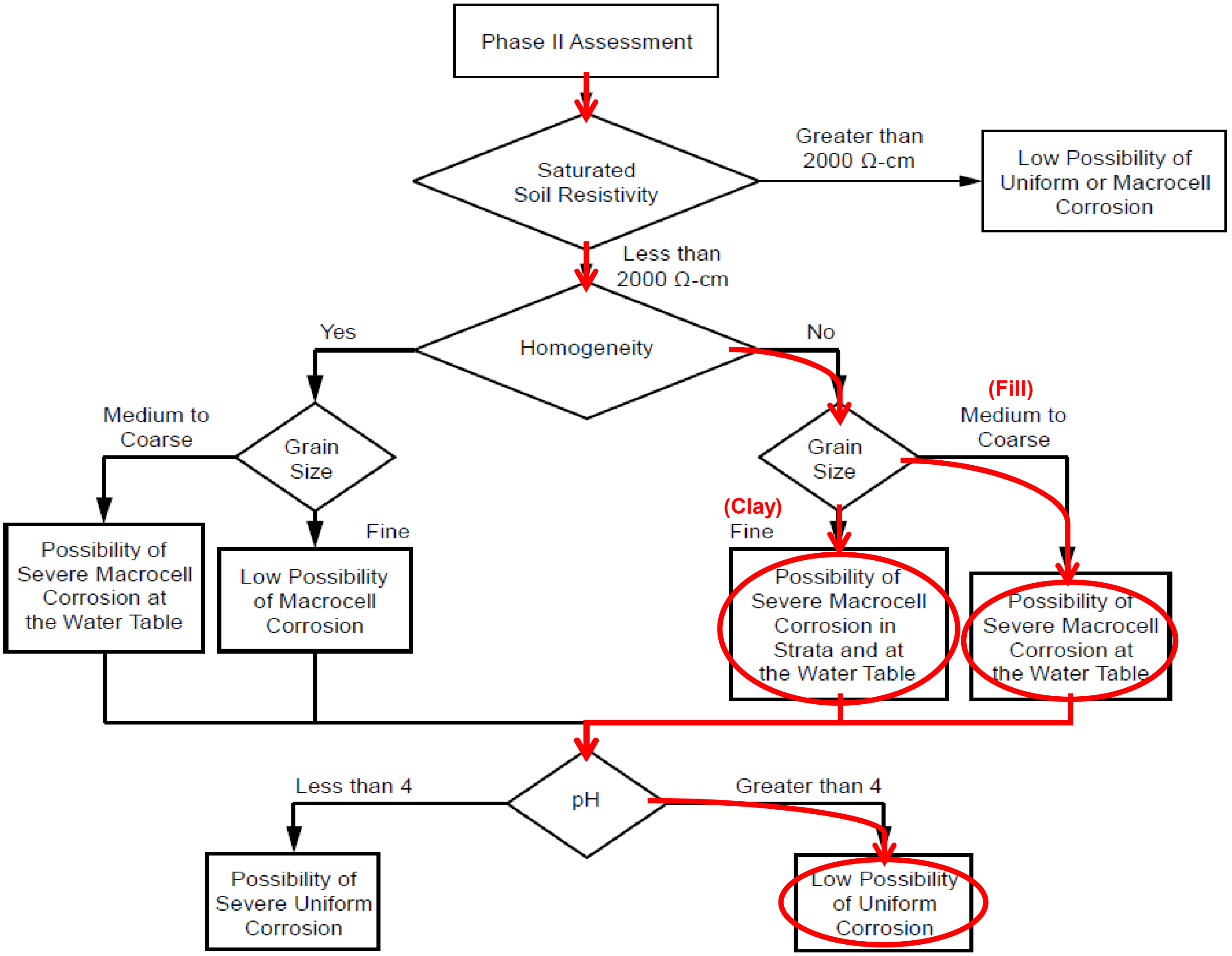

Figure 3—Determination of the Possibility for General and/or Macrocell Corrosion Based on Soil Analysis

Step 2: use AASHTO flowchart Figure 4 (Ref. 3) to determine the recommended corrosion assessment procedure for the existing piles.

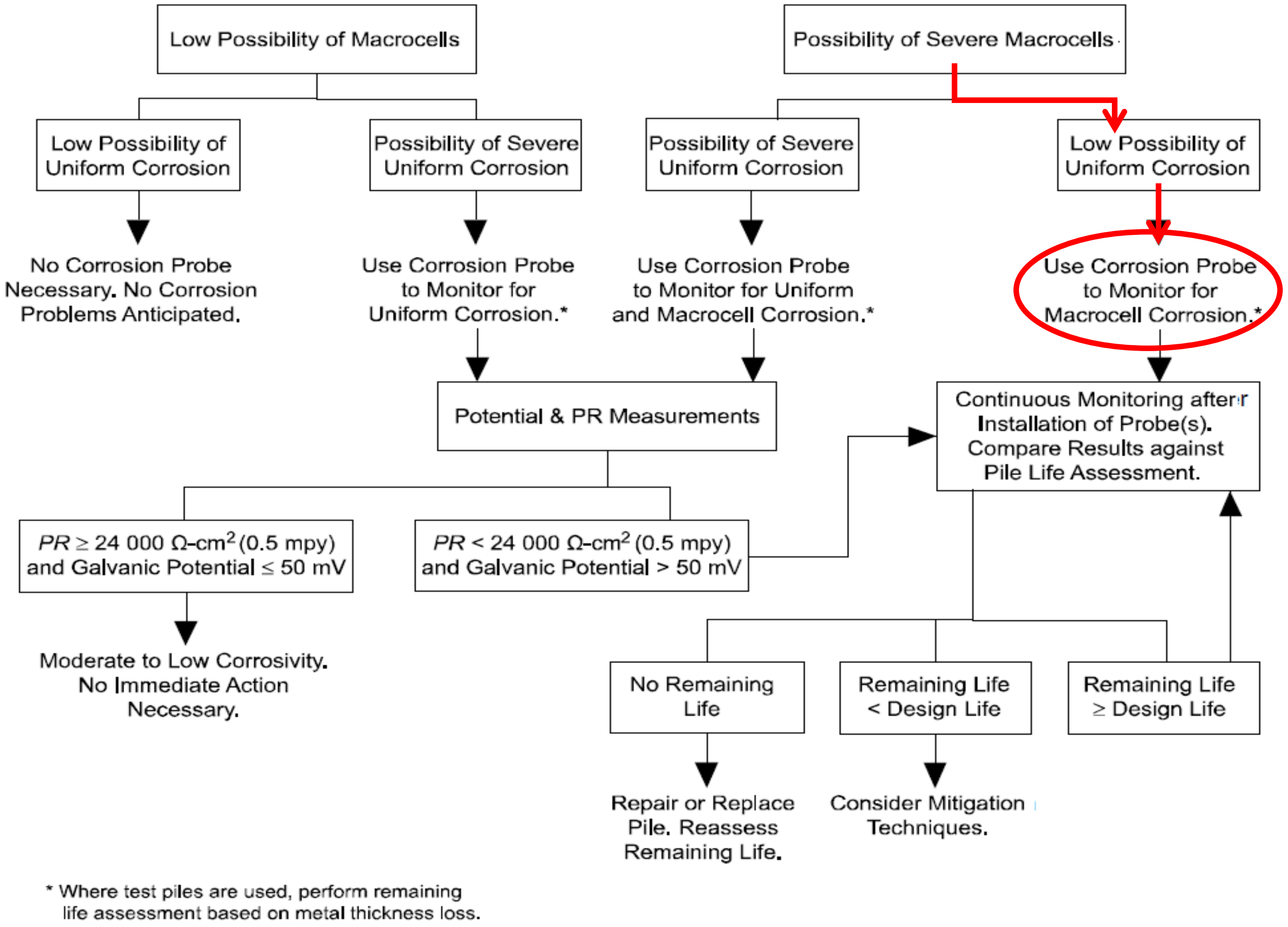

Figure 4—Determination of the Necessity for Electrochemical Testing, Corrosion Monitoring, and Mitigation

Based on the corrosivity lab test results, the AASHTO Standard Practice recommends using a corrosion probe to monitor for macrocell corrosion.

Since a corrosion probe is not currently installed at site, several approximate corrosion rates from FHWA and NCHRP will be used to estimate corrosion loss of the existing piles.

CALCULATIONS

Date: 8/1/2024
Project No.: 31404817.004
Subject: Soil Corrosivity Evaluation for Bridge Foundations
Project Title: MaineDOT Hogan Rd Bridge Phase II

Made by: KAR
Checked by: DEB
Reviewed by: CCB

C. Estimate corrosion loss of the pile steel and calculate the resulting reduced pile section area.

Linear methods:

Per FHWA GEC-12 Driven Pile Foundations (Ref. 4), Section 6.12.1.1, for steel piles buried in fill or disturbed natural soils, a conservative estimate of the corrosion rate is 0.003 inches per year.

Corrosion loss per year = 0.003 inches

Per NCHRP Report 675 LRFD Metal Loss (Ref 5), Figure 8, for plain steel elements and soil resistivity less than 3,000 ohm-cm, an average corrosion rate is 13.5 μm/yr = 0.0005 in/yr.

Corrosion loss per year = 0.0005 inches
Corrosion loss per year with FS of 2 = 0.0011 inches

Estimate the total corrosion loss that the existing piles may experience over the desired remaining service life of 50 years:

Desired remaining service life = 50 years (2024 to 2074)

	NCHRP	FHWA		
Linear rate of corrosion loss =	0.0011	0.003	inches per year	per steel face or side
Future corrosion loss =	0.05	0.15	inches over 50 years	

Non-linear methods:

Per FHWA Corrosion of Soil Reinforcements (Ref. 6), Section 2.4.a, a general conclusion of studies on underground corrosion is that the rate of corrosion is greatest in the first few years of burial and then levels off to a steady but significantly lower rate. Thus, both Ref. 5 and Ref. 6 suggest a non-linear model of steel corrosion loss:

$$x = kt^{n}$$ (Ref. 5, Equation 1)

in which *x* is the loss of steel thickness per side, *k* is a constant dependent on soil conditions (i.e., soil corrosivity), *t* is time in years, and *n* is a constant dependent on steel conditions (i.e., galvanized or plain).

k = 50 μm (Ref. 5, Table 5, for pH > 5 and resistivity between 700 and 2,000 ohm-cm)
n = 0.8 (Ref. 5 Eq. 3 for plain steel elements; Ref. 6 pg. 2-25 for non-galvanized steel)
t = 50 years
x = 1143 μm over the next 50 years
x = 0.05 inches over the next 50 years
(future)

Pile section loss:

Using the non-linear total corrosion loss per side = 0.05 inches over the next 50 years

Pile size: HP 10x42 (Ref. 7, Sheet 10)

	Intact Section		
Depth, d =	9.70	in	(Ref. 8)
Width, b_f =	10.1	in	(Ref. 8)
Web thickness, t_w =	0.415	in	(Ref. 8)
Flange thickness, t_f =	0.420	in	(Ref. 8)
Fillet area =	0.2	in^2	
Section area, A =	12.4	in^2	(Ref. 8)

	Corroded Section	
Depth, d =	9.60	in
Width, b_f =	10.0	in
Web thickness, t_w =	0.315	in
Flange thickness, t_f =	0.320	in
Fillet area =	0.2	in^2
Section area, A =	9.4	in^2

CONCLUSIONS

Based on the corrosivity lab test results, the AASHTO Standard Practice recommends using a corrosion probe to monitor for macrocell corrosion. Since a corrosion probe is not currently installed at site, several approximate corrosion rates from FHWA and NCHRP were used to estimate corrosion loss of the existing piles. Using a non-linear corrosion loss of 0.05 inches per side over the next 50 years (desired future life), the estimated corroded section area of the HP 10x42 piles would be 9.4 in^2, representing 76% of the intact section area. Per the AASHTO Standard Practice, the anticipated corrosion pattern is macrocell rather than uniform, so this estimated corroded section area may vary throughout the pile length.

Attachment 4

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Abutment 1 (old Abutment 2) - Strength, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

OBJECTIVE

Determine if Abutment 1 located at Sta. 109+43 (old Abutment 2), corroded HP 10x42 steel piles provide adequate resistance based on geometry from the as-built 1983 record plans and the strength loading provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3), and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Abutment 1 (existing Abutment 2) (Pile 16), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. PDR Draft 2-3-2023 Hogan Road DDI (Bridge Portion Only) 018595.10
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. Loading provided by VHB in Computation package titled 1983 Bridge Foundation Loads, dated 4/5/2024
11. WSP Existing Abutment 1 FB-Multipier Soil Properties
12. WSP calculation package titled Equivalent rectangular Abutment 1 cross section, dated 5/1/2024
13. WSP Boring logs, included as an appendix with the Geotechnical Report.
14. WSP compressive strength and elastic moduli laboratory testing results, dated November 16, 2023, included as an appendix with the Geotechnical Report.
15. WSP calculation package titled Pile Drivability at Abutment 2 (existing Abutment 1) - HP 10X42
16. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 8/1/2024.
17. Email from VHB on 10/10/24, requesting a resultant load move 0.71 feet in the positive x-direction.

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength output
4. FB-Multipier Service output

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
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ASSUMPTIONS

1. The HP 10x42 steel H-piles are being evaluated based on the as-built conditions and geometry shown in Ref. 5.
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used,
3. The loads provided by VHB (Ref.10) are applied at the abutment centroid, at the bottom of the pile cap. Abutment
4. All piles have reinforced pile tips (Ref. 5) and (Ref. 8).
5. The I-95 facing piles are battered at 3 on 12 (0.25) away from the abutment (Ref. 5).
6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.
7. FB-Multipier does not allow for angled geometry, the pile cap is tapered at the wingwall per (Ref. 5) but is modeled as a continuous rectangle in FB-Multipier. The wingwall piles are also skewed on the record plans, a limitation of FB-Multipier is that H-piles can only be oriented in the weak x or weak y direction they cannot be skewed. The skewed wingwall piles are oriented such to match the orientation of the rest of the piles and the road side wingwall piles are battered at a 3 on 12 to match the record plans.
8. The pile cap was modeled in FB-Multipier with a stiffness equivalent to a rectangular cross-section with the same moment of inertia as the existing abutment cross-section (Ref. 12).
9. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.
10. A φdyn of 0.5 is used to determine the factored pile resistance as requested by Maine DOT using (Ref. 7 Table 10.5.5.2.3-1).
11. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcemnt in FB-Multipier.

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	50	years	(per Maine DOT)
Total corrosion loss =	0.05	inches	(Ref. 16)

Pile size: HP 10x42

Intact Section				Corroded Section			(ref. 16)
Depth, d =	9.700	in	(Ref. 4, Table 1-4)	Depth, d =	9.6	in	
Width, b_f =	10.100	in	(Ref. 4, Table 1-4)	Width, b_f =	10.0	in	
Web thickness, t_w =	0.415	in	(Ref. 4, Table 1-4)	Web thickness, t_w =	0.315	in	
Flange thickness, t_f =	0.420	in	(Ref. 4, Table 1-4)	Flange thickness, t_f =	0.320	in	
Fillet area =	0.2	in^2		Fillet area =	0.2	in^2	
Section area =	12.4	in^2	(Ref. 4, Table 1-4)	Section area =	9.4	in^2	
Section area =	0.09	ft^2		Section area =	0.07	ft^2	
				Moment of inertia about the y-axis, I_y =	157	in^4	
				Moment of inertia about the x-axis, I_x =	53	in^4	
				Radius of gyration about the y-axis, r_y =	4.08	in	
				Radius of gyration about the x-axis, r_x =	2.38	in	

CALCULATIONS

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Elastic section modulus about the y-axis, Sy = 32.7 in^3
Elastic section modulus about the x-axis, Sx = 10.7 in^3
Plastic section modulus about the y-axis, Zy = 36.0 in^3
Plastic section modulus about the x-axis, Zx = 16.2 in^3

Steel yield stress, F_y = 36 ksi (Grade A36) Sheet 3 of 53 (Ref. 5)
Steel elastic modulus, E_{st} = 29,000 ksi
Top of pile cap elevation = 109.2 ft Sheet 10 of 53 (Ref. 5)
Pile cap thickness = 3 ft Sheet 10 of 53 (Ref. 5)
Pile cap midplane elevation = 107.7 ft
elevation = 107.7 ft
Base of pile cap elevation = 106.2 ft
Pile length = 13.4 ft

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Fill	1	Cohesive	106.2	104.2	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesionless	104.2	96.9	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	3	Cohesionless	96.9	95.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	4	Rock	95.3	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Sta. 109+43.83 in Profile E-E' (Sheet X Interpretive Subsurface Cross Section E-E').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model: (Pile 16)
Maximum axial force in the piles, P_u = 117.3 kips
Maximum bending moment in the piles, $M_x = M_2$ = 5.5 kip-ft = 66.1 in-kips
Maximum bending moment in the piles, $M_y = M_3$ = 83.2 kip-ft = 997.9 in-kips
Maximum shear force in the piles, V_u = 21.6 kips
Depth below pile head to fixity = 11.6 ft = Elev. 96.11 ft
Depth below pile cap to fixity = 10.1 ft = Elev. 96.11 ft

C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.07	ft^3/ft	(from Part A)
δ/ϕ_f =	0.75		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.79		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.04		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.46		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Fill	Glacial Till	
Friction angle, ϕ_f =	34	35	degrees (Ref. 6)
δ =	26	26	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	0.99	1.04	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.86	0.85	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.125	0.851	ksf (at midpoint of soil layer using stratigraphy in Part A)
f_s =	0.047	0.342	ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		
	Top	Bottom	
Undrained shear strength, s_u =	1.343	1.343	ksf (Ref. 6)
Adhesion factor, a =	1.00	1.00	(Ref. 7 Fig. 10.7.3.8.6b-1, sand
f_s =	1.343	1.343	ksf

Nominal shaft resistance:

$R_s = f_s A_s$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = (d × 2) + (b_f × 2) = 39.2 in = 3.3 ft (from Part A)

	Fill	Stiff Clay	Glacial Till		
Thickness =	2.0	7.3	1.6	ft	(from stratigraphy in Part A)
A_s =	6.5	23.8	5.2	ft^2	
R_s per layer =	0.3	32.0	0.2	kips	

Total R_s = 32.6 kips

Unit toe resistance, using the Kulhawy and Goodman method for hard rock:

$q_p = 0.33 q_u$ for RQD $\leq$ 70%
$q_p = 0.80 q_u$ for RQD = 100% (Ref. 3, Section 7.2.1.4.2)

where:

Rock quality designation, RQD = 9 % (Ref. 13, minimum RQD in borings with lab testing)
Unconfined compressive strength, q_u = 908 ksf (Ref. 14, minimum UCS from lab test data)

q_p = 299.6 ksf

Nominal toe resistance:

$R_p = q_p A_p$ (Ref. 7, Eq. 10.7.3.8.6a-3)

where:

Pile toe area, A_p = 0.09 ft^2 (from Part A)

Total R_p = 25.8 kips (bedrock)

Nominal and factored geotechnical resistance:

$R_n = R_s + R_p$ (Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = 58 kips (bedrock)

$$R_r = \varphi_{stat} R_n$$ (Ref. 7, Eq. 10.7.3.8.6a-1)

where:

(Ref. 7 Tbl. 10.5.5.2.3-1)

$\varphi_{stat} =$ 0.35 for shaft and toe resistance of cohesive soil by the α-method

$\varphi_{stat} =$ 0.45 for shaft and toe resistance of cohesionless soil by the Nordlund method

$\varphi_{stat} =$ 0.45 for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Fill	Stiff Clay	Glacial Till	Pile Tip (Bedrock)	
R_s or R_p =	0.3	32.0	0.2	25.8	kips
φ_{stat} =	0.45	0.35	0.45	0.45	(Ref. 7, Table 10.5.5.2.3-1)
R_r per layer =	0.1	11.2	0.1	11.6	

Total R_r = 23 kips (bedrock)

Wave Equation Analysis

Nominal and factored geotechnical resistance obained from GRWEAP at 100% of the steel yield strength (Ref. 15):

R_n = 287 kips (Ref. 15)

φ_{dyn} = 0.5 (Assumption 10)

R_f = 144 kips

Check that the geotechnical axial resistance Obtained from GRLWEAP (Ref. 15) is sufficient to support axial load on pile:

P_u = 117 kips (from Part B)

R_r = 144 kips > P_u = 117 kips OK

D. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K = 0.65 (Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)

Unbraced length, l = 121.13 in (from Part B)

Minimum radius of gyration, r_s = 2.38 in (from Part A)

Check: Kl/r_s = 33 < 120 OK

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Abutment 1 (old Abutment 2) - Strength, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Local buckling and equivalent nominal yield resistance :

$\frac{b}{t} \le \lambda_r$ (Ref. 7, Eq. 6.9.4.2.1-1) $0.56\sqrt{\frac{E_{st}}{F_y}}$ (Ref. 7, Table 6.9.4.2.1-1)
(Assumption 9)

where:

Half Flange width, b_f =	5.0	in	(from Part A)
Flange thickness, t_f =	0.320	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	36	ksi	(from Part A)

Check: $\frac{b}{t} =$ 15.63 < 15.89 $= 0.56\sqrt{\frac{E_{st}}{F_y}}$

The pile section is a non-slender element (Ref. 7, Article 6.9.4.1)

Calculate equivalent nominal yield resistance:

$P_o = F_y A_g$ (Ref. 7, Article 6.9.4.1)

Equivalent nominal yield resistance, P_o = 339 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$

for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	9.4	in^2	(from Part A)
Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, l =	121.13	in	(from Part B)
Minimum radius of gyration, r_s =	2.38	in	(from Part A)

P_e (flexural) = 2,463 kips

Elastic and flexural torsional buckling resistance:

$\frac{D}{t_w} \le 150$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	8.960	in	D= d - (2 × tf)
Web thickness, t_w =	0.315	in	(from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Date:	10/25/2024	**Made by:**	DEB
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Nominal and factored structural resistance in axial compression:

$$P_n = P_o 0.658^{P_o/P_e}$$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$$P_n = 0.877P_e$$ (Ref. 7, Eq. 6.9.4.1.1-2)

P_o / P_e = 0.14 < 2.25, so use Eq. 6.9.4.1.1-1

P_n = 320 kips

$$P_{r_{axial}} = \varphi_c P_n$$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c = 0.50 (Ref. 7, Article 6.5.4.2))

P_r = 160 kips

$$P_{r_{axial\,/\,flexural}} = \varphi_c P_n$$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c = 0.70 (Ref. 7, Article 6.5.4.2)) (Assumption X.)

$P_{r,\ axial/\ flexural}$= 224 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

P_u = 117 kips (from Part B)

$P_{r,\ axial}$ = 160 kips > P_u = 117 kips OK

E. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

λ_f = 15.63

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

λ_{pf} = 10.79

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

λr_f = 23.56

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	36	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	10.7	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.7	in^3	(from Part A)
Plastic section modulus about x, Z_x =	16.2	in^3	(from Part A)
Plastic section modulus about y, Z_y =	36.0	in^3	(from Part A)

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Abutment 1 (old Abutment 2) - Strength, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	508	1,251	in-kips

φ_f = 1.00 (Ref. 7, Article 6.5.4.2)

M_{rx} = 508 in-kips

M_{ry} = 1,251 in-kips

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} = 66 in-kips (from Part B)

M_{uy} = 998 in-kips (from Part B)

M_{rx} =	508	in-kips	>	M_{ux} =	66	in-kips	OK
M_{ry} =	1,251	in-kips	>	M_{uy} =	998	in-kips	OK

F. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

if $P_u/P_{r,\text{ axial/ flexural}} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

if $P_u/P_{r,\text{ axial/ flexural}} \geq 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\text{ axial/ flexural}}$ = 0.5 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 1.3 > 1.0 Not OK

G. Determine the nominal and factored structural resistance of a single pile in shear.

$$V_p = 0.58F_y d_w t_w$$

(Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$

(Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	36	ksi	(from Part A)
Web depth, d_w =	8.960	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.315	in	(from Part A)

$$\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_y}}$$

then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	8.960	in	D= d - (2 × tf)
t_w =	0.315	in	(from Part A)

$1.12\sqrt{\dfrac{Ek}{F_y}}$ = 71.1 ksi (Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 71.1 C = 1.0

$V_p =$ 59 kips
$V_n =$ 59 kips

$\varphi_v =$ 1.00 (Ref. 7, Article 6.5.4.2)
$V_r =$ 59 kips

Check that the structural shear resistance is sufficient to support shear load on pile:

(Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 21.6 kips (from Part B)

$V_r =$ 59 kips > $V_u =$ 21.6 kips OK

CONCLUSIONS

The results of the analysis indicate under proposed strength loading conditions with an estimated pile length of 13.0 feet, a maximum axial force of 117.3 kips, the corresponding strong axis (y-axis) moment of 83.2 ft-kips (997.9 in-kips) occurs in the HP10x42 piles at Abutment 1 (old Abutment 2) in Pile 16 (the maximum axial load condition) of the 1983 portion of the Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity. The axial and flexural interaction ratio for Pile 16 exceeded the limiting value of 1.0 This evaluation considers section loss for an additional 50 year design life, this evaluation does not consider downdrag. A summary table is presented below.

Abutment 1 (old Abutment 2, Pile 16) Summary Table		
Pile Length (ft)		13
Depth below Pile Head to Fixity (ft)		12
Geotechnical Axial Resistance (kips)	Nominal	287
	Factored	144
Structural Axial Compression Resistance (kips)	Nominal	320
	Factored	160
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	42
	Factored	42
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	104
	Factored	104
Combined Axial and Flexural Interaction Ratio		1.3
Shear Resistance (kips)	Nominal	59
	Factored	59

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

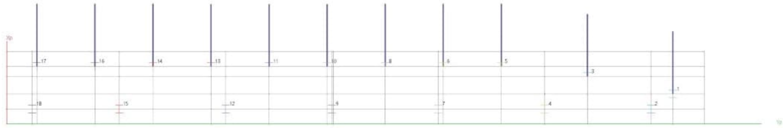


Figure 1b: Profile View of Pile Cap (Transverse)

Note: The pile cap was modeled with an equivalent thickness of 8.2 feet to account for the overall abutment stiffness.

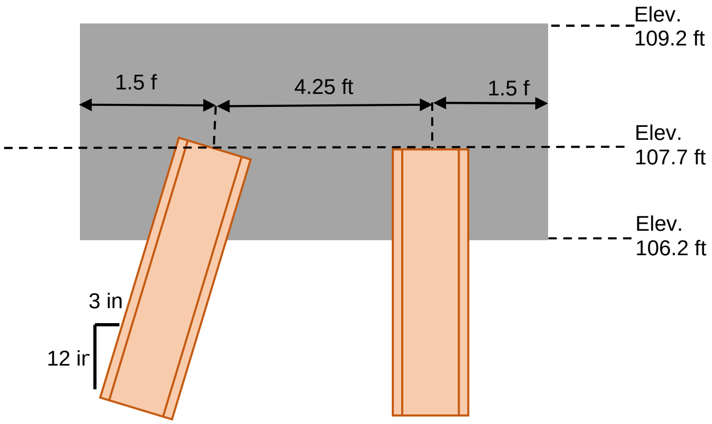


Figure 1c: 3D View of Pile Model

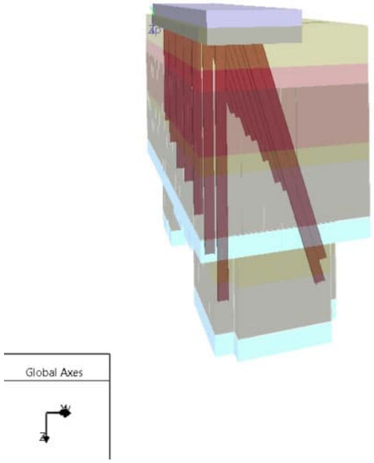


Figure 1d: 3D View of Model Loading

Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Existing Fill	1	Cohesionless	106.2	104.2	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	34	127	-	-	-	-	0.65		0.35	46		-	-	-
Stiff Clay	2	Cohesive	104.2	96.9	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1340	1340	-	-	-
Glacial Till	3	Cohesionless	96.9	95.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	333		-	-	-
Bedrock	4	Rock	95.3	90.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		3332.9	287.0	908,000

Notes:

1. The in-situ soil stratigraphy is based on Sta. 109+43.83 in Profile E-E' (Sheet X Interpretive Subsurface Cross Section E-E'), At the existing Abutment 2, for 15 foot length piles.

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

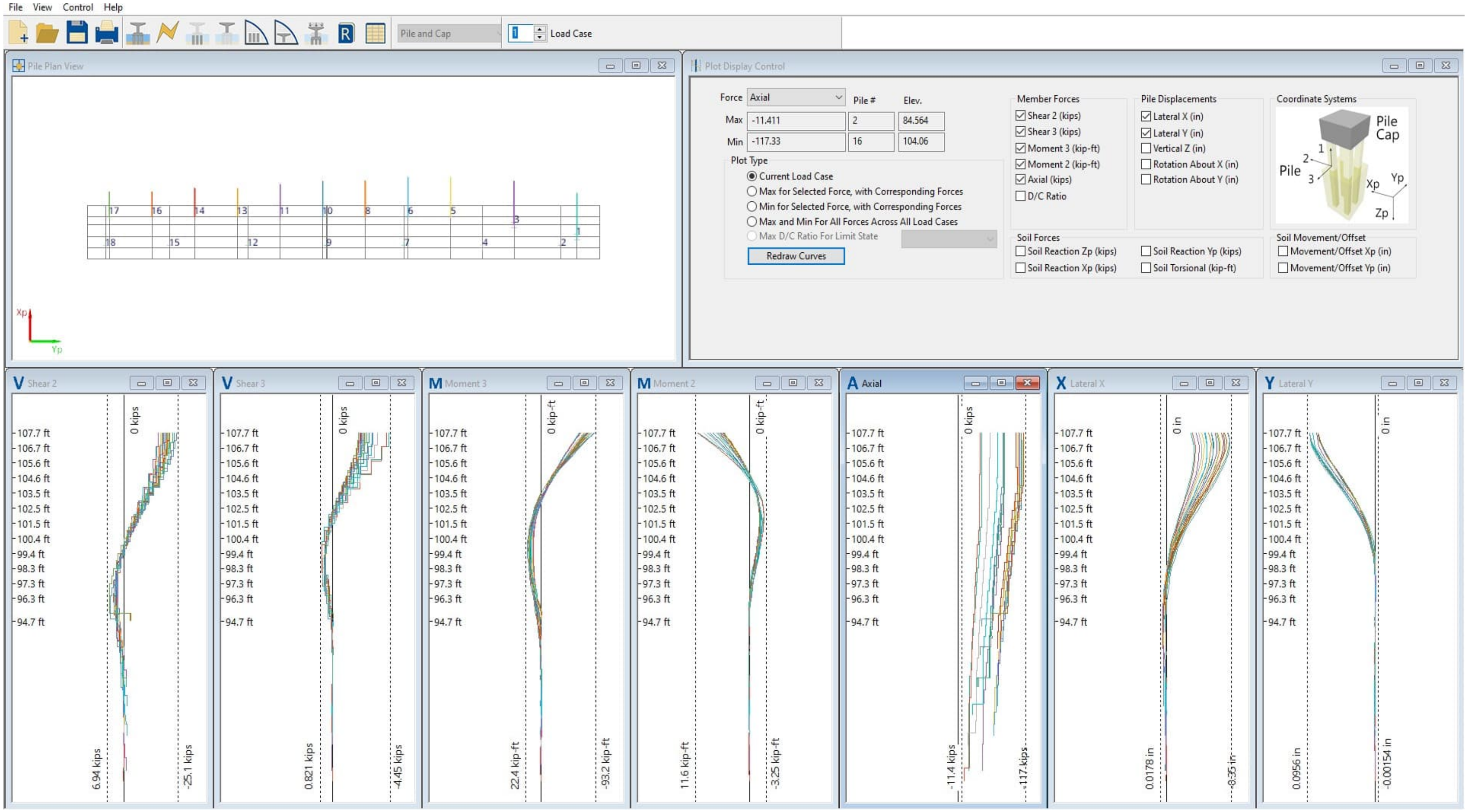
6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

Prepared: DEB
Checked: ATM
Reviewed: CCB

Strength



Force: Axial

		Pile #	Elev.
Max	-11.411	2	84.564
Min	-117.33	16	104.06

Plot Type
- ● Current Load Case
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Cases
- ○ Max D/C Ratio For Limit State

Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

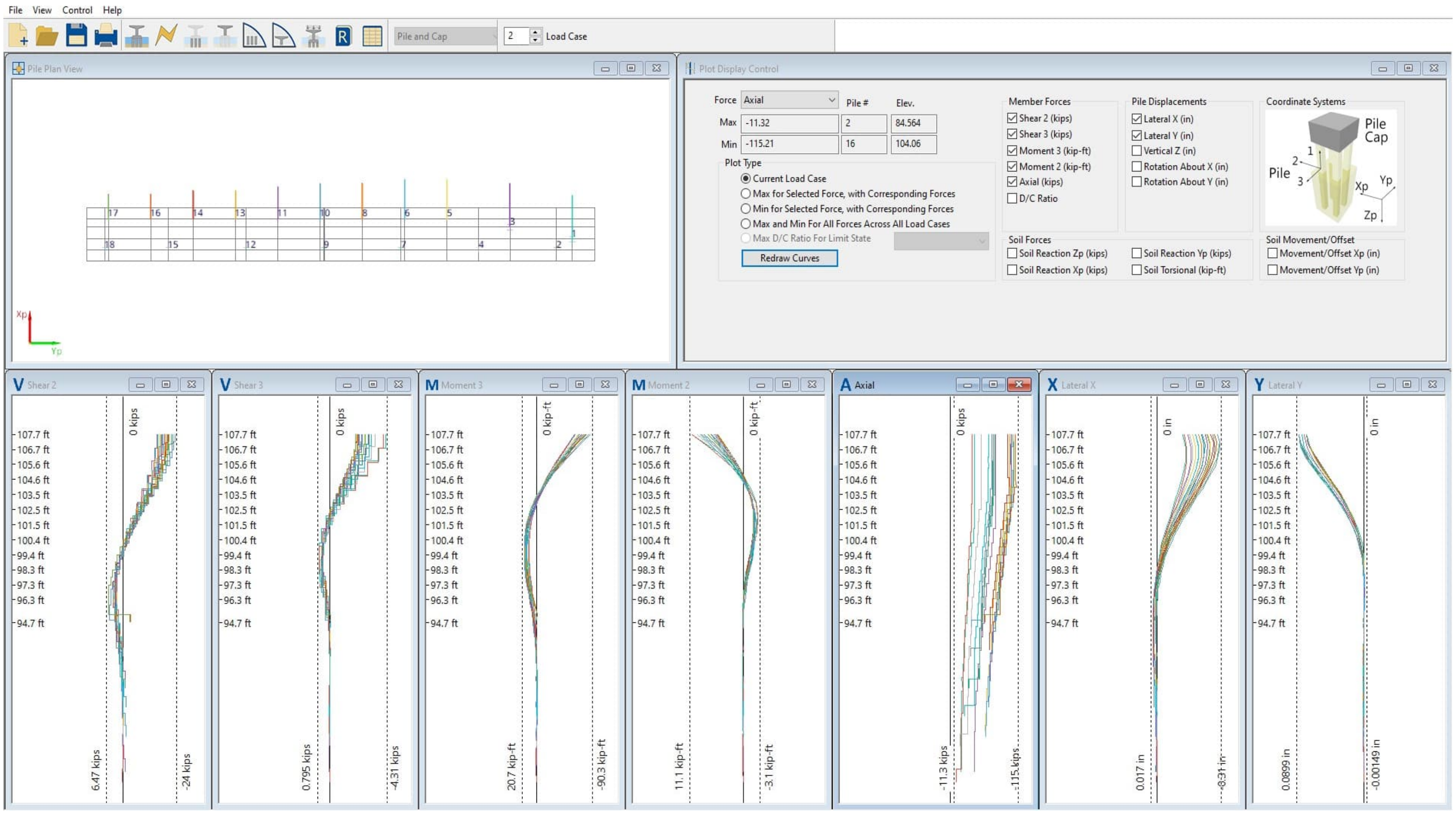
Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Coordinate Systems

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Soil Movement/Offset
- ☐ Movement/Offset Xp (in)
- ☐ Movement/Offset Yp (in)

Service



Load Case: 2

Force: Axial

	Value	Pile #	Elev.
Max	-11.32	2	84.564
Min	-115.21	16	104.06

Plot Type
- ● Current Load Case
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Cases
- ○ Max D/C Ratio For Limit State

Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Soil Movement/Offset
- ☐ Movement/Offset Xp (in)
- ☐ Movement/Offset Yp (in)

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Abutment 1 (old Abutment 2) - Strength, with Downdrag, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

OBJECTIVE

Determine if Abutment 1 located at Sta. 109+43 (old Abutment 2), corroded HP 10x42 steel piles provide adequate resistance based on geometry from the as-built 1983 record plans and the strength loading provided by VHB, and includes the estimated strength I downdrag loading.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3), and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Abutment 1 (existing Abutment 2) (Pile 16), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. PDR Draft 2-3-2023 Hogan Road DDI (Bridge Portion Only) 018595.10
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. Loading provided by VHB in Computation package titled 1983 Bridge Foundation Loads, dated 4/5/2024
11. WSP Existing Abutment 1 FB-Multipier Soil Properties
12. WSP calculation package titled Equivalent rectangular Abutment 1 cross section, dated 5/1/2024
13. WSP Boring logs, included as an appendix with the Geotechnical Report.
14. WSP compressive strength and elastic moduli laboratory testing results, dated November 16, 2023, included as an appendix with the Geotechnical Report.
15. WSP calculation package titled Pile Drivability at Abutment 2 (existing Abutment 1) - HP 10X42
16. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 8/1/2024.
17. Email from VHB on 10/10/24, requesting a resultant load move 0.71 feet in the positive x-direction.
18. WSP calculation package titled Existing Pile Downdrag at Existing Abutment 2 (new Abutment 1), dated 10/24/2024.

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength output
4. FB-Multipier Service output

ASSUMPTIONS

1. The HP 10x42 steel H-piles are being evaluated based on the as-built conditions and geometry shown in Ref. 5.
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no
3. The loads provided by VHB (Ref.10) are applied at the abutment centroid, at the bottom of the pile cap. Abutment designations
4. All piles have reinforced pile tips (Ref. 5) and (Ref. 8).

5. The I-95 facing piles are battered at 3 on 12 (0.25) away from the abutment (Ref. 5).

6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.

7. FB-Multipier does not allow for angled geometry, the pile cap is tapered at the wingwall per (Ref. 5) but is modeled as a continuous rectangle in FB-Multipier. The wingwall piles are also skewed on the record plans, a limitation of FB-Multipier is that H-piles can only be oriented in the weak x or weak y direction they cannot be skewed. The skewed wingwall piles are oriented such to match the orientation of the rest of the piles and the road side wingwall piles are battered at a 3 on 12 to match the record plans.

8. The pile cap was modeled in FB-Multipier with a stiffness equivalent to a rectangular cross-section with the same moment of inertia as the existing abutment cross-section (Ref. 12).

9. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

10. A φdyn of 0.5 is used to determine the factored pile resistance as requested by Maine DOT using (Ref. 7 Table 10.5.5.2.3-1).

11. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods
Design life = 50 years (per Maine DOT)
Total corrosion loss = 0.05 inches (Ref. 16)

Pile size: HP 10x42

Intact Section				Corroded Section		(ref. 16)
Depth, d =	9.700	in	(Ref. 4, Table 1-4)	Depth, d =	9.6	in
Width, b_f =	10.100	in	(Ref. 4, Table 1-4)	Width, b_f =	10.0	in
Web thickness, t_w =	0.415	in	(Ref. 4, Table 1-4)	Web thickness, t_w =	0.315	in
Flange thickness, t_f =	0.420	in	(Ref. 4, Table 1-4)	Flange thickness, t_f =	0.320	in
Fillet area =	0.2	in^2		Fillet area =	0.2	in^2
Section area =	12.4	in^2	(Ref. 4, Table 1-4)	Section area =	9.4	in^2
Section area =	0.09	ft^2		Section area =	0.07	ft^2
				Moment of inertia about the y-axis, I_y =	157	in^4
				Moment of inertia about the x-axis, I_x =	53	in^4
				Radius of gyration about the y-axis, r_y =	4.08	in
				Radius of gyration about the x-axis, r_x =	2.38	in
				Elastic section modulus about the y-axis, Sy =	32.7	in^3
				Elastic section modulus about the x-axis, Sx =	10.7	in^3
				Plastic section modulus about the y-axis, Zy =	36.0	in^3
				Plastic section modulus about the x-axis, Zx =	16.2	in^3

Steel yield stress, F_y = 36 ksi (Grade A36) Sheet 3 of 53 (Ref. 5)
Steel elastic modulus, E_{st} = 29,000 ksi
Top of pile cap elevation = 109.2 ft Sheet 10 of 53 (Ref. 5)
Pile cap thickness = 3 ft Sheet 10 of 53 (Ref. 5)
Pile cap midplane elevation = 107.7 ft
elevation = 107.7 ft
Base of pile cap elevation = 106.2 ft
Pile length = 13.4 ft

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
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Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Fill	1	Cohesive	106.2	104.2	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesionless	104.2	96.9	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	3	Cohesionless	96.9	95.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	4	Rock	95.3	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Sta. 109+43.83 in Profile E-E' (Sheet X Interpretive Subsurface Cross Section E-E').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model: (Pile 16)

Maximum axial force in the piles (includes 40.8 kip downdrag load), Pu = 158.6 kips

Maximum bending moment in the piles, $M_x = M_2$ = 5.5 kip-ft = 66.1 in-kips

Maximum bending moment in the piles, $M_y = M_3$ = 83.2 kip-ft = 997.9 in-kips

Maximum shear force in the piles, V_u = 21.6 kips

Depth below pile head to fixity = 11.6 ft = Elev. 96.11 ft

Depth below pile cap to fixity = 10.1 ft = Elev. 96.11 ft

C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega}$$

(Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V = 0.07 ft^3/ft (from Part A)

δ/ϕ_f = 0.75 (Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"

Angle of pile taper from vertical, ω = 0 degrees (piles are not tapered)

Coefficient K_δ for ϕ_f of 30° = 0.79 (Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)

Coefficient K_δ for ϕ_f of 35° = 1.04 (Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)

Coefficient K_δ for ϕ_f of 40° = 1.46 (Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Fill	Glacial Till	
Friction angle, ϕ_f =	34	35	degrees (Ref. 6)
δ =	26	26	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	0.99	1.04	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.86	0.85	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.125	0.851	ksf (at midpoint of soil layer using stratigraphy in Part A)
f_s =	0.047	0.342	ksf

CALCULATIONS

Date: 10/25/2024
Project No.: 31404817.004
Subject: Rehabilitated Abutment 1 (old Abutment 2) - Strength, with Downdrag, Corroded Section
Project Title: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME

Made by: DEB
Checked by: JEF
Reviewed by: CCB

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		
	Top	Bottom	
Undrained shear strength, s_u =	1.343	1.343	ksf (Ref. 6)
Adhesion factor, a =	1.00	1.00	(Ref. 7 Fig. 10.7.3.8.6b-1, sand
f_s =	1.343	1.343	ksf

Nominal shaft resistance:

$$R_s = f_s A_s$$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = (d × 2) + (b_f × 2) = 39.2 in = 3.3 ft (from Part A)

	Fill	Stiff Clay	Glacial Till	
Thickness =	2.0	7.3	1.6	ft (from stratigraphy in Part A)
A_s =	6.5	23.8	5.2	ft^2
R_s per layer =	0.3	32.0	0.2	kips

Total R_s = 32.6 kips

Unit toe resistance, using the Kulhawy and Goodman method for hard rock:

$$q_p = 0.33 q_u \text{ for RQD} \leq 70\%$$
$$q_p = 0.80 q_u \text{ for RQD} = 100\%$$ (Ref. 3, Section 7.2.1.4.2)

where:

Rock quality designation, RQD = 9 % (Ref. 13, minimum RQD in borings with lab testing)
Unconfined compressive strength, q_u = 908 ksf (Ref. 14, minimum UCS from lab test data)

q_p = 299.6 ksf

Nominal toe resistance:

$$R_p = q_p A_p$$ (Ref. 7, Eq. 10.7.3.8.6a-3)

where:

Pile toe area, A_p = 0.09 ft^2 (from Part A)

Total R_p = 25.8 kips (bedrock)

Nominal and factored geotechnical resistance:

$$R_n = R_s + R_p$$ (Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = 58 kips (bedrock)

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Abutment 1 (old Abutment 2) - Strength, with Downdrag, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

$$R_r = \varphi_{stat} R_n$$ (Ref. 7, Eq. 10.7.3.8.6a-1)

where:

(Ref. 7 Tbl. 10.5.5.2.3-1)

φ_{stat} = 0.35 for shaft and toe resistance of cohesive soil by the α-method

φ_{stat} = 0.45 for shaft and toe resistance of cohesionless soil by the Nordlund method

φ_{stat} = 0.45 for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Fill	Stiff Clay	Glacial Till	Pile Tip (Bedrock)	
R_s or R_p =	0.3	32.0	0.2	25.8	kips
φ_{stat} =	0.45	0.35	0.45	0.45	(Ref. 7, Table 10.5.5.2.3-1)
R_r per layer =	0.1	11.2	0.1	11.6	

Total R_r = 23 kips (bedrock)

Wave Equation Analysis

Nominal and factored geotechnical resistance obtained from GRWEAP at 100% of the steel yield strength (Ref. 15):

R_n = 287 kips (Ref. 15)

φ_{dyn} = 0.5 (Assumption 10)

R_f = 144 kips

Check that the geotechnical axial resistance Obtained from GRLWEAP (Ref. 15) is sufficient to support axial load on pile:

P_u = 159 kips (from Part B)

R_r = 144 kips < P_u = 159 kips Not OK

D. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K = 0.65 (Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)

Unbraced length, l = 121.13 in (from Part B)

Minimum radius of gyration, r_s = 2.38 in (from Part A)

Check: Kl/r_s = 33 < 120 OK

Local buckling and equivalent nominal yield resistance :

$\frac{b}{t} \leq \lambda_r$ (Ref. 7, Eq. 6.9.4.2.1-1) $0.56\sqrt{\frac{E_{st}}{F_y}}$ (Ref. 7, Table 6.9.4.2.1-1)
(Assumption 9)

where:

Half Flange width, b_f =	5.0	in	(from Part A)
Flange thickness, t_f =	0.320	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	36	ksi	(from Part A)

Check: $\frac{b}{t}$ = 15.63 < 15.89 $= 0.56\sqrt{\frac{E_{st}}{F_y}}$

The pile section is a non-slender element (Ref. 7, Article 6.9.4.1)

Calculate equivalent nominal yield resistance:

$P_o = F_y A_g$ (Ref. 7, Article 6.9.4.1)

Equivalent nominal yield resistance, P_o = 339 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$

for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	9.4	in^2	(from Part A)
Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, l =	121.13	in	(from Part B)
Minimum radius of gyration, r_s =	2.38	in	(from Part A)

P_e (flexural) = 2,463 kips

Elastic and flexural torsional buckling resistance:

$\frac{D}{t_w} \leq 150$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	8.960	in	D= d - (2 × tf)
Web thickness, t_w =	0.315	in	(from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$$P_n = P_o 0.658^{P_o/P_e}$$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$$P_n = 0.877P_e$$ (Ref. 7, Eq. 6.9.4.1.1-2)

$P_o / P_e =$ 0.14 < 2.25, so use Eq. 6.9.4.1.1-1

$P_n =$ 320 kips

$$P_{r_{axial}} = \varphi_c P_n$$ (Ref. 7 Eq. 6.9.2.1-1)

$\varphi_c =$ 0.50 (Ref. 7, Article 6.5.4.2))

$P_r =$ 160 kips

$$P_{r_{axial/flexural}} = \varphi_c P_n$$ (Ref. 7 Eq. 6.9.2.1-1)

$\varphi_c =$ 0.70 (Ref. 7, Article 6.5.4.2)) (Assumption X.)

$P_{r,\ axial/\ flexural} =$ 224 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

$P_u =$ 159 kips (from Part B)

$P_{r,\ axial} =$ 160 kips > $P_u =$ 159 kips OK

E. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3) (Ref. 7, Eqn 6.12.2.2.1-4) (Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_f = \frac{b_f}{2t_f} \qquad \lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}} \qquad \lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

$\lambda_f =$ 15.63 $\lambda_{pf} =$ 10.79 $\lambda r_f =$ 23.56

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	36	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	10.7	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.7	in^3	(from Part A)
Plastic section modulus about x, Z_x =	16.2	in^3	(from Part A)
Plastic section modulus about y, Z_y =	36.0	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	508	1,251	in-kips

φ_f = 1.00 (Ref. 7, Article 6.5.4.2)
M_{rx} = 508 in-kips
M_{ry} = 1,251 in-kips

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} = 66 in-kips (from Part B)
M_{uy} = 998 in-kips (from Part B)

M_{rx} = 508 in-kips > M_{ux} = 66 in-kips OK
M_{ry} = 1,251 in-kips > M_{uy} = 998 in-kips OK

F. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r}+\left(\frac{M_{ux}}{M_{rx}}+\frac{M_{uy}}{M_{ry}}\right)\le 1.0$$ if $P_u/P_{r,\ \text{axial/ flexural}} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r}+\frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}}+\frac{M_{uy}}{M_{ry}}\right)\le 1.0$$ if $P_u/P_{r,\ \text{axial/ flexural}} \geq 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\ \text{axial/ flexural}}$ = 0.7 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 1.5 > 1.0 Not OK

G. Determine the nominal and factored structural resistance of a single pile in shear.

$$V_p = 0.58F_y d_w t_w$$ (Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y = 36 ksi (from Part A)
Web depth, d_w = 8.960 in $d_w = d - (2 \times t_f)$
Web thickness, t_w = 0.315 in (from Part A)

$$\frac{D}{t_w}\le 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k = 5.0 (Ref. 7, Article 6.10.9.2)
D = 8.960 in D= d - (2 × tf)
t_w = 0.315 in (from Part A)

$$1.12\sqrt{\frac{Ek}{F_y}} =$$ 71.1 ksi (Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 71.1 C = 1.0

$V_p =$ 59 kips

$V_n =$ 59 kips

$\varphi_v =$ 1.00 (Ref. 7, Article 6.5.4.2)

$V_r =$ 59 kips

Check that the structural shear resistance is sufficient to support shear load on pile:

(Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 21.6 kips (from Part B)

$V_r =$ 59 kips $>$ $V_u =$ 21.6 kips OK

CONCLUSIONS

The results of the analysis indicate under proposed strength loading conditions with an estimated pile length of 13.0 feet, a maximum axial force of 158.6 kips including downdrag occurs in Pile 16. This axial force exceeds the factored geotechnical resistance and when evaluated with the corresponding strong axis (y-axis) moment of 83.2 ft-kips (997.9 in-kips) the limiting axial and flexural interaction ratio of 1.0 is exceeded. The estimated pile length is sufficient for the piles to achieve fixity. This evaluation considers section loss for an additional 50-year design life. A summary table is presented below.

Abutment 1 (old Abutment 2, Pile 16) Summary Table		
Pile Length (ft)		13
Depth below Pile Head to Fixity (ft)		12
Geotechnical Axial Resistance (kips)	Nominal	287
	Factored	144
Structural Axial Compression Resistance (kips)	Nominal	320
	Factored	160
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	42
	Factored	42
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	104
	Factored	104
Combined Axial and Flexural Interaction Ratio		1.5
Shear Resistance (kips)	Nominal	59
	Factored	59

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

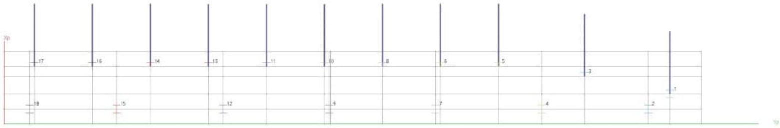


Figure 1b: Profile View of Pile Cap (Transverse)

Note: The pile cap was modeled with an equivalent thickness of 8.2 feet to account for the overall abutment stiffness.

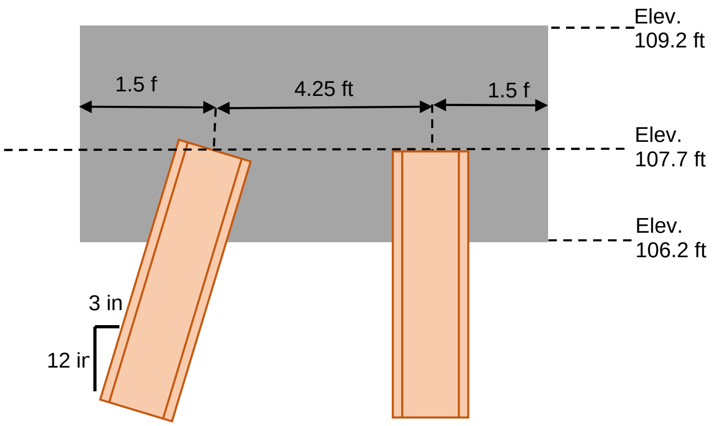


Figure 1c: 3D View of Pile Model

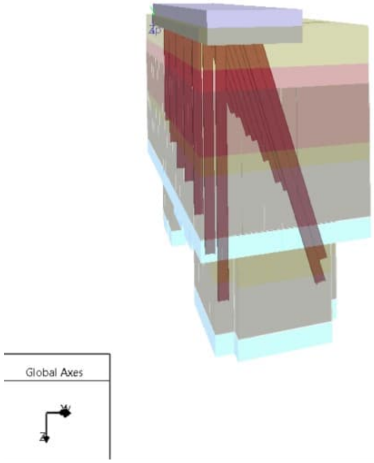


Figure 1d: 3D View of Model Loading

Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Existing Fill	1	Cohesionless	106.2	104.2	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	34	127	-	-	-	-	0.65		0.35	46		-	-	-
Stiff Clay	2	Cohesive	104.2	96.9	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1340	1340	-	-	-
Glacial Till	3	Cohesionless	96.9	95.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	333		-	-	-
Bedrock	4	Rock	95.3	90.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		3332.9	287.0	908,000

Notes:

1. The in-situ soil stratigraphy is based on Sta. 109+43.83 in Profile E-E' (Sheet X Interpretive Subsurface Cross Section E-E'), At the existing Abutment 2, for 15 foot length piles.

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

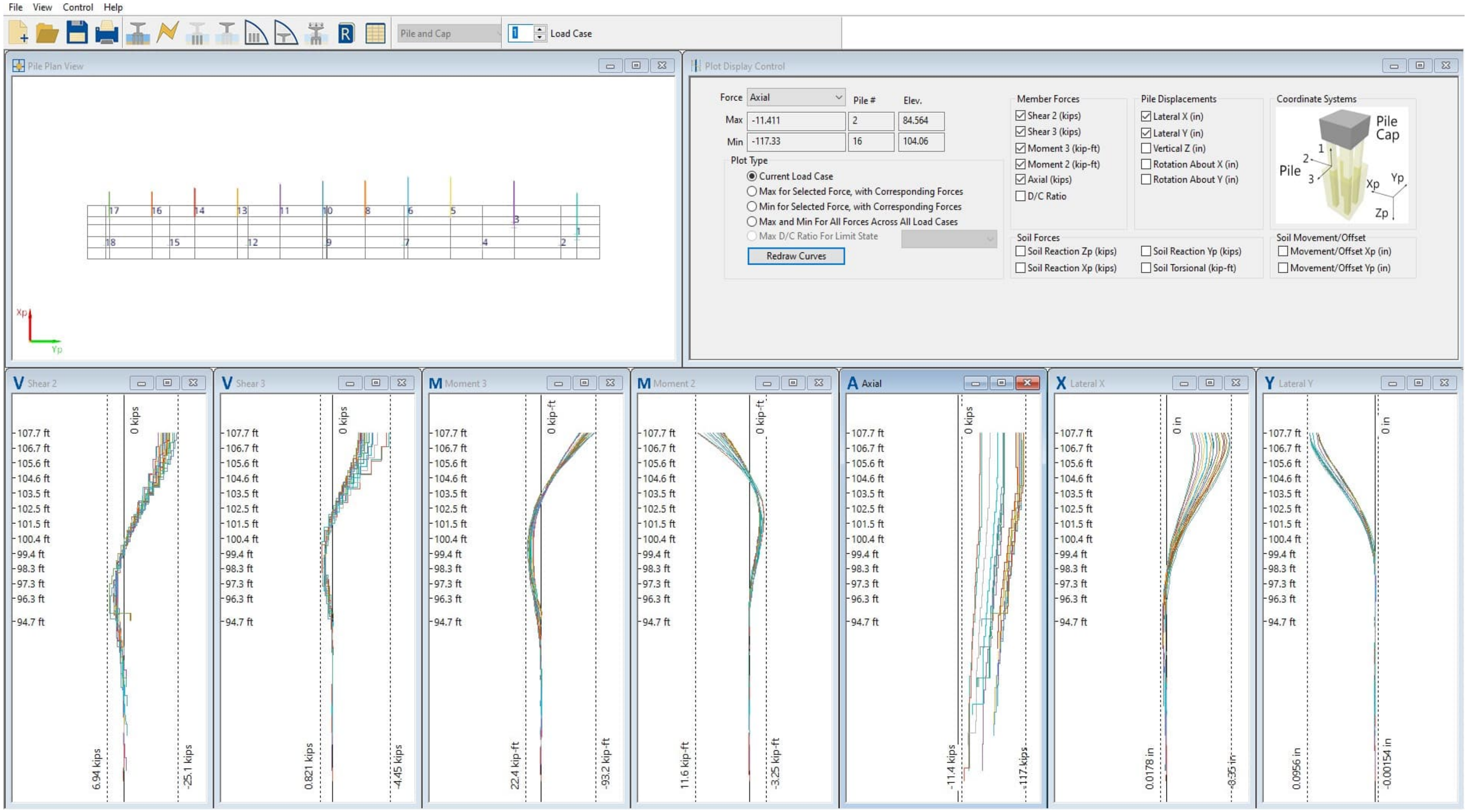
6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

Prepared: DEB
Checked: ATM
Reviewed: CCB

Strength



Load Case: 1

Force: Axial

		Pile #	Elev.
Max	-11.411	2	84.564
Min	-117.33	16	104.06

Plot Type
- ● Current Load Case
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Cases
- ○ Max D/C Ratio For Limit State

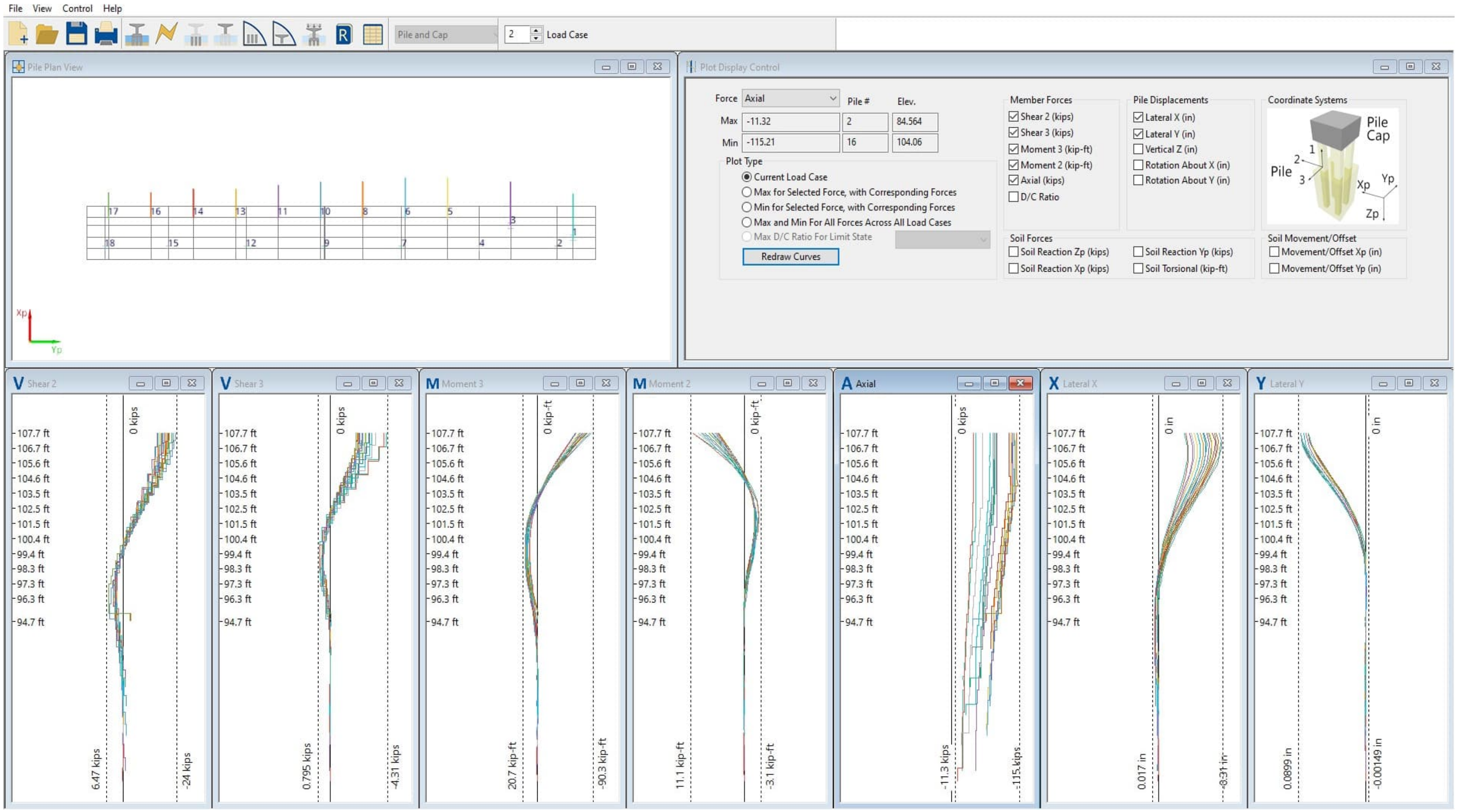
Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Soil Movement/Offset
- ☐ Movement/Offset Xp (in)
- ☐ Movement/Offset Yp (in)

Service



Load Case: 2

Force: Axial

	Value	Pile #	Elev.
Max	-11.32	2	84.564
Min	-115.21	16	104.06

Plot Type
- ● Current Load Case
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Cases
- ○ Max D/C Ratio For Limit State

Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Soil Movement/Offset
- ☐ Movement/Offset Xp (in)
- ☐ Movement/Offset Yp (in)

CALCULATIONS

Date:	10/15/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Abutment 2 (old Abutment 1) - Strength, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

OBJECTIVE

Determine if Abutment 2, located at Sta. 113+91 (old Abutment 1), corroded HP 10x42 steel piles provide adequate resistance based on geometry from the as-built 1983 record plans and the strength loading provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3), and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Abutment 1 (Pile 6), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. PDR Draft 2-3-2023 Hogan Road DDI (Bridge Portion Only) 018595.10
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. Loading provided by VHB in Computation package titled 1983 Bridge Foundation Loads, dated 4/5/2024
11. WSP Existing Abutment 1 FB-Multipier Soil Properties
12. WSP calculation package titled Equivalent rectangular Abutment 1 cross section, dated 5/1/2024
13. WSP Boring logs, included as an appendix with the Geotechnical Report.
14. WSP compressive strength and elastic moduli laboratory testing results, dated November 16, 2023, included as an appendix with the Geotechnical Report.
15. WSP calculation package titled Pile Drivability at Abutment 1 (existing Abutment 2) - HP 10X42
16. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 8/1/2024.
17. Email from VHB on 10/10/24, requesting a resultant load move 0.71 feet in the positive x-direction.

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength output
4. FB-Multipier Service output

ASSUMPTIONS

1. The HP 10x42 steel H-piles are being evaluated based on the as-built conditions and geometry shown in Ref. 5.
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The loads provided by VHB (Ref.10) are applied at the abutment centroid, at the bottom of the pile cap. Abutment designations are based on VHB designations in Ref. 10.
4. All piles have reinforced pile tips (Ref. 5) and (Ref. 8).
5. The I-95 facing piles are battered at 3 on 12 (0.25) away from the abutment (Ref. 5).

6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.

7. FB-Multipier does not allow for angled geometry, the pile cap is tapered at the wingwall per (Ref. 5) but is modeled as a continuous rectangle in FB-Multipier. The wingwall piles are also skewed on the record plans, a limitation of FB-Multipier is that H-piles can only be oriented in the weak x or weak y direction they cannot be skewed. The skewed wingwall piles are oriented such to match the orientation of the rest of the piles and the road side wingwall piles are battered at a 3 on 12 to match the record plans.

8. The pile cap was modeled in FB-Multipier with a stiffness equivalent to a rectangular cross-section with the same moment of inertia as the existing abutment cross-section (Ref. 12).

9. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

10. A φdyn of 0.5 is used to determine the factored pile resistance as requested by Maine DOT using (Ref. 7 Table 10.5.5.2.3-1).

11. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcemnt in FB-Multipier.

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	50	years	(per Maine DOT)
Total corrosion loss =	0.05	inches	(Ref. 16)

Pile size: HP 10x42

Intact Section			Corroded Section			(ref. 16)
Depth, d =	9.700	in (Ref. 4, Table 1-4)	Depth, d =	9.6	in	
Width, b_f =	10.100	in (Ref. 4, Table 1-4)	Width, b_f =	10.0	in	
Web thickness, t_w =	0.415	in (Ref. 4, Table 1-4)	Web thickness, t_w =	0.315	in	
Flange thickness, t_f =	0.420	in (Ref. 4, Table 1-4)	Flange thickness, t_f =	0.320	in	
Fillet area =	0.2	in^2	Fillet area =	0.2	in^2	
Section area =	12.4	in^2 (Ref. 4, Table 1-4)	Section area =	9.4	in^2	
Section area =	0.09	ft^2	Section area =	0.07	ft^2	
			Moment of inertia about the pile y-axis, I_y =	157	in^4	
			Moment of inertia about the pile x-axis, I_x =	53	in^4	
			Radius of gyration about the pile y-axis, r_y =	4.08	in	
			Radius of gyration about the pile x-axis, r_x =	2.38	in	
			Elastic section modulus about the pile y-axis, Sy =	32.7	in^3	
			Elastic section modulus about the pile x-axis, Sx =	10.7	in^3	
			Plastic section modulus about the pile y-axis, Zy =	36.0	in^3	
			Plastic section modulus about the pile x-axis, Zx =	16.2	in^3	

Steel yield stress, F_y =	36	ksi	(Grade A36) Sheet 3 of 53 (Ref. 5)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	104.0	ft	Sheet 10 of 53 (Ref. 5)
Pile cap thickness =	3	ft	Sheet 10 of 53 (Ref. 5)
Pile cap midplane elevation =	102.5	ft	

Pile head elevation = as-modeled pile cap midplane elevation =	102.5	ft
Base of pile cap elevation =	101.0	ft
Pile length =	42.2	ft

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Fill	1	Cohesive	101.0	94.7	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesionless	94.7	82.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Medium Clay	3	Cohesive	82.1	81.4	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	4	Cohesionless	81.4	61.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	5	Rock	61.3	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Sta. 113+91.73 in Profile F-F' (Sheet X Interpretive Subsurface Cross Section F-F').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model:

	(Pile 6)					
Maximum axial force in the piles, P_u =	117.0	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	5.4	kip-ft	=		64.6	in-kips
Maximum bending moment in the piles, $M_y = M_3$ =	85.4	kip-ft	=		1024.5	in-kips
Maximum shear force in the piles, V_u =	20.4	kips				
Depth below pile head to fixity =	14.2	ft	=	Elev.	88.33	ft
Depth below pile cap to fixity =	12.7	ft	=	Elev.	88.33	ft

C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip elevation.

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega} \qquad \text{(Ref. 7, Eq. 10.7.3.8.6f-1)}$$

where:

Pile soil displacement, V =	0.07	ft^3/ft	(from Part A)
δ/ϕ_f =	0.75		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.79		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.04		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.46		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

CALCULATIONS

Date:	10/15/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Abutment 2 (old Abutment 1) - Strength, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

	Fill	Glacial Till	
Friction angle, ϕ_f =	34	35	degrees (Ref. 6)
δ =	26	26	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	0.99	1.04	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.86	0.85	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.394	2.733	ksf (at midpoint of soil layer using stratigraphy in Part A)

f_s =	0.144	1.068	ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		Medium Clay		
	Top	Bottom	Top	Bottom	
Undrained shear strength, s_u =	1.343	1.343	0.888	0.888	ksf (Ref. 6)
Adhesion factor, a =	1.00	1.00	1.00	1.00	(Ref. 7 Fig. 10.7.3.8.6b-1, sand over clay, 10b<D<20b)
f_s =	1.343	1.343	0.888	0.888	ksf

Nominal shaft resistance:

$$R_s = f_s A_s$$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = (d × 2) + (b_f × 2) = 39.2 in = 3.3 ft (from Part A)

	Fill	Stiff Clay	Medium Clay	Glacial Till	
Thickness =	6.3	12.6	0.7	20.1	ft (from stratigraphy in Part A)
A_s =	20.6	41.2	2.3	65.7	ft^2

R_s per layer =	3.0	55.3	2.0	70.1	kips

Total R_s = 130.4 kips

Unit toe resistance, using the Kulhawy and Goodman method for hard rock:

$$q_p = 0.33 q_u \text{ for RQD} \leq 70\%$$
$$q_p = 0.80 q_u \text{ for RQD} = 100\%$$

(Ref. 3, Section 7.2.1.4.2)

where:

Rock quality designation, RQD = 9 % (Ref. 13, minimum RQD in borings with lab testing)

Unconfined compressive strength, q_u = 908 ksf (Ref. 14, minimum UCS from lab test data)

q_p = 299.6 ksf

Nominal toe resistance:

$$R_p = q_p A_p$$ (Ref. 7, Eq. 10.7.3.8.6a-3)

where:

Pile toe area, A_p = 0.09 ft^2 (from Part A)

Total R_p = $\boxed{25.8}$ kips (bedrock)

Nominal and factored geotechnical resistance:

$$R_n = R_s + R_p$$ (Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = $\boxed{156}$ kips (bedrock)

$$R_r = \varphi_{stat} R_n$$ (Ref. 7, Eq. 10.7.3.8.6a-1)

where: (Ref. 7 Tbl. 10.5.5.2.3-1)

φ_{stat} = 0.35 for shaft and toe resistance of cohesive soil by the α-method

φ_{stat} = 0.45 for shaft and toe resistance of cohesionless soil by the Nordlund method

φ_{stat} = 0.45 for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Fill	Stiff Clay	Medium Clay	Glacial Till	Pile Tip (Bedrock)	
R_s or R_p =	3.0	55.3	2.0	70.1	25.8	kips
φ_{stat} =	0.45	0.35	0.35	0.45	0.45	(Ref. 7, Table 10.5.5.2.3-1)
R_r per layer =	1.3	19.3	0.7	31.6	11.6	

Total R_r = $\boxed{65}$ kips (bedrock)

Wave Equation Analysis

Nominal and factored geotechnical resistance obained from GRWEAP at 100% of the steel yield strength (Ref. 15):

R_n = 328 kips (Ref. 15)

φ_{dyn} = 0.5 Assumption 10

R_f = 164 kips

Check that the geotechnical axial resistance Obtained from GRLWEAP (Ref. 15) is sufficient to support axial load on pile:

P_u = 117 kips (from Part B)

R_r = 164 kips > P_u = 117 kips OK

D. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, l =	151.99	in	(from Part B)
Minimum radius of gyration, r_s =	2.38	in	(from Part A)

Check: Kl/r_s = 42 < 120 OK

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \le \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $$0.56\sqrt{\frac{E_{st}}{F_y}}$$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 9)

where:

Half Flange width, b_f =	5.0	in	(from Part A)
Flange thickness, t_f =	0.320	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	36	ksi	(from Part A)

Check: $$\frac{b}{t} = 15.63 < 15.89 = 0.56\sqrt{\frac{E_{st}}{F_y}}$$

The pile section is a non-slender element (Ref. 7, Article 6.9.4.1)

Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1)

Equivalent nominal yield resistance, P_o = 339 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	9.4	in^2	(from Part A)
Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head du embedment into pile cap)
Unbraced length, l =	151.99	in	(from Part B)
Minimum radius of gyration, r_s =	2.38	in	(from Part A)

P_e (flexural) = 1,565 kips

Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \le 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	8.960	in	$D = d - (2 \times tf)$
Web thickness, t_w =	0.315	in	(from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$P_n = P_o 0.658^{P_o/P_e}$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$P_n = 0.877P_e$ (Ref. 7, Eq. 6.9.4.1.1-2)

$(P_o / P_{e})_{top} =$ 0.22 < 2.25, so use Eq. 6.9.4.1.1-1

$P_{n,\,top} =$ 310 kips

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

$\varphi_c =$ 0.50 (Ref. 7, Article 6.5.4.2))

$P_{r,axial} =$ 155 kips

$P_{r_{axial\,/\,flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

$\varphi_c =$ 0.70 (Ref. 7, Article 6.5.4.2))

$P_{r,\,axial/\,flexural} =$ 217 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

$P_u =$ 117 kips (from Part B)

$P_{r,\,axial} =$ 155 kips > $P_u =$ 117 kips OK

E. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

$\lambda_f =$ 15.63

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

$\lambda_{pf} =$ 10.79

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

$\lambda r_f =$ 23.56

Nominal and factored structural resistance in flexure:

$M_n = M_p = 1.5F_yS_y$ if $\lambda_f \le \lambda_{pf}$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$

(Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, $F_y =$	36	ksi	(from Part A)
Steel elastic modulus, $E_{st} =$	29,000	ksi	(from Part A)
Elastic section modulus about x, $S_x =$	10.7	in^3	(from Part A)
Elastic section modulus about y, $S_y =$	32.7	in^3	(from Part A)
Plastic section modulus about x, $Z_x =$	16.2	in^3	(from Part A)
Plastic section modulus about y, $Z_y =$	36.0	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	508	1,251	in-kips

φ_f = 1.00 (Ref. 7, Article 6.5.4.2)

M_{rx} = 508 in-kips

M_{ry} = 1,251 in-kips

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} = 65 in-kips (from Part B)

M_{uy} = 1025 in-kips (from Part B)

M_{rx} =	508	in-kips	>	M_{ux} =	65	in-kips	OK
M_{ry} =	1,251	in-kips	>	M_{uy} =	1025	in-kips	OK

F. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

if $P_u/P_{r,\text{ axial/ flexural}} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

if $P_u/P_{r,\text{ axial/ flexural}} \geq 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\text{ axial/ flexural}}$ = 0.5 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 1.4 > 1.0 Not OK

G. Determine the nominal and factored structural resistance of a single pile in shear.

$$V_p = 0.58F_y d_w t_w$$

(Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$

(Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	36	ksi	(from Part A)
Web depth, d_w =	8.960	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.315	in	(from Part A)

$$\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_y}}$$

then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	8.960	in	D= d - (2 × tf)
t_w =	0.315	in	(from Part A)

$$1.12\sqrt{\frac{Ek}{F_y}} = 71.1 \text{ ksi}$$

(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 71.1 C = 1.0

V_p = 59 kips

V_n = 59 kips

Date:	10/15/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Abutment 2 (old Abutment 1) - Strength, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

$\varphi_v =$ 1.00 (Ref. 7, Article 6.5.4.2)

$V_r =$ 59 kips

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_u \leq \varphi_v V_n$$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 20.4 kips (from Part B)

$V_r =$ 59 kips > $V_u =$ 20.4 kips OK

CONCLUSIONS

The results of the analysis indicate under proposed strength loading conditions with an estimated pile length of 42.0 feet, a maximum axial force of 116.96 kips, the corresponding strong axis (y-axis) moment of 85.4 ft-kips (1024.5 in-kips) occurs in the HP10x42 piles at Abutment 2 (old Abutment 1) in Pile 6 of the 1983 portion of the Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity. The axial and flexural interaction ratio for Pile 6 exceeded the limiting value of 1.0. This evaluation considers section loss for an addtional 50 year design life, this evaluation does not consider downdrag. A summary table is presented below.

Abutment 2 (old Abutment 1, Pile 6) Summary Table		
Pile Length (ft)		42
Depth below Pile Head to Fixity (ft)		14
Geotechnical Axial Resistance (kips)	Nominal	328
	Factored	164
Structural Axial Compression Resistance (kips)	Nominal	310
	Factored	155
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	42
	Factored	42
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	104
	Factored	104
Combined Axial and Flexural Interaction Ratio		1.4
Shear Resistance (kips)	Nominal	59
	Factored	59

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

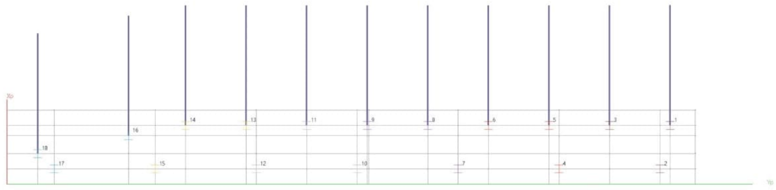


Figure 1b: Profile View of Pile Cap (Transverse)

Note: The pile cap was modeled with an equivalent thickness of 8.0 feet to account for the overall abutment stiffness.

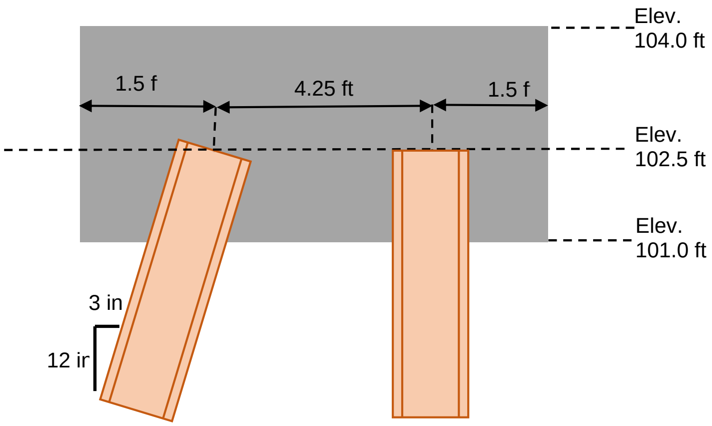


Figure 1c: 3D View of Pile Model

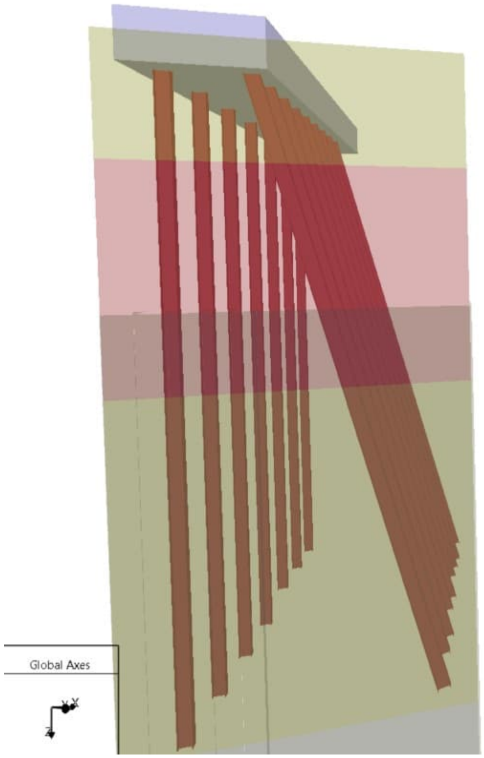
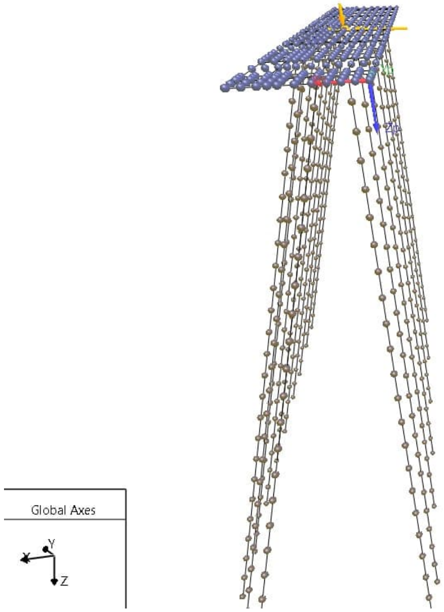


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) ^{1}		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) ^{2}	Friction Angle (°) ^{2}	Subgrade Modulus (pci) ^{3}	Undrained Shear Strength (psf) ^{2}		Major Principal Strain @ 50% ^{4}	Major Principal Strain @ 100% ^{4}	Shear Modulus (ksi) ^{3}		Poisson's Ratio ^{3}	Nominal Unit Skin Friction and Torsional Shear Stress (psf) ^{5,6,7}		Nominal Unit Tip Resistance (ksf) ^{6,7}	Nominal Tip Resistance (kips)	UCS (psf) ^{8}
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Existing Fill	1	Cohesionless	101.0	94.7	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	34	127	-	-	-	-	0.65		0.35	144		-	-	-
Stiff Clay	2	Cohesive	94.7	82.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1340	1340	-	-	-
Medium Clay	3	Cohesive	82.1	81.4	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	121	-	-	888	888	0.010	0.030	1.44	1.44	0.45	888	888	-	-	-
Glacial Till	4	Cohesionless	81.4	61.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	1068		-	-	-
Bedrock	5	Rock	61.3	57.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		3809.0	328.0	908,000

Notes:

1. The in-situ soil stratigraphy is based on Sta. 113+91.73 in Profile F-F' (Sheet X Interpretive Subsurface Cross Section F-F'), At the existing Abutment 1, for 41 foot length piles.

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

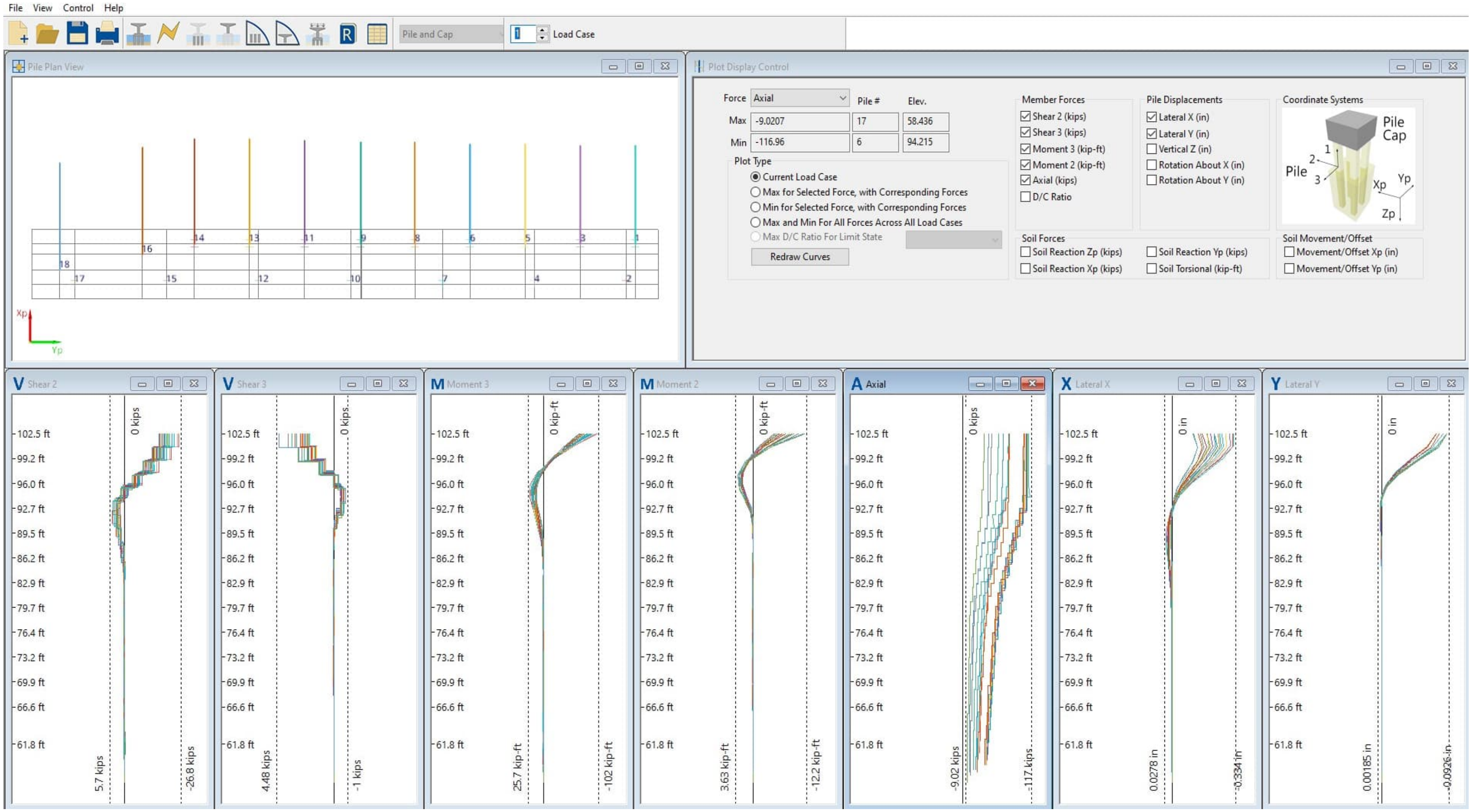
6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

Prepared:	DEB
Checked:	ATM
Reviewed:	CCB

Strength



Load Case: 1

Force: Axial

		Pile #	Elev.
Max	-9.0207	17	58.436
Min	-116.96	6	94.215

Plot Type
- ● Current Load Case
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Cases
- ○ Max D/C Ratio For Limit State

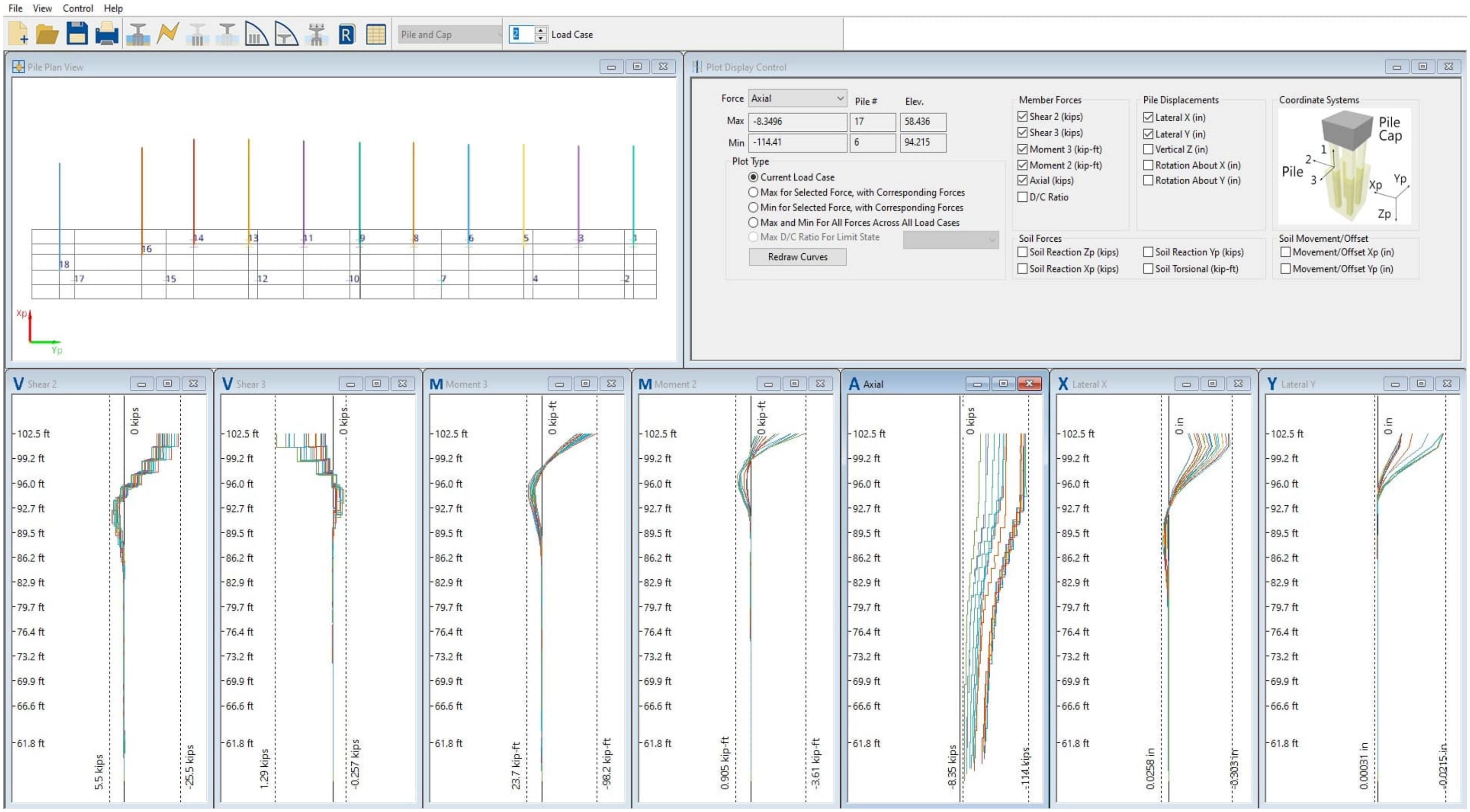
Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Soil Movement/Offset
- ☐ Movement/Offset Xp (in)
- ☐ Movement/Offset Yp (in)

Service



Load Case: 2

Pile and Cap

Force: Axial

	Force	Pile #	Elev.
Max	-8.3496	17	58.436
Min	-114.41	6	94.215

Plot Type
- ● Current Load Case
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Cases
- ○ Max D/C Ratio For Limit State

Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Soil Movement/Offset
- ☐ Movement/Offset Xp (in)
- ☐ Movement/Offset Yp (in)

CALCULATIONS

Date:	10/15/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Abutment 2 (old Abutment 1) - Strength, with Downdrag, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

OBJECTIVE

Determine if Abutment 2, located at Sta. 113+91 (old Abutment 1), corroded HP 10x42 steel piles provide adequate resistance based on geometry from the as-built 1983 record plans and the strength loading provided by VHB, and includes the estimated strength I downdrag loading.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3), and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Abutment 1 (Pile 6), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. PDR Draft 2-3-2023 Hogan Road DDI (Bridge Portion Only) 018595.10
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. Loading provided by VHB in Computation package titled 1983 Bridge Foundation Loads, dated 4/5/2024
11. WSP Existing Abutment 1 FB-Multipier Soil Properties
12. WSP calculation package titled Equivalent rectangular Abutment 1 cross section, dated 5/1/2024
13. WSP Boring logs, included as an appendix with the Geotechnical Report.
14. WSP compressive strength and elastic moduli laboratory testing results, dated November 16, 2023, included as an appendix with the Geotechnical Report.
15. WSP calculation package titled Pile Drivability at Abutment 1 (existing Abutment 2) - HP 10X42
16. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 8/1/2024.
17. Email from VHB on 10/10/24, requesting a resultant load move 0.71 feet in the positive x-direction.
18. WSP calculation package titled Existing Pile Downdrag at Existing Abutment 2 (new Abutment 1), dated 10/24/2024.

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength output
4. FB-Multipier Service output

ASSUMPTIONS

1. The HP 10x42 steel H-piles are being evaluated based on the as-built conditions and geometry shown in Ref. 5.
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The loads provided by VHB (Ref.10) are applied at the abutment centroid, at the bottom of the pile cap. Abutment designations are based on VHB designations in Ref. 10.
4. All piles have reinforced pile tips (Ref. 5) and (Ref. 8).
5. The I-95 facing piles are battered at 3 on 12 (0.25) away from the abutment (Ref. 5).

6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.

7. FB-Multipier does not allow for angled geometry, the pile cap is tapered at the wingwall per (Ref. 5) but is modeled as a continuous rectangle in FB-Multipier. The wingwall piles are also skewed on the record plans, a limitation of FB-Multipier is that H-piles can only be oriented in the weak x or weak y direction they cannot be skewed. The skewed wingwall piles are oriented such to match the orientation of the rest of the piles and the road side wingwall piles are battered at a 3 on 12 to match the record plans.

8. The pile cap was modeled in FB-Multipier with a stiffness equivalent to a rectangular cross-section with the same moment of inertia as the existing abutment cross-section (Ref. 12).

9. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

10. A φdyn of 0.5 is used to determine the factored pile resistance as requested by Maine DOT using (Ref. 7 Table 10.5.5.2.3-1).

11. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	50	years	(per Maine DOT)
Total corrosion loss =	0.05	inches	(Ref. 16)

Pile size: HP 10x42

	Intact Section				Corroded Section		(ref. 16)
Depth, d =	9.700	in	(Ref. 4, Table 1-4)	Depth, d =	9.6	in	
Width, b_f =	10.100	in	(Ref. 4, Table 1-4)	Width, b_f =	10.0	in	
Web thickness, t_w =	0.415	in	(Ref. 4, Table 1-4)	Web thickness, t_w =	0.315	in	
Flange thickness, t_f =	0.420	in	(Ref. 4, Table 1-4)	Flange thickness, t_f =	0.320	in	
Fillet area =	0.2	in^2		Fillet area =	0.2	in^2	
Section area =	12.4	in^2	(Ref. 4, Table 1-4)	Section area =	9.4	in^2	
Section area =	0.09	ft^2		Section area =	0.07	ft^2	
				Moment of inertia about the pile y-axis, I_y =	157	in^4	
				Moment of inertia about the pile x-axis, I_x =	53	in^4	
				Radius of gyration about the pile y-axis, r_y =	4.08	in	
				Radius of gyration about the pile x-axis, r_x =	2.38	in	
				Elastic section modulus about the pile y-axis, Sy =	32.7	in^3	
				Elastic section modulus about the pile x-axis, Sx =	10.7	in^3	
				Plastic section modulus about the pile y-axis, Zy =	36.0	in^3	
				Plastic section modulus about the pile x-axis, Zx =	16.2	in^3	

Steel yield stress, F_y =	36	ksi	(Grade A36) Sheet 3 of 53 (Ref. 5)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	104.0	ft	Sheet 10 of 53 (Ref. 5)
Pile cap thickness =	3	ft	Sheet 10 of 53 (Ref. 5)
Pile cap midplane elevation =	102.5	ft	

Pile head elevation = as-modeled pile cap midplane elevation =	102.5	ft
Base of pile cap elevation =	101.0	ft
Pile length =	42.2	ft

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Fill	1	Cohesive	101.0	94.7	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesionless	94.7	82.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Medium Clay	3	Cohesive	82.1	81.4	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	4	Cohesionless	81.4	61.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	5	Rock	61.3	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Sta. 113+91.73 in Profile F-F' (Sheet X Interpretive Subsurface Cross Section F-F').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model: (Pile 6)

Maximum axial force in the piles (includes 70.4 kip downdrag load), P_u =	188.1	kips			
Maximum bending moment in the piles, $M_x = M_2$ =	5.4	kip-ft	=	64.6	in-kips
Maximum bending moment in the piles, $M_y = M_3$ =	85.4	kip-ft	=	1024.5	in-kips
Maximum shear force in the piles, V_u =	20.4	kips			
Depth below pile head to fixity =	14.2	ft	=	Elev. 88.33	ft
Depth below pile cap to fixity =	12.7	ft	=	Elev. 88.33	ft

C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip elevation.

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.07	ft^3/ft	(from Part A)
δ/ϕ_f =	0.75		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.79		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.04		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.46		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Fill	Glacial Till	
Friction angle, ϕ_f =	34	35	degrees (Ref. 6)
δ =	26	26	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	0.99	1.04	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.86	0.85	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.394	2.733	ksf (at midpoint of soil layer using stratigraphy in Part A)
f_s =	0.144	1.068	ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		Medium Clay		
	Top	Bottom	Top	Bottom	
Undrained shear strength, s_u =	1.343	1.343	0.888	0.888	ksf (Ref. 6)
Adhesion factor, a =	1.00	1.00	1.00	1.00	(Ref. 7 Fig. 10.7.3.8.6b-1, sand over clay, 10b<D<20b)
f_s =	1.343	1.343	0.888	0.888	ksf

Nominal shaft resistance:

$$R_s = f_s A_s$$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = (d × 2) + (b_f × 2) = 39.2 in = 3.3 ft (from Part A)

	Fill	Stiff Clay	Medium Clay	Glacial Till	
Thickness =	6.3	12.6	0.7	20.1	ft (from stratigraphy in Part A)
A_s =	20.6	41.2	2.3	65.7	ft^2
R_s per layer =	3.0	55.3	2.0	70.1	kips

Total R_s = 130.4 kips

Unit toe resistance, using the Kulhawy and Goodman method for hard rock:

$$q_p = 0.33 q_u \text{ for RQD} \leq 70\%$$
$$q_p = 0.80 q_u \text{ for RQD} = 100\%$$ (Ref. 3, Section 7.2.1.4.2)

where:

Rock quality designation, RQD = 9 % (Ref. 13, minimum RQD in borings with lab testing)
Unconfined compressive strength, q_u = 908 ksf (Ref. 14, minimum UCS from lab test data)

q_p = 299.6 ksf

Nominal toe resistance:

$R_p = q_p A_p$ (Ref. 7, Eq. 10.7.3.8.6a-3)

where:

Pile toe area, A_p = 0.09 ft^2 (from Part A)

Total R_p = $\boxed{25.8}$ kips (bedrock)

Nominal and factored geotechnical resistance:

$R_n = R_s + R_p$ (Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = $\boxed{156}$ kips (bedrock)

$R_r = \varphi_{stat} R_n$ (Ref. 7, Eq. 10.7.3.8.6a-1)

where: (Ref. 7 Tbl. 10.5.5.2.3-1)

φ_{stat} = 0.35 for shaft and toe resistance of cohesive soil by the α-method
φ_{stat} = 0.45 for shaft and toe resistance of cohesionless soil by the Nordlund method
φ_{stat} = 0.45 for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Fill	Stiff Clay	Medium Clay	Glacial Till	Pile Tip (Bedrock)	
R_s or R_p =	3.0	55.3	2.0	70.1	25.8	kips
φ_{stat} =	0.45	0.35	0.35	0.45	0.45	(Ref. 7, Table 10.5.5.2.3-1)
R_r per layer =	1.3	19.3	0.7	31.6	11.6	

Total R_r = $\boxed{65}$ kips (bedrock)

Wave Equation Analysis

Nominal and factored geotechnical resistance obtained from GRWEAP at 100% of the steel yield strength (Ref. 15):

R_n = 328 kips (Ref. 15)

φ_{dyn} = 0.5 Assumption 10

R_f = 164 kips

Check that the geotechnical axial resistance Obtained from GRLWEAP (Ref. 15) is sufficient to support axial load on pile:

P_u = 188 kips (from Part B)

R_r = 164 kips < P_u = 188 kips Not OK

D. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K = 0.65 (Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, ℓ = 151.99 in (from Part B)
Minimum radius of gyration, r_s = 2.38 in (from Part A)

Check: $K\ell/r_s$ = 42 < 120 OK

Local buckling and equivalent nominal yield resistance :

$\frac{b}{t} \leq \lambda_r$ (Ref. 7, Eq. 6.9.4.2.1-1) $0.56\sqrt{\frac{E_{st}}{F_y}}$ (Ref. 7, Table 6.9.4.2.1-1)
(Assumption 9)

where:

Half Flange width, b_f = 5.0 in (from Part A)
Flange thickness, t_f = 0.320 in (from Part A)
Steel elastic modulus, E_{st} = 29,000 ksi (from Part A)
Steel yield stress, F_y = 36 ksi (from Part A)

Check: $\frac{b}{t}$ = 15.63 < 15.89 $= 0.56\sqrt{\frac{E_{st}}{F_y}}$

The pile section is a non-slender element (Ref. 7, Article 6.9.4.1)

Calculate equivalent nominal yield resistance:

$P_o = F_y A_g$ (Ref. 7, Article 6.9.4.1)

Equivalent nominal yield resistance, P_o = 339 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{K\ell}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} = 29,000 ksi (from Part A)
Cross-sectional area, A_g = 9.4 in^2 (from Part A)
Effective length factor, K = 0.65 (Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due embedment into pile cap)
Unbraced length, ℓ = 151.99 in (from Part B)
Minimum radius of gyration, r_s = 2.38 in (from Part A)

P_e (flexural) = 1,565 kips

Elastic and flexural torsional buckling resistance:

$\frac{D}{t_w} \leq 150$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D = 8.960 in D= d - (2 × tf)
Web thickness, t_w = 0.315 in (from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$P_n = P_o 0.658^{P_o/P_e}$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$P_n = 0.877P_e$ (Ref. 7, Eq. 6.9.4.1.1-2)

$(P_o / P_{e)top}$ = 0.22 < 2.25, so use Eq. 6.9.4.1.1-1

$P_{n,\ top}$ = 310 kips

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c = 0.50 (Ref. 7, Article 6.5.4.2))

$P_{r,axial}$ = 155 kips

$P_{r_{axial\,/\,flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c = 0.70 (Ref. 7, Article 6.5.4.2))

$P_{r,\ axial/\ flexural}$= 217 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

P_u = 188 kips (from Part B)

$P_{r,\ axial}$ = 155 kips < P_u = 188 kips Not OK

E. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

λ_f = 15.63

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

λ_{pf} = 10.79

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

λr_f = 23.56

Nominal and factored structural resistance in flexure:

$M_n = M_p = 1.5F_yS_y$ if $\lambda_f \le \lambda_{pf}$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$

(Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y = 36 ksi (from Part A)

Steel elastic modulus, E_{st} = 29,000 ksi (from Part A)

Elastic section modulus about x, S_x = 10.7 in^3 (from Part A)

Elastic section modulus about y, S_y = 32.7 in^3 (from Part A)

Plastic section modulus about x, Z_x = 16.2 in^3 (from Part A)

Plastic section modulus about y, Z_y = 36.0 in^3 (from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	508	1,251	in-kips

φ_f = 1.00 (Ref. 7, Article 6.5.4.2)

M_{rx} = 508 in-kips

M_{ry} = 1,251 in-kips

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} = 65 in-kips (from Part B)

M_{uy} = 1025 in-kips (from Part B)

M_{rx} =	508	in-kips	>	M_{ux} =	65	in-kips	OK
M_{ry} =	1,251	in-kips	>	M_{uy} =	1025	in-kips	OK

F. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$ if $P_u/P_{r,\text{ axial/ flexural}} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$ if $P_u/P_{r,\text{ axial/ flexural}} \geq 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\text{ axial/ flexural}}$ = 0.9 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 1.7 > 1.0 Not OK

G. Determine the nominal and factored structural resistance of a single pile in shear.

$$V_p = 0.58F_y d_w t_w$$ (Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	36	ksi	(from Part A)
Web depth, d_w =	8.960	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.315	in	(from Part A)

$$\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	8.960	in	D= d - (2 × tf)
t_w =	0.315	in	(from Part A)

$$1.12\sqrt{\frac{Ek}{F_y}} =$$ 71.1 ksi (Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 71.1 C = 1.0

V_p = 59 kips

V_n = 59 kips

$\varphi_v =$ 1.00 (Ref. 7, Article 6.5.4.2)

$V_r =$ 59 kips

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_u \leq \varphi_v V_n$$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 20.4 kips (from Part B)

$V_r =$ 59 kips > $V_u =$ 20.4 kips OK

CONCLUSIONS

The results of the analysis indicate under proposed strength loading conditions with an estimated pile length of 42.2 feet, a maximum axial force of 188.1 kips including downdrag occurs in Pile 6. This axial force exceeds the factored geotechnical resistance and when evaluated with the corresponding strong axis (y-axis) moment of 85.4 ft-kips (1024.5 in-kips) the limiting axial and flexural interaction ratio of 1.0 is exceeded. The estimated pile length is sufficient for the piles to achieve fixity. This evaluation considers section loss for an additional 50-year design life. A summary table is presented below.

Abutment 2 (old Abutment 1, Pile 6) Summary Table		
Pile Length (ft)		42
Depth below Pile Head to Fixity (ft)		14
Geotechnical Axial Resistance (kips)	Nominal	328
	Factored	164
Structural Axial Compression Resistance (kips)	Nominal	310
	Factored	155
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	42
	Factored	42
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	104
	Factored	104
Combined Axial and Flexural Interaction Ratio		1.7
Shear Resistance (kips)	Nominal	59
	Factored	59

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

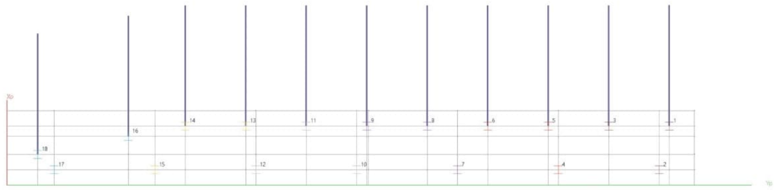


Figure 1b: Profile View of Pile Cap (Transverse)

Note: The pile cap was modeled with an equivalent thickness of 8.0 feet to account for the overall abutment stiffness.

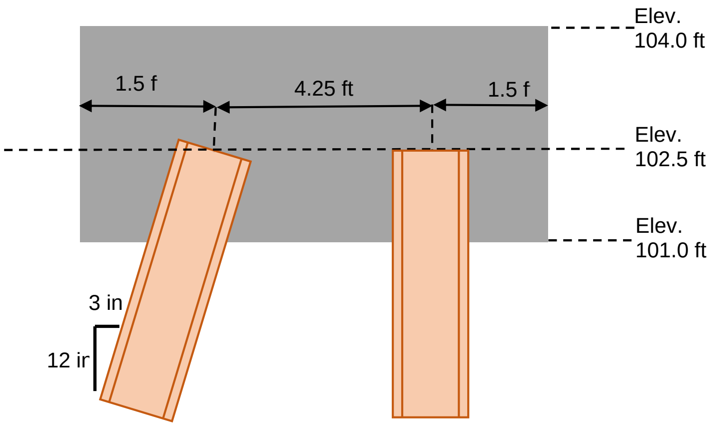


Figure 1c: 3D View of Pile Model

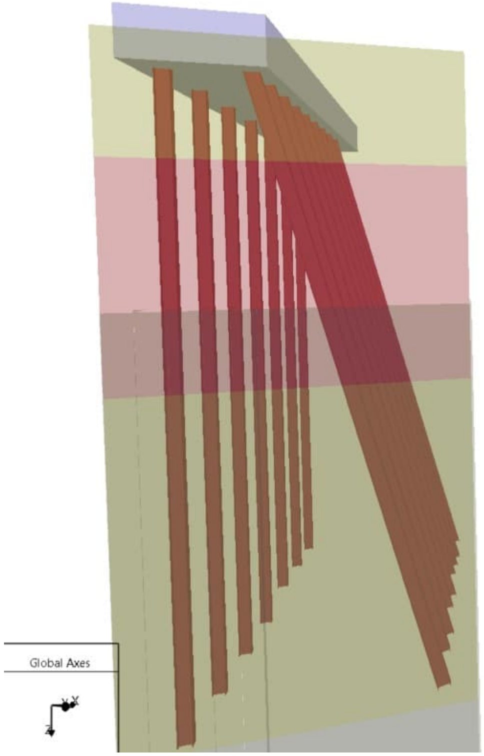
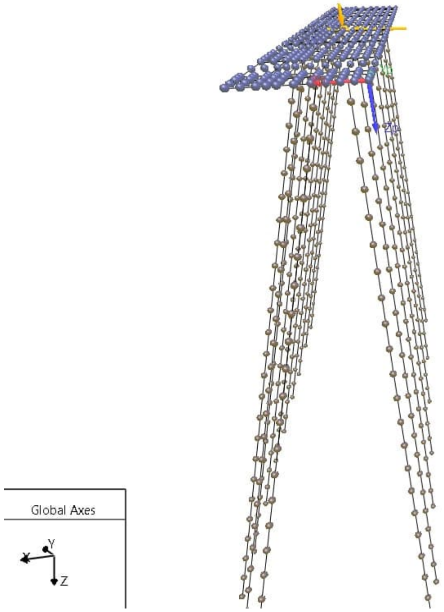


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Existing Fill	1	Cohesionless	101.0	94.7	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	34	127	-	-	-	-	0.65		0.35	144		-	-	-
Stiff Clay	2	Cohesive	94.7	82.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1340	1340	-	-	-
Medium Clay	3	Cohesive	82.1	81.4	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	121	-	-	888	888	0.010	0.030	1.44	1.44	0.45	888	888	-	-	-
Glacial Till	4	Cohesionless	81.4	61.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	1068		-	-	-
Bedrock	5	Rock	61.3	57.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		3809.0	328.0	908,000

Notes:

1. The in-situ soil stratigraphy is based on Sta. 113+91.73 in Profile F-F' (Sheet X Interpretive Subsurface Cross Section F-F'), At the existing Abutment 1, for 41 foot length piles.

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

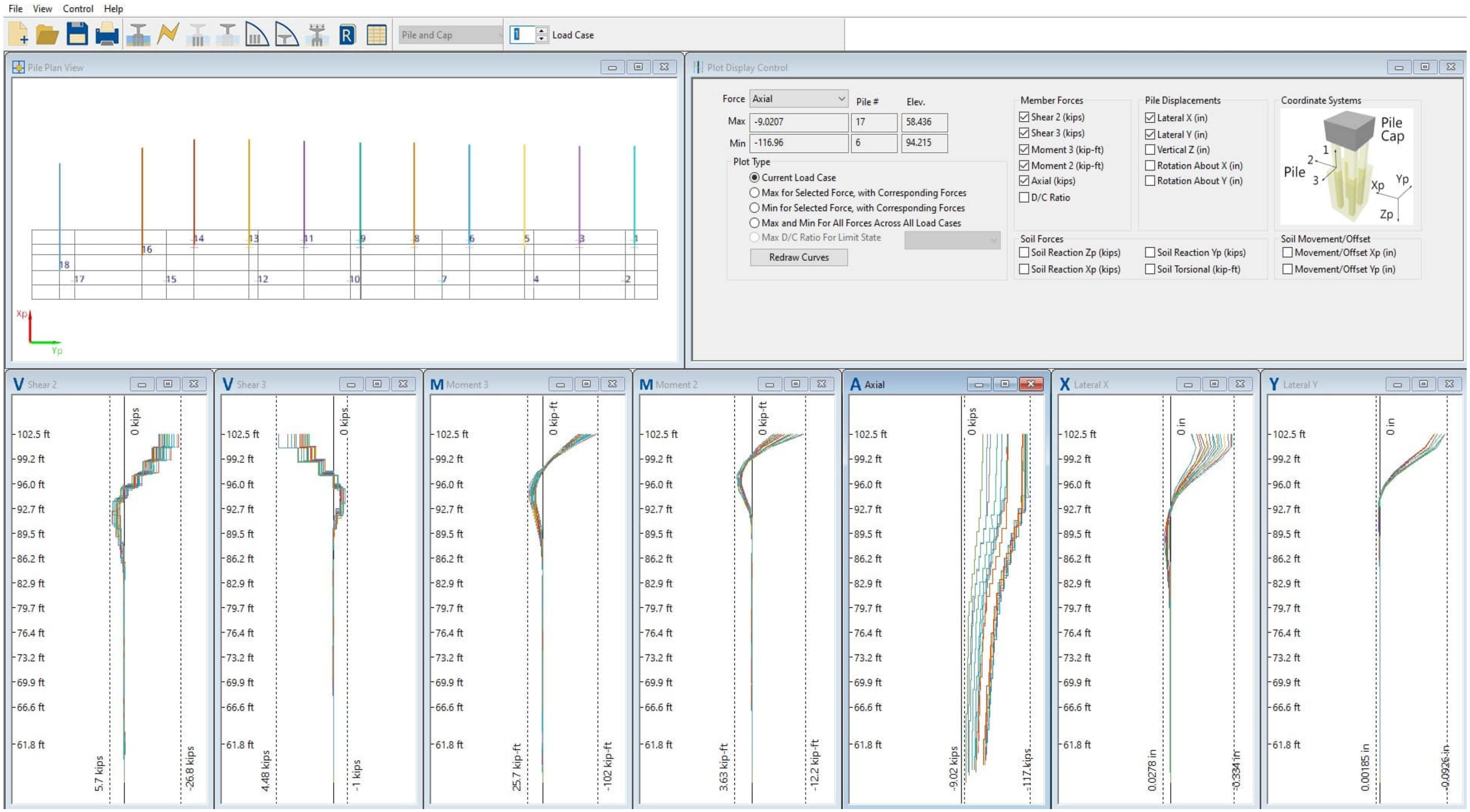
6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

Prepared: DEB
Checked: ATM
Reviewed: CCB

Strength



Force: Axial

		Pile #	Elev.
Max	-9.0207	17	58.436
Min	-116.96	6	94.215

Plot Type
● Current Load Case
○ Max for Selected Force, with Corresponding Forces
○ Min for Selected Force, with Corresponding Forces
○ Max and Min For All Forces Across All Load Cases
○ Max D/C Ratio For Limit State

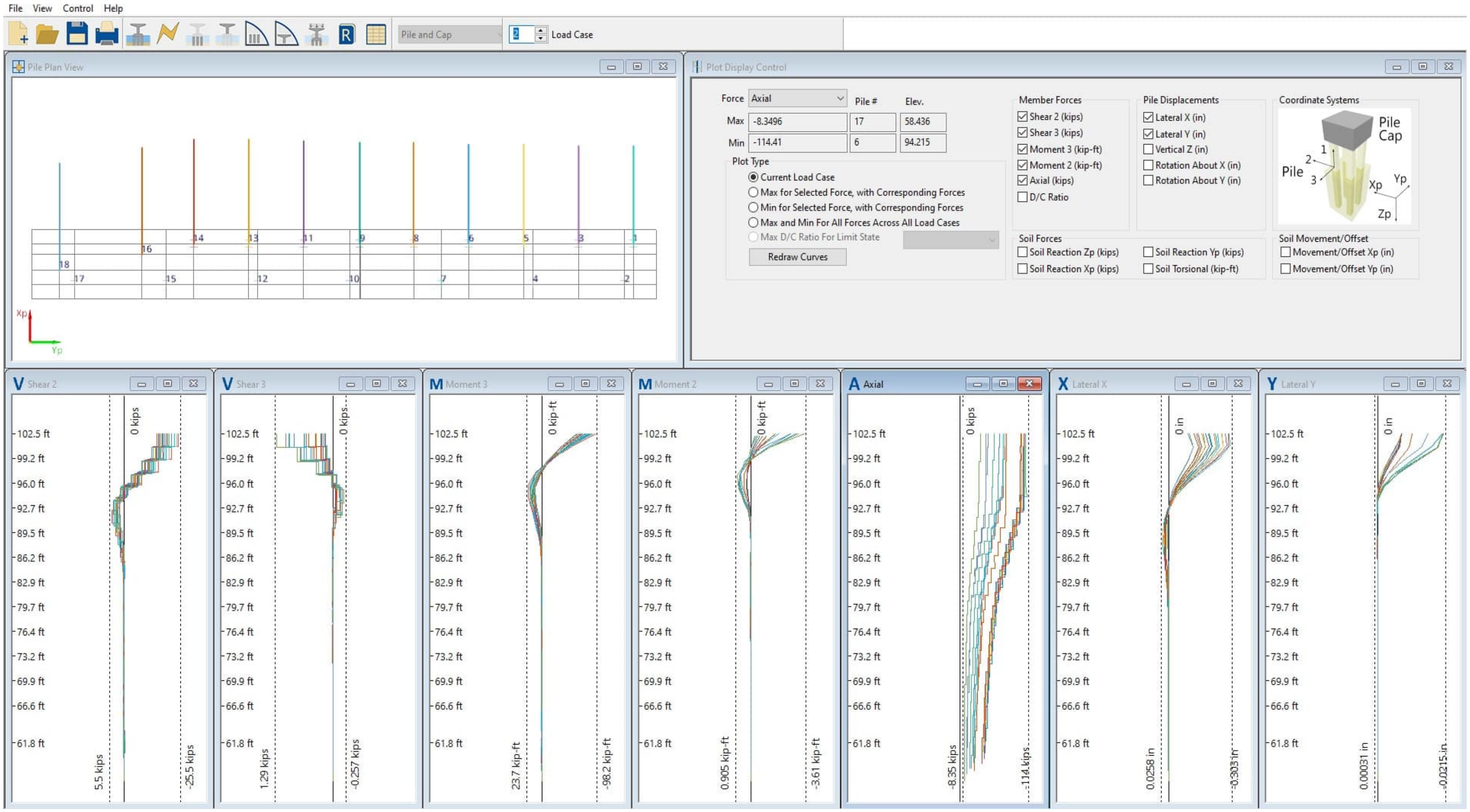
Member Forces
☑ Shear 2 (kips)
☑ Shear 3 (kips)
☑ Moment 3 (kip-ft)
☑ Moment 2 (kip-ft)
☑ Axial (kips)
☐ D/C Ratio

Pile Displacements
☑ Lateral X (in)
☑ Lateral Y (in)
☐ Vertical Z (in)
☐ Rotation About X (in)
☐ Rotation About Y (in)

Soil Forces
☐ Soil Reaction Zp (kips)
☐ Soil Reaction Xp (kips)
☐ Soil Reaction Yp (kips)
☐ Soil Torsional (kip-ft)

Soil Movement/Offset
☐ Movement/Offset Xp (in)
☐ Movement/Offset Yp (in)

Service



Load Case: 2

Force: Axial

		Pile #	Elev.
Max	-8.3496	17	58.436
Min	-114.41	6	94.215

Plot Type
- ● Current Load Case
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Cases
- ○ Max D/C Ratio For Limit State

Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Coordinate Systems

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Soil Movement/Offset
- ☐ Movement/Offset Xp (in)
- ☐ Movement/Offset Yp (in)

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Pier 5 (old Pier 1) H-Pile Analysis - Strength 1 Load Case 1, Intact Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

OBJECTIVE

Determine if Pier 5 located at Sta. 113+13 (old Pier 1), intact HP 10x42 steel piles provide adequate resistance based on geometry from the as-built 1983 record plans, and existing Case 1 loading provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3), and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. PDR Draft 2-3-2023 Hogan Road DDI (Bridge Portion Only) 018595.10
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB titled "Bangor Hogan Road 1983 Existing Bridge Pier 5 Bearing Loads", Dated 9/11/2024.
12. WSP Existing Pier 1 FB-Multipier Soil Properties
13. Email from VHB received on Thursday September 12th, 2024 with subject line "Bangor Hogan Road - 1983 Bridge Pier 5 pile foundation evaluation"
14. WSP calculation package titled Pile Drivability at Pier 5 (existing Pier 1) - HP 10X42

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I Case 1 output
4. FB-Multipier Service I Case 1 output
5. Existing Pier 5 Loads

ASSUMPTIONS

1. The HP 10x42 steel H-piles are being evaluated based on the as-built conditions and geometry shown in Ref. 5.
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 11) were applied at the top of the pier cap at the locations of the corresponding bridge girder bearings. Girder designations are based on VHB designations in Ref. 11.
4. All piles have reinforced pile tips (Ref. 5) and (Ref. 8).
5. All piles are battered at 3 on 12 (0.25) in the positive or negative x-direction.

6. Pile fixity is assumed to occur at the depth where the bending moment is less than 1 kip-ft.
7. Downdrag loading has not been considered in this analysis.
8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcemnt in FB-Multipier.
9. A vertical earth load EV was added based on the difference between the existing ground surface El. of 94.3 feet and the top of Pier 5 footing of El. 90.5, resulting in 3.8 feet of overburden soil. The resulting earth pressure was distributed evenly as point loads at the center of the pile locations along the pile cap.
10. A φdyn of 0.5 is used to determine the factored pile resistance as requested by Maine DOT using (Ref. 7 Table 10.5.5.2.3-1).

CALCULATION

A. Select the pile section parameters.

Pile size: HP 10x42

	Intact Section		
Depth, d =	9.700	in	(Ref. 4, Table 1-4)
Width, b_f =	10.100	in	(Ref. 4, Table 1-4)
Web thickness, t_w =	0.415	in	(Ref. 4, Table 1-4)
Flange thickness, t_f =	0.420	in	(Ref. 4, Table 1-4)
Fillet area =	0.2	in^2	
Section area =	12.4	in^2	(Ref. 4, Table 1-4)
Section area =	0.09	ft^2	
Moment of inertia about the x-axis, I_x =	207	in^4	
Moment of inertia about the y-axis, I_y =	72	in^4	
Radius of gyration about the x-axis, r_x =	4.08	in	
Radius of gyration about the y-axis, r_y =	2.41	in	
Elastic section modulus about the x-axis, S_x =	42.6	in^3	
Elastic section modulus about the y-axis, S_y =	14.3	in^3	
Plastic section modulus about the x-axis, Z_x =	47.5	in^3	
Plastic section modulus about the y-axis, Z_y =	21.8	in^3	

Steel yield stress, F_y =	36	ksi	(Grade A36) Sheet 3 of 53 (Ref. 5)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	90.5	ft	Sheet 13 of 53 (Ref. 5)
Pile cap thickness =	3	ft	Sheet 13 of 53 (Ref. 5)
Pile cap midplane elevation =	89.0	ft	
Pile head elevation = as-modeled pile cap midplane elevation =	89.0	ft	
Base of pile cap elevation =	87.5	ft	
Pile length =	28.6	ft	

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Pier 5 (old Pier 1) H-Pile Analysis - Strength 1 Load Case 1, Intact Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Stiff Clay	1	Cohesive	89.0	82.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	2	Cohesionless	82.1	61.4	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	3	Rock	61.4	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Sta. 113+13 in Profile F-F' (Sheet X Interpretive Subsurface Cross Section G-G').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model:

	(Pile 14)				
Maximum axial force in the piles, P_u =	134.0	kips			
Maximum bending moment in the piles, $M_x = M_2$ =	3.7	kip-ft	=	44.5	in-kips
Maximum bending moment in the piles, $M_y = M_3$ =	0.5	kip-ft	=	5.6	in-kips
Maximum shear force in the piles, V_u =	1.1	kips			
Depth below pile head to fixity =	2.8	ft	=	Elev. 86.17	ft
Depth below pile cap to fixity =	1.3	ft	=	Elev. 86.17	ft

C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip elevation.

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega}$$

(Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.09	ft^3/ft	(from Part A)
δ/ϕ_f =	0.75		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-pile
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.83		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.11		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.62		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Glacial Till	
Friction angle, ϕ_f =	35	degrees (Ref. 6)
δ =	26	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.11	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.85	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	1.111	ksf (at midpoint of soil layer using stratigraphy in Part A)

f_s = 0.464 ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		
	Top	Bottom	
Undrained shear strength, s_u =	1.343	1.343	ksf (Ref. 6)
Adhesion factor, a =	0.88	0.88	(Ref. 7 Fig. 10.7.3.8.6b-1, firm to stiff clay, D >=20b)
f_s =	1.182	1.182	ksf

Nominal shaft resistance:

$$R_s = f_s A_s$$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = (d × 2) + (b_f × 2) = 39.6 in = 3.3 ft (from Part A)

	Stiff Clay	Glacial Till	
Thickness =	6.9	20.7	ft (from stratigraphy in Part A)
A_s =	22.8	68.3	ft^2
R_s per layer =	26.9	31.7	kips

Total R_s = 58.6 kips

Unit toe resistance, using the Kulhawy and Goodman method:

$$q_p = 0.33q_u \text{ for RQD} \leq 70\%$$
$$q_p = 0.80q_u \text{ for RQD} = 100\%$$

(Ref. 3, Section 7.2.1.4.1)

where:

Rock quality designation, RQD = 9 % (Ref. 9, minimum RQD in borings with lab testing)
Unconfined compressive strength, q_u = 908 ksf (Ref. 10, minimum UCS from lab test data)

q_p = $\boxed{299.6}$ ksf

Nominal toe resistance:

$$R_p = q_p A_p$$

(Ref. 7, Eq. 10.7.3.8.6a-3)

where:

Pile toe area, A_p = 0.09 ft^2 (from Part A)

Total R_p = $\boxed{25.8}$ kips (bedrock)

Nominal and factored geotechnical resistance:

$$R_n = R_s + R_p$$

(Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = $\boxed{84}$ kips (bedrock)

$$R_r = \varphi_{stat} R_n$$

(Ref. 7, Eq. 10.7.3.8.6a-1)

where:	φ_{stat} =	0.35	for shaft and toe resistance of cohesive soil by the α-method
(Ref. 7 Tbl. 10.5.5.2.3-1)	φ_{stat} =	0.45	for shaft and toe resistance of cohesionless soil by the Nordlund method
	φ_{stat} =	0.45	for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Stiff Clay	Glacial Till	Pile Tip (Bedrock)	
R_s or R_p =	26.9	31.7	25.8	kips
φ_{stat} =	0.35	0.45	0.45	(Ref. 7, Table 10.5.5.2.3-1)
R_r per layer =	9.4	14.3	11.6	

Total R_r = $\boxed{35}$ kips (bedrock)

Wave Equation Analysis

Nominal and factored geotechnical resistance obtained from GRWEAP at 100% of the steel yield strength (Ref. 14):

$R_n =$ 310 kips (Ref. 14)

$\varphi_{dyn} =$ 0.5 (Assumption 10)

$R_f =$ 155 kips

Check that the geotechnical axial resistance is sufficient to support axial load on pile:

$P_u =$ 134 kips (from Part B)

$R_r =$ 155 kips $>$ $P_u =$ 134 kips OK

D. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K = 0.65 (Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)

Unbraced length, l = 16.00 in (from Part B)

Minimum radius of gyration, r_s = 2.41 in (from Part A)

Check: $Kl/r_s =$ 4 $<$ 120 OK

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \leq \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $$0.56\sqrt{\frac{E_{st}}{F_y}}$$ (Ref. 7, Table 6.9.4.2.1-1)
(Assumption 9)

where:

Half Flange width, b_f = 5.1 in (from Part A)

Flange thickness, t_f = 0.420 in (from Part A)

Steel elastic modulus, E_{st} = 29,000 ksi (from Part A)

Steel yield stress, F_y = 36 ksi (from Part A)

Check:

$$\frac{b}{t} = 12.02 < 15.89 = 0.56\sqrt{\frac{E_{st}}{F_y}}$$

The pile section is a non-slender element (Ref. 7, Article 6.9.4.1)

Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1)

Equivalent nominal yield resistance, P_o = 446 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	12.4	in^2	(from Part A)
Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translatior condition at pile head due to embedment into pile cap)
Unbraced length, l =	16.00	in	(from Part B)
Minimum radius of gyration, r_s =	2.41	in	(from Part A)
P_e (flexural) =	190,930	kips	

Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	8.860	in	D= d - (2 × tf)
Web thickness, t_w =	0.415	in	(from Part A)

Check: D/t_w = 21 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$P_n = P_o 0.658^{P_o/P_e}$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$P_n = 0.877P_e$ (Ref. 7, Eq. 6.9.4.1.1-2)

$P_o / P_e =$ 0.00 < 2.25, so use Eq. 6.9.4.1.1-1

$P_n =$ 446 kips

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

$\varphi_c =$ 0.50 (Ref. 7, Article 6.5.4.2))

$P_r =$ 223 kips

$P_{r_{axial\,/\,flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

$\varphi_c =$ 0.70 (Ref. 7, Article 6.5.4.2))

$P_{r,\text{ axial/ flexural}} =$ 312 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

$P_u =$ 134 kips (from Part B)

$P_{r,\text{ axial}} =$ 223 kips > $P_u =$ 134 kips OK

E. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

$\lambda_f =$ 12.02

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

$\lambda_{pf} =$ 10.79

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

$\lambda r_f =$ 23.56

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \leq \lambda_{pf}$$

(Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \leq \lambda_{rf}$$

(Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	36	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	42.6	in^3	(from Part A)
Elastic section modulus about y, S_y =	14.3	in^3	(from Part A)
Plastic section modulus about x, Z_x =	47.5	in^3	(from Part A)
Plastic section modulus about y, Z_y =	21.8	in^3	(from Part A)

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Pier 5 (old Pier 1) H-Pile Analysis - Strength 1 Load Case 1, Intact Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	1,693	759	in-kips

φ_f = 1.00 (Ref. 7, Article 6.5.4.2)

M_{rx} = 1,693 in-kips

M_{ry} = 759 in-kips

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} = 44 in-kips (from Part B)

M_{uy} = 6 in-kips (from Part B)

M_{rx} =	1,693	in-kips	>	M_{ux} =	44	in-kips	OK
M_{ry} =	759	in-kips	>	M_{uy} =	6	in-kips	OK

F. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

if $P_u/P_{r,\ axial/\ flexural} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

if $P_u/P_{r,\ axial/\ flexural} \geq 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\ axial/\ flexural}$ = 0.4 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.5 < 1.0 OK

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
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G. Determine the nominal and factored structural resistance of a single pile in shear.

$$V_p = 0.58 F_y d_w t_w$$ (Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	36	ksi	(from Part A)
Web depth, d_w =	8.860	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.415	in	(from Part A)

$$\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	8.860	in	D= d - (2 × tf)
t_w =	0.415	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	71.1	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 21 < 71.1 C = 1.0

V_p =	77	kips	
V_n =	77	kips	
φ_v =	1.00		(Ref. 7, Article 6.5.4.2)
V_r =	77	kips	
V_u =	1.1	kips	(from Part B)

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_u \leq \varphi_v V_n$$ (Ref. 7, Eq. 6.10.9.1-1)

V_u = 1.1 kips (from Part B)

V_r = 77 kips > V_u = 1.1 kips OK

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Pier 5 (old Pier 1) H-Pile Analysis - Strength 1 Load Case 1, Intact Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

CONCLUSIONS

The results of the analysis indicate that under the Case 1, Strength I load case, and an estimated pile length of 29 feet, a maximum axial force of 134 kips, the corresponding strong axis (x-axis) moment of 44.5 in-kips (3.7 ft-kips) occurs in the HP10x42 piles at Pier 5 (old Pier 1) in Pile 14 of the 1983 portion of the Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity. This evaluation does not consider downdrag. A summary table is presented below.

Pier 5 (Old Pier 1, Pile 14) Summary Table		
Pile Length (ft)		29
Depth below Pile Head to Fixity (ft)		3
Geotechnical Axial Resistance (kips)	Nominal	310
	Factored	155
Structural Axial Resistance (kips)	Nominal	446
	Factored	223
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	141
	Factored	141
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	63
	Factored	63
Combined Axial and Flexural Interaction Ratio		0.5
Shear Resistance (kips)	Nominal	77
	Factored	77

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

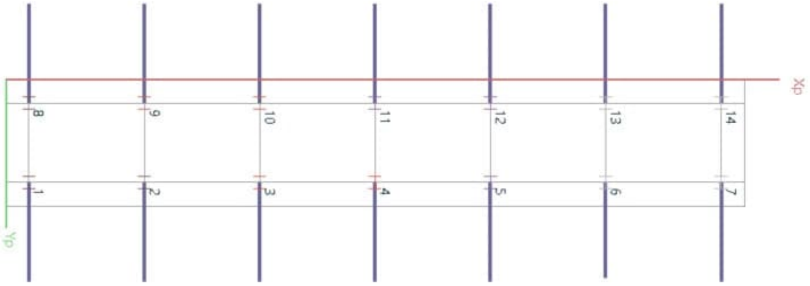


Figure 1b: Profile View of Pile Cap (Transverse)

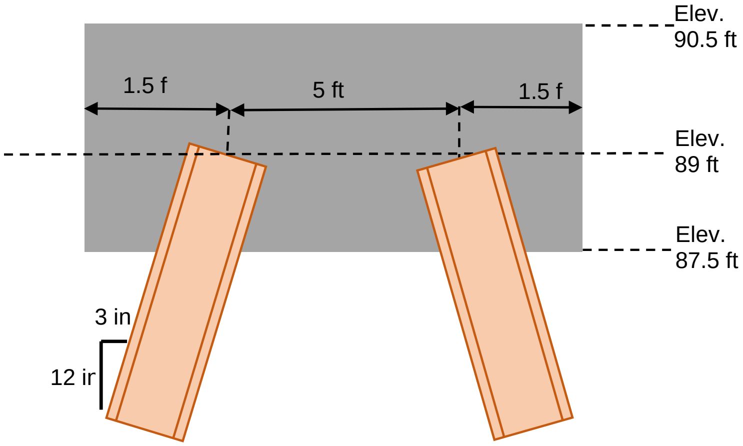


Figure 1c: 3D View of Pile Model

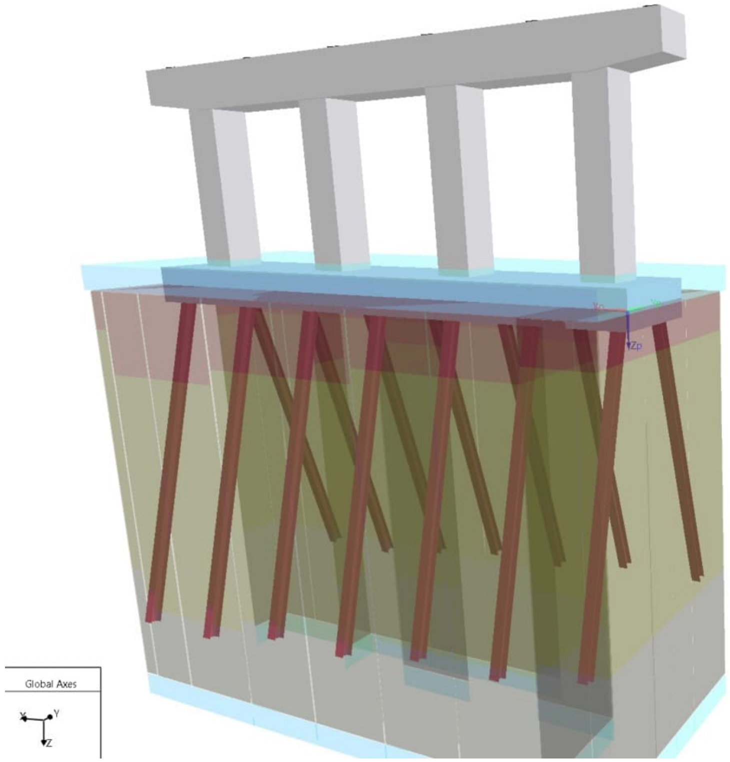
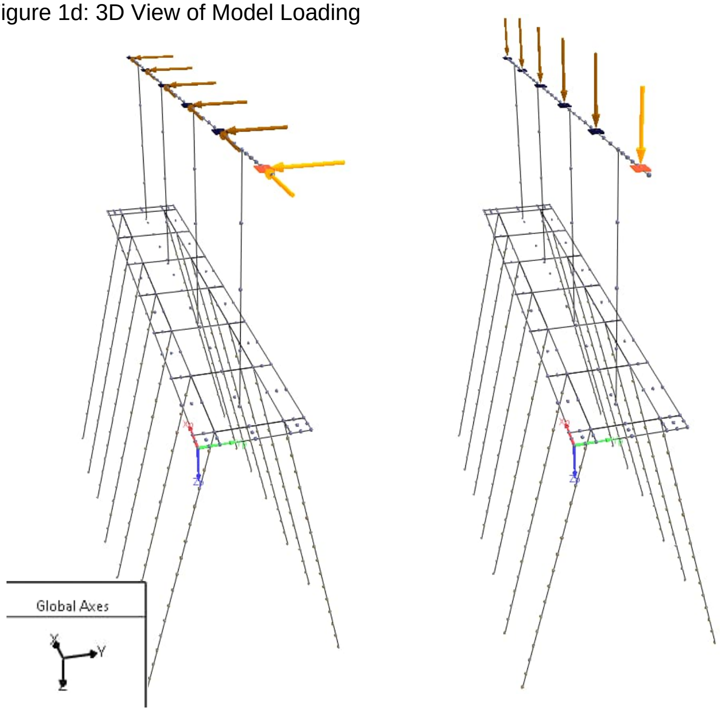


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Stiff Clay	1	Cohesive	89.0	82.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1182	1182	-	-	-
Glacial Till	2	Cohesionless	82.1	61.4	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	464		-	-	-
Bedrock	3	Rock	61.4	55.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		3600	310	908,000

Notes:

1. The in-situ soil stratigraphy is based on Sta. 113+31 in Profile G-G' (Sheet X Interpretive Subsurface Cross Section G-G'), At the existing Pier 1, for 31 foot length piles.

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

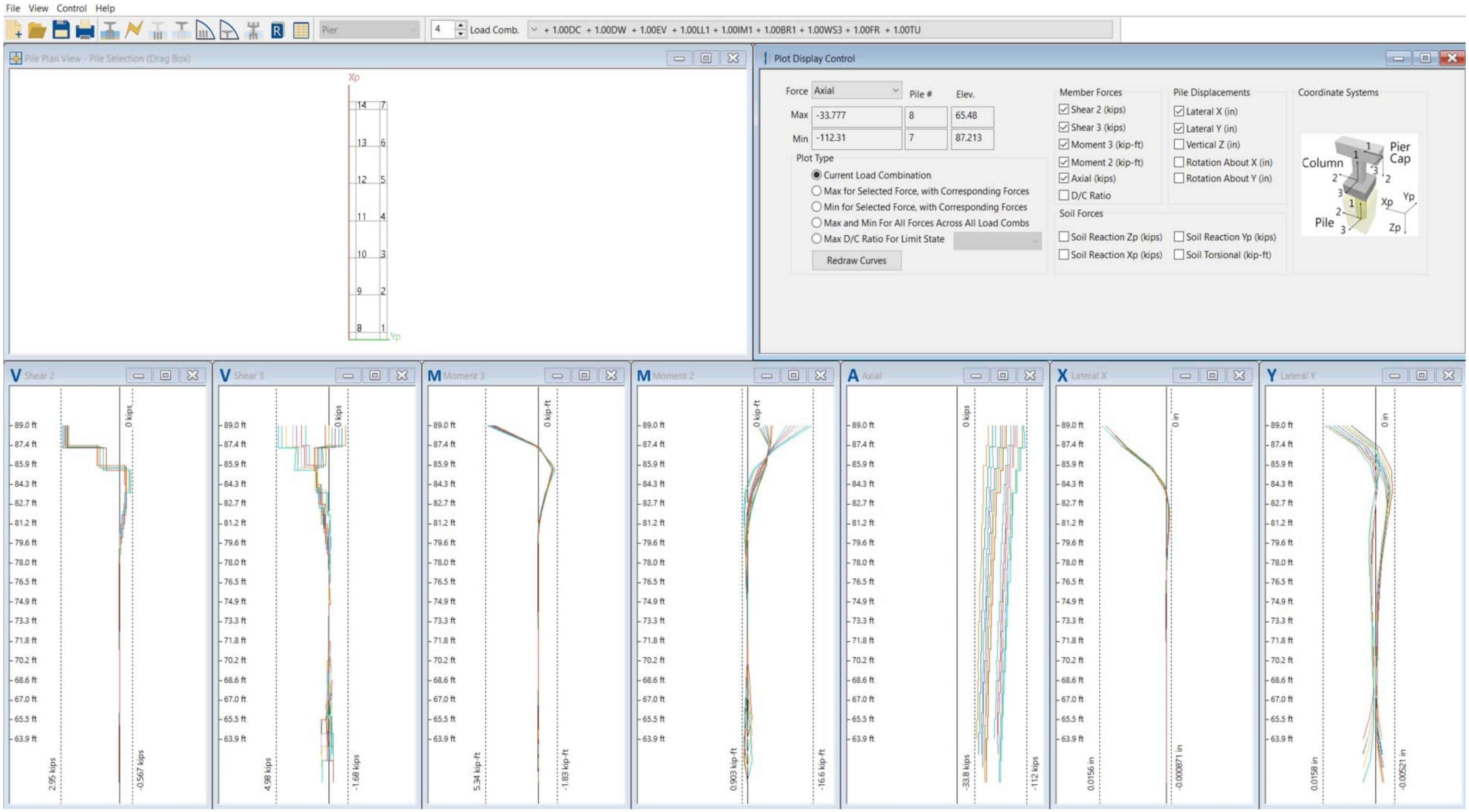
6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

7. Unit tip resistance values for rock obtained from WSP calculation package titled Pile Drivability at Pier 5 (existing Pier 1) - HP 10x42.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

Prepared: DEB
Checked: ATM
Reviewed: CCB

Case 1 Service I



File View Control Help

Pier | 4 | Load Comb. | + 1.00DC + 1.00DW + 1.00EV + 1.00LL1 + 1.00IM1 + 1.00BR1 + 1.00WS3 + 1.00FR + 1.00TU

Pile Plan View - Pile Selection (Drag Box)

Plot Display Control

Force: Axial

	Force	Pile #	Elev.
Max	-33.777	8	65.48
Min	-112.31	7	87.213

Plot Type

- ● Current Load Combination
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Combs
- ○ Max D/C Ratio For Limit State

Redraw Curves

Member Forces

- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements

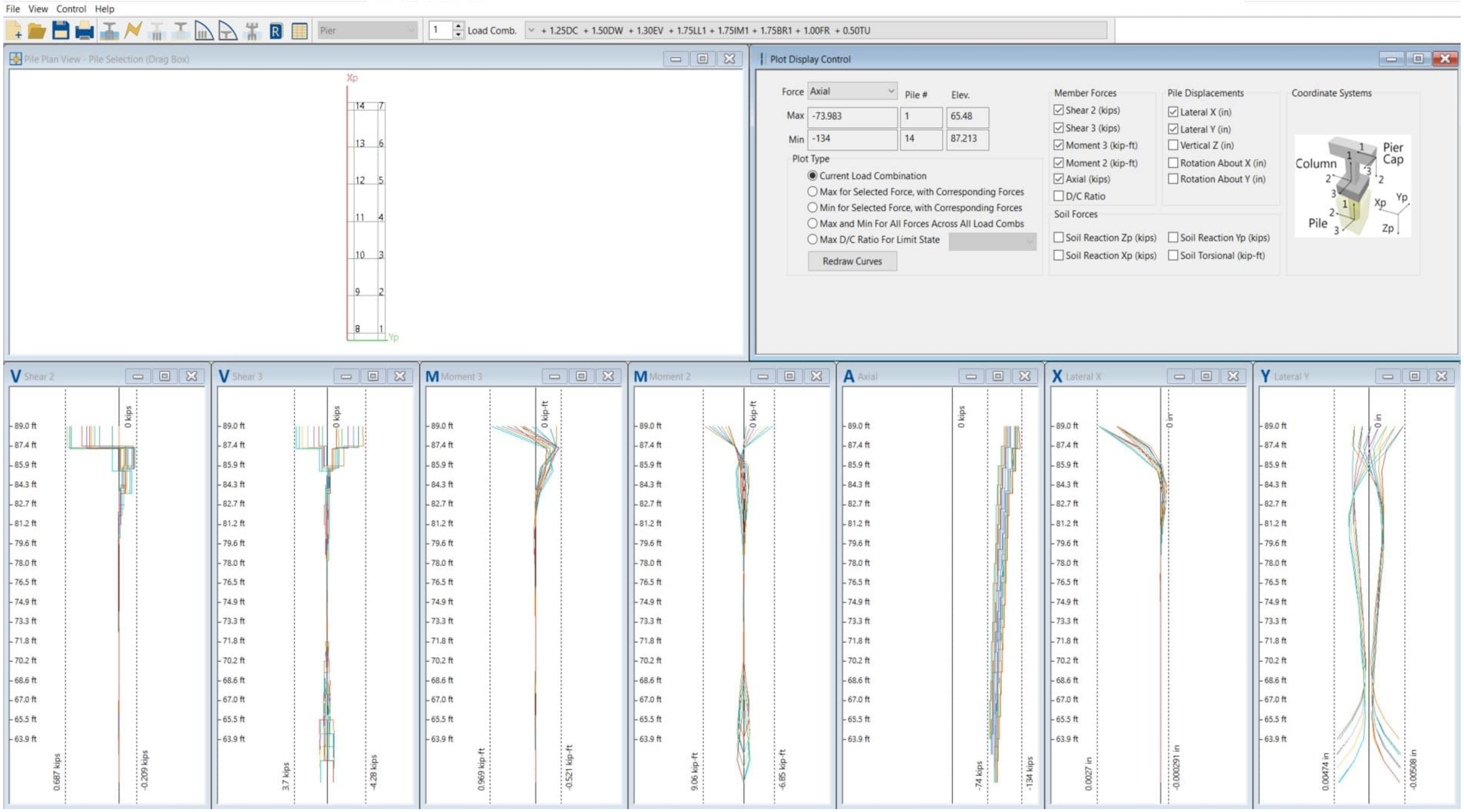
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces

- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Torsional (kip-ft)

Coordinate Systems

Case 1 Strength I



1 Load Comb. + 1.25DC + 1.50DW + 1.30EV + 1.75LL1 + 1.75IM1 + 1.75BR1 + 1.00FR + 0.50TU

Force: Axial

	Force	Pile #	Elev.
Max	-73.983	1	65.48
Min	-134	14	87.213

Plot Type
- ● Current Load Combination
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Combs
- ○ Max D/C Ratio For Limit State

Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Bangor Hogan Road 1983 Existing Bridge Pier 5 Bearing Loads

Unfactored Bearing Loads **9/11/2024**

Girder	Governing Force	DC	DW	HL93	TU	FR	TU	FR
					Case 1		Case 2	
G1	FX (kips)			0.0 k	10.2 k	1.7 k	-17.6 k	-1.7 k
G2	FX (kips)			-3.9 k	9.5 k	2.2 k	-16.4 k	-2.2 k
G3	FX (kips)			-5.1 k	8.8 k	2.1 k	-15.2 k	-2.1 k
G4	FX (kips)			-5.7 k	8.3 k	2.1 k	-14.3 k	-2.1 k
G5	FX (kips)			-5.1 k	7.9 k	2.1 k	-13.7 k	-2.1 k
G6	FX (kips)			-5.1 k	7.6 k	1.6 k	-13.2 k	-1.6 k
G1	FY (kips)			-2.8 k	9.3 k	2.0 k	-16.1 k	-2.0 k
G2	FY (kips)			-2.2 k	8.7 k	2.4 k	-15.0 k	-2.4 k
G3	FY (kips)			-3.7 k	8.1 k	2.4 k	-13.9 k	-2.4 k
G4	FY (kips)			-4.2 k	7.6 k	2.3 k	-13.1 k	-2.3 k
G5	FY (kips)			-3.6 k	7.2 k	2.4 k	-12.5 k	-2.4 k
G6	FY (kips)			0.0 k	7.0 k	1.8 k	-12.1 k	-1.8 k
G1	FZ (kips)	72.9 k	11.2 k	2.8 k				
G2	FZ (kips)	85.3 k	20.3 k	29.4 k				
G3	FZ (kips)	84.3 k	20.5 k	66.4 k				
G4	FZ (kips)	84.1 k	20.6 k	71.6 k				
G5	FZ (kips)	84.5 k	20.6 k	64.7 k				
G6	FZ (kips)	68.9 k	12.2 k	25.1 k				

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Pier 5 (old Pier 1) H-Pile Analysis - Strength 1 Load Case 2, Intact Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

OBJECTIVE

Determine if Pier 5 located at Sta. 113+13 (old Pier 1), intact HP 10x42 steel piles provide adequate resistance based on geometry from the as-built 1983 record plans, and existing Case 2 loading provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3), and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. PDR Draft 2-3-2023 Hogan Road DDI (Bridge Portion Only) 018595.10
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB titled "Bangor Hogan Road 1983 Existing Bridge Pier 5 Bearing Loads", Dated 9/11/2024.
12. WSP Existing Pier 1 FB-Multipier Soil Properties
13. Email from VHB received on Thursday September 12th, 2024 with subject line "Bangor Hogan Road - 1983 Bridge Pier 5 pile foundation evaluation"
14. WSP calculation package titled Pile Drivability at Pier 5 (existing Pier 1) - HP 10X42

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I Case 2 output
4. FB-Multipier Service I Case 2 output
5. Existing Pier 5 Loads

ASSUMPTIONS

1. The HP 10x42 steel H-piles are being evaluated based on the as-built conditions and geometry shown in Ref. 5.
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 11) were applied at the top of the pier cap at the locations of the corresponding bridge girder bearings. Girder designations are based on VHB designations in Ref. 11.
4. All piles have reinforced pile tips (Ref. 5) and (Ref. 8).
5. All piles are battered at 3 on 12 (0.25) in the positive or negative x-direction.

6. Pile fixity is assumed to occur at the depth where the bending moment is less than 1 kip-ft.
7. Downdrag loading has not been considered in this analysis.
8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcemnt in FB-Multipier.
9. A vertical earth load EV was added based on the difference between the existing ground surface El. of 94.3 feet and the top of Pier 5 footing of El. 90.5, resulting in 3.8 feet of overburden soil. The resulting earth pressure was distributed evenly as point loads at the center of the pile locations along the pile cap.
10. A φdyn of 0.5 is used to determine the factored pile resistance as requested by Maine DOT using (Ref. 7 Table 10.5.5.2.3-1).

CALCULATION

A. Select the pile section parameters.

Pile size: HP 10x42

	Intact Section		
Depth, d =	9.700	in	(Ref. 4, Table 1-4)
Width, b_f =	10.100	in	(Ref. 4, Table 1-4)
Web thickness, t_w =	0.415	in	(Ref. 4, Table 1-4)
Flange thickness, t_f =	0.420	in	(Ref. 4, Table 1-4)
Fillet area =	0.2	in^2	
Section area =	12.4	in^2	(Ref. 4, Table 1-4)
Section area =	0.09	ft^2	
Moment of inertia about the x-axis, I_x =	207	in^4	
Moment of inertia about the y-axis, I_y =	72	in^4	
Radius of gyration about the x-axis, r_x =	4.08	in	
Radius of gyration about the y-axis, r_y =	2.41	in	
Elastic section modulus about the x-axis, S_x =	42.6	in^3	
Elastic section modulus about the y-axis, S_y =	14.3	in^3	
Plastic section modulus about the x-axis, Z_x =	47.5	in^3	
Plastic section modulus about the y-axis, Z_y =	21.8	in^3	

Steel yield stress, F_y =	36	ksi	(Grade A36) Sheet 3 of 53 (Ref. 5)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	90.5	ft	Sheet 13 of 53 (Ref. 5)
Pile cap thickness =	3	ft	Sheet 13 of 53 (Ref. 5)
Pile cap midplane elevation =	89.0	ft	
Pile head elevation = as-modeled pile cap midplane elevation =	89.0	ft	
Base of pile cap elevation =	87.5	ft	
Pile length =	27.4	ft	

CALCULATIONS

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Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Stiff Clay	1	Cohesive	89.0	83.4	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	2	Cohesionless	83.4	62.6	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	3	Rock	62.6	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Sta. 113+13 in Profile F-F' (Sheet X Interpretive Subsurface Cross Section G-G').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model: (Pile 12)

Maximum axial force in the piles, P_u = 163.8 kips

Maximum bending moment in the piles, $M_x = M_2$ = 30.4 kip-ft = 364.3 in-kips

Maximum bending moment in the piles, $M_y = M_3$ = 11.6 kip-ft = 138.7 in-kips

Maximum shear force in the piles, V_u = 6.2 kips

Depth below pile head to fixity = 6.4 ft = Elev. 82.62 ft

Depth below pile cap to fixity = 4.9 ft = Elev. 82.62 ft

C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip elevation.

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.09	ft^3/ft	(from Part A)
δ/ϕ_f =	0.75		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-pile
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.83		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.11		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.62		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Glacial Till	
Friction angle, ϕ_f =	35	degrees (Ref. 6)
δ =	26	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.11	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.85	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and $\delta/\varphi f$)
Vertical effective stress, σ'_v =	1.037	ksf (at midpoint of soil layer using stratigraphy in Part A)

f_s = 0.433 ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		
	Top	Bottom	
Undrained shear strength, s_u =	1.343	1.343	ksf (Ref. 6)
Adhesion factor, a =	0.88	0.88	(Ref. 7 Fig. 10.7.3.8.6b-1, firm to stiff clay, D >=20b)
f_s =	1.182	1.182	ksf

Nominal shaft resistance:

$$R_s = f_s A_s$$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = $(d \times 2) + (b_f \times 2)$ = 39.6 in = 3.3 ft (from Part A)

	Stiff Clay	Glacial Till	
Thickness =	5.6	20.8	ft (from stratigraphy in Part A)
A_s =	18.5	68.6	ft^2
R_s per layer =	21.8	29.7	kips

Total R_s = 51.6 kips

Unit toe resistance, using the Kulhawy and Goodman method:

$$q_p = 0.33 q_u \text{ for RQD} \leq 70\%$$
$$q_p = 0.80 q_u \text{ for RQD} = 100\%$$

(Ref. 3, Section 7.2.1.4.1)

where:

Rock quality designation, RQD = 9 % (Ref. 9, minimum RQD in borings with lab testing)
Unconfined compressive strength, q_u = 908 ksf (Ref. 10, minimum UCS from lab test data)

q_p = $\boxed{299.6}$ ksf

Nominal toe resistance:

$$R_p = q_p A_p$$

(Ref. 7, Eq. 10.7.3.8.6a-3)

where:

Pile toe area, A_p = 0.09 ft^2 (from Part A)

Total R_p = $\boxed{25.8}$ kips (bedrock)

Nominal and factored geotechnical resistance:

$$R_n = R_s + R_p$$

(Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = $\boxed{77}$ kips (bedrock)

$$R_r = \varphi_{stat} R_n$$

(Ref. 7, Eq. 10.7.3.8.6a-1)

where: (Ref. 7 Tbl. 10.5.5.2.3-1)

φ_{stat} = 0.35 for shaft and toe resistance of cohesive soil by the α-method
φ_{stat} = 0.45 for shaft and toe resistance of cohesionless soil by the Nordlund method
φ_{stat} = 0.45 for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Stiff Clay	Glacial Till	Pile Tip (Bedrock)	
R_s or R_p =	21.8	29.7	25.8	kips
φ_{stat} =	0.35	0.45	0.45	(Ref. 7, Table 10.5.5.2.3-1)
R_r per layer =	7.6	13.4	11.6	

Total R_r = $\boxed{33}$ kips (bedrock)

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Pier 5 (old Pier 1) H-Pile Analysis - Strength 1 Load Case 2, Intact Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Wave Equation Analysis

Nominal and factored geotechnical resistance obtained from GRWEAP at 100% of the steel yield strength (Ref. 14):

$R_n =$ 310 kips (Ref. 14)

$\varphi_{dyn} =$ 0.5 (Assumption 10)

$R_f =$ 155 kips

Check that the geotechnical axial resistance is sufficient to support axial load on pile:

$P_u =$ 164 kips (from Part B)

$R_r =$ 155 kips < $P_u =$ 164 kips Not OK

D. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K = 0.65 (Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)

Unbraced length, l = 58.62 in (from Part B)

Minimum radius of gyration, r_s = 2.41 in (from Part A)

Check: $Kl/r_s =$ 16 < 120 OK

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \leq \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $$0.56\sqrt{\frac{E_{st}}{F_y}}$$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 9)

where:

Half Flange width, b_f = 5.1 in (from Part A)

Flange thickness, t_f = 0.420 in (from Part A)

Steel elastic modulus, E_{st} = 29,000 ksi (from Part A)

Steel yield stress, F_y = 36 ksi (from Part A)

Check:

$$\frac{b}{t} = 12.02 < 15.89 = 0.56\sqrt{\frac{E_{st}}{F_y}}$$

The pile section is a non-slender element (Ref. 7, Article 6.9.4.1)

Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1)

Equivalent nominal yield resistance, P_o = 446 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	12.4	in^2	(from Part A)
Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translatior condition at pile head due to embedment into pile cap)
Unbraced length, l =	58.62	in	(from Part B)
Minimum radius of gyration, r_s =	2.41	in	(from Part A)
P_e (flexural) =	14,230	kips	

Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	8.860	in	D= d - (2 × tf)
Web thickness, t_w =	0.415	in	(from Part A)

Check: D/t_w = 21 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$$P_n = P_o 0.658^{P_o/P_e}$$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$$P_n = 0.877 P_e$$ (Ref. 7, Eq. 6.9.4.1.1-2)

P_o / P_e = 0.03 < 2.25, so use Eq. 6.9.4.1.1-1

P_n = 441 kips

$$P_{r_{axial}} = \varphi_c P_n$$ (Ref. 7 Eq. 6.9.2.1-1)

$\varphi_c =$ 0.50 (Ref. 7, Article 6.5.4.2))
$P_r =$ 220 kips

$$P_{r_{axial\,/\,flexural}} = \varphi_c P_n$$ (Ref. 7 Eq. 6.9.2.1-1)

$\varphi_c =$ 0.70 (Ref. 7, Article 6.5.4.2))
$P_{r,\ axial/\ flexural} =$ 308 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

$P_u =$ 164 kips (from Part B)

$P_{r,\ axial} =$ 220 kips > $P_u =$ 164 kips OK

E. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

$\lambda_f =$ 12.02

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

$\lambda_{pf} =$ 10.79

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

$\lambda r_f =$ 23.56

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \leq \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \leq \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	36	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	42.6	in^3	(from Part A)
Elastic section modulus about y, S_y =	14.3	in^3	(from Part A)
Plastic section modulus about x, Z_x =	47.5	in^3	(from Part A)
Plastic section modulus about y, Z_y =	21.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	1,693	759	in-kips

φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	1,693	in-kips	
M_{ry} =	759	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} =	364	in-kips	(from Part B)
M_{uy} =	139	in-kips	(from Part B)

M_{rx} =	1,693	in-kips	>	M_{ux} =	364	in-kips	OK
M_{ry} =	759	in-kips	>	M_{uy} =	139	in-kips	OK

F. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$ if $P_u/P_{r,\text{ axial/ flexural}} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$ if $P_u/P_{r,\text{ axial/ flexural}} \geq 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\text{ axial/ flexural}}$ = 0.5 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.9 < 1.0 OK

G. Determine the nominal and factored structural resistance of a single pile in shear.

$V_p = 0.58F_y d_w t_w$ (Ref. 7, 6.10.9.2-2))

$V_n = CV_p = V_{cr}$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	36	ksi	(from Part A)
Web depth, d_w =	8.860	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.415	in	(from Part A)

$$\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	8.860	in	D= d - (2 × tf)
t_w =	0.415	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	71.1	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 21 < 71.1 C = 1.0

V_p =	77	kips	
V_n =	77	kips	
φ_v =	1.00		(Ref. 7, Article 6.5.4.2)
V_r =	77	kips	
V_u =	6.2	kips	(from Part B)

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

V_u = 6.2 kips (from Part B)

V_r = 77 kips > V_u = 6.2 kips OK

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Pier 5 (old Pier 1) H-Pile Analysis - Strength 1 Load Case 2, Intact Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

CONCLUSIONS

The results of the analysis indicate that under the Case 2, Strength I load case, and an estimated pile length of 27 feet, a maximum axial force of 163.8 kips, the corresponding strong axis (x-axis) moment of 364.3 in-kips (30.4 ft-kips) occurs in the HP10x42 piles at Pier 5 (old Pier 1) in Pile 12 of the 1983 portion of the Hogan Road Bridge. The corresponding axial force of 163.8 kips (determined using FB-MultiPier) is greater than the geotechnical resistance of the existing HP 10x42 piles determined from a wave equation analysis using the intact pile section properties, typical pile driving equipment used during the time of construction, and soil and rock conditions at the site. The estimated pile length is sufficient for the piles to achieve fixity. This evaluation does not consider downdrag. A summary table is presented below.

Pier 5 (Old Pier 1, Pile 12) Summary Table		
Pile Length (ft)		27
Depth below Pile Head to Fixity (ft)		6
Geotechnical Axial Resistance (kips)	Nominal	310
	Factored	155
Structural Axial Resistance (kips)	Nominal	441
	Factored	220
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	141
	Factored	141
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	63
	Factored	63
Combined Axial and Flexural Interaction Ratio		0.9
Shear Resistance (kips)	Nominal	77
	Factored	77

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

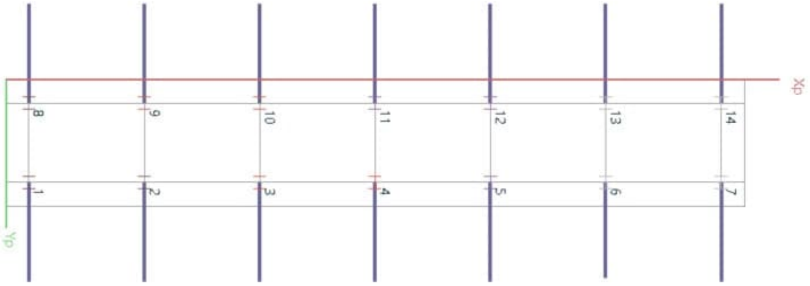


Figure 1b: Profile View of Pile Cap (Transverse)

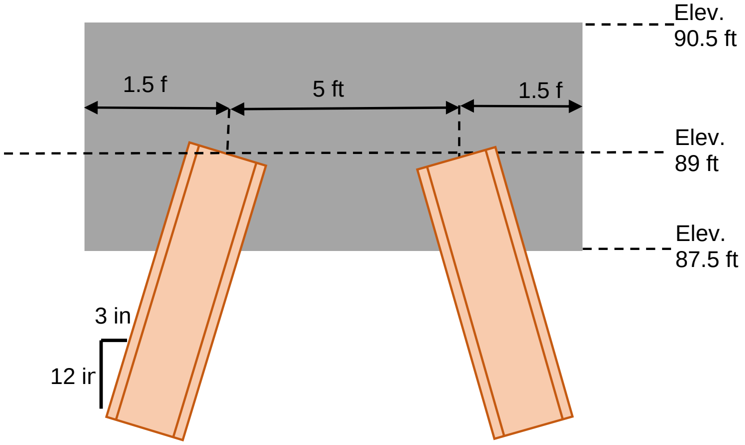


Figure 1c: 3D View of Pile Model

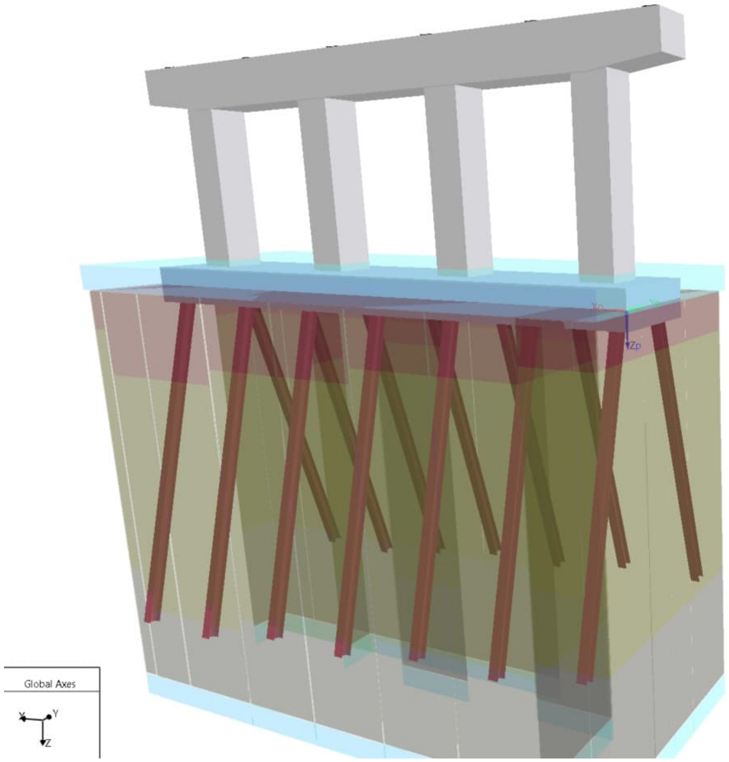
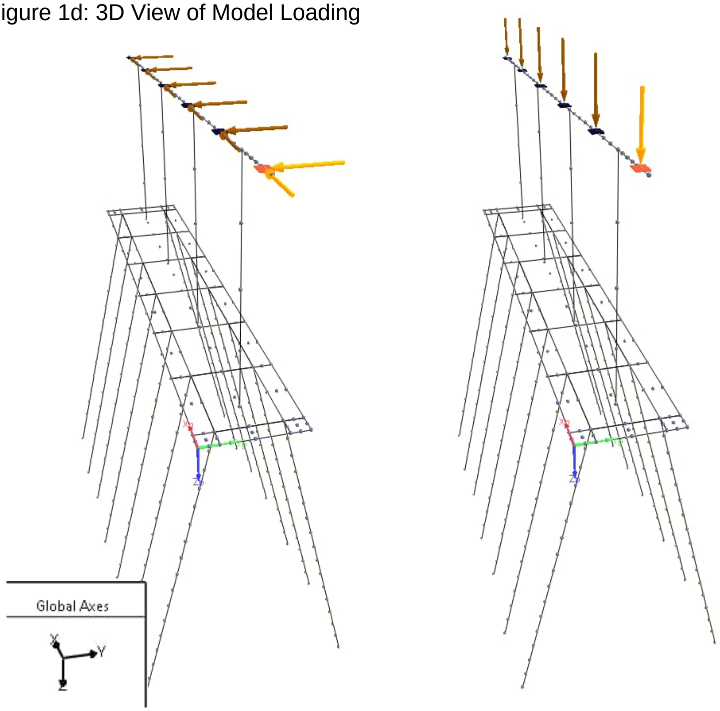


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Stiff Clay	1	Cohesive	89.0	83.4	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1182	1182	-	-	-
Glacial Till	2	Cohesionless	83.4	62.6	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	433		-	-	-
Bedrock	3	Rock	62.6	55.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		299.6	16.9	908,000

Notes:

1. The in-situ soil stratigraphy is based on Sta. 113+31 in Profile G-G' (Sheet X Interpretive Subsurface Cross Section G-G'), At the existing Pier 1, for 29 foot length piles.

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

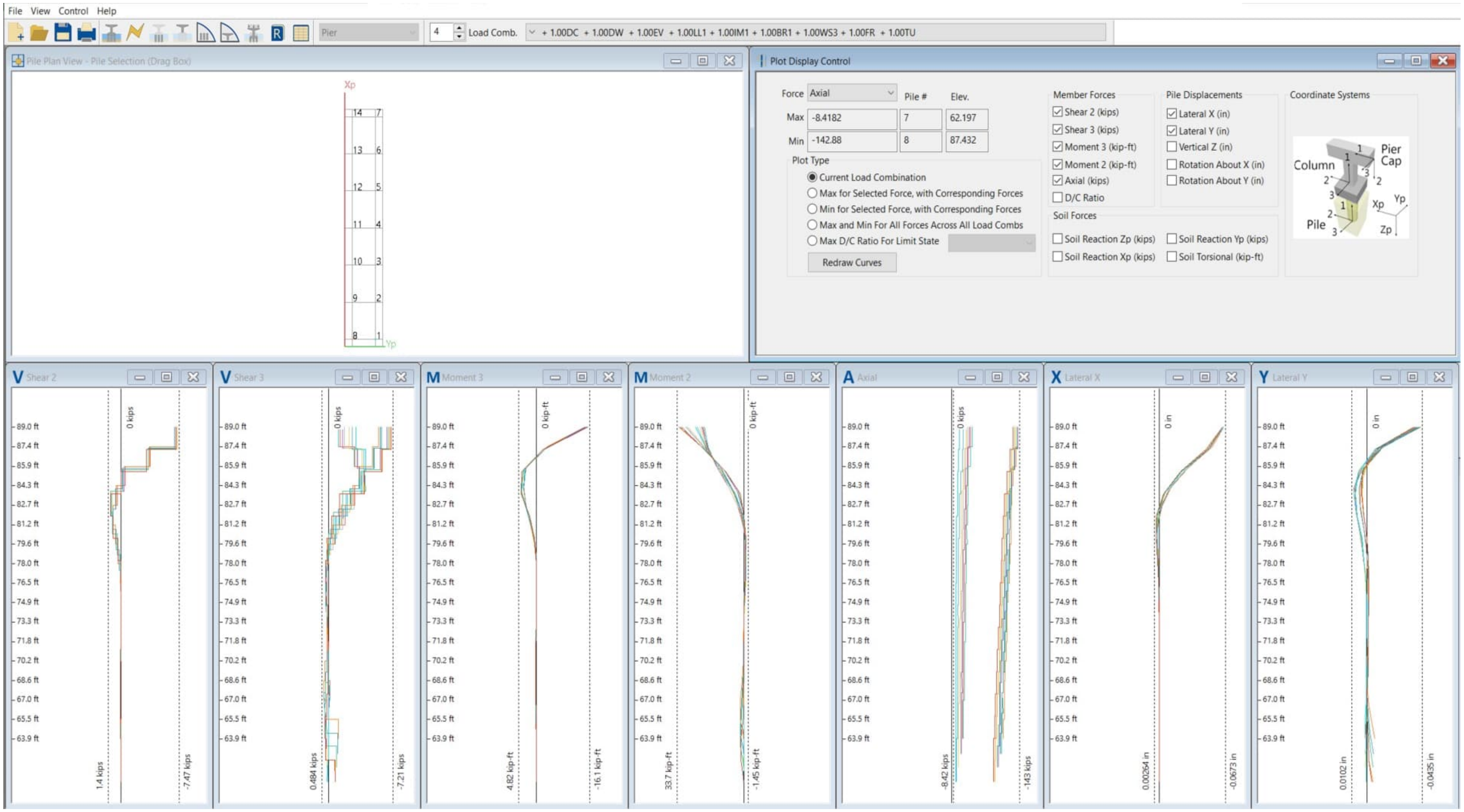
6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

Prepared: DEB
Checked: ATM
Reviewed: CCB

Case 2 Service I



Pier

4 Load Comb. + 1.00DC + 1.00DW + 1.00EV + 1.00LL1 + 1.00IM1 + 1.00BR1 + 1.00WS3 + 1.00FR + 1.00TU

Force: Axial

	Force	Pile #	Elev.
Max	-8.4182	7	62.197
Min	-142.88	8	87.432

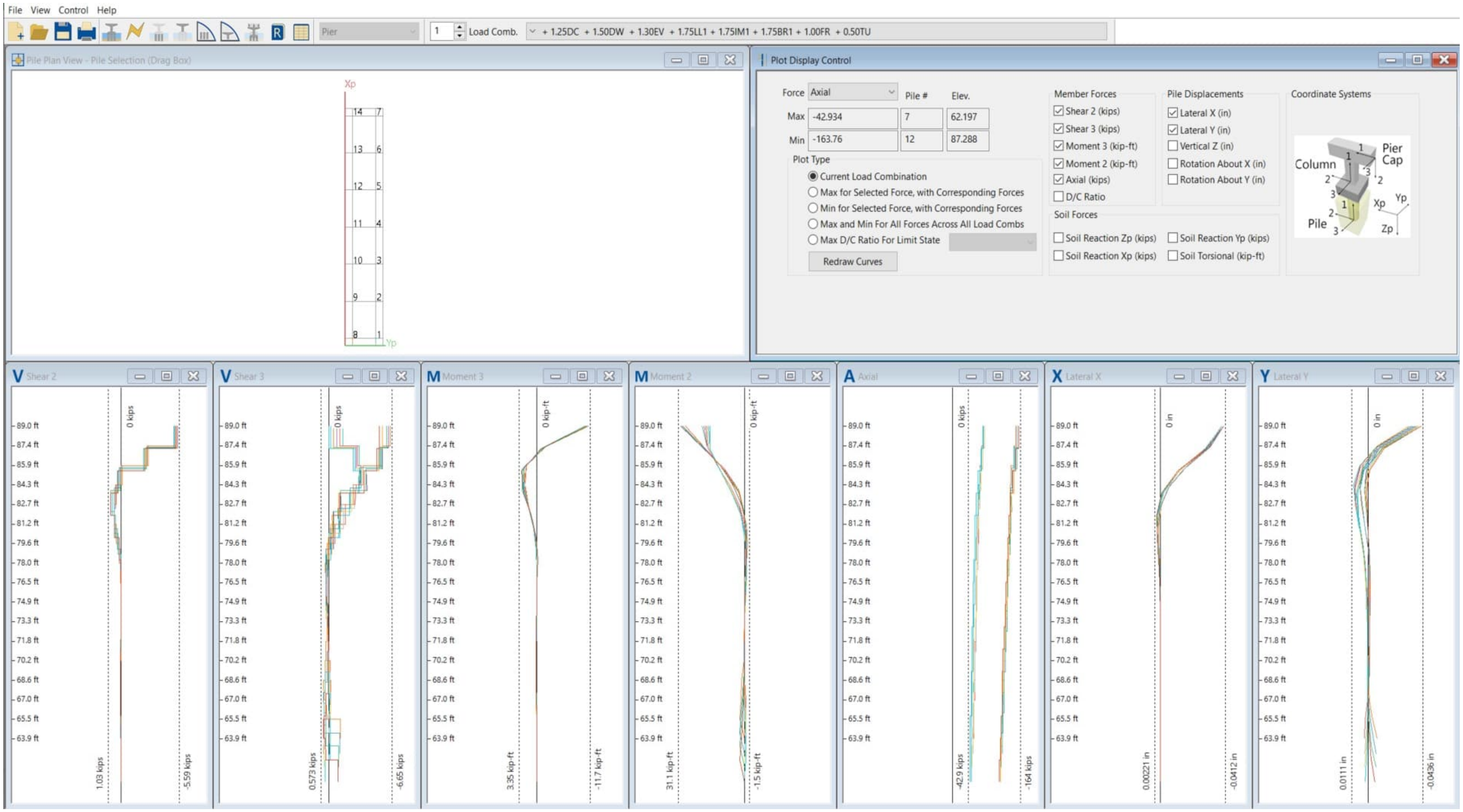
Plot Type
- ● Current Load Combination
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Combs
- ○ Max D/C Ratio For Limit State

Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Case 2 Strength I



File View Control Help

Pier

1 Load Comb. + 1.25DC + 1.50DW + 1.30EV + 1.75LL1 + 1.75IM1 + 1.75BR1 + 1.00FR + 0.50TU

Pile Plan View - Pile Selection (Drag Box)

Plot Display Control

Force: Axial

		Pile #	Elev.
Max	-42.934	7	62.197
Min	-163.76	12	87.288

Plot Type
- ● Current Load Combination
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Combs
- ○ Max D/C Ratio For Limit State

Redraw Curves

Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Coordinate Systems

Bangor Hogan Road 1983 Existing Bridge Pier 5 Bearing Loads

Unfactored Bearing Loads **9/11/2024**

Girder	Governing Force	DC	DW	HL93	TU	FR	TU	FR
					Case 1		Case 2	
G1	FX (kips)			0.0 k	10.2 k	1.7 k	-17.6 k	-1.7 k
G2	FX (kips)			-3.9 k	9.5 k	2.2 k	-16.4 k	-2.2 k
G3	FX (kips)			-5.1 k	8.8 k	2.1 k	-15.2 k	-2.1 k
G4	FX (kips)			-5.7 k	8.3 k	2.1 k	-14.3 k	-2.1 k
G5	FX (kips)			-5.1 k	7.9 k	2.1 k	-13.7 k	-2.1 k
G6	FX (kips)			-5.1 k	7.6 k	1.6 k	-13.2 k	-1.6 k
G1	FY (kips)			-2.8 k	9.3 k	2.0 k	-16.1 k	-2.0 k
G2	FY (kips)			-2.2 k	8.7 k	2.4 k	-15.0 k	-2.4 k
G3	FY (kips)			-3.7 k	8.1 k	2.4 k	-13.9 k	-2.4 k
G4	FY (kips)			-4.2 k	7.6 k	2.3 k	-13.1 k	-2.3 k
G5	FY (kips)			-3.6 k	7.2 k	2.4 k	-12.5 k	-2.4 k
G6	FY (kips)			0.0 k	7.0 k	1.8 k	-12.1 k	-1.8 k
G1	FZ (kips)	72.9 k	11.2 k	2.8 k				
G2	FZ (kips)	85.3 k	20.3 k	29.4 k				
G3	FZ (kips)	84.3 k	20.5 k	66.4 k				
G4	FZ (kips)	84.1 k	20.6 k	71.6 k				
G5	FZ (kips)	84.5 k	20.6 k	64.7 k				
G6	FZ (kips)	68.9 k	12.2 k	25.1 k				

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Pier 5 (old Pier 1) H-Pile Analysis - Strength 1 Load Case 1, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

OBJECTIVE

Determine if Pier 5 located at Sta. 113+13 (old Pier 1), corroded HP 10x42 steel piles provide adequate resistance based on geometry from the as-built 1983 record plans, Pier 5 rehab details and Case 1 loading provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) to evaluate the geotechnical and structural resistance of the individual piles.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
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10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Existing Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 8/1/2024.
14. WSP calculation package titled Pile Drivability at Pier 5 (existing Pier 1) - HP 10X42
15. Email from VHB received on Wednesday July 31st, 2024 with subject line "Bangor Hogan Road - 1983 Bridge - Pier 5 bearing forces for FB Multi-pier Analysis by WSP"

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I Case 1 output
4. FB-Multipier Service I Case 1 output
5. Rehabilitation Pier 5 loads

ASSUMPTIONS

1. The HP 10x42 steel H-piles are being evaluated based on the as-built conditions and geometry shown in Ref. 5.
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).

3. The provided loads (Ref. 11) were applied at the bottom of the pile cap at the centroid of the pile cap. Abutment designations are based on VHB designations in Ref. 11.
4. All piles have reinforced pile tips (Ref. 5) and (Ref. 8).
5. All piles are battered at 3 on 12 (0.25) in the positive or negative x-direction.
6. Pile fixity is assumed to occur at the depth where the bending moment is less than 1 kip-ft.
7. Downdrag loading has not been considered in this analysis.
8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcemnt in FB-Multipier.
9. A vertical earth load EV was added based on the difference between the existing ground surface El. of 94.3 feet and the top of Pier 5 footing of El. 90.5, resulting in 3.8 feet of overburden soil. The resultant earth pressure was distributed evenly as point loads at the center of the pile locations along the pile cap.
10. The proposed pier cap build up was accounted for in the FB-Multipier model by increasing the pier cap thickness to account for the added self weight.
11. A φdyn of 0.5 is used to determine the factored pile resistance as requested by Maine DOT using (Ref. 7 Table 10.5.5.2.3-1).

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	50	years	(per Maine DOT)
Total corrosion loss =	0.05	inches	(Ref. 13)

Pile size: HP 10x42

Intact Section				Corroded Section		(ref. 13)
Depth, d =	9.700	in	(Ref. 4, Table 1-4)	Depth, d =	9.6	in
Width, b_f =	10.100	in	(Ref. 4, Table 1-4)	Width, b_f =	10.0	in
Web thickness, t_w =	0.415	in	(Ref. 4, Table 1-4)	Web thickness, t_w =	0.315	in
Flange thickness, t_f =	0.420	in	(Ref. 4, Table 1-4)	Flange thickness, t_f =	0.320	in
Fillet area =	0.2	in^2		Fillet area =	0.2	in^2
Section area =	12.4	in^2	(Ref. 4, Table 1-4)	Section area =	9.4	in^2
Section area =	0.09	ft^2		Section area =	0.07	ft^2
				Moment of inertia about the x-axis, I_x =	157	in^4
				Moment of inertia about the y-axis, I_y =	53	in^4
				Radius of gyration about the x-axis, r_x =	4.08	in
				Radius of gyration about the y-axis, r_y =	2.38	in
				Elastic section modulus about the x-axis, S_x =	32.7	in^3
				Elastic section modulus about the y-axis, S_y =	10.7	in^3
				Plastic section modulus about the x-axis, Z_x =	36.0	in^3
				Plastic section modulus about the y-axis, Z_y =	16.2	in^3

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Steel yield stress, F_y = 36 ksi (Grade A36) Sheet 3 of 53 (Ref. 5)

Steel elastic modulus, E_{st} = 29,000 ksi

Top of pile cap elevation = 90.5 ft Sheet 13 of 53 (Ref. 5)

Pile cap thickness = 3 ft Sheet 13 of 53 (Ref. 5)

Pile cap midplane elevation = 89.0 ft

elevation = 89.0 ft

Base of pile cap elevation = 87.5 ft

Pile length = 28.6 ft

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Stiff Clay	1	Cohesive	89.0	82.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	2	Cohesionless	82.1	61.4	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	3	Rock	61.4	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Sta. 113+13 in Profile F-F' (Sheet X Interpretive Subsurface Cross Section G-G').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model: (Pile 14)

Maximum axial force in the piles, P_u = 146.6 kips

Maximum bending moment in the piles, $M_x = M_2$ = 15.6 kip-ft = 187.4 in-kips

Maximum bending moment in the piles, $M_y = M_3$ = 3.7 kip-ft = 44.0 in-kips

Maximum shear force in the piles, V_u = 4.3 kips

Depth below pile head to fixity = 4.3 ft = Elev. 84.67 ft

Depth below pile cap to fixity = 2.8 ft = Elev. 84.67 ft

C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.07	ft^3/ft	(from Part A)
δ/ϕ_f =	0.75		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.79		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.04		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.46		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Glacial Till	
Friction angle, ϕ_f =	35	degrees (Ref. 6)
δ =	26	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.04	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.85	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	1.111	ksf (at midpoint of soil layer using stratigraphy in Part A)

$$f_s = \alpha s_u$$

f_s = 0.434 ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		
	Top	Bottom	
Undrained shear strength, s_u =	1.343	1.343	ksf (Ref. 6)
Adhesion factor, a =	0.88	0.88	(Ref. 7 Fig. 10.7.3.8.6b-1, sand
f_s =	1.182	1.182	ksf

CALCULATIONS

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Nominal shaft resistance:

$R_s = f_s A_s$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = (d × 2) + (b_f × 2) = 39.2 in = 3.3 ft (from Part A)

	Stiff Clay	Glacial Till	
Thickness =	6.9	20.7	ft (from stratigraphy in Part A)
A_s =	22.5	67.6	ft^2
R_s per layer =	26.6	29.4	kips

Total R_s = 56.0 kips

Unit toe resistance, using the Kulhawy and Goodman method:

$q_p = 0.33 q_u$ for RQD ≤ 70%
$q_p = 0.80 q_u$ for RQD = 100%
(Ref. 3, Section 7.2.1.4.1)

where:

Rock quality designation, RQD = 9 % (Ref. 9, minimum RQD in borings with lab testing)
Unconfined compressive strength, q_u = 908 ksf (Ref. 10, minimum UCS from lab test data)

q_p = 299.6 ksf

Nominal toe resistance:

$R_p = q_p A_p$ (Ref. 7, Eq. 10.7.3.8.6a-3)

where:

$R_n = R_s + R_p$

Pile toe area, A_p = 0.09 ft^2 (from Part A)

Total R_p = 25.8 kips (bedrock)

Nominal and factored geotechnical resistance:

$R_n = R_s + R_p$ (Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = 82 kips (bedrock)

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$$R_r = \varphi_{stat} R_n$$ (Ref. 7, Eq. 10.7.3.8.6a-1)

where: (Ref. 7 Tbl. 10.5.5.2.3-1)

$\varphi_{stat} =$ 0.35 for shaft and toe resistance of cohesive soil by the α-method

$\varphi_{stat} =$ 0.45 for shaft and toe resistance of cohesionless soil by the Nordlund method

$\varphi_{stat} =$ 0.45 for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Stiff Clay	Glacial Till	Pile Tip (Bedrock)	
R_s or R_p =	26.6	29.4	25.8	kips
φ_{stat} =	0.35	0.45	0.45	(Ref. 7, Table 10.5.5.2.3-1)
R_r per layer =	9.3	13.2	11.6	

Total R_r = 34 kips (bedrock)

Wave Equation Analysis

Nominal and factored geotechnical resistance obained from GRWEAP at 100% of the steel yield strength (Ref. 14):

$R_n =$ 310 kips (Ref. 14)

$\varphi_{dyn} =$ 0.5 (Assumption 11)

$R_f =$ 155 kips

Check that the geotechnical axial resistance Obtained from GRLWEAP (Ref. 15) is sufficient to support axial load on pile:

$P_u =$ 147 kips (from Part B)

$R_r =$ 155 kips > $P_u =$ 147 kips OK

D. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, l =	34.01	in	(from Part B)
Minimum radius of gyration, r_s =	2.38	in	(from Part A)

Check: Kl/r_s = 9 < 120 OK

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \leq \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $$0.56\sqrt{\frac{E_{st}}{F_y}}$$ (Ref. 7, Table 6.9.4.2.1-1)
(Assumption 9)

where:

Half Flange width, b_f =	5.0	in	(from Part A)
Flange thickness, t_f =	0.320	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	36	ksi	(from Part A)

Check: $$\frac{b}{t} = 15.63 < 15.89 = 0.56\sqrt{\frac{E_{st}}{F_y}}$$

The pile section is a non-slender element (Ref. 7, Article 6.9.4.1)

Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1)

Equivalent nominal yield resistance, P_o = 339 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	9.4	in^2	(from Part A)
Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, l =	34.01	in	(from Part B)
Minimum radius of gyration, r_s =	2.38	in	(from Part A)

P_e (flexural) = 31,252 kips

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Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D = 8.960 in D= d - (2 × tf)

Web thickness, t_w = 0.315 in (from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling

Nominal and factored structural resistance in axial compression:

$$P_n = P_o 0.658^{P_o/P_e}$$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$$P_n = 0.877 P_e$$ (Ref. 7, Eq. 6.9.4.1.1-2)

P_o / P_e = 0.01 < 2.25, so use Eq. 6.9.4.1.1-1

P_n = 338 kips

$$P_{r_{axial}} = \varphi_c P_n$$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c = 0.50 (Ref. 7, Article 6.5.4.2))

P_r = 169 kips

$$P_{r_{axial/flexural}} = \varphi_c P_n$$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c = 0.70 (Ref. 7, Article 6.5.4.2))

$P_{r,\ axial/\ flexural}$= 236 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

P_u = 147 kips (from Part B)

$P_{r,\ axial}$ = 169 kips > P_u = 147 kips OK

CALCULATIONS

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E. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

$\lambda_f =$ 15.63

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

$\lambda_{pf} =$ 10.79

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

$\lambda r_f =$ 23.56

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \leq \lambda_{pf}$$

(Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \leq \lambda_{rf}$$

(Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	36	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	32.7	in^3	(from Part A)
Elastic section modulus about y, S_y =	10.7	in^3	(from Part A)
Plastic section modulus about x, Z_x =	36.0	in^3	(from Part A)
Plastic section modulus about y, Z_y =	16.2	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	1,251	508	in-kips

φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	1,251	in-kips	
M_{ry} =	508	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} =	187	in-kips	(from Part B)
M_{uy} =	44	in-kips	(from Part B)

M_{rx} =	1,251	in-kips	>	M_{ux} =	187	in-kips	OK
M_{ry} =	508	in-kips	>	M_{uy} =	44	in-kips	OK

F. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$ if $P_u/P_{r,\ axial/\ flexural} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$ if $P_u/P_{r,\ axial/\ flexural} \geq 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\ axial/\ flexural}$ = 0.6 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.8 < 1.0 OK

G. Determine the nominal and factored structural resistance of a single pile in shear.

$$V_n = CV_p = V_{cr}$$

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_p = 0.58F_y d_w t_w$$ (Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	36	ksi	(from Part A)
Web depth, d_w =	8.960	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.315	in	(from Part A)

$$\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	8.960	in	D= d - (2 × tf)
t_w =	0.315	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	71.1	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 71.1 C = 1.0

V_p =	59	kips
V_n =	59	kips

$$V_u \leq \varphi_v V_n$$

φ_v =	1.00		(Ref. 7, Article 6.5.4.2)
V_r =	59	kips	

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_u \leq \varphi_v V_n$$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 4.3 kips (from Part B)

$V_r =$ 59 kips > $V_u =$ 4.3 kips OK

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 1, and an estimated pile length of 29 feet, a maximum axial force of 146.6 kips , the corresponding strong axis (x-axis) moment of 187.4 in-kips (15.6 ft-kips) occurs in the HP10x42 piles at Pier 5 (old Pier 1) in Pile 14 of the 1983 portion of the Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity. This evaluation considers section loss for an addtional 50 year design life, this evaluation does not consider downdrag. A summary table is presented below.

Old Pier 1 (new Pier 5, Pile 14) Summary Table		
Pile Length (ft)		29
Depth below Pile Head to Fixity (ft)		4
Geotechnical Axial Resistance (kips)	Nominal	310
	Factored	155
Structural Axial Resistance (kips)	Nominal	338
	Factored	169
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	104
	Factored	104
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	42
	Factored	42
Combined Axial and Flexural Interaction Ratio		0.8
Shear Resistance (kips)	Nominal	59
	Factored	59

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

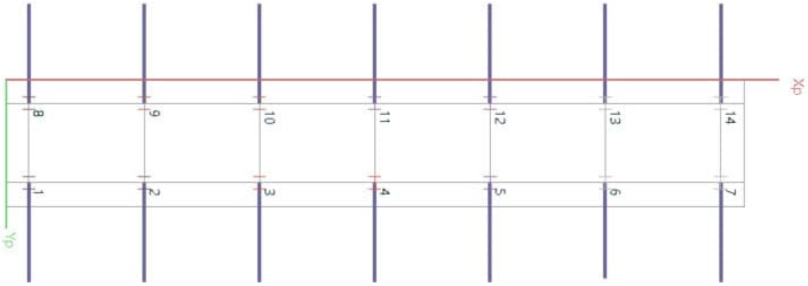


Figure 1b: Profile View of Pile Cap (Transverse)

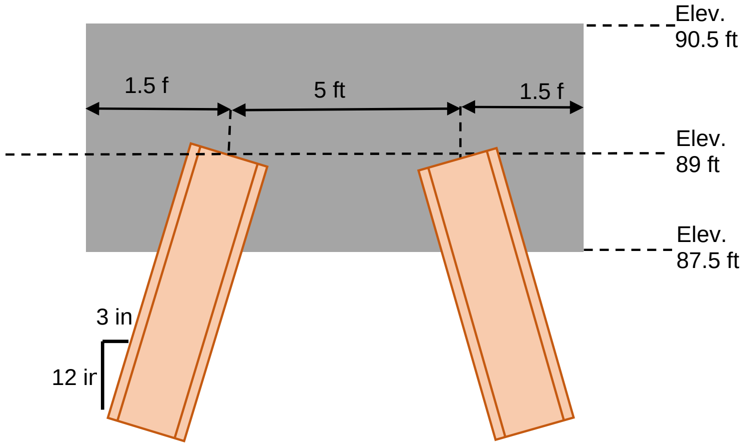


Figure 1c: 3D View of Pile Model

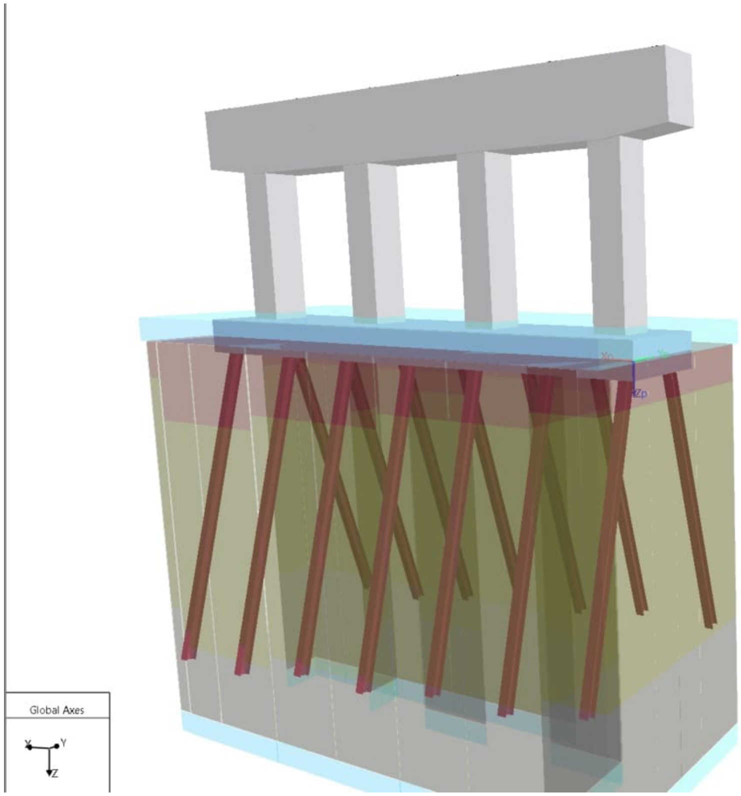
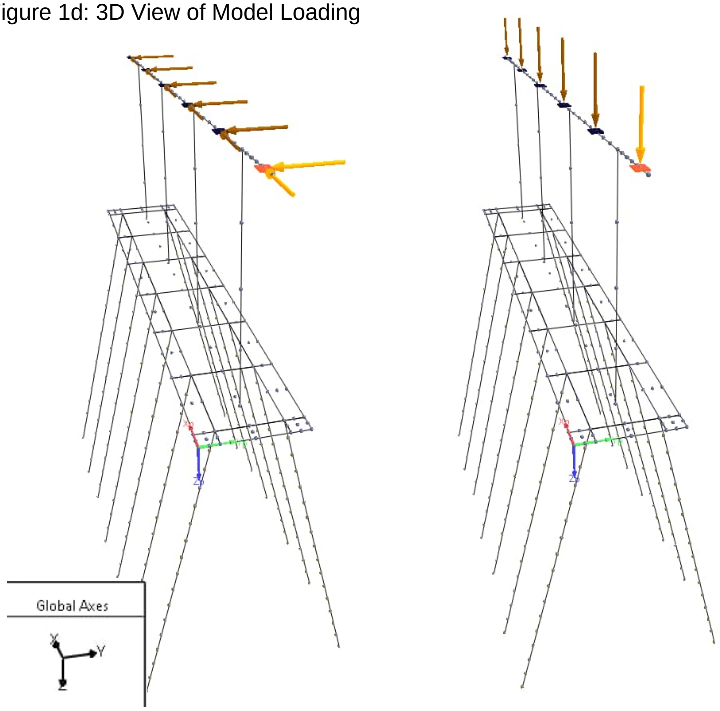


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Stiff Clay	1	Cohesive	89.0	82.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1182	1182	-	-	-
Glacial Till	2	Cohesionless	82.1	61.4	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	434		-	-	-
Bedrock	3	Rock	61.4	55.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		3600	310	908,000

Notes:

1. The in-situ soil stratigraphy is based on Sta. 113+31 in Profile G-G' (Sheet X Interpretive Subsurface Cross Section G-G'), At the existing Pier 1, for 31 foot length piles.

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

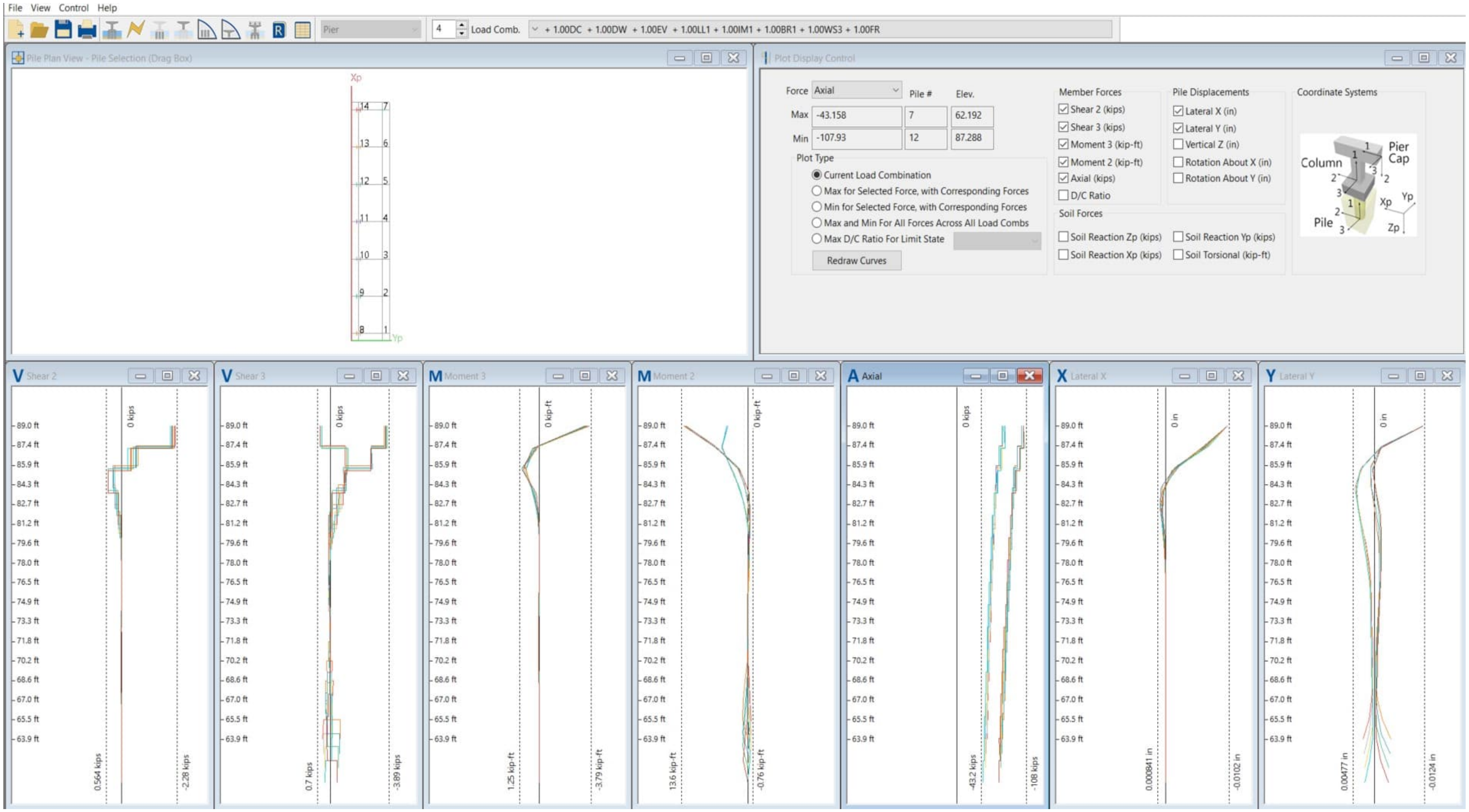
6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

7. Unit tip resistance values for rock obtained from WSP calculation package titled Pile Drivability at Pier 5 (existing Pier 1) - HP 10x42.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

Prepared:	DEB
Checked:	ATM
Reviewed:	CCB

Case 1 Service I



Load Comb. 4: + 1.00DC + 1.00DW + 1.00EV + 1.00LL1 + 1.00IM1 + 1.00BR1 + 1.00WS3 + 1.00FR

Force: Axial

	Force	Pile #	Elev.
Max	-43.158	7	62.192
Min	-107.93	12	87.288

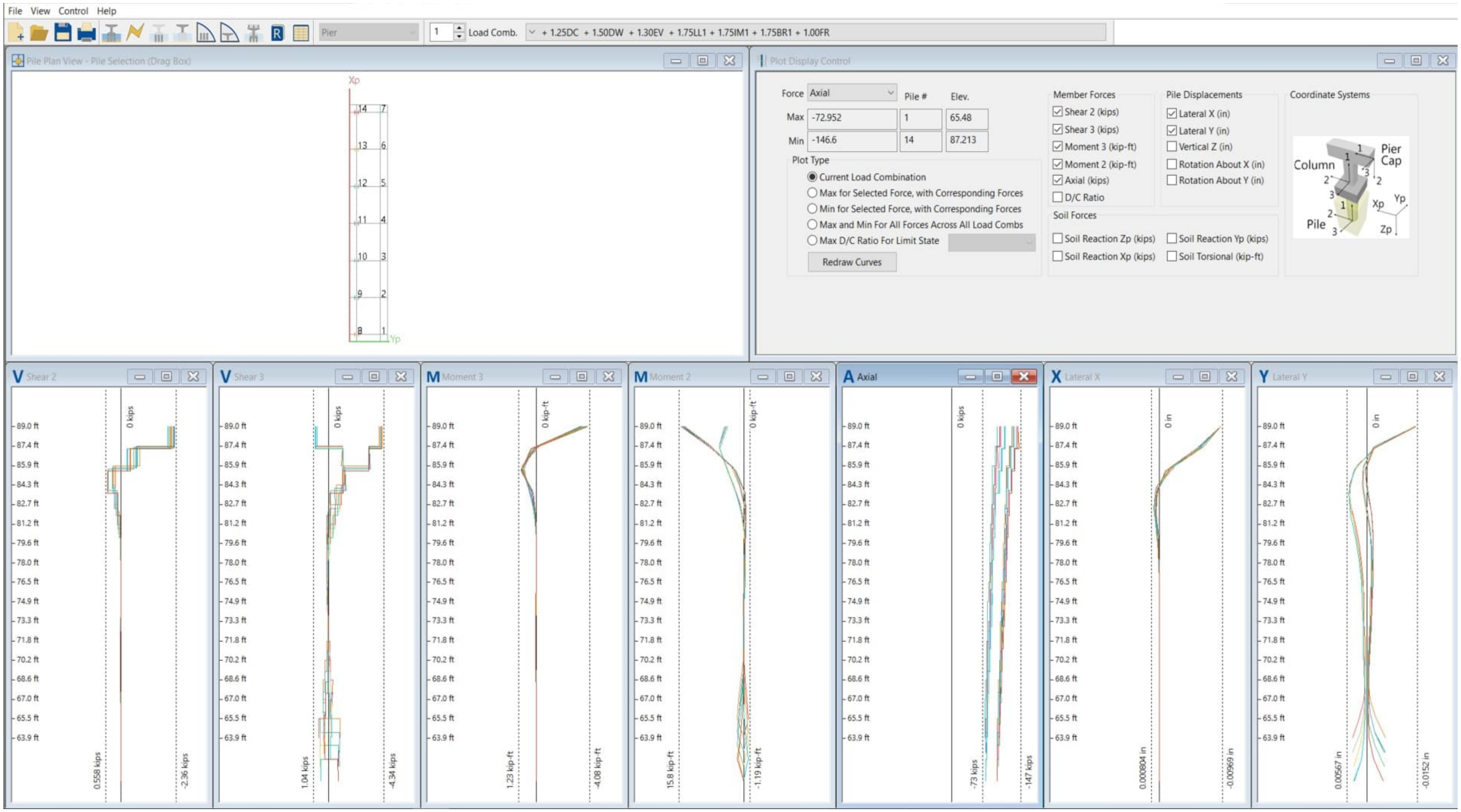
Plot Type
- ● Current Load Combination
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Combs
- ○ Max D/C Ratio For Limit State

Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Case 1 Strength I



1 Load Comb. + 1.25DC + 1.50DW + 1.30EV + 1.75LL1 + 1.75IM1 + 1.75BR1 + 1.00FR

Force: Axial

	Force	Pile #	Elev.
Max	-72.952	1	65.48
Min	-146.6	14	87.213

Plot Type
- ● Current Load Combination
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Combs
- ○ Max D/C Ratio For Limit State

Redraw Curves

Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

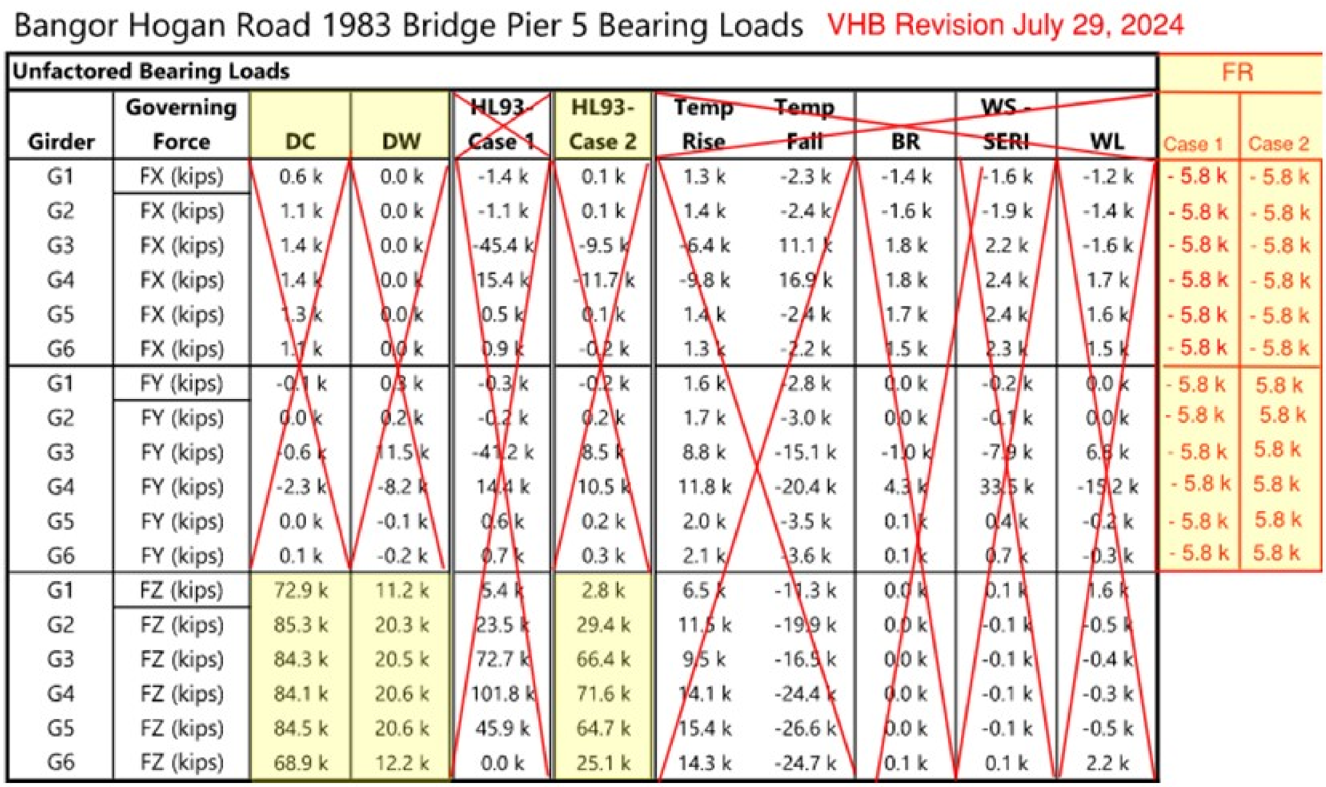
Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Received form VHB on July 22, 2024

Bangor Hogan Road 1983 Bridge Pier 5 Bearing Loads

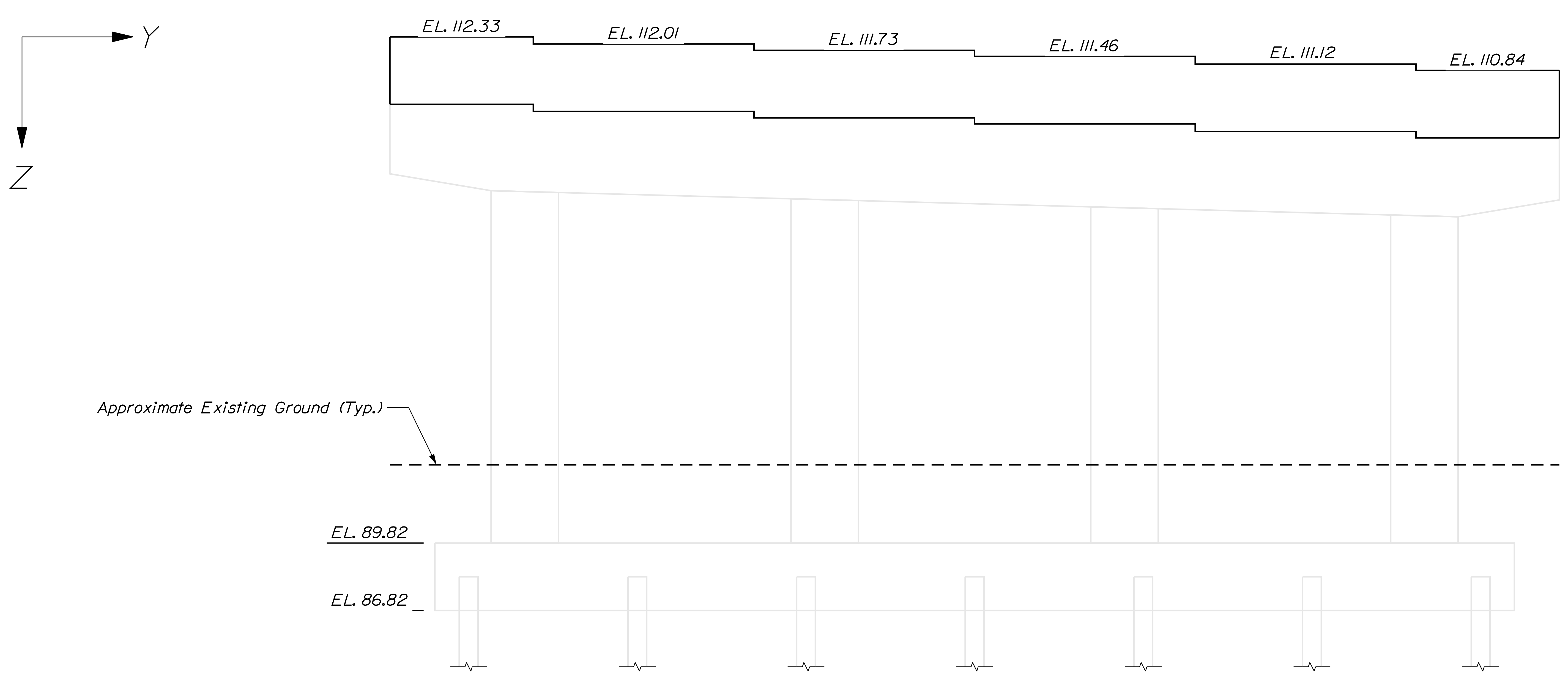
Unfactored Bearing Loads										
Girder	**Governing Force**	**DC**	**DW**	**HL93-Case 1**	**HL93-Case 2**	**Temp Rise**	**Temp Fall**	**BR**	**WS - SERI**	**WL**
G1	FX (kips)	0.6 k	0.0 k	-1.4 k	0.1 k	1.3 k	-2.3 k	-1.4 k	-1.6 k	-1.2 k
G2	FX (kips)	1.1 k	0.0 k	-1.1 k	0.1 k	1.4 k	-2.4 k	-1.6 k	-1.9 k	-1.4 k
G3	FX (kips)	1.4 k	0.0 k	-45.4 k	-9.5 k	-6.4 k	11.1 k	1.8 k	2.2 k	-1.6 k
G4	FX (kips)	1.4 k	0.0 k	15.4 k	-11.7 k	-9.8 k	16.9 k	1.8 k	2.4 k	1.7 k
G5	FX (kips)	1.3 k	0.0 k	0.5 k	0.1 k	1.4 k	-2.4 k	1.7 k	2.4 k	1.6 k
G6	FX (kips)	1.1 k	0.0 k	0.9 k	-0.2 k	1.3 k	-2.2 k	1.5 k	2.3 k	1.5 k
G1	FY (kips)	-0.1 k	0.3 k	-0.3 k	-0.2 k	1.6 k	-2.8 k	0.0 k	-0.2 k	0.0 k
G2	FY (kips)	0.0 k	0.2 k	-0.2 k	0.2 k	1.7 k	-3.0 k	0.0 k	-0.1 k	0.0 k
G3	FY (kips)	-0.6 k	11.5 k	-41.2 k	8.5 k	8.8 k	-15.1 k	-1.0 k	-7.9 k	6.8 k
G4	FY (kips)	-2.3 k	-8.2 k	14.4 k	10.5 k	11.8 k	-20.4 k	4.3 k	33.5 k	-15.2 k
G5	FY (kips)	0.0 k	-0.1 k	0.6 k	0.2 k	2.0 k	-3.5 k	0.1 k	0.4 k	-0.2 k
G6	FY (kips)	0.1 k	-0.2 k	0.7 k	0.3 k	2.1 k	-3.6 k	0.1 k	0.7 k	-0.3 k
G1	FZ (kips)	72.9 k	11.2 k	5.4 k	2.8 k	6.5 k	-11.3 k	0.0 k	0.1 k	1.6 k
G2	FZ (kips)	85.3 k	20.3 k	23.5 k	29.4 k	11.5 k	-19.9 k	0.0 k	-0.1 k	-0.5 k
G3	FZ (kips)	84.3 k	20.5 k	72.7 k	66.4 k	9.5 k	-16.5 k	0.0 k	-0.1 k	-0.4 k
G4	FZ (kips)	84.1 k	20.6 k	101.8 k	71.6 k	14.1 k	-24.4 k	0.0 k	-0.1 k	-0.3 k
G5	FZ (kips)	84.5 k	20.6 k	45.9 k	64.7 k	15.4 k	-26.6 k	0.0 k	-0.1 k	-0.5 k
G6	FZ (kips)	68.9 k	12.2 k	0.0 k	25.1 k	14.3 k	-24.7 k	0.1 k	0.1 k	2.2 k



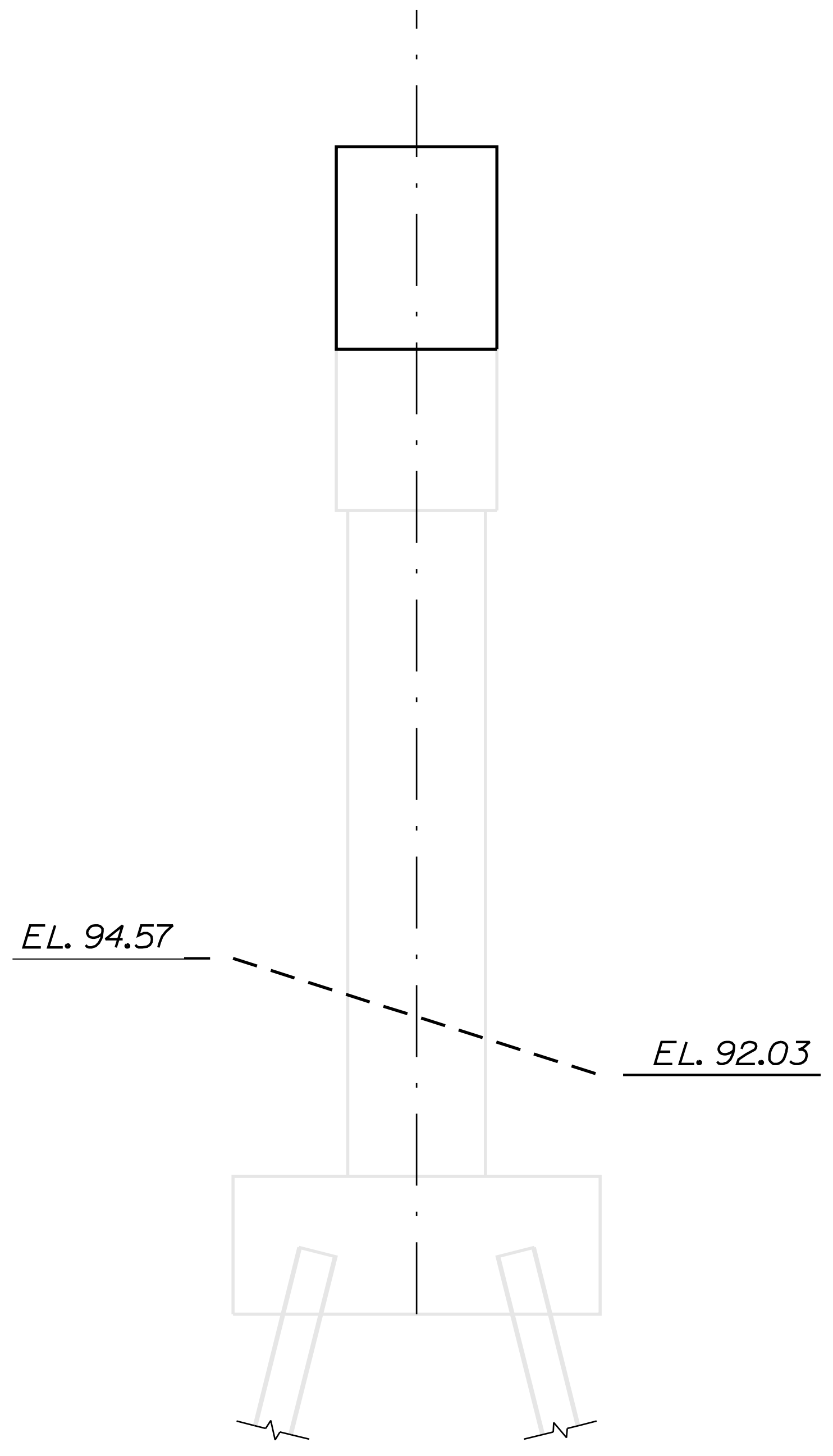
Bangor Hogan Road 1983 Bridge Pier 5 Bearing Loads VHB Revision July 29, 2024

Unfactored Bearing Loads											FR	
Girder	**Governing Force**	**DC**	**DW**	~~**HL93-Case 1**~~	**HL93-Case 2**	~~**Temp Rise**~~	~~**Temp Fall**~~	~~**BR**~~	~~**WS - SERI**~~	~~**WL**~~	Case 1	Case 2
G1	FX (kips)	~~0.6 k~~	~~0.0 k~~	~~-1.4 k~~	~~0.1 k~~	~~1.3 k~~	~~-2.3 k~~	~~-1.4 k~~	~~-1.6 k~~	~~-1.2 k~~	- 5.8 k	- 5.8 k
G2	FX (kips)	~~1.1 k~~	~~0.0 k~~	~~-1.1 k~~	~~0.1 k~~	~~1.4 k~~	~~-2.4 k~~	~~-1.6 k~~	~~-1.9 k~~	~~-1.4 k~~	- 5.8 k	- 5.8 k
G3	FX (kips)	~~1.4 k~~	~~0.0 k~~	~~-45.4 k~~	~~-9.5 k~~	~~-6.4 k~~	~~11.1 k~~	~~1.8 k~~	~~2.2 k~~	~~-1.6 k~~	- 5.8 k	- 5.8 k
G4	FX (kips)	~~1.4 k~~	~~0.0 k~~	~~15.4 k~~	~~-11.7 k~~	~~-9.8 k~~	~~16.9 k~~	~~1.8 k~~	~~2.4 k~~	~~1.7 k~~	- 5.8 k	- 5.8 k
G5	FX (kips)	~~1.3 k~~	~~0.0 k~~	~~0.5 k~~	~~0.1 k~~	~~1.4 k~~	~~-2.4 k~~	~~1.7 k~~	~~2.4 k~~	~~1.6 k~~	- 5.8 k	- 5.8 k
G6	FX (kips)	~~1.1 k~~	~~0.0 k~~	~~0.9 k~~	~~-0.2 k~~	~~1.3 k~~	~~-2.2 k~~	~~1.5 k~~	~~2.3 k~~	~~1.5 k~~	- 5.8 k	- 5.8 k
G1	FY (kips)	~~-0.1 k~~	~~0.3 k~~	~~-0.3 k~~	~~-0.2 k~~	~~1.6 k~~	~~-2.8 k~~	~~0.0 k~~	~~-0.2 k~~	~~0.0 k~~	- 5.8 k	5.8 k
G2	FY (kips)	~~0.0 k~~	~~0.2 k~~	~~-0.2 k~~	~~0.2 k~~	~~1.7 k~~	~~-3.0 k~~	~~0.0 k~~	~~-0.1 k~~	~~0.0 k~~	- 5.8 k	5.8 k
G3	FY (kips)	~~-0.6 k~~	~~11.5 k~~	~~-41.2 k~~	~~8.5 k~~	~~8.8 k~~	~~-15.1 k~~	~~-1.0 k~~	~~-7.9 k~~	~~6.8 k~~	- 5.8 k	5.8 k
G4	FY (kips)	~~-2.3 k~~	~~-8.2 k~~	~~14.4 k~~	~~10.5 k~~	~~11.8 k~~	~~-20.4 k~~	~~4.3 k~~	~~33.5 k~~	~~-15.2 k~~	- 5.8 k	5.8 k
G5	FY (kips)	~~0.0 k~~	~~-0.1 k~~	~~0.6 k~~	~~0.2 k~~	~~2.0 k~~	~~-3.5 k~~	~~0.1 k~~	~~0.4 k~~	~~-0.2 k~~	- 5.8 k	5.8 k
G6	FY (kips)	~~0.1 k~~	~~-0.2 k~~	~~0.7 k~~	~~0.3 k~~	~~2.1 k~~	~~-3.6 k~~	~~0.1 k~~	~~0.7 k~~	~~-0.3 k~~	- 5.8 k	5.8 k
G1	FZ (kips)	72.9 k	11.2 k	~~5.4 k~~	2.8 k	~~6.5 k~~	~~-11.3 k~~	~~0.0 k~~	~~0.1 k~~	~~1.6 k~~		
G2	FZ (kips)	85.3 k	20.3 k	~~23.5 k~~	29.4 k	~~11.5 k~~	~~-19.9 k~~	~~0.0 k~~	~~-0.1 k~~	~~-0.5 k~~		
G3	FZ (kips)	84.3 k	20.5 k	~~72.7 k~~	66.4 k	~~9.5 k~~	~~-16.5 k~~	~~0.0 k~~	~~-0.1 k~~	~~-0.4 k~~		
G4	FZ (kips)	84.1 k	20.6 k	~~101.8 k~~	71.6 k	~~14.1 k~~	~~-24.4 k~~	~~0.0 k~~	~~-0.1 k~~	~~-0.3 k~~		
G5	FZ (kips)	84.5 k	20.6 k	~~45.9 k~~	64.7 k	~~15.4 k~~	~~-26.6 k~~	~~0.0 k~~	~~-0.1 k~~	~~-0.5 k~~		
G6	FZ (kips)	68.9 k	12.2 k	~~0.0 k~~	25.1 k	~~14.3 k~~	~~-24.7 k~~	~~0.1 k~~	~~0.1 k~~	~~2.2 k~~		

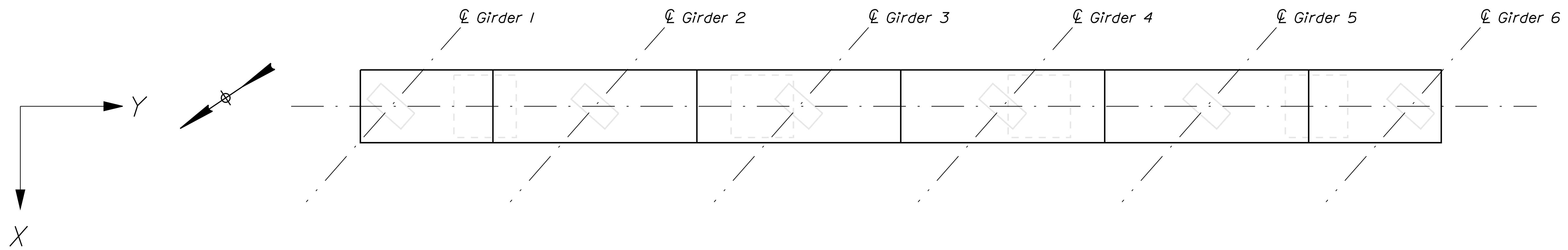
NOTE:
INFORMATION ON THIS SHEET IS FOR
PIER 5 MODELING PURPOSES ONLY.



PROPOSED PIER ELEVATION



PROPOSED PIER SECTION



PROPOSED PIER CAP PLAN

EXISTING FOOTING PLAN

Filename: ...\031_EB_Existing_Pier_5.dgn
Division: HIGHWAY
Username: jharris
Date:7/22/2024

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

BRIDGE PLANS

WIN
18595.10

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

PROJ. MANAGER	R. MOULTON	BY	DATE
DESIGN-DETAILED			7/12/24
CHECKED-REVIEWED	YP	SMH	7/12/24
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD BRIDGE
I-95

BANGOR PENOBSCOT

EASTBOUND BRIDGE PIER 5
REHAB DETAILS

SHEET NUMBER

B1

OF 1

CALCULATIONS

Date: 10/25/2024
Made by: DEB
Project No.: 31404817.004
Checked by: JEF
Subject: Rehabilitated Pier 5 (old Pier 1) H-Pile Analysis - Strength 1 Load Case 2, Corroded Section
Reviewed by: CCB
Project Title: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME

OBJECTIVE

Determine if Pier 5 located at Sta. 113+13 (old Pier 1), corroded HP 10x42 steel piles provide adequate resistance based on geometry from the as-built 1983 record plans, Pier 5 rehab details and Case 2 loading provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) to evaluate the geotechnical and structural resistance of the individual piles.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. PDR Draft 2-3-2023 Hogan Road DDI (Bridge Portion Only) 018595.10
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Existing Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 8/1/2024.
14. WSP calculation package titled Pile Drivability at Pier 5 (existing Pier 1) - HP 10X42
15. Email from VHB received on Wednesday July 31st, 2024 with subject line "Bangor Hogan Road - 1983 Bridge - Pier 5 bearing forces for FB Multi-pier Analysis by WSP"

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I Case 2 output
4. FB-Multipier Service I Case 2 output
5. Rehabilitation Pier 5 loads

ASSUMPTIONS

1. The HP 10x42 steel H-piles are being evaluated based on the as-built conditions and geometry shown in Ref. 5.

2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 11) were applied at the bottom of the pile cap at the centroid of the pile cap. Abutment designations are based on VHB designations in Ref. 11.
4. All piles have reinforced pile tips (Ref. 5) and (Ref. 8).
5. All piles are battered at 3 on 12 (0.25) in the positive or negative x-direction.
6. Pile fixity is assumed to occur at the depth where the bending moment is less than 1 kip-ft.
7. Downdrag loading has not been considered in this analysis.
8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcemnt in FB-Multipier.
9. A vertical earth load EV was added based on the difference between the existing ground surface El. of 94.3 feet and the top of Pier 5 footing of El. 90.5, resulting in 3.8 feet of overburden soil. The resultant earth pressure was distributed evenly as point loads at the center of the pile locations along the pile cap.
10. The proposed pier cap build up was accounted for in the FB-Multipier model by increasing the pier cap thickness to account for the added self weight.
11. A φdyn of 0.5 is used to determine the factored pile resistance as requested by Maine DOT using (Ref. 7 Table 10.5.5.2.3-1).

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	50	years	(per Maine DOT)
Total corrosion loss =	0.05	inches	(Ref. 13)

Pile size: HP 10x42

Intact Section				Corroded Section		(ref. 13)
Depth, d =	9.700	in	(Ref. 4, Table 1-4)	Depth, d =	9.6	in
Width, b_f =	10.100	in	(Ref. 4, Table 1-4)	Width, b_f =	10.0	in
Web thickness, t_w =	0.415	in	(Ref. 4, Table 1-4)	Web thickness, t_w =	0.315	in
Flange thickness, t_f =	0.420	in	(Ref. 4, Table 1-4)	Flange thickness, t_f =	0.320	in
Fillet area =	0.2	in^2		Fillet area =	0.2	in^2
Section area =	12.4	in^2	(Ref. 4, Table 1-4)	Section area =	9.4	in^2
Section area =	0.09	ft^2		Section area =	0.07	ft^2
				Moment of inertia about the x-axis, I_x =	157	in^4
				Moment of inertia about the y-axis, I_y =	53	in^4
				Radius of gyration about the x-axis, r_x =	4.08	in
				Radius of gyration about the y-axis, r_y =	2.38	in

Elastic section modulus about the x-axis, S_x = 32.7 in^3
Elastic section modulus about the y-axis, S_y = 10.7 in^3
Plastic section modulus about the x-axis, Z_x = 36.0 in^3
Plastic section modulus about the y-axis, Z_y = 16.2 in^3

Steel yield stress, F_y = 36 ksi (Grade A36) Sheet 3 of 53 (Ref. 5)

Steel elastic modulus, E_{st} = 29,000 ksi
Top of pile cap elevation = 90.5 ft Sheet 13 of 53 (Ref. 5)
Pile cap thickness = 3 ft Sheet 13 of 53 (Ref. 5)
Pile cap midplane elevation = 89.0 ft
elevation = 89.0 ft
Base of pile cap elevation = 87.5 ft

Pile length = 28.6 ft

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Stiff Clay	1	Cohesive	89.0	82.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	2	Cohesionless	82.1	61.4	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	3	Rock	61.4	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Sta. 113+13 in Profile F-F' (Sheet X Interpretive Subsurface Cross Section G-G').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model: (Pile 14)
Maximum axial force in the piles, P_u = 156.1 kips
Maximum bending moment in the piles, $M_x = M_2$ = 15.4 kip-ft = 184.3 in-kips
Maximum bending moment in the piles, $M_y = M_3$ = 3.6 kip-ft = 42.7 in-kips
Maximum shear force in the piles, V_u = 4.1 kips
Depth below pile head to fixity = 4.4 ft = Elev. 84.57 ft
Depth below pile cap to fixity = 2.9 ft = Elev. 84.57 ft

C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis.

Static Analysis
Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.07	ft^3/ft	(from Part A)
δ/ϕ_f =	0.75		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-pile
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.79		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.04		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.46		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Glacial Till	
Friction angle, ϕ_f =	35	degrees (Ref. 6)
δ =	26	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.04	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.85	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	1.111	ksf (at midpoint of soil layer using stratigraphy in Part A)

$$f_s = \alpha s_u$$

f_s =	0.434	ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		
	Top	Bottom	
Undrained shear strength, s_u =	1.343	1.343	ksf (Ref. 6)
Adhesion factor, a =	0.88	0.88	(Ref. 7 Fig. 10.7.3.8.6b-1, sand
f_s =	1.182	1.182	ksf

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Pier 5 (old Pier 1) H-Pile Analysis - Strength 1 Load Case 2, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Nominal shaft resistance:

$R_s = f_s A_s$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = (d × 2) + (b_f × 2) = 39.2 in = 3.3 ft (from Part A)

	Stiff Clay	Glacial Till	
Thickness =	6.9	20.7	ft (from stratigraphy in Part A)
A_s =	22.5	67.6	ft^2
R_s per layer =	26.6	29.4	kips

Total R_s = 56.0 kips

Unit toe resistance, using the Kulhawy and Goodman method:

$q_p = 0.33q_u$ for RQD ≤ 70%
$q_p = 0.80q_u$ for RQD = 100%
(Ref. 3, Section 7.2.1.4.1)

where:

Rock quality designation, RQD = 9 % (Ref. 9, minimum RQD in borings with lab testing)
Unconfined compressive strength, q_u = 908 ksf (Ref. 10, minimum UCS from lab test data)

q_p = 299.6 ksf

Nominal toe resistance:

$R_p = q_p A_p$ (Ref. 7, Eq. 10.7.3.8.6a-3)

where:

$R_n = R_s + R_p$ Pile toe area, A_p = 0.09 ft^2 (from Part A)

Total R_p = 25.8 kips (bedrock)

Nominal and factored geotechnical resistance:

$R_n = R_s + R_p$ (Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = 82 kips (bedrock)

$$R_r = \varphi_{stat} R_n$$ (Ref. 7, Eq. 10.7.3.8.6a-1)

where:

(Ref. 7 Tbl. 10.5.5.2.3-1)

$\varphi_{stat} =$ 0.35 for shaft and toe resistance of cohesive soil by the α-method

$\varphi_{stat} =$ 0.45 for shaft and toe resistance of cohesionless soil by the Nordlund method

$\varphi_{stat} =$ 0.45 for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Stiff Clay	Glacial Till	Pile Tip (Bedrock)	
R_s or R_p =	26.6	29.4	25.8	kips
φ_{stat} =	0.35	0.45	0.45	(Ref. 7, Table 10.5.5.2.3-1)
R_r per layer =	9.3	13.2	11.6	

Total R_r = 34 kips (bedrock)

Wave Equation Analysis

Nominal and factored geotechnical resistance obained from GRWEAP at 100% of the steel yield strength (Ref. 14):

$R_n =$ 310 kips (Ref. 14)

$\varphi_{dyn} =$ 0.5 (Assumption 11)

$R_f =$ 155 kips

Check that the geotechnical axial resistance Obtained from GRLWEAP (Ref. 15) is sufficient to support axial load on pile:

$P_u =$ 156 kips (from Part B)

$R_r =$ 155 kips < $P_u =$ 156 kips Not OK

D. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K = 0.65 (Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)

Unbraced length, l = 35.18 in (from Part B)

Minimum radius of gyration, r_s = 2.38 in (from Part A)

Check: Kl/r_s = 10 < 120 OK

CALCULATIONS

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Pier 5 (old Pier 1) H-Pile Analysis - Strength 1 Load Case 2, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \leq \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $$0.56\sqrt{\frac{E_{st}}{F_y}}$$ (Ref. 7, Table 6.9.4.2.1-1)
(Assumption 9)

where:

Half Flange width, b_f =	5.0	in	(from Part A)
Flange thickness, t_f =	0.320	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	36	ksi	(from Part A)

Check:

$$\frac{b}{t} = \quad 15.63 \quad < \quad 15.89 \quad = 0.56\sqrt{\frac{E_{st}}{F_y}}$$

The pile section is a non-slender element (Ref. 7, Article 6.9.4.1)

Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1)

Equivalent nominal yield resistance, P_o = 339 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	9.4	in^2	(from Part A)
Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, l =	35.18	in	(from Part B)
Minimum radius of gyration, r_s =	2.38	in	(from Part A)

P_e (flexural) = 29,207 kips

Date:	10/25/2024	**Made by:**	DEB
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Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D = 8.960 in $D = d - (2 \times t_f)$

Web thickness, t_w = 0.315 in (from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional

Nominal and factored structural resistance in axial compression:

$$P_n = P_o 0.658^{P_o/P_e}$$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$$P_n = 0.877 P_e$$ (Ref. 7, Eq. 6.9.4.1.1-2)

P_o / P_e = 0.01 < 2.25, so use Eq. 6.9.4.1.1-1

P_n = 338 kips

$$P_{r_{axial}} = \varphi_c P_n$$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c = 0.50 (Ref. 7, Article 6.5.4.2))

P_r = 169 kips

$$P_{r_{axial/flexural}} = \varphi_c P_n$$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c = 0.70 (Ref. 7, Article 6.5.4.2))

$P_{r,\ axial/\ flexural}$ = 236 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

P_u = 156 kips (from Part B)

P_r = 169 kips > P_u = 156 kips OK

E. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)	(Ref. 7, Eqn 6.12.2.2.1-4)	(Ref. 7, Eqn 6.12.2.2.1-5)
$\lambda_f = \frac{b_f}{2t_f}$	$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$	$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$
λ_f = 15.63	λ_{pf} = 10.79	λr_f = 23.56

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$

(Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1-\left(1-\frac{S_y}{Z_y}\right)\left(\frac{\lambda_f-\lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$

(Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	36	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	32.7	in^3	(from Part A)
Elastic section modulus about y, S_y =	10.7	in^3	(from Part A)
Plastic section modulus about x, Z_x =	36.0	in^3	(from Part A)
Plastic section modulus about y, Z_y =	16.2	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	1,251	508	in-kips

φ_f = 1.00 (Ref. 7, Article 6.5.4.2)

M_{rx} = 1,251 in-kips

M_{ry} = 508 in-kips

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} = 184 in-kips (from Part B)

M_{uy} = 43 in-kips (from Part B)

M_{rx} =	1,251	in-kips	>	M_{ux} =	184	in-kips	OK
M_{ry} =	508	in-kips	>	M_{uy} =	43	in-kips	OK

F. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r}+\left(\frac{M_{ux}}{M_{rx}}+\frac{M_{uy}}{M_{ry}}\right)\leq 1.0$$ if $P_u/P_{r,\text{ axial/ flexural}} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r}+\frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}}+\frac{M_{uy}}{M_{ry}}\right)\leq 1.0$$ if $P_u/P_{r,\text{ axial/ flexural}} \geq 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

P_u/P_r = 0.7 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.9 < 1.0 OK

G. Determine the nominal and factored structural resistance of a single pile in shear.

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_p = 0.58F_y d_w t_w$$ (Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y = 36 ksi (from Part A)

Web depth, d_w = 8.960 in $d_w = d - (2 \times t_f)$

Web thickness, t_w = 0.315 in (from Part A)

$$\frac{D}{t_w}\leq 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k = 5.0 (Ref. 7, Article 6.10.9.2)

D = 8.960 in D= d - (2 × tf)

t_w = 0.315 in (from Part A)

$$1.12\sqrt{\frac{Ek}{F_y}} =$$ 71.1 ksi (Ref. 7, Eq. 6.10.9.3.2-4)

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Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Check: $D/t_w = 28 < 71.1$ $C = 1.0$

$V_p = 59$ kips

$V_n = 59$ kips

$V_u \leq \varphi_v V_n$

$\varphi_v = 1.00$ (Ref. 7, Article 6.5.4.2)

$V_r = 59$ kips

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u = 4.1$ kips (from Part B)

$V_r = 59$ kips $>$ $V_u = 4.1$ kips OK

Date:	10/25/2024	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	JEF
Subject:	Rehabilitated Pier 5 (old Pier 1) H-Pile Analysis - Strength 1 Load Case 2, Corroded Section	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 2, and an estimated pile length of 29 feet, a maximum axial force of 156.1 kips, the corresponding strong axis (x-axis) moment of 184.3 in-kips (15.4 ft-kips) occurs in the HP10x42 piles at Pier 5 (old Pier 1) in Pile 14 of the 1983 portion of the Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity. The corresponding axial force of 156.1 kips (determined using FB-MultiPier) is greater than the geotechnical resistance of the existing corroded HP 10x42 piles determined from a wave equation analysis using the intact pile section properties, typical pile driving equipment used during the time of construction, and soil and rock conditions at the site. This evaluation considers section loss for an addtional 50 year design life, this evaluation does not consider downdrag. A summary table is presented below.

Old Pier 1 (new Pier 5, Pile 14) Summary Table		
Pile Length (ft)		29
Depth below Pile Head to Fixity (ft)		4
Geotechnical Axial Resistance (kips)	Nominal	310
	Factored	155
Structural Axial Resistance (kips)	Nominal	338
	Factored	169
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	104
	Factored	104
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	42
	Factored	42
Combined Axial and Flexural Interaction Ratio		0.9
Shear Resistance (kips)	Nominal	59
	Factored	59

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

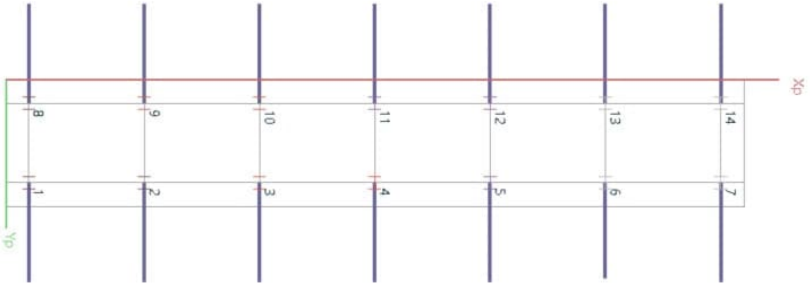


Figure 1b: Profile View of Pile Cap (Transverse)

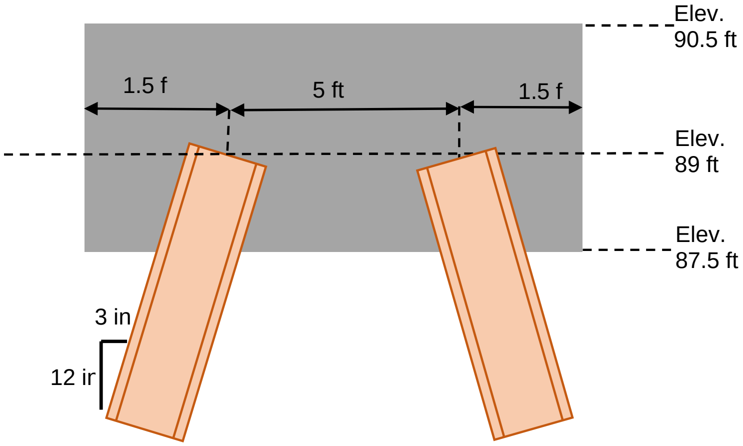


Figure 1c: 3D View of Pile Model

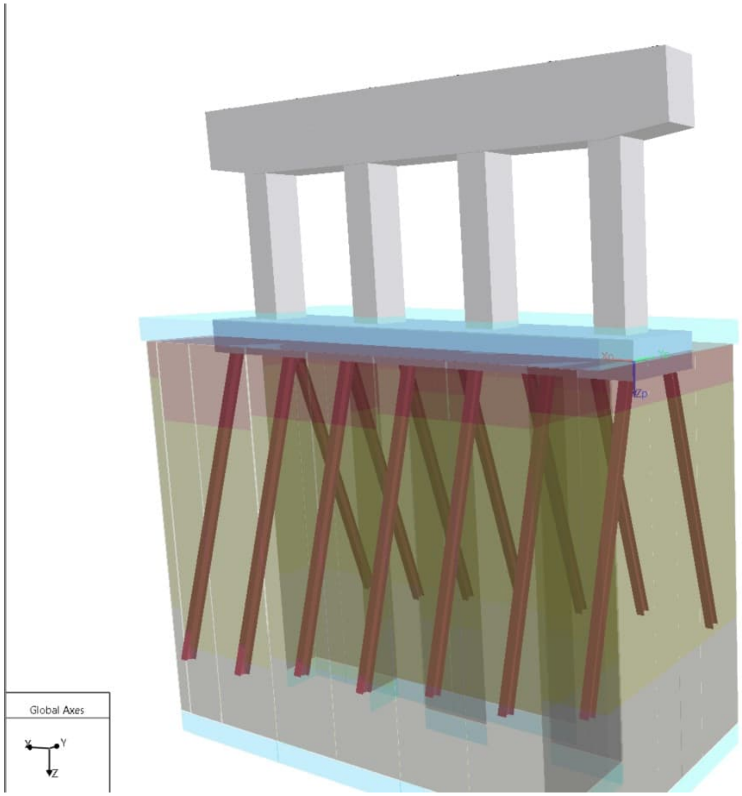
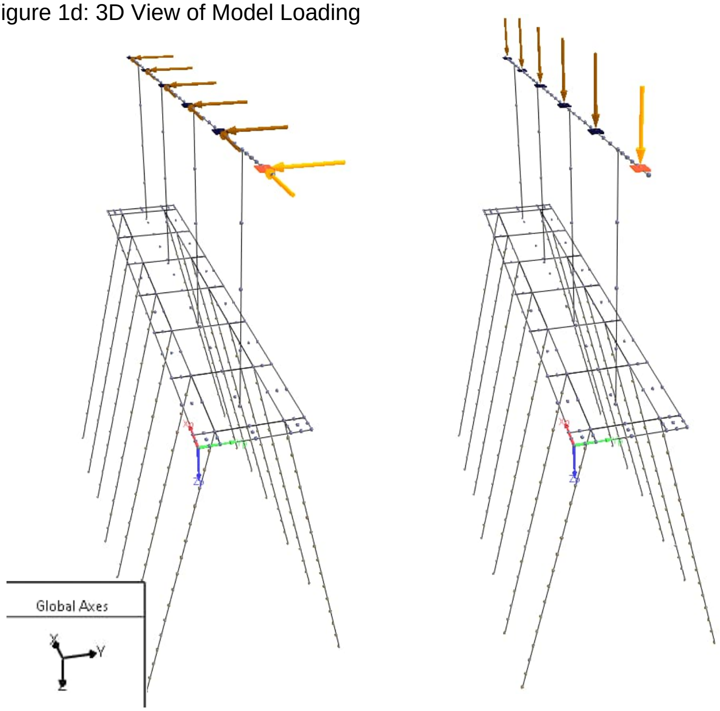


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

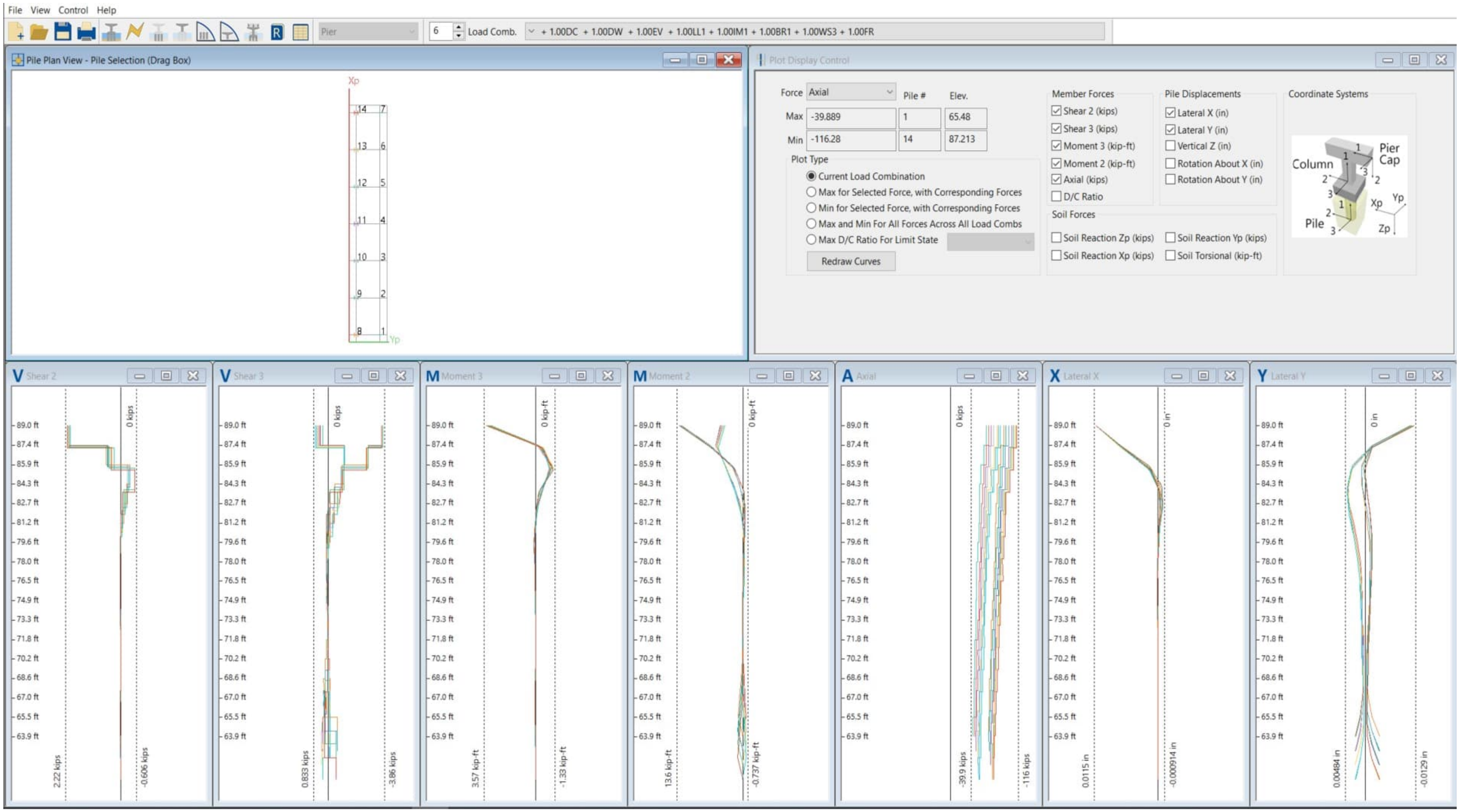
Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Stiff Clay	1	Cohesive	89.0	82.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1182	1182	-	-	-
Glacial Till	2	Cohesionless	82.1	61.4	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	434		-	-	-
Bedrock	3	Rock	61.4	55.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		3600	310	908,000

Notes:

1. The in-situ soil stratigraphy is based on Sta. 113+31 in Profile G-G' (Sheet X Interpretive Subsurface Cross Section G-G'), At the existing Pier 1, for 31 foot length piles.
2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.
3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).
4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.
5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).
6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).
7. Unit tip resistance values for rock obtained from WSP calculation package titled Pile Drivability at Pier 5 (existing Pier 1) - HP 10x42.
8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

Prepared:	DEB
Checked:	ATM
Reviewed:	CCB

Case 2 Service I



6 Load Comb. + 1.00DC + 1.00DW + 1.00EV + 1.00LL1 + 1.00IM1 + 1.00BR1 + 1.00WS3 + 1.00FR

	Force	Pile #	Elev.
	Axial		
Max	-39.889	1	65.48
Min	-116.28	14	87.213

Plot Type
- ● Current Load Combination
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Combs
- ○ Max D/C Ratio For Limit State

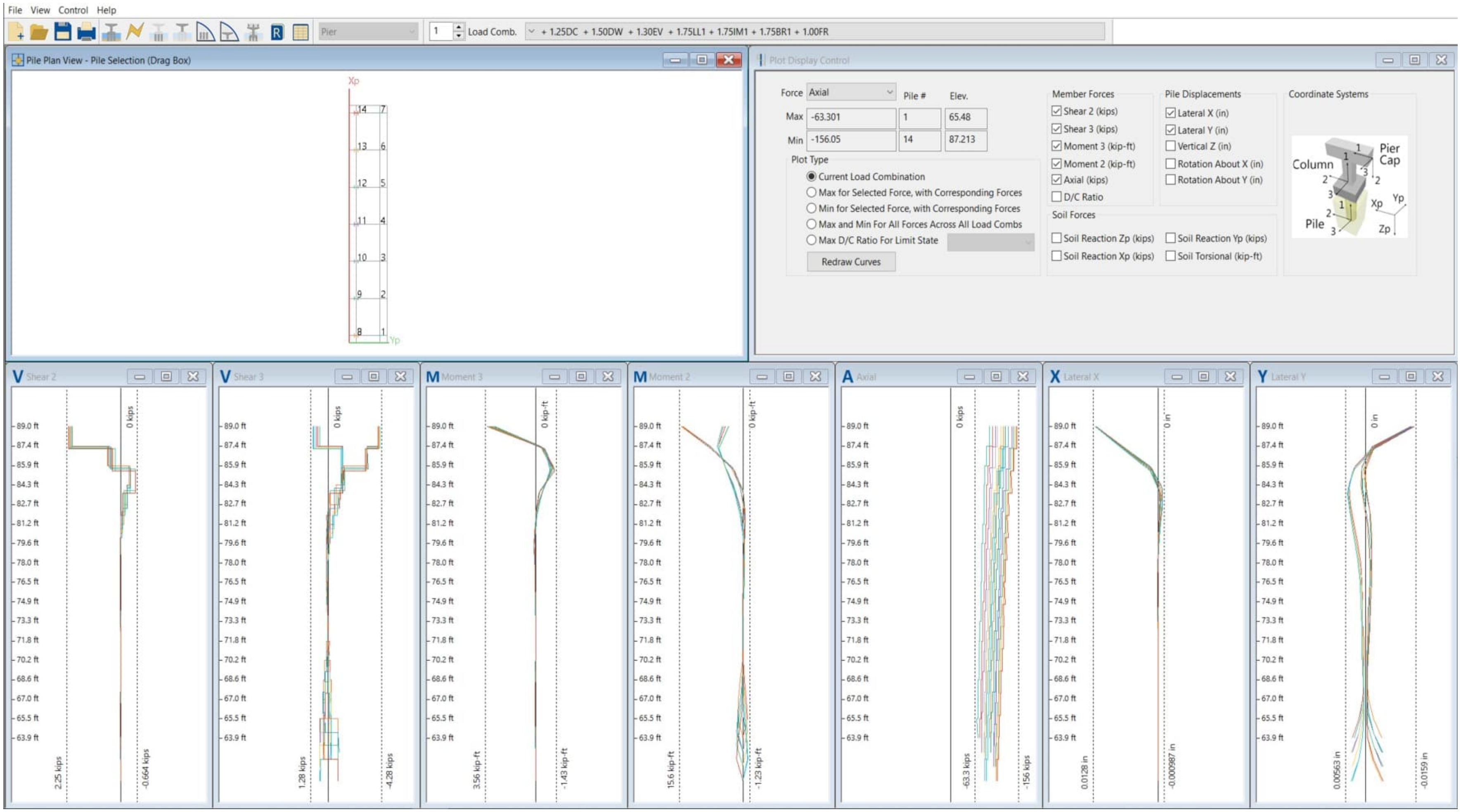
Redraw Curves

Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

Case 2 Strength I



Pier

1 Load Comb. + 1.25DC + 1.50DW + 1.30EV + 1.75LL1 + 1.75IM1 + 1.75BR1 + 1.00FR

Force: Axial

		Pile #	Elev.
Max	-63.301	1	65.48
Min	-156.05	14	87.213

Plot Type
- ● Current Load Combination
- ○ Max for Selected Force, with Corresponding Forces
- ○ Min for Selected Force, with Corresponding Forces
- ○ Max and Min For All Forces Across All Load Combs
- ○ Max D/C Ratio For Limit State

Member Forces
- ☑ Shear 2 (kips)
- ☑ Shear 3 (kips)
- ☑ Moment 3 (kip-ft)
- ☑ Moment 2 (kip-ft)
- ☑ Axial (kips)
- ☐ D/C Ratio

Pile Displacements
- ☑ Lateral X (in)
- ☑ Lateral Y (in)
- ☐ Vertical Z (in)
- ☐ Rotation About X (in)
- ☐ Rotation About Y (in)

Soil Forces
- ☐ Soil Reaction Zp (kips)
- ☐ Soil Reaction Xp (kips)
- ☐ Soil Reaction Yp (kips)
- ☐ Soil Torsional (kip-ft)

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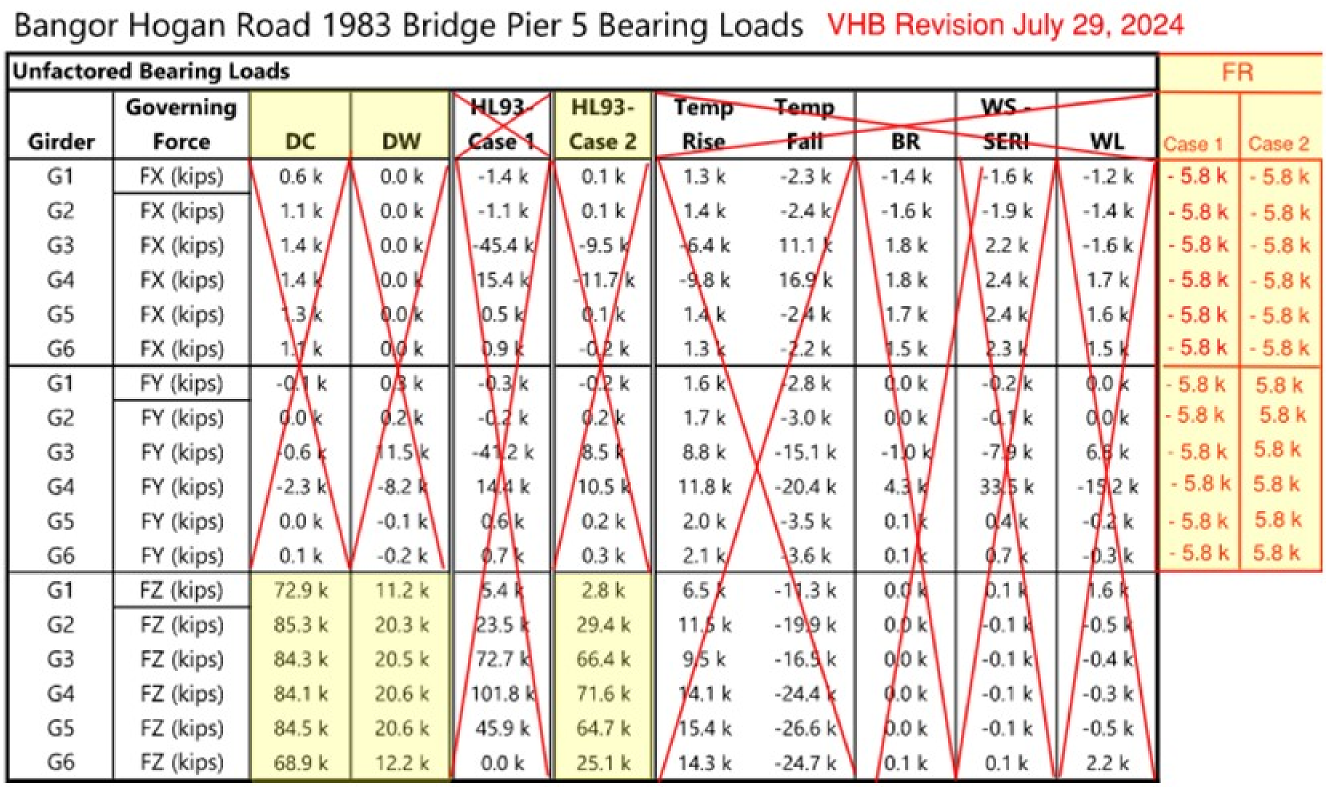
Bangor Hogan Road 1983 Bridge Pier 5 Bearing Loads

Unfactored Bearing Loads

Girder	Governing Force	DC	DW	HL93-Case 1	HL93-Case 2	Temp Rise	Temp Fall	BR	WS - SERI	WL
G1	FX (kips)	0.6 k	0.0 k	-1.4 k	0.1 k	1.3 k	-2.3 k	-1.4 k	-1.6 k	-1.2 k
G2	FX (kips)	1.1 k	0.0 k	-1.1 k	0.1 k	1.4 k	-2.4 k	-1.6 k	-1.9 k	-1.4 k
G3	FX (kips)	1.4 k	0.0 k	-45.4 k	-9.5 k	-6.4 k	11.1 k	1.8 k	2.2 k	-1.6 k
G4	FX (kips)	1.4 k	0.0 k	15.4 k	-11.7 k	-9.8 k	16.9 k	1.8 k	2.4 k	1.7 k
G5	FX (kips)	1.3 k	0.0 k	0.5 k	0.1 k	1.4 k	-2.4 k	1.7 k	2.4 k	1.6 k
G6	FX (kips)	1.1 k	0.0 k	0.9 k	-0.2 k	1.3 k	-2.2 k	1.5 k	2.3 k	1.5 k
G1	FY (kips)	-0.1 k	0.3 k	-0.3 k	-0.2 k	1.6 k	-2.8 k	0.0 k	-0.2 k	0.0 k
G2	FY (kips)	0.0 k	0.2 k	-0.2 k	0.2 k	1.7 k	-3.0 k	0.0 k	-0.1 k	0.0 k
G3	FY (kips)	-0.6 k	11.5 k	-41.2 k	8.5 k	8.8 k	-15.1 k	-1.0 k	-7.9 k	6.8 k
G4	FY (kips)	-2.3 k	-8.2 k	14.4 k	10.5 k	11.8 k	-20.4 k	4.3 k	33.5 k	-15.2 k
G5	FY (kips)	0.0 k	-0.1 k	0.6 k	0.2 k	2.0 k	-3.5 k	0.1 k	0.4 k	-0.2 k
G6	FY (kips)	0.1 k	-0.2 k	0.7 k	0.3 k	2.1 k	-3.6 k	0.1 k	0.7 k	-0.3 k
G1	FZ (kips)	72.9 k	11.2 k	5.4 k	2.8 k	6.5 k	-11.3 k	0.0 k	0.1 k	1.6 k
G2	FZ (kips)	85.3 k	20.3 k	23.5 k	29.4 k	11.5 k	-19.9 k	0.0 k	-0.1 k	-0.5 k
G3	FZ (kips)	84.3 k	20.5 k	72.7 k	66.4 k	9.5 k	-16.5 k	0.0 k	-0.1 k	-0.4 k
G4	FZ (kips)	84.1 k	20.6 k	101.8 k	71.6 k	14.1 k	-24.4 k	0.0 k	-0.1 k	-0.3 k
G5	FZ (kips)	84.5 k	20.6 k	45.9 k	64.7 k	15.4 k	-26.6 k	0.0 k	-0.1 k	-0.5 k
G6	FZ (kips)	68.9 k	12.2 k	0.0 k	25.1 k	14.3 k	-24.7 k	0.1 k	0.1 k	2.2 k

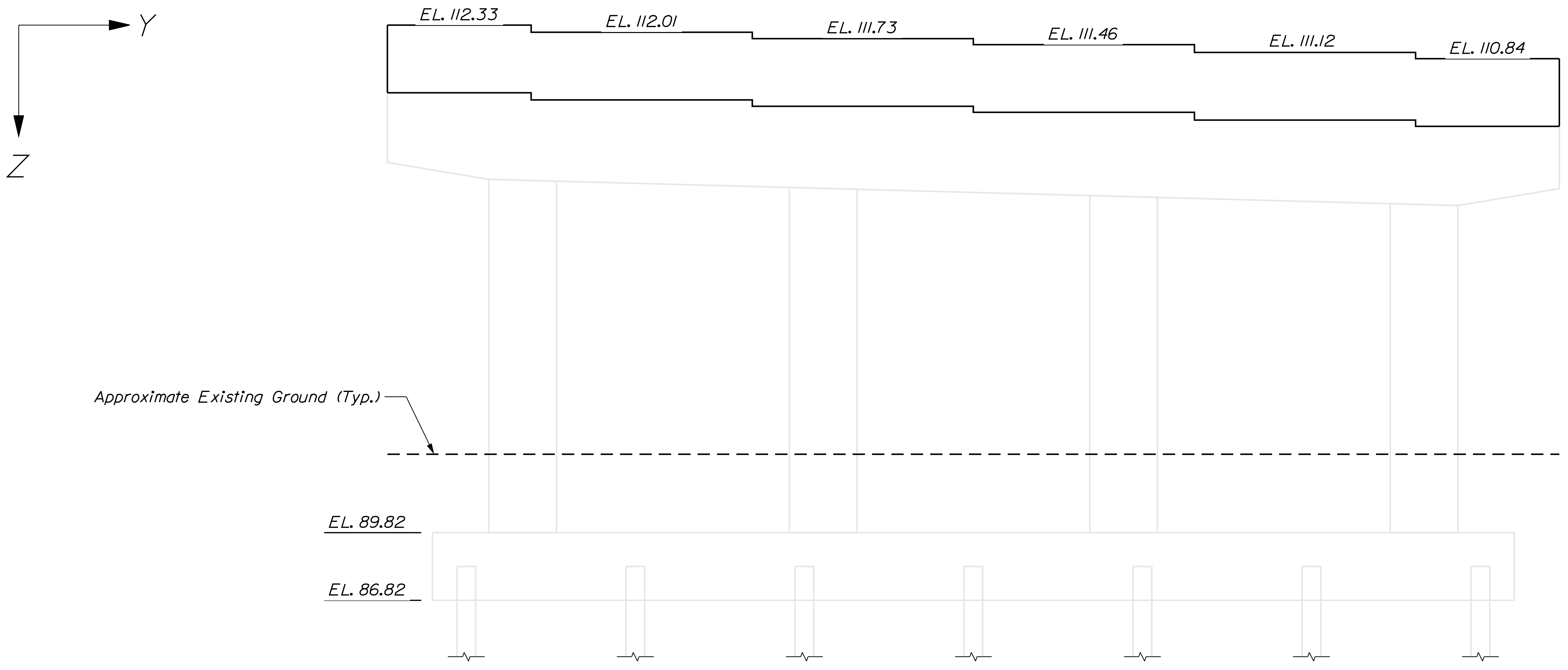


Bangor Hogan Road 1983 Bridge Pier 5 Bearing Loads VHB Revision July 29, 2024

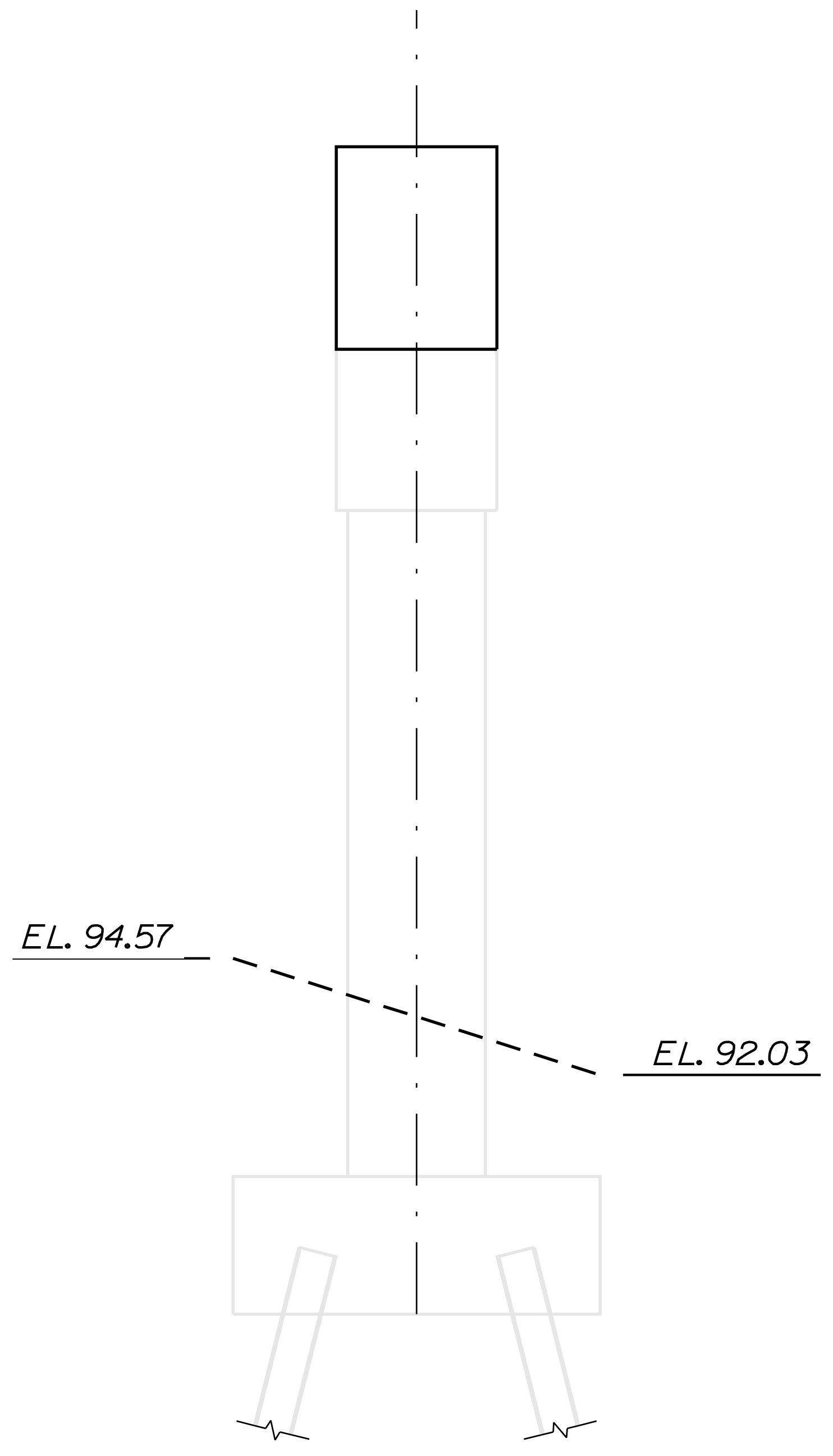
Unfactored Bearing Loads

Girder	Governing Force	DC	DW	~~HL93-Case 1~~	HL93-Case 2	~~Temp Rise~~	~~Temp Fall~~	BR	~~WS - SERI~~	WL	FR Case 1	FR Case 2
G1	FX (kips)	~~0.6 k~~	~~0.0 k~~	~~-1.4 k~~	~~0.1 k~~	~~1.3 k~~	~~-2.3 k~~	~~-1.4 k~~	~~-1.6 k~~	~~-1.2 k~~	- 5.8 k	- 5.8 k
G2	FX (kips)	~~1.1 k~~	~~0.0 k~~	~~-1.1 k~~	~~0.1 k~~	~~1.4 k~~	~~-2.4 k~~	~~-1.6 k~~	~~-1.9 k~~	~~-1.4 k~~	- 5.8 k	- 5.8 k
G3	FX (kips)	~~1.4 k~~	~~0.0 k~~	~~-45.4 k~~	~~-9.5 k~~	~~-6.4 k~~	~~11.1 k~~	~~1.8 k~~	~~2.2 k~~	~~-1.6 k~~	- 5.8 k	- 5.8 k
G4	FX (kips)	~~1.4 k~~	~~0.0 k~~	~~15.4 k~~	~~-11.7 k~~	~~-9.8 k~~	~~16.9 k~~	~~1.8 k~~	~~2.4 k~~	~~1.7 k~~	- 5.8 k	- 5.8 k
G5	FX (kips)	~~1.3 k~~	~~0.0 k~~	~~0.5 k~~	~~0.1 k~~	~~1.4 k~~	~~-2.4 k~~	~~1.7 k~~	~~2.4 k~~	~~1.6 k~~	- 5.8 k	- 5.8 k
G6	FX (kips)	~~1.1 k~~	~~0.0 k~~	~~0.9 k~~	~~-0.2 k~~	~~1.3 k~~	~~-2.2 k~~	~~1.5 k~~	~~2.3 k~~	~~1.5 k~~	- 5.8 k	- 5.8 k
G1	FY (kips)	~~-0.1 k~~	~~0.3 k~~	~~-0.3 k~~	~~-0.2 k~~	~~1.6 k~~	~~-2.8 k~~	~~0.0 k~~	~~-0.2 k~~	~~0.0 k~~	- 5.8 k	5.8 k
G2	FY (kips)	~~0.0 k~~	~~0.2 k~~	~~-0.2 k~~	~~0.2 k~~	~~1.7 k~~	~~-3.0 k~~	~~0.0 k~~	~~-0.1 k~~	~~0.0 k~~	- 5.8 k	5.8 k
G3	FY (kips)	~~-0.6 k~~	~~11.5 k~~	~~-41.2 k~~	~~8.5 k~~	~~8.8 k~~	~~-15.1 k~~	~~-1.0 k~~	~~-7.9 k~~	~~6.8 k~~	- 5.8 k	5.8 k
G4	FY (kips)	~~-2.3 k~~	~~-8.2 k~~	~~14.4 k~~	~~10.5 k~~	~~11.8 k~~	~~-20.4 k~~	~~4.3 k~~	~~33.5 k~~	~~-15.2 k~~	- 5.8 k	5.8 k
G5	FY (kips)	~~0.0 k~~	~~-0.1 k~~	~~0.6 k~~	~~0.2 k~~	~~2.0 k~~	~~-3.5 k~~	~~0.1 k~~	~~0.4 k~~	~~-0.2 k~~	- 5.8 k	5.8 k
G6	FY (kips)	~~0.1 k~~	~~-0.2 k~~	~~0.7 k~~	~~0.3 k~~	~~2.1 k~~	~~-3.6 k~~	~~0.1 k~~	~~0.7 k~~	~~-0.3 k~~	- 5.8 k	5.8 k
G1	FZ (kips)	72.9 k	11.2 k	~~5.4 k~~	2.8 k	~~6.5 k~~	~~-11.3 k~~	~~0.0 k~~	~~0.1 k~~	~~1.6 k~~		
G2	FZ (kips)	85.3 k	20.3 k	~~23.5 k~~	29.4 k	~~11.5 k~~	~~-19.9 k~~	~~0.0 k~~	~~-0.1 k~~	~~-0.5 k~~		
G3	FZ (kips)	84.3 k	20.5 k	~~72.7 k~~	66.4 k	~~9.5 k~~	~~-16.5 k~~	~~0.0 k~~	~~-0.1 k~~	~~-0.4 k~~		
G4	FZ (kips)	84.1 k	20.6 k	~~101.8 k~~	71.6 k	~~14.1 k~~	~~-24.4 k~~	~~0.0 k~~	~~-0.1 k~~	~~-0.3 k~~		
G5	FZ (kips)	84.5 k	20.6 k	~~45.9 k~~	64.7 k	~~15.4 k~~	~~-26.6 k~~	~~0.0 k~~	~~-0.1 k~~	~~-0.5 k~~		
G6	FZ (kips)	68.9 k	12.2 k	~~0.0 k~~	25.1 k	~~14.3 k~~	~~-24.7 k~~	~~0.1 k~~	~~0.1 k~~	~~2.2 k~~		

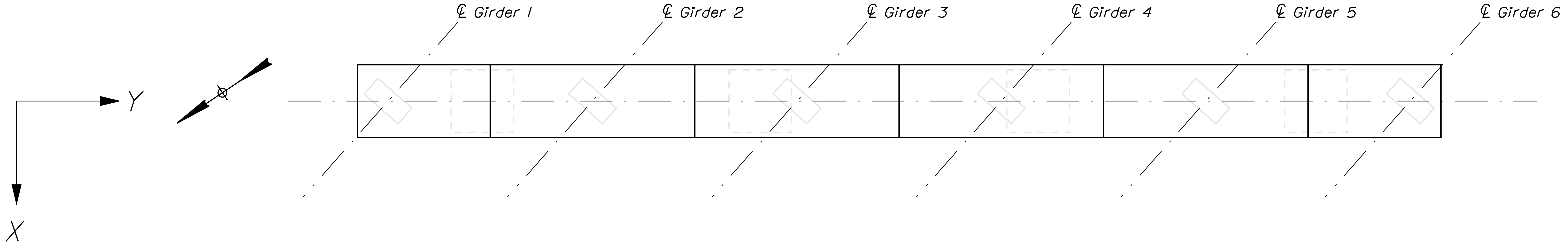
NOTE:
INFORMATION ON THIS SHEET IS FOR
PIER 5 MODELING PURPOSES ONLY.



PROPOSED PIER ELEVATION



PROPOSED PIER SECTION



PROPOSED PIER CAP PLAN

EXISTING FOOTING PLAN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

PROJ. MANAGER	R. MOULTON	BY	DATE
DESIGN-DETAILED			7/12/24
CHECKED-REVIEWED	YP	SMH	7/12/24
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD BRIDGE
I-95
BANGOR PENOBSCOT

EASTBOUND BRIDGE PIER 5
REHAB DETAILS

SHEET NUMBER
B1
OF 1

Filename: ...\031_EB_Existing_Pier_5.dgn Division: HIGHWAY Username: jharris Date:7/22/2024

Attachment 5

CALCULATIONS

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 1 (Old Abutment 2) - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

OBJECTIVE

Evaluate the downdrag load acting on the HP 10x42 steel pile with the maximum loading at the Abutment 1 (Old Abutment 2) (Pile 16) of the bridge carrying Hogan Road Eastbound over I-95. This analysis uses estimated settlement from 1983 to present-day.

METHOD

Use APile (Ref. 1) to estimate the shaft resistance contributing to downdrag.

REFERENCES

1. Ensoft, Inc. APILE Version 2019.9.11 - Offshore. Release date October 15, 2021.
2. Maine Department of Transportation Bridge Program. Preliminary Design Report: Hogan Road / I-95 Bridge #5823 over Interstate 95, Exit 187 Diverging Diamond Interchange, Bangor, Maine, Federal Project #1859510, WIN 018595.10. Draft 2-3-2023.
3. WSP calculation titled "Hogan Rd Highway Settlement Embankment 1983 10 28 24.xlsx", dated 10/28/2024.
4. WSP Interpretive Subsurface Profile E-E', included as part of the Geotechnical Design Report.
5. WSP calculations of interpreted soil parameters based on field and laboratory data.
6. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
7. AASHTO LRFD Bridge Design Specifications, 9th Edition. 2020.
8. WSP pile resistance calculation and FB-Multipier model for Abutment 1 (old Abutment 2), dated 10/25/2024
9. VHB computation package titled "1983 Bridge Foundation Loads", dated 4/5/2024.

ATTACHMENTS

1. Profile of estimated settlement beneath Abutment 1 (Old Abutment 2) at 41 years after embankment widening construction at the location of the pile with maximum axial loading
2. Output from APile analysis

ASSUMPTIONS

1. The widened portion of the existing Abutment 1 (Old Abutment 2) is supported on eighteen HP10x42 steel piles, based on the as-built bridge widening plans dated 1983 in Ref. 2, Appendix C. The downdrag is evaluated for the pile with maximum axial loading based on the proposed abutment renovation loads provided by VHB (Ref. 9) and distributed to the individual piles by WSP using FB-MultiPier (Ref. 8). The estimated pile length at this location as shown in the 1983 as-built plans (Ref. 2, Appendix C, Sheet 13) is 13.4 feet below the base of the abutment. Based on WSP's interpreted subsurface profile along the existing abutment (Ref. 4), the piles are assumed to have been driven into weathered bedrock.

2. The downdrag depth is based on the total settlement estimated to have occurred up to present-day due to the 1983 embankment widening at the Abutment 1 (Old Abutment 2) at the location of the pile with maximum loading (Ref. 3).

3. The FHWA automated computation method provided in APile is used for the software computations of unit load transfers and axial pile capacity.

4. Since APile requires soil layers to be defined to a depth of at least the pile tip plus two pile diameters, the bedrock is modeled as "sand" with a friction angle of 50 degrees.

5. Downdrag is calculated with an uncorroded pile section to represent the maximum possible downdrag load.

6. Abutment designations are based on VHB designations in Ref. 9.

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 1 (Old Abutment 2) - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CALCULATION

1. Establish soil profile and pile properties.

The soil profile used in the downdrag analysis is based on Ref. 4 at the centerline of the Abutment 1 (Old Abutment 2). The soil properties used in the downdrag analysis are taken from Ref. 5 and are based on correlations to field test and laboratory test data.

Base of Abutment 1 elevation =	106.2	feet	(Ref. 2, Appendix C, Sheet 11)
Water table elevation =	102.4	feet	(Ref. 4)

Soil Layer	Elevation (feet)		Depth below Abutment (feet)		Effective Unit Weight (pcf)	Friction Angle (°)	Undrained Shear Strength (psf)	
	Top	Bottom	Top	Bottom			Top	Bottom
Existing Fill	106.2	104.2	0.0	2.0	125	34	-	-
Stiff Clay	104.2	102.4	2.0	3.8	122	-	1340	1340
Stiff Clay	102.4	96.9	3.8	9.3	59.6	-	1340	1340
Glacial Till	96.9	95.3	9.3	10.9	67.6	35	-	-
Bedrock	95.3	-	10.9	-	95.6	50	-	-

Pile steel elastic modulus =	29,000,000	psi	(typical value for steel)
Pile length =	13.4	ft	(Assumption 1)
Pile section box perimeter =	39.6	in	(using pile flange width and depth from Ref. 6)
Pile section tip area =	12.4	in^2	(Ref. 6)
Pile nominal weight =	0.042	kip/ft	(Ref. 6)

2. Identify the depth below the abutment to which settlement is large enough to fully develop downdrag loading.

Per Ref. 7 Article 3.11.8, downdrag can be assumed to fully develop for the length of pile with settlement equal to or greater than 0.4 inches. As calculated in Ref. 3 - Attachment 10, settlement beneath Abutment 1 (Old Abutment 2) up to present-day (41 years after construction) due to construction of the widened bridge abutment and approach embankment is estimated to be greater than or equal to 0.4 inches down to an elevation of 97.2 feet (depth of 9.0 feet below the base of the abutment). Thus, downdrag loading is assumed to be fully developed for the upper 9.0 feet of the pile below the base of the abutment.

Per Ref. 7 Article 10.7.1.6.2 for piles end bearing on rock downdrag shall be considered at the strength and extreme limit state.

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 1 (Old Abutment 2) - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

3. Use APile to estimate shaft resistance and total downdrag load.

Downdrag Load Factors:
(Ref. 7, Tables 3.4.1-1 and 3.4.1-2)

Soil Type	Strength I
Sand	1.05
Clay	1.40

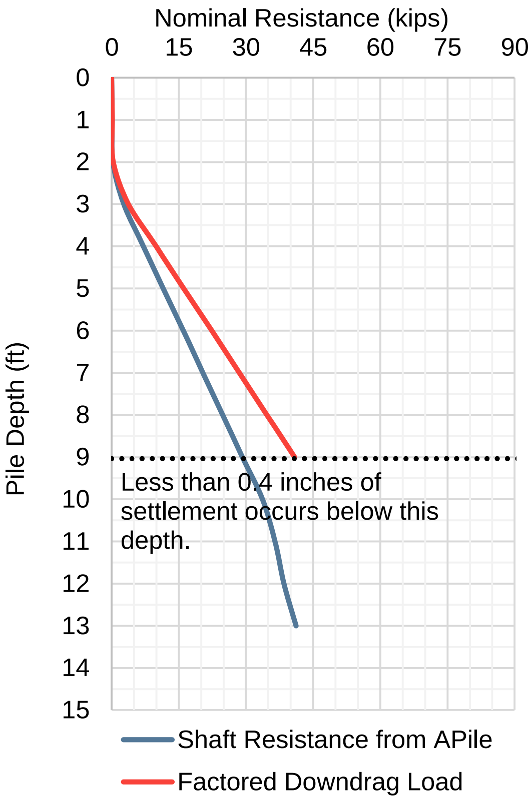
Abutment Loads:

	Strength I	
Per Pile	117.33	kips

(Ref. 8)

Shaft resistance and downdrag load for each foot of pile length:

Pile Depth below Base of Abutment	Soil Type from APile	Shaft Resistance from APile	Factored Downdrag Load	Total Factored Axial Load*
(feet)		(kips)	(kips)	(kips)
0	Sand	0.0	0.0	117.3
1	Sand	0.1	0.1	117.5
2	Clay	0.3	0.4	117.8
3	Clay	2.7	3.7	121.2
4	Clay	7.1	9.9	127.4
5	Clay	11.5	16.1	133.6
6	Clay	16.0	22.4	139.9
7	Clay	20.4	28.5	146.1
8	Clay	24.8	34.7	152.4
9	**Clay**	**29.2**	**40.8**	**158.6**
10	Sand	33.7		117.8
11	Sand	36.5		117.8
12	Sand	38.5		117.8
13	Sand	41.2		117.9

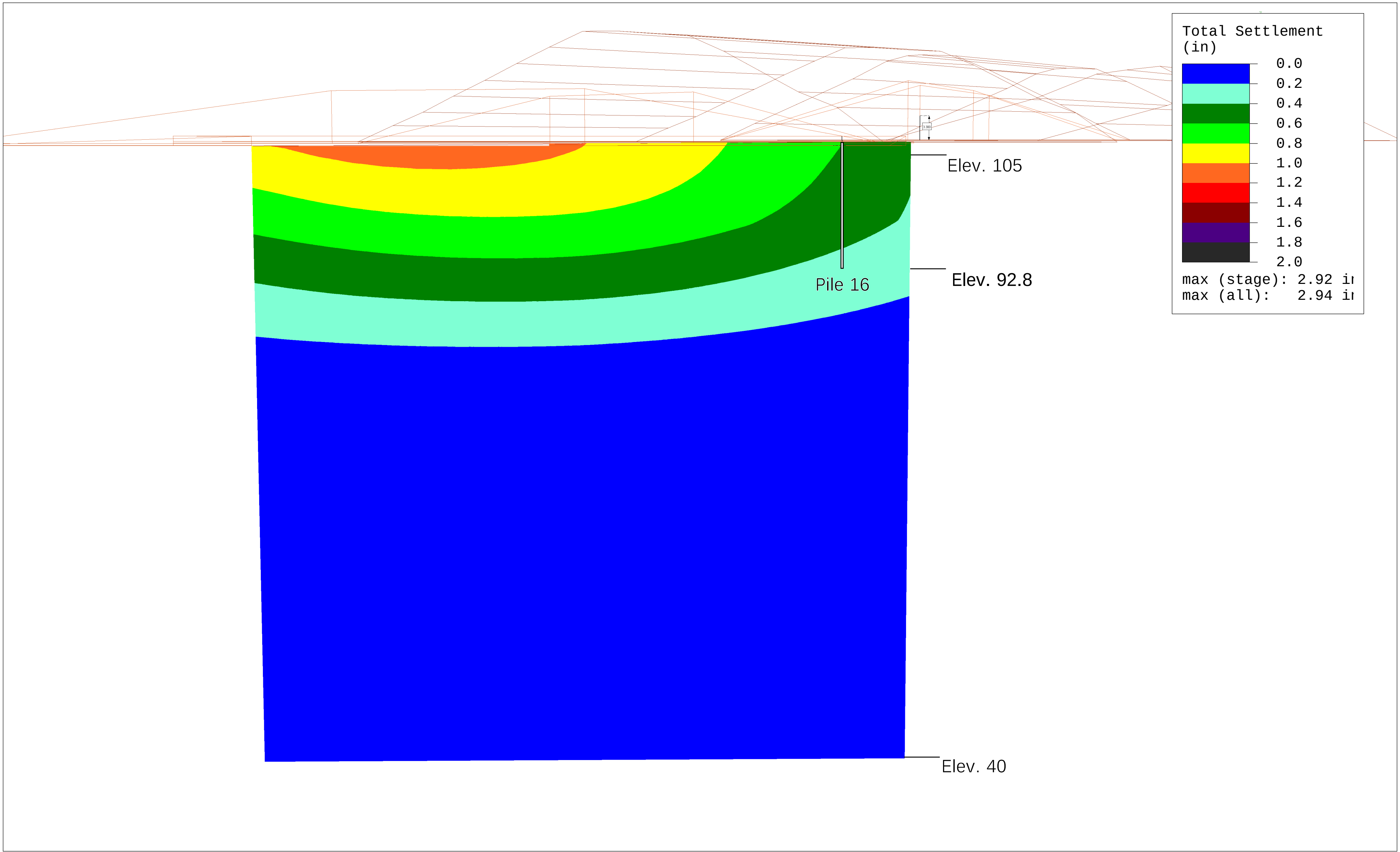


*Strength I, including Substructure Loading, Downdrag, and Pile Weight

Strength I factored downdrag load = 40.8 kips per pile

CONCLUSIONS

Based on the analysis of settlement between 1983 and present-day, downdrag is estimated to have developed along the upper 9 feet of pile 16 below the base of the Abutment 1 (Old Abutment 2). A factored downdrag load of 40.8 kips per pile for the Strength I limit state was calculated.



wsp	*Project* MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.004			
	Analysis Description Total Settlement along Abutment 1 (Old Abutment 2) (Sta. 109+43) at Present-Day (41 years after embankment widening)			
	Drawn By LMP	*Checked By* KAR	*Reviewed By* CCB	*Scale* As shown
SETTLE3 5.023	*Date* 11/1/2024	*File Name* Hogan Rd Highway Interchange 1958 surface 1983Fill 10 24 -v3.s3z		Attachment 1

Attachment 2

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

                      APILE for Windows, Version 2019.9.11

                       Serial Number :  264781183

                       A Program for Analyzing the Axial Capacity
                       and Short-term Settlement of Driven Piles
                                 under Axial Loading.
                     (c) Copyright ENSOFT, Inc., 1987-2019
                           All Rights Reserved

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


     This program is licensed to :

     WSP
     APILE Global, Global License

     Path to file locations      : C:\Users\USLP710461\WSP O365\MaineDOT Bangor
Hogan Road Bridge & DDI - Project Files\5 Technical Work\GDR Pile & Abutment
Design\Existing Bridge - Downdrag\APile Models\Abutement 1 Pile 16 - Max loading\
     Name of input data file     : HP10x42_Abutment1 Pile 16.ap9d
     Name of output file         : HP10x42_Abutment1 Pile 16.ap9o
     Name of plot output file    : HP10x42_Abutment1 Pile 16.ap9p

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

               Date:  October 24, 2024     Time:  10:40:33



1
          *********************
          * INPUT INFORMATION *
          *********************



          Existing Pile Downdrag at  Abutment 2

          DESIGNER   : LMP

          JOB NUMBER : 31404817.004


          METHOD FOR UNIT LOAD TRANSFERS :
```

```
- FHWA (Federal Highway Administration)
  Unfactored Unit Side Friction and Unit Side Resistance are used.



COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)


TYPE OF LOADING :
- COMPRESSION


PILE TYPE :

H-Pile/Steel Pile


DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY     = 0.290E+08 PSI
- CROSS SECTION AREA        =     12.40 IN2


NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL     =     13.40 FT.
- BATTER ANGLE              =      0.00 DEG
- PILE STICKUP LENGTH, PSL  =      0.00 FT.
- ZERO FRICTION LENGTH, ZFL =      0.00 FT.
- PERIMETER OF PILE         =     39.60 IN.
- TIP AREA OF PILE          =     12.40 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION       =      1.00 FT.



SOIL INFORMATIONS :



                  LATERAL    EFFECTIVE    FRICTION      Nq
          SOIL    EARTH      UNIT         ANGLE       FACTOR
 DEPTH    TYPE    PRESSURE   WEIGHT       DEGREES      FHWA
  FT.                        LB/FT^3
   0.00   SAND      0.80*      125.00        34.00     55.60**
   2.00   SAND      0.80*      125.00        34.00     55.60**
   2.00   CLAY      0.80*      122.00         0.00      4.80**
   3.80   CLAY      0.80*      122.00         0.00      4.80**
   3.80   CLAY      0.80*       59.60         0.00      4.80**
```

```
   9.30   CLAY      0.80*       59.60         0.00      4.80**
   9.30   SAND      0.80*       67.60        35.00     64.00**
  10.90   SAND      0.80*       67.60        35.00     64.00**
  10.90   SAND      0.80*       95.60        50.00    475.00**
  25.00   SAND      0.80*       95.60        50.00    475.00**

*  VALUE ASSUMED BY THE PROGRAM
** VALUE ESTIMATED BY THE PROGRAM BASED ON FRICTION ANGLE



MAXIMUM   MAXIMUM   UNDISTURB  REMOLDED
UNIT      UNIT      SHEAR      SHEAR     BLOW   UNIT SKIN  UNIT END
FRICTION  BEARING   STRENGTH   STRENGTH  COUNT  FRICTION   BEARING
  KSF       KSF        KSF       KSF               KSF       KSF
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
  WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
  PLAN TO LIMIT THE COMPUTED DATA.



         LRFD FACTOR  LRFD FACTOR
          ON UNIT      ON UNIT
 DEPTH    FRICTION     BEARING
  FT.
   0.00    1.000        1.000
   2.00    1.000        1.000
   2.00    1.000        1.000
   3.80    1.000        1.000
   3.80    1.000        1.000
   9.30    1.000        1.000
   9.30    1.000        1.000
  10.90    1.000        1.000
  10.90    1.000        1.000
  25.00    1.000        1.000


1
          **********************
```

```
  * COMPUTATION RESULT *
  **********************



                    **********************
                    *  FED. HWY. METHOD  *
                    **********************

     PILE          SKIN          END       ULTIMATE
  PENETRATION   FRICTION       BEARING     CAPACITY
      FT.          KIP           KIP          KIP
      0.00           0.0           0.2           0.2
      1.00           0.1           0.4           0.5
      2.00           0.3           0.8           1.1
      3.00           2.7           1.0           3.7
      4.00           7.1           1.0           8.2
      5.00          11.5           1.0          12.6
      6.00          16.0           1.0          17.0
      7.00          20.4           1.0          21.4
      8.00          24.8           1.0          25.9
      9.00          29.2           1.4          30.7
     10.00          33.7           7.2          40.9
     11.00          36.5          17.5          54.0
     12.00          38.5          28.5          67.0
     13.00          41.2          36.3          77.5


    NOTES:
    - AN ASTERISK IS PLACED IN THE END-BEARING COLUMN
      IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
      OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.



        *************************************************
        * COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
        * CURVES FOR AXIAL LOADING                      *
        *************************************************


T-Z CURVE   NO. OF   DEPTH TO CURVE   LOAD TRANSFER   PILE MOVEMENT
   NO.      POINTS        FT.              PSI             IN.

    1         10      0.4167E-01
                                       0.0000E+00      0.0000E+00
                                       0.4336E-02      0.2017E-01
                                       0.7226E-02      0.3908E-01
                                       0.1084E-01      0.7185E-01
                                       0.1301E-01      0.1008E+00
                                       0.1445E-01      0.1261E+00
```

```
                                   0.1445E-01      0.2521E+00
                                   0.1445E-01      0.3782E+00
                                   0.1445E-01      0.6303E+00
                                   0.1445E-01      0.2521E+01
2         10      0.1000E+01
                                   0.0000E+00      0.0000E+00
                                   0.1041E+00      0.2017E-01
                                   0.1734E+00      0.3908E-01
                                   0.2601E+00      0.7185E-01
                                   0.3122E+00      0.1008E+00
                                   0.3469E+00      0.1261E+00
                                   0.3469E+00      0.2521E+00
                                   0.3469E+00      0.3782E+00
                                   0.3469E+00      0.6303E+00
                                   0.3469E+00      0.2521E+01
3         10      0.1958E+01
                                   0.0000E+00      0.0000E+00
                                   0.2038E+00      0.2017E-01
                                   0.3396E+00      0.3908E-01
                                   0.5094E+00      0.7185E-01
                                   0.6113E+00      0.1008E+00
                                   0.6793E+00      0.1261E+00
                                   0.6793E+00      0.2521E+00
                                   0.6793E+00      0.3782E+00
                                   0.6793E+00      0.6303E+00
                                   0.6793E+00      0.2521E+01
4         10      0.2042E+01
                                   0.0000E+00      0.0000E+00
                                   0.3158E+00      0.2017E-01
                                   0.5263E+00      0.3908E-01
                                   0.7894E+00      0.7185E-01
                                   0.9473E+00      0.1008E+00
                                   0.1053E+01      0.1261E+00
                                   0.9473E+00      0.2521E+00
                                   0.9473E+00      0.3782E+00
                                   0.9473E+00      0.6303E+00
                                   0.9473E+00      0.2521E+01
5         10      0.2900E+01
                                   0.0000E+00      0.0000E+00
                                   0.2533E+01      0.2017E-01
                                   0.4222E+01      0.3908E-01
                                   0.6333E+01      0.7185E-01
                                   0.7600E+01      0.1008E+00
                                   0.8444E+01      0.1261E+00
                                   0.7600E+01      0.2521E+00
                                   0.7600E+01      0.3782E+00
                                   0.7600E+01      0.6303E+00
                                   0.7600E+01      0.2521E+01
6         10      0.3758E+01
                                   0.0000E+00      0.0000E+00
                                   0.2792E+01      0.2017E-01
```

```
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 7         10      0.3842E+01
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 8         10      0.6550E+01
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 9         10      0.9258E+01
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
10         10      0.9342E+01
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.9306E+01      0.2521E+00
                                    0.9306E+01      0.3782E+00
                                    0.9306E+01      0.6303E+00
```

```
                                    0.9306E+01      0.2521E+01
11         10      0.1010E+02
                                    0.0000E+00      0.0000E+00
                                    0.2595E+01      0.2017E-01
                                    0.4325E+01      0.3908E-01
                                    0.6487E+01      0.7185E-01
                                    0.7784E+01      0.1008E+00
                                    0.8649E+01      0.1261E+00
                                    0.8649E+01      0.2521E+00
                                    0.8649E+01      0.3782E+00
                                    0.8649E+01      0.6303E+00
                                    0.8649E+01      0.2521E+01
12         10      0.1086E+02
                                    0.0000E+00      0.0000E+00
                                    0.1101E+01      0.2017E-01
                                    0.1836E+01      0.3908E-01
                                    0.2753E+01      0.7185E-01
                                    0.3304E+01      0.1008E+00
                                    0.3671E+01      0.1261E+00
                                    0.3671E+01      0.2521E+00
                                    0.3671E+01      0.3782E+00
                                    0.3671E+01      0.6303E+00
                                    0.3671E+01      0.2521E+01
13         10      0.1094E+02
                                    0.0000E+00      0.0000E+00
                                    0.9372E+00      0.2017E-01
                                    0.1562E+01      0.3908E-01
                                    0.2343E+01      0.7185E-01
                                    0.2812E+01      0.1008E+00
                                    0.3124E+01      0.1261E+00
                                    0.3124E+01      0.2521E+00
                                    0.3124E+01      0.3782E+00
                                    0.3124E+01      0.6303E+00
                                    0.3124E+01      0.2521E+01
14         10      0.1795E+02
                                    0.0000E+00      0.0000E+00
                                    0.1799E+01      0.2017E-01
                                    0.2999E+01      0.3908E-01
                                    0.4499E+01      0.7185E-01
                                    0.5398E+01      0.1008E+00
                                    0.5998E+01      0.1261E+00
                                    0.5998E+01      0.2521E+00
                                    0.5998E+01      0.3782E+00
                                    0.5998E+01      0.6303E+00
                                    0.5998E+01      0.2521E+01
15         10      0.2496E+02
                                    0.0000E+00      0.0000E+00
                                    0.1799E+01      0.2017E-01
                                    0.2999E+01      0.3908E-01
                                    0.4499E+01      0.7185E-01
                                    0.5398E+01      0.1008E+00
```

```
                                        0.5998E+01      0.1261E+00
                                        0.5998E+01      0.2521E+00
                                        0.5998E+01      0.3782E+00
                                        0.5998E+01      0.6303E+00
                                        0.5998E+01      0.2521E+01


 TIP   LOAD         TIP MOVEMENT
     KIP                IN.

 0.0000E+00          0.0000E+00
 0.2271E+01          0.6303E-02
 0.4542E+01          0.1261E-01
 0.9084E+01          0.2521E-01
 0.1817E+02          0.1639E+00
 0.2725E+02          0.5294E+00
 0.3270E+02          0.9202E+00
 0.3633E+02          0.1261E+01
 0.3633E+02          0.1891E+01
 0.3633E+02          0.2521E+01


               LOAD VERSUS SETTLEMENT CURVE
               ****************************



 TOP  LOAD     TOP MOVEMENT       TIP  LOAD      TIP MOVEMENT
    KIP             IN.               KIP            IN.
0.1053E+00      0.1326E-03      0.3603E-01      0.1000E-03
0.1053E+01      0.1326E-02      0.3603E+00      0.1000E-02
0.5264E+01      0.6628E-02      0.1802E+01      0.5000E-02
0.1058E+02      0.1326E-01      0.3603E+01      0.1000E-01
0.2082E+02      0.2648E-01      0.7206E+01      0.2000E-01
0.3656E+02      0.6101E-01      0.1071E+02      0.5000E-01
0.4718E+02      0.9401E-01      0.1267E+02      0.8000E-01
0.5276E+02      0.1156E+00      0.1398E+02      0.1000E+00
0.5925E+02      0.2184E+00      0.1907E+02      0.2000E+00
0.6556E+02      0.5215E+00      0.2652E+02      0.5000E+00
0.7006E+02      0.8235E+00      0.3103E+02      0.8000E+00
0.7259E+02      0.1025E+01      0.3355E+02      0.1000E+01
0.7537E+02      0.2026E+01      0.3633E+02      0.2000E+01
```

CALCULATIONS

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 1 (Old Abutment 2)- Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

OBJECTIVE

Evaluate the downdrag load acting on the HP 10x42 steel pile closest to the maximum settlement point on the Abutment 1 (Old Abutment 2) (Pile 7) of the bridge carrying Hogan Road Eastbound over I-95. This analysis uses estimated settlement from 1983 to present-day.

METHOD

Use APile (Ref. 1) to estimate the shaft resistance contributing to downdrag.

REFERENCES

1. Ensoft, Inc. APILE Version 2019.9.11 - Offshore. Release date October 15, 2021.
2. Maine Department of Transportation Bridge Program. Preliminary Design Report: Hogan Road / I-95 Bridge #5823 over Interstate 95, Exit 187 Diverging Diamond Interchange, Bangor, Maine, Federal Project #1859510, WIN 018595.10. Draft 2-3-2023.
3. WSP calculation titled "Hogan Rd Highway Settlement Embankment 1983 10 28 24.xlsx", dated 10/28/2024.
4. WSP Interpretive Subsurface Profile E-E', included as part of the Geotechnical Design Report.
5. WSP calculations of interpreted soil parameters based on field and laboratory data.
6. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
7. AASHTO LRFD Bridge Design Specifications, 9th Edition. 2020.
8.WSP pile resistance calculation and FB-Multipier model for Abutment 1 (old Abutment 2), dated 10/25/2024
9. VHB computation package titled "1983 Bridge Foundation Loads", dated 4/5/2024.

ATTACHMENTS

1. Profile of estimated settlement beneath Abutment 1 (Old Abutment 2) at 41 years after embankment widening construction at the location of the pile with maximum axial loading
2. Output from APile analysis

ASSUMPTIONS

1. The widened portion of the existing Abutment 1 (Old Abutment 2) is supported on eighteen HP10x42 steel piles, based on the as-built bridge widening plans dated 1983 in Ref. 2, Appendix C. The downdrag is evaluated for the pile closest to the maximum settlement. The estimated pile length at this location as shown in the 1983 as-built plans (Ref. 2, Appendix C, Sheet 13) is 13.4 feet below the base of the abutment. Based on WSP's interpreted subsurface profile along the existing abutment (Ref. 4), the piles are assumed to have been driven into weathered bedrock.
2. The downdrag depth is based on the total settlement estimated to have occurred up to present-day due to the 1983 embankment widening at the Abutment 1 (Old Abutment 2) at the maximum settlement location on the Abutment 1 (Ref. 3).
3. The FHWA automated computation method provided in APile is used for the software computations of unit load transfers and axial pile capacity.
4. Since APile requires soil layers to be defined to a depth of at least the pile tip plus two pile diameters, the bedrock is modeled as "sand" with a friction angle of 50 degrees.
5. Downdrag is calculated with an uncorroded pile section to represent the maximum possible downdrag load.
6. Abutment designations are based on VHB designations in Ref. 9.

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 1 (Old Abutment 2)- Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CALCULATION

1. Establish soil profile and pile properties.

The soil profile used in the downdrag analysis is based on Ref. 4 at the centerline of the Abutment 1 (Old Abutment 2). The soil properties used in the downdrag analysis are taken from Ref. 5 and are based on correlations to field test and laboratory test data.

Base of Abutment 1 elevation = 106.2 feet (Ref. 2, Appendix C, Sheet 11)
Water table elevation = 102.4 feet (Ref. 4)

Soil Layer	Elevation (feet)		Depth below Abutment (feet)		Effective Unit Weight (pcf)	Friction Angle (°)	Undrained Shear Strength (psf)	
	Top	Bottom	Top	Bottom			Top	Bottom
Existing Fill	106.2	103.1	0.0	3.1	125	34	-	-
Stiff Clay	103.1	102.4	3.1	3.8	122	-	1340	1340
Stiff Clay	102.4	92.4	3.8	13.8	59.6	-	1340	1340
Glacial Till	92.4	89.2	13.8	17.0	67.6	35	-	-
Bedrock	89.2	-	17.0	-	95.6	50	-	-

Pile steel elastic modulus = 29,000,000 psi (typical value for steel)
Pile length = 19.5 ft (Assumption 1)
Pile section box perimeter = 39.6 in (using pile flange width and depth from Ref. 6)
Pile section tip area = 12.4 in^2 (Ref. 6)
Pile nominal weight = 0.042 kip/ft (Ref. 6)

2. Identify the depth below the abutment to which settlement is large enough to fully develop downdrag loading.

Per Ref. 7 Article 3.11.8, downdrag can be assumed to fully develop for the length of pile with settlement equal to or greater than 0.4 inches. As calculated in Ref. 3 - Attachment 10, settlement beneath Abutment 1 (Old Abutment 2) up to present-day (41 years after construction) due to construction of the widened bridge abutment and approach embankment is estimated to be greater than or equal to 0.4 inches down to an elevation of 89.8 feet (depth of 16.4 feet below the base of the abutment). Thus, downdrag loading is assumed to be fully developed for the upper 16.4 feet of the pile below the base of the abutment.

Per Ref. 7 Article 10.7.1.6.2 for piles end bearing on rock downdrag shall be considered at the strength and extreme limit state.

CALCULATIONS

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 1 (Old Abutment 2)- Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

3. Use APile to estimate shaft resistance and total downdrag load.

Downdrag Load Factors:
(Ref. 7, Tables 3.4.1-1 and 3.4.1-2)

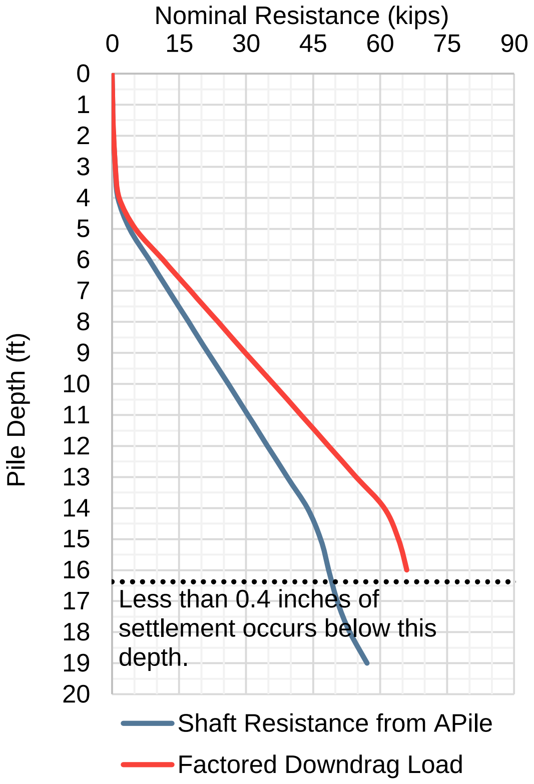
Soil Type	Strength I
Sand	1.05
Clay	1.40

Abutment Loads:

	Strength I		
Per Pile	70.3	kips	(Ref. 8)

Shaft resistance and downdrag load for each foot of pile length:

Pile Depth below Base of Abutment (feet)	Soil Type from APile	Shaft Resistance from APile (kips)	Factored Downdrag Load (kips)	Total Factored Axial Load* (kips)
0	Sand	0.0	0.0	70.3
1	Sand	0.1	0.1	70.4
2	Sand	0.3	0.3	70.7
3	Sand	0.7	0.7	71.2
4	Clay	1.3	1.6	72.0
5	Clay	3.9	5.2	75.7
6	Clay	8.3	11.4	81.9
7	Clay	12.7	17.5	88.1
8	Clay	17.1	23.7	94.3
9	Clay	21.5	29.9	100.5
10	Clay	26.0	36.2	106.9
11	Clay	30.4	42.3	113.1
12	Clay	34.8	48.5	119.3
13	Clay	39.2	54.6	125.5
14	Clay	43.7	60.9	131.8
15	Sand	46.7	64.1	135.0
16	**Sand**	**48.5**	**66.0**	**136.9**
17	Sand	50.4		71.0
18	Sand	53.2		71.1
19	Sand	57.1		71.1



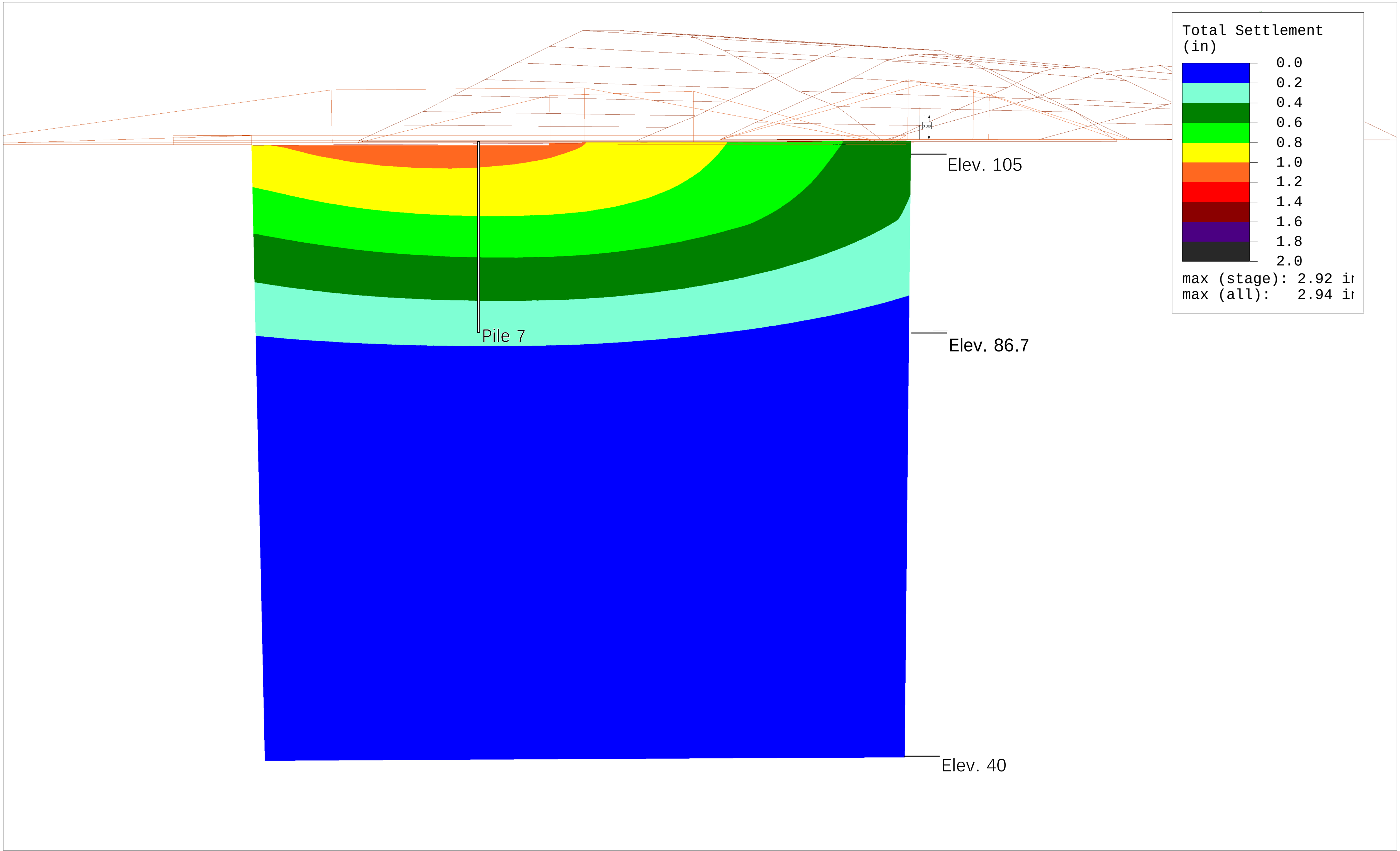
Less than 0.4 inches of settlement occurs below this depth.

*Strength I, including Substructure Loading, Downdrag, and Pile Weight

Strength I factored downdrag load = 66.0 kips per pile

CONCLUSIONS

Based on the analysis of settlement between 1983 and present-day, downdrag is estimated to have developed along the upper 16.4 feet of pile 7 below the base of the Abutment 1 (Old Abutment 2). A factored downdrag load of 66.0 kips per pile for the Strength I limit state was calculated.



SETTLE3 5.023

Project MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.004

Analysis Description Total Settlement along Abutment 1 (Old Abutment 2) (Sta. 109+43) at Present-Day (41 years after embankment widening)

Drawn By LMP	*Checked By* KAR	*Reviewed By* CCB	*Scale* As shown
Date 11/1/2024	*File Name* Hogan Rd Highway Interchange 1958 surface 1983Fill 10 24 -v3.s3z		

Attachment 1

Attachment 2

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

                      APILE for Windows, Version 2019.9.11

                       Serial Number :  264781183

                       A Program for Analyzing the Axial Capacity
                       and Short-term Settlement of Driven Piles
                                 under Axial Loading.
                     (c) Copyright ENSOFT, Inc., 1987-2019
                           All Rights Reserved

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


     This program is licensed to :

     WSP
     APILE Global, Global License

     Path to file locations      : C:\Users\USLP710461\WSP O365\MaineDOT Bangor
Hogan Road Bridge & DDI - Project Files\5 Technical Work\GDR Pile & Abutment
Design\Existing Bridge - Downdrag\APile Models\Abutement 1 Pile 16 - Max loading\
     Name of input data file     : HP10x42_Abutment1 Pile 7.ap9d
     Name of output file         : HP10x42_Abutment1 Pile 7.ap9o
     Name of plot output file    : HP10x42_Abutment1 Pile 7.ap9p

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

               Date:  October 25, 2024     Time:  14:57:39



1
          *********************
          * INPUT INFORMATION *
          *********************



          Existing Pile Downdrag at  Abutment 2

          DESIGNER   : LMP

          JOB NUMBER : 31404817.004


          METHOD FOR UNIT LOAD TRANSFERS :
```

```
- FHWA (Federal Highway Administration)
  Unfactored Unit Side Friction and Unit Side Resistance are used.



COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)


TYPE OF LOADING :
- COMPRESSION


PILE TYPE :

H-Pile/Steel Pile


DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY     = 0.290E+08 PSI
- CROSS SECTION AREA        =     12.40 IN2


NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL     =     19.50 FT.
- BATTER ANGLE              =      0.00 DEG
- PILE STICKUP LENGTH, PSL  =      0.00 FT.
- ZERO FRICTION LENGTH, ZFL =      0.00 FT.
- PERIMETER OF PILE         =     39.60 IN.
- TIP AREA OF PILE          =     12.40 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION       =      1.00 FT.



SOIL INFORMATIONS :



                  LATERAL    EFFECTIVE    FRICTION      Nq
          SOIL    EARTH      UNIT         ANGLE        FACTOR
 DEPTH    TYPE    PRESSURE   WEIGHT       DEGREES       FHWA
  FT.                        LB/FT^3
   0.00   SAND      0.80*     125.00        34.00     55.60**
   3.10   SAND      0.80*     125.00        34.00     55.60**
   3.10   CLAY      0.80*     122.00         0.00      4.80**
   3.80   CLAY      0.80*     122.00         0.00      4.80**
   3.80   CLAY      0.80*      59.60         0.00      4.80**
```

```
  13.80   CLAY      0.80*       59.60         0.00      4.80**
  13.80   SAND      0.80*       67.60        35.00     64.00**
  17.00   SAND      0.80*       67.60        35.00     64.00**
  17.00   SAND      0.80*       95.60        50.00    475.00**
  25.00   SAND      0.80*       95.60        50.00    475.00**

*  VALUE ASSUMED BY THE PROGRAM
** VALUE ESTIMATED BY THE PROGRAM BASED ON FRICTION ANGLE



MAXIMUM   MAXIMUM   UNDISTURB  REMOLDED
UNIT      UNIT      SHEAR      SHEAR     BLOW   UNIT SKIN  UNIT END
FRICTION  BEARING   STRENGTH   STRENGTH  COUNT  FRICTION   BEARING
  KSF       KSF        KSF       KSF                KSF       KSF
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
  WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
  PLAN TO LIMIT THE COMPUTED DATA.



         LRFD FACTOR  LRFD FACTOR
          ON UNIT      ON UNIT
 DEPTH    FRICTION     BEARING
  FT.
   0.00    1.000       1.000
   3.10    1.000       1.000
   3.10    1.000       1.000
   3.80    1.000       1.000
   3.80    1.000       1.000
  13.80    1.000       1.000
  13.80    1.000       1.000
  17.00    1.000       1.000
  17.00    1.000       1.000
  25.00    1.000       1.000


1
          **********************
```

```
  * COMPUTATION RESULT *
  **********************



                    *********************
                    *  FED. HWY. METHOD  *
                    *********************

     PILE          SKIN          END       ULTIMATE
  PENETRATION   FRICTION       BEARING     CAPACITY
      FT.          KIP           KIP          KIP
      0.00           0.0           0.2          0.2
      1.00           0.1           0.4          0.4
      2.00           0.3           0.8          1.1
      3.00           0.7           1.1          1.8
      4.00           1.3           1.2          2.5
      5.00           3.9           1.1          5.0
      6.00           8.3           1.0          9.3
      7.00          12.7           1.0         13.7
      8.00          17.1           1.0         18.2
      9.00          21.5           1.0         22.6
     10.00          26.0           1.0         27.0
     11.00          30.4           1.0         31.4
     12.00          34.8           1.0         35.9
     13.00          39.2           1.7         40.9
     14.00          43.7           2.8         46.4
     15.00          46.7           4.0         50.7
     16.00          48.5          12.2         60.7
     17.00          50.4          26.0         76.4
     18.00          53.2          40.6         93.9
     19.00          57.1          50.8        107.9

    NOTES:
    - AN ASTERISK IS PLACED IN THE END-BEARING COLUMN
      IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
      OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.



        *************************************************
        * COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
        * CURVES FOR AXIAL LOADING                      *
        *************************************************


T-Z CURVE   NO. OF   DEPTH TO CURVE   LOAD TRANSFER   PILE MOVEMENT
   NO.      POINTS        FT.              PSI             IN.

    1         10      0.4167E-01
```

```
                                   0.0000E+00      0.0000E+00
                                   0.4336E-02      0.2017E-01
                                   0.7226E-02      0.3908E-01
                                   0.1084E-01      0.7185E-01
                                   0.1301E-01      0.1008E+00
                                   0.1445E-01      0.1261E+00
                                   0.1445E-01      0.2521E+00
                                   0.1445E-01      0.3782E+00
                                   0.1445E-01      0.6303E+00
                                   0.1445E-01      0.2521E+01
2         10      0.1550E+01
                                   0.0000E+00      0.0000E+00
                                   0.1613E+00      0.2017E-01
                                   0.2688E+00      0.3908E-01
                                   0.4032E+00      0.7185E-01
                                   0.4839E+00      0.1008E+00
                                   0.5376E+00      0.1261E+00
                                   0.5376E+00      0.2521E+00
                                   0.5376E+00      0.3782E+00
                                   0.5376E+00      0.6303E+00
                                   0.5376E+00      0.2521E+01
3         10      0.3058E+01
                                   0.0000E+00      0.0000E+00
                                   0.3182E+00      0.2017E-01
                                   0.5304E+00      0.3908E-01
                                   0.7956E+00      0.7185E-01
                                   0.9547E+00      0.1008E+00
                                   0.1061E+01      0.1261E+00
                                   0.1061E+01      0.2521E+00
                                   0.1061E+01      0.3782E+00
                                   0.1061E+01      0.6303E+00
                                   0.1061E+01      0.2521E+01
4         10      0.3142E+01
                                   0.0000E+00      0.0000E+00
                                   0.3269E+00      0.2017E-01
                                   0.5449E+00      0.3908E-01
                                   0.8173E+00      0.7185E-01
                                   0.9807E+00      0.1008E+00
                                   0.1090E+01      0.1261E+00
                                   0.9807E+00      0.2521E+00
                                   0.9807E+00      0.3782E+00
                                   0.9807E+00      0.6303E+00
                                   0.9807E+00      0.2521E+01
5         10      0.3600E+01
                                   0.0000E+00      0.0000E+00
                                   0.3746E+00      0.2017E-01
                                   0.6243E+00      0.3908E-01
                                   0.9365E+00      0.7185E-01
                                   0.1124E+01      0.1008E+00
                                   0.1249E+01      0.1261E+00
                                   0.1124E+01      0.2521E+00
```

```
                                    0.1124E+01      0.3782E+00
                                    0.1124E+01      0.6303E+00
                                    0.1124E+01      0.2521E+01
 6         10      0.4058E+01
                                    0.0000E+00      0.0000E+00
                                    0.5548E+00      0.2017E-01
                                    0.9247E+00      0.3908E-01
                                    0.1387E+01      0.7185E-01
                                    0.1664E+01      0.1008E+00
                                    0.1849E+01      0.1261E+00
                                    0.1664E+01      0.2521E+00
                                    0.1664E+01      0.3782E+00
                                    0.1664E+01      0.6303E+00
                                    0.1664E+01      0.2521E+01
 7         10      0.4142E+01
                                    0.0000E+00      0.0000E+00
                                    0.7527E+00      0.2017E-01
                                    0.1255E+01      0.3908E-01
                                    0.1882E+01      0.7185E-01
                                    0.2258E+01      0.1008E+00
                                    0.2509E+01      0.1261E+00
                                    0.2258E+01      0.2521E+00
                                    0.2258E+01      0.3782E+00
                                    0.2258E+01      0.6303E+00
                                    0.2258E+01      0.2521E+01
 8         10      0.8950E+01
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 9         10      0.1376E+02
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
10         10      0.1384E+02
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
```

```
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.9306E+01      0.2521E+00
                                    0.9306E+01      0.3782E+00
                                    0.9306E+01      0.6303E+00
                                    0.9306E+01      0.2521E+01
11         10      0.1540E+02
                                    0.0000E+00      0.0000E+00
                                    0.1121E+01      0.2017E-01
                                    0.1868E+01      0.3908E-01
                                    0.2802E+01      0.7185E-01
                                    0.3363E+01      0.1008E+00
                                    0.3737E+01      0.1261E+00
                                    0.3737E+01      0.2521E+00
                                    0.3737E+01      0.3782E+00
                                    0.3737E+01      0.6303E+00
                                    0.3737E+01      0.2521E+01
12         10      0.1696E+02
                                    0.0000E+00      0.0000E+00
                                    0.1215E+01      0.2017E-01
                                    0.2025E+01      0.3908E-01
                                    0.3038E+01      0.7185E-01
                                    0.3646E+01      0.1008E+00
                                    0.4051E+01      0.1261E+00
                                    0.4051E+01      0.2521E+00
                                    0.4051E+01      0.3782E+00
                                    0.4051E+01      0.6303E+00
                                    0.4051E+01      0.2521E+01
13         10      0.1704E+02
                                    0.0000E+00      0.0000E+00
                                    0.1265E+01      0.2017E-01
                                    0.2109E+01      0.3908E-01
                                    0.3163E+01      0.7185E-01
                                    0.3796E+01      0.1008E+00
                                    0.4218E+01      0.1261E+00
                                    0.4218E+01      0.2521E+00
                                    0.4218E+01      0.3782E+00
                                    0.4218E+01      0.6303E+00
                                    0.4218E+01      0.2521E+01
14         10      0.2100E+02
                                    0.0000E+00      0.0000E+00
                                    0.2516E+01      0.2017E-01
                                    0.4193E+01      0.3908E-01
                                    0.6289E+01      0.7185E-01
                                    0.7547E+01      0.1008E+00
                                    0.8386E+01      0.1261E+00
                                    0.8386E+01      0.2521E+00
                                    0.8386E+01      0.3782E+00
                                    0.8386E+01      0.6303E+00
                                    0.8386E+01      0.2521E+01
```

```
   15         10      0.2496E+02
                                        0.0000E+00      0.0000E+00
                                        0.2516E+01      0.2017E-01
                                        0.4193E+01      0.3908E-01
                                        0.6289E+01      0.7185E-01
                                        0.7547E+01      0.1008E+00
                                        0.8386E+01      0.1261E+00
                                        0.8386E+01      0.2521E+00
                                        0.8386E+01      0.3782E+00
                                        0.8386E+01      0.6303E+00
                                        0.8386E+01      0.2521E+01


 TIP   LOAD        TIP MOVEMENT
     KIP                IN.

 0.0000E+00         0.0000E+00
 0.3175E+01         0.6303E-02
 0.6350E+01         0.1261E-01
 0.1270E+02         0.2521E-01
 0.2540E+02         0.1639E+00
 0.3810E+02         0.5294E+00
 0.4572E+02         0.9202E+00
 0.5080E+02         0.1261E+01
 0.5080E+02         0.1891E+01
 0.5080E+02         0.2521E+01


               LOAD VERSUS SETTLEMENT CURVE
               ****************************



 TOP  LOAD     TOP MOVEMENT       TIP  LOAD      TIP MOVEMENT
    KIP             IN.              KIP              IN.
0.1563E+00      0.1702E-03      0.5037E-01      0.1000E-03
0.1563E+01      0.1702E-02      0.5037E+00      0.1000E-02
0.7844E+01      0.8516E-02      0.2519E+01      0.5000E-02
0.1577E+02      0.1706E-01      0.5037E+01      0.1000E-01
0.3045E+02      0.3384E-01      0.1007E+02      0.2000E-01
0.5268E+02      0.7338E-01      0.1497E+02      0.5000E-01
0.6725E+02      0.1096E+00      0.1772E+02      0.8000E-01
0.7485E+02      0.1330E+00      0.1955E+02      0.1000E+00
0.8272E+02      0.2383E+00      0.2665E+02      0.2000E+00
0.9193E+02      0.5447E+00      0.3708E+02      0.5000E+00
0.9822E+02      0.8488E+00      0.4337E+02      0.8000E+00
0.1018E+03      0.1051E+01      0.4691E+02      0.1000E+01
0.1056E+03      0.2054E+01      0.5080E+02      0.2000E+01
```

CALCULATIONS

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 1 (Old Abutment 2) - Pile with minimum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

OBJECTIVE

Evaluate the downdrag load acting on the HP 10x42 steel pile closest to the minimum settlement point on the Abutment 1 (Old Abutment 2) (Pile 17) of the bridge carrying Hogan Road Eastbound over I-95. This analysis uses estimated settlement from 1983 to present-day.

METHOD

Use APile (Ref. 1) to estimate the shaft resistance contributing to downdrag.

REFERENCES

1. Ensoft, Inc. APILE Version 2019.9.11 - Offshore. Release date October 15, 2021.
2. Maine Department of Transportation Bridge Program. Preliminary Design Report: Hogan Road / I-95 Bridge #5823 over Interstate 95, Exit 187 Diverging Diamond Interchange, Bangor, Maine, Federal Project #1859510, WIN 018595.10. Draft 2-3-2023.
3. WSP calculation titled "Hogan Rd Highway Settlement Embankment 1983 10 28 24.xlsx", dated 10/28/2024.
4. WSP Interpretive Subsurface Profile E-E', included as part of the Geotechnical Design Report.
5. WSP calculations of interpreted soil parameters based on field and laboratory data.
6. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
7. AASHTO LRFD Bridge Design Specifications, 9th Edition. 2020.
8. WSP pile resistance calculation and FB-Multipier model for Abutment 1 (old Abutment 2), dated 10/25/2024
9. VHB computation package titled "1983 Bridge Foundation Loads", dated 4/5/2024.

ATTACHMENTS

1. Profile of estimated settlement beneath Abutment 1 (Old Abutment 2) at 41 years after embankment widening construction at the location of the pile with maximum axial loading
2. Output from APile analysis

ASSUMPTIONS

1. The widened portion of the existing Abutment 1 (Old Abutment 2) is supported on eighteen HP10x42 steel piles, based on the as-built bridge widening plans dated 1983 in Ref. 2, Appendix C. The downdrag is evaluated for the pile closest to the minimum settlement. The estimated pile length at this location as shown in the 1983 as-built plans (Ref. 2, Appendix C, Sheet 13) is 13.4 feet below the base of the abutment. Based on WSP's interpreted subsurface profile along the existing abutment (Ref. 4), the piles are assumed to have been driven into weathered bedrock.

2. The downdrag depth is based on the total settlement estimated to have occurred up to present-day due to the 1983 embankment widening at the Abutment 1 (Old Abutment 2) at the minimum settlement location on the Abutment 1 (Ref. 3).

3. The FHWA automated computation method provided in APile is used for the software computations of unit load transfers and axial pile capacity.

4. Since APile requires soil layers to be defined to a depth of at least the pile tip plus two pile diameters, the bedrock is modeled as "sand" with a friction angle of 50 degrees.

5. Downdrag is calculated with an uncorroded pile section to represent the maximum possible downdrag load.

6. Abutment designations are based on VHB designations in Ref. 9.

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 1 (Old Abutment 2) - Pile with minimum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CALCULATION

1. Establish soil profile and pile properties.

The soil profile used in the downdrag analysis is based on Ref. 4 at the centerline of the Abutment 1 (Old Abutment 2). The soil properties used in the downdrag analysis are taken from Ref. 5 and are based on correlations to field test and laboratory test data.

Base of Abutment 1 elevation =	106.2	feet	(Ref. 2, Appendix C, Sheet 11)
Water table elevation =	102.4	feet	(Ref. 4)

Soil Layer	Elevation (feet)		Depth below Abutment (feet)		Effective Unit Weight (pcf)	Friction Angle (°)	Undrained Shear Strength (psf)	
	Top	Bottom	Top	Bottom			Top	Bottom
Existing Fill	106.2	104.2	0.0	2.0	125	34	-	-
Stiff Clay	104.2	102.4	2.0	3.8	122	-	1340	1340
Stiff Clay	102.4	96.9	3.8	9.3	59.6	-	1340	1340
Glacial Till	96.9	95.3	9.3	10.9	67.6	35	-	-
Bedrock	95.3	-	10.9	-	95.6	50	-	-

Pile steel elastic modulus =	29,000,000	psi	(typical value for steel)
Pile length =	13.4	ft	(Assumption 1)
Pile section box perimeter =	39.6	in	(using pile flange width and depth from Ref. 6)
Pile section tip area =	12.4	in^2	(Ref. 6)
Pile nominal weight =	0.042	kip/ft	(Ref. 6)

2. Identify the depth below the abutment to which settlement is large enough to fully develop downdrag loading.

Per Ref. 7 Article 3.11.8, downdrag can be assumed to fully develop for the length of pile with settlement equal to or greater than 0.4 inches. As calculated in Ref. 3 - Attachment 10, settlement beneath Abutment 1 (Old Abutment 2) up to present-day (41 years after construction) due to construction of the widened bridge abutment and approach embankment is estimated to be greater than or equal to 0.4 inches down to an elevation of 102.5 feet (depth of 3.7 feet below the base of the abutment). Thus, downdrag loading is assumed to be fully developed for the upper 3.7 feet of the pile below the base of the abutment.

Per Ref. 7 Article 10.7.1.6.2 for piles end bearing on rock downdrag shall be considered at the strength and extreme limit state.

CALCULATIONS

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 1 (Old Abutment 2) - Pile with minimum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

3. Use APile to estimate shaft resistance and total downdrag load.

Downdrag Load Factors:
(Ref. 7, Tables 3.4.1-1 and 3.4.1-2)

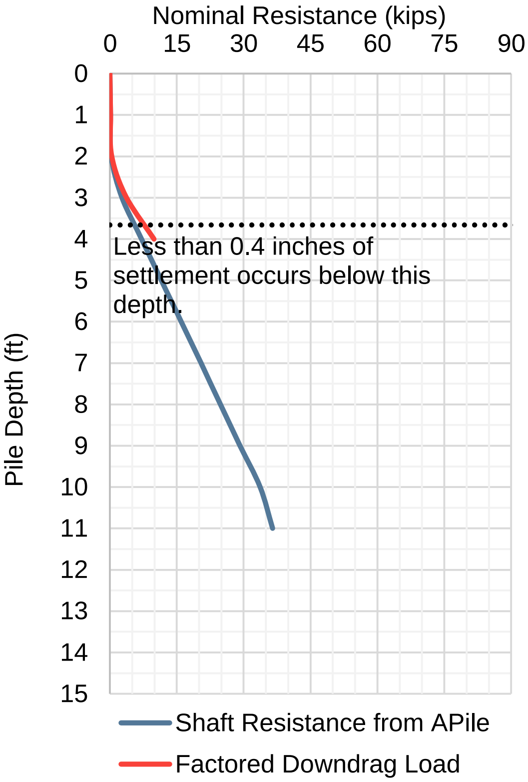
Soil Type	Strength I
Sand	1.05
Clay	1.40

Abutment Loads:

	Strength I		
Per Pile	113	kips	(Ref. 8)

Shaft resistance and downdrag load for each foot of pile length:

Pile Depth below Base of Abutment	Soil Type from APile	Shaft Resistance from APile	Factored Downdrag Load	Total Factored Axial Load*
(feet)		(kips)	(kips)	(kips)
0	Sand	0.0	0.0	113.0
1	Sand	0.1	0.1	113.1
2	Clay	0.3	0.4	113.5
3	Clay	2.7	3.7	116.9
4	**Clay**	**7.1**	**9.9**	**123.1**
5	Clay	11.5		113.2
6	Clay	16.0		113.3
7	Clay	20.4		113.3
8	Clay	24.8		113.3
9	Clay	29.2		113.4
10	Sand	33.7		113.4
11	Sand	36.5		113.5
12	Sand	38.5		113.5
13	Sand	41.2		113.5

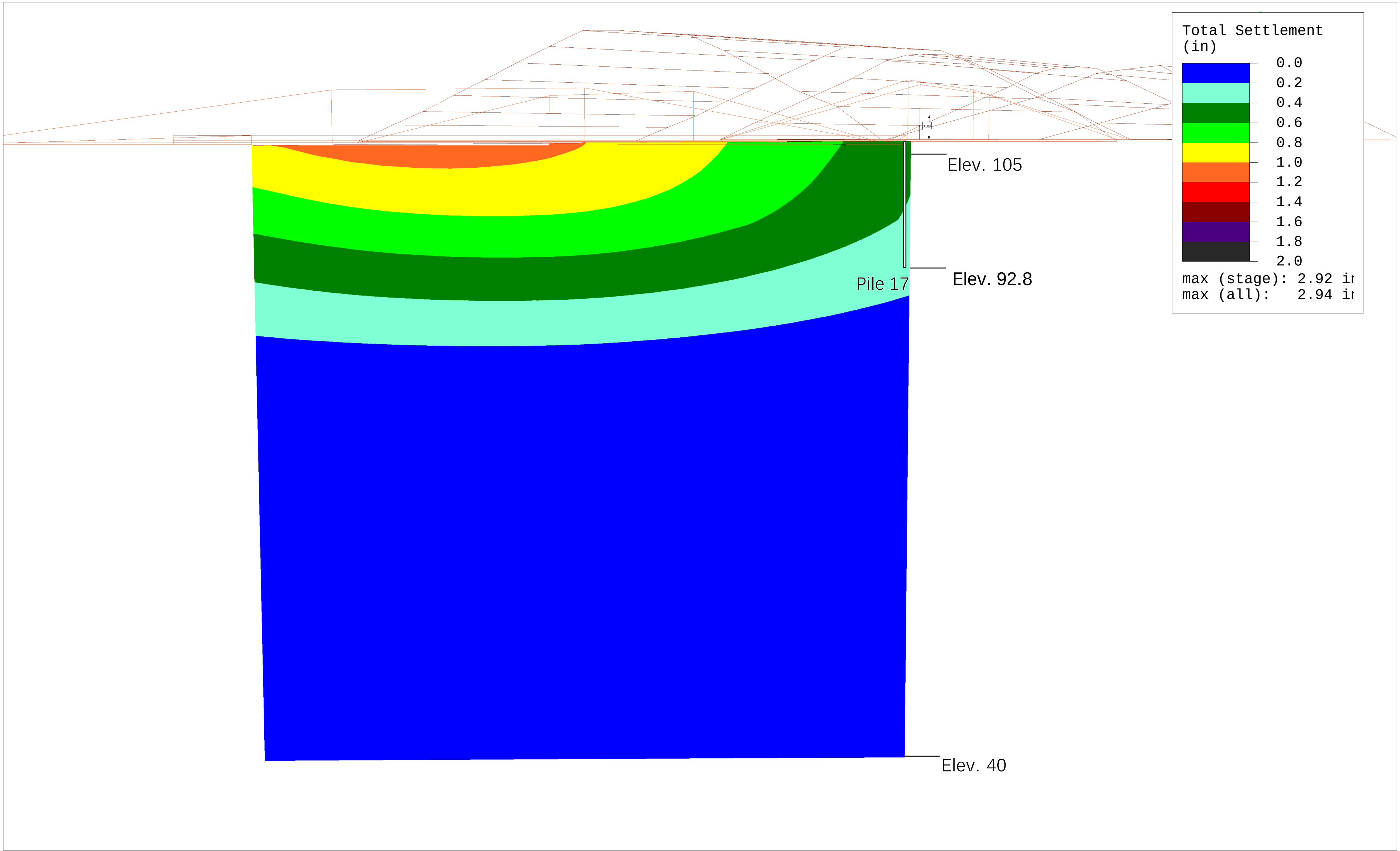


*Strength I, including Substructure Loading, Downdrag, and Pile Weight

Strength I factored downdrag load = 9.9 kips per pile

CONCLUSIONS

Based on the analysis of settlement between 1983 and present-day, downdrag is estimated to have developed along the upper 3.7 feet of pile 17 below the base of the Abutment 1 (Old Abutment 2). A factored downdrag load of 9.9 kips per pile for the Strength I limit state was calculated.



Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.004
Analysis Description	Total Settlement along Abutment 1 (Old Abutment 2) (Sta. 109+43) at Present-Day (41 years after embankment widening)

Drawn By	LMP	Checked By	KAR	Reviewed By	CCB	Scale	As shown
Date	11/1/2024	File Name	Hogan Rd Highway Interchange 1958 surface 1983Fill 10 24 -v3.s3z				

Attachment 1

SETTLE3 5.023

Attachment 2

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

                     APILE for Windows, Version 2019.9.11

                      Serial Number :  264781183

                      A Program for Analyzing the Axial Capacity
                      and Short-term Settlement of Driven Piles
                                under Axial Loading.
                    (c) Copyright ENSOFT, Inc., 1987-2019
                          All Rights Reserved

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


     This program is licensed to :

     WSP
     APILE Global, Global License

     Path to file locations      : C:\Users\USLP710461\WSP O365\MaineDOT Bangor
Hogan Road Bridge & DDI - Project Files\5 Technical Work\GDR Pile & Abutment
Design\Existing Bridge - Downdrag\APile Models\Abutement 1 Pile 16 - Max loading\
     Name of input data file     : HP10x42_Abutment1 Pile 16.ap9d
     Name of output file         : HP10x42_Abutment1 Pile 16.ap9o
     Name of plot output file    : HP10x42_Abutment1 Pile 16.ap9p

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

               Date:  October 24, 2024     Time:  10:40:33



1
          *********************
          * INPUT INFORMATION *
          *********************



          Existing Pile Downdrag at  Abutment 2

          DESIGNER   : LMP

          JOB NUMBER : 31404817.004


          METHOD FOR UNIT LOAD TRANSFERS :
```

```
- FHWA (Federal Highway Administration)
  Unfactored Unit Side Friction and Unit Side Resistance are used.



COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)


TYPE OF LOADING :
- COMPRESSION


PILE TYPE :

H-Pile/Steel Pile


DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY     = 0.290E+08 PSI
- CROSS SECTION AREA        =     12.40 IN2


NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL     =     13.40 FT.
- BATTER ANGLE              =      0.00 DEG
- PILE STICKUP LENGTH, PSL  =      0.00 FT.
- ZERO FRICTION LENGTH, ZFL =      0.00 FT.
- PERIMETER OF PILE         =     39.60 IN.
- TIP AREA OF PILE          =     12.40 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION       =      1.00 FT.



SOIL INFORMATIONS :



                  LATERAL    EFFECTIVE    FRICTION      Nq
          SOIL    EARTH      UNIT         ANGLE       FACTOR
 DEPTH    TYPE    PRESSURE   WEIGHT       DEGREES      FHWA
  FT.                        LB/FT^3
   0.00   SAND      0.80*      125.00        34.00     55.60**
   2.00   SAND      0.80*      125.00        34.00     55.60**
   2.00   CLAY      0.80*      122.00         0.00      4.80**
   3.80   CLAY      0.80*      122.00         0.00      4.80**
   3.80   CLAY      0.80*       59.60         0.00      4.80**
```

```
  9.30   CLAY      0.80*       59.60         0.00      4.80**
  9.30   SAND      0.80*       67.60        35.00     64.00**
 10.90   SAND      0.80*       67.60        35.00     64.00**
 10.90   SAND      0.80*       95.60        50.00    475.00**
 25.00   SAND      0.80*       95.60        50.00    475.00**

*  VALUE ASSUMED BY THE PROGRAM
** VALUE ESTIMATED BY THE PROGRAM BASED ON FRICTION ANGLE



MAXIMUM   MAXIMUM   UNDISTURB  REMOLDED
UNIT      UNIT      SHEAR      SHEAR     BLOW   UNIT SKIN  UNIT END
FRICTION  BEARING   STRENGTH   STRENGTH  COUNT  FRICTION   BEARING
  KSF       KSF        KSF       KSF               KSF       KSF
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
  WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
  PLAN TO LIMIT THE COMPUTED DATA.



         LRFD FACTOR  LRFD FACTOR
          ON UNIT      ON UNIT
 DEPTH    FRICTION     BEARING
  FT.
   0.00    1.000        1.000
   2.00    1.000        1.000
   2.00    1.000        1.000
   3.80    1.000        1.000
   3.80    1.000        1.000
   9.30    1.000        1.000
   9.30    1.000        1.000
  10.90    1.000        1.000
  10.90    1.000        1.000
  25.00    1.000        1.000


1
          **********************
```

```
 * COMPUTATION RESULT *
 **********************



                   **********************
                   *  FED. HWY. METHOD  *
                   **********************

    PILE          SKIN          END       ULTIMATE
 PENETRATION   FRICTION       BEARING     CAPACITY
     FT.          KIP           KIP         KIP
     0.00           0.0           0.2         0.2
     1.00           0.1           0.4         0.5
     2.00           0.3           0.8         1.1
     3.00           2.7           1.0         3.7
     4.00           7.1           1.0         8.2
     5.00          11.5           1.0        12.6
     6.00          16.0           1.0        17.0
     7.00          20.4           1.0        21.4
     8.00          24.8           1.0        25.9
     9.00          29.2           1.4        30.7
    10.00          33.7           7.2        40.9
    11.00          36.5          17.5        54.0
    12.00          38.5          28.5        67.0
    13.00          41.2          36.3        77.5


   NOTES:
   - AN ASTERISK IS PLACED IN THE END-BEARING COLUMN
     IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
     OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.



       *************************************************
       * COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
       * CURVES FOR AXIAL LOADING                      *
       *************************************************


T-Z CURVE   NO. OF   DEPTH TO CURVE   LOAD TRANSFER   PILE MOVEMENT
   NO.      POINTS        FT.              PSI             IN.

    1         10      0.4167E-01
                                       0.0000E+00      0.0000E+00
                                       0.4336E-02      0.2017E-01
                                       0.7226E-02      0.3908E-01
                                       0.1084E-01      0.7185E-01
                                       0.1301E-01      0.1008E+00
                                       0.1445E-01      0.1261E+00
```

```
                                   0.1445E-01      0.2521E+00
                                   0.1445E-01      0.3782E+00
                                   0.1445E-01      0.6303E+00
                                   0.1445E-01      0.2521E+01
2         10      0.1000E+01
                                   0.0000E+00      0.0000E+00
                                   0.1041E+00      0.2017E-01
                                   0.1734E+00      0.3908E-01
                                   0.2601E+00      0.7185E-01
                                   0.3122E+00      0.1008E+00
                                   0.3469E+00      0.1261E+00
                                   0.3469E+00      0.2521E+00
                                   0.3469E+00      0.3782E+00
                                   0.3469E+00      0.6303E+00
                                   0.3469E+00      0.2521E+01
3         10      0.1958E+01
                                   0.0000E+00      0.0000E+00
                                   0.2038E+00      0.2017E-01
                                   0.3396E+00      0.3908E-01
                                   0.5094E+00      0.7185E-01
                                   0.6113E+00      0.1008E+00
                                   0.6793E+00      0.1261E+00
                                   0.6793E+00      0.2521E+00
                                   0.6793E+00      0.3782E+00
                                   0.6793E+00      0.6303E+00
                                   0.6793E+00      0.2521E+01
4         10      0.2042E+01
                                   0.0000E+00      0.0000E+00
                                   0.3158E+00      0.2017E-01
                                   0.5263E+00      0.3908E-01
                                   0.7894E+00      0.7185E-01
                                   0.9473E+00      0.1008E+00
                                   0.1053E+01      0.1261E+00
                                   0.9473E+00      0.2521E+00
                                   0.9473E+00      0.3782E+00
                                   0.9473E+00      0.6303E+00
                                   0.9473E+00      0.2521E+01
5         10      0.2900E+01
                                   0.0000E+00      0.0000E+00
                                   0.2533E+01      0.2017E-01
                                   0.4222E+01      0.3908E-01
                                   0.6333E+01      0.7185E-01
                                   0.7600E+01      0.1008E+00
                                   0.8444E+01      0.1261E+00
                                   0.7600E+01      0.2521E+00
                                   0.7600E+01      0.3782E+00
                                   0.7600E+01      0.6303E+00
                                   0.7600E+01      0.2521E+01
6         10      0.3758E+01
                                   0.0000E+00      0.0000E+00
                                   0.2792E+01      0.2017E-01
```

```
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 7         10      0.3842E+01
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 8         10      0.6550E+01
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 9         10      0.9258E+01
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
10         10      0.9342E+01
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.9306E+01      0.2521E+00
                                    0.9306E+01      0.3782E+00
                                    0.9306E+01      0.6303E+00
```

```
                                            0.9306E+01      0.2521E+01
11          10      0.1010E+02
                                            0.0000E+00      0.0000E+00
                                            0.2595E+01      0.2017E-01
                                            0.4325E+01      0.3908E-01
                                            0.6487E+01      0.7185E-01
                                            0.7784E+01      0.1008E+00
                                            0.8649E+01      0.1261E+00
                                            0.8649E+01      0.2521E+00
                                            0.8649E+01      0.3782E+00
                                            0.8649E+01      0.6303E+00
                                            0.8649E+01      0.2521E+01
12          10      0.1086E+02
                                            0.0000E+00      0.0000E+00
                                            0.1101E+01      0.2017E-01
                                            0.1836E+01      0.3908E-01
                                            0.2753E+01      0.7185E-01
                                            0.3304E+01      0.1008E+00
                                            0.3671E+01      0.1261E+00
                                            0.3671E+01      0.2521E+00
                                            0.3671E+01      0.3782E+00
                                            0.3671E+01      0.6303E+00
                                            0.3671E+01      0.2521E+01
13          10      0.1094E+02
                                            0.0000E+00      0.0000E+00
                                            0.9372E+00      0.2017E-01
                                            0.1562E+01      0.3908E-01
                                            0.2343E+01      0.7185E-01
                                            0.2812E+01      0.1008E+00
                                            0.3124E+01      0.1261E+00
                                            0.3124E+01      0.2521E+00
                                            0.3124E+01      0.3782E+00
                                            0.3124E+01      0.6303E+00
                                            0.3124E+01      0.2521E+01
14          10      0.1795E+02
                                            0.0000E+00      0.0000E+00
                                            0.1799E+01      0.2017E-01
                                            0.2999E+01      0.3908E-01
                                            0.4499E+01      0.7185E-01
                                            0.5398E+01      0.1008E+00
                                            0.5998E+01      0.1261E+00
                                            0.5998E+01      0.2521E+00
                                            0.5998E+01      0.3782E+00
                                            0.5998E+01      0.6303E+00
                                            0.5998E+01      0.2521E+01
15          10      0.2496E+02
                                            0.0000E+00      0.0000E+00
                                            0.1799E+01      0.2017E-01
                                            0.2999E+01      0.3908E-01
                                            0.4499E+01      0.7185E-01
                                            0.5398E+01      0.1008E+00
```

```
                                        0.5998E+01      0.1261E+00
                                        0.5998E+01      0.2521E+00
                                        0.5998E+01      0.3782E+00
                                        0.5998E+01      0.6303E+00
                                        0.5998E+01      0.2521E+01


 TIP   LOAD        TIP MOVEMENT
     KIP               IN.

 0.0000E+00         0.0000E+00
 0.2271E+01         0.6303E-02
 0.4542E+01         0.1261E-01
 0.9084E+01         0.2521E-01
 0.1817E+02         0.1639E+00
 0.2725E+02         0.5294E+00
 0.3270E+02         0.9202E+00
 0.3633E+02         0.1261E+01
 0.3633E+02         0.1891E+01
 0.3633E+02         0.2521E+01


               LOAD VERSUS SETTLEMENT CURVE
               ****************************



 TOP  LOAD     TOP MOVEMENT       TIP  LOAD      TIP MOVEMENT
    KIP             IN.              KIP             IN.
0.1053E+00      0.1326E-03      0.3603E-01      0.1000E-03
0.1053E+01      0.1326E-02      0.3603E+00      0.1000E-02
0.5264E+01      0.6628E-02      0.1802E+01      0.5000E-02
0.1058E+02      0.1326E-01      0.3603E+01      0.1000E-01
0.2082E+02      0.2648E-01      0.7206E+01      0.2000E-01
0.3656E+02      0.6101E-01      0.1071E+02      0.5000E-01
0.4718E+02      0.9401E-01      0.1267E+02      0.8000E-01
0.5276E+02      0.1156E+00      0.1398E+02      0.1000E+00
0.5925E+02      0.2184E+00      0.1907E+02      0.2000E+00
0.6556E+02      0.5215E+00      0.2652E+02      0.5000E+00
0.7006E+02      0.8235E+00      0.3103E+02      0.8000E+00
0.7259E+02      0.1025E+01      0.3355E+02      0.1000E+01
0.7537E+02      0.2026E+01      0.3633E+02      0.2000E+01
```

CALCULATIONS

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 2 (Old Abutment 1) - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

OBJECTIVE

Evaluate the downdrag load acting on the HP 10x42 steel pile with the maximum loading at the Abutment 2 (Old Abutment 1) (Pile 6) of the bridge carrying Hogan Road Eastbound over I-95. This analysis uses estimated settlement from 1983 to present-day.

METHOD

Use APile (Ref. 1) to estimate the shaft resistance contributing to downdrag.

REFERENCES

1. Ensoft, Inc. APILE Version 2019.9.11 - Offshore. Release date October 15, 2021.
2. Maine Department of Transportation Bridge Program. Preliminary Design Report: Hogan Road / I-95 Bridge #5823 over Interstate 95, Exit 187 Diverging Diamond Interchange, Bangor, Maine, Federal Project #1859510, WIN 018595.10. Draft 2-3-2023.
3. WSP calculation titled "Hogan Rd Highway Settlement Embankment 1983 10 28 24.xlsx", dated 10/28/2024.
4. WSP Interpretive Subsurface Profile E-E', included as part of the Geotechnical Design Report.
5. WSP calculations of interpreted soil parameters based on field and laboratory data.
6. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
7. AASHTO LRFD Bridge Design Specifications, 9th Edition. 2020.
8. WSP pile resistance calculation and FB-Multipier model for Abutment 2 (old Abutment 1), dated 10/25/2024
9. VHB computation package titled "1983 Bridge Foundation Loads", dated 4/5/2024.

ATTACHMENTS

1. Profile of estimated settlement beneath Abutment 2 (Old Abutment 1) at 41 years after embankment widening construction at the location of the pile with maximum axial loading
2. Output from APile analysis

ASSUMPTIONS

1. The widened portion of the existing Abutment 2 (Old Abutment 1) is supported on eighteen HP10x42 steel piles, based on the as-built bridge widening plans dated 1983 in Ref. 2, Appendix C. The downdrag is evaluated for the pile with maximum axial loading based on the proposed abutment renovation loads provided by VHB (Ref. 9) and distributed to the individual piles by WSP using FB-MultiPier (Ref. 8). The estimated pile length at this location as shown in the 1983 as-built plans (Ref. 2, Appendix C, Sheet 13) is 42.2 feet below the base of the abutment. Based on WSP's interpreted subsurface profile along the existing abutment (Ref. 4), the piles are assumed to have been driven into weathered bedrock.

2. The downdrag depth is based on the total settlement estimated to have occurred up to present-day due to the 1983 embankment widening at the Abutment 2 (Old Abutment 1) at the location of the pile with maximum loading (Ref. 3).

3. The FHWA automated computation method provided in APile is used for the software computations of unit load transfers and axial pile capacity.

4. Since APile requires soil layers to be defined to a depth of at least the pile tip plus two pile diameters, the bedrock is modeled as "sand" with a friction angle of 50 degrees.

5. Downdrag is calculated with an uncorroded pile section to represent the maximum possible downdrag load.

6. Abutment designations are based on VHB designations in Ref. 9.

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 2 (Old Abutment 1) - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CALCULATION

1. Establish soil profile and pile properties.

The soil profile used in the downdrag analysis is based on Ref. 4 at the centerline of the Abutment 2 (Old Abutment 1). The soil properties used in the downdrag analysis are taken from Ref. 5 and are based on correlations to field test and laboratory test data.

Base of Abutment 1 elevation =	101.0	feet	(Ref. 2, Appendix C, Sheet 10)
Water table elevation =	87.1	feet	(Ref. 4)

Soil Layer	Elevation (feet)		Depth below Abutment (feet)		Effective Unit Weight (pcf)	Friction Angle (°)	Undrained Shear Strength (psf)	
	Top	Bottom	Top	Bottom			Top	Bottom
Existing Fill	101.0	94.7	0.0	6.3	125	34	-	-
Stiff Clay	94.7	87.1	6.3	13.9	122	-	1340	1340
Stiff Clay	87.1	82.1	13.9	18.9	59.6	-	1340	1340
Medium Clay	82.1	81.6	18.9	19.4	58.6	-	888	888
Glacial Till	81.6	61.3	19.4	39.7	67.6	35	-	-
Bedrock	61.3	-	39.7	-	95.6	50	-	-

Pile steel elastic modulus =	29,000,000	psi	(typical value for steel)
Pile length =	42.2	ft	(Assumption 1)
Pile section box perimeter =	39.6	in	(using pile flange width and depth from Ref. 6)
Pile section tip area =	12.4	in^2	(Ref. 6)
Pile nominal weight =	0.042	kip/ft	(Ref. 6)

2. Identify the depth below the abutment to which settlement is large enough to fully develop downdrag loading.

Per Ref. 7 Article 3.11.8, downdrag can be assumed to fully develop for the length of pile with settlement equal to or greater than 0.4 inches. As calculated in Ref. 3 - Attachment 10, settlement beneath Abutment 2 (Old Abutment 1) up to present-day (41 years after construction) due to construction of the widened bridge abutment and approach embankment is estimated to be greater than or equal to 0.4 inches down to an elevation of 83.4 feet (depth of 17.6 feet below the base of the abutment). Thus, downdrag loading is assumed to be fully developed for the upper 17.6 feet of the pile below the base of the abutment.

Per Ref. 7 Article 10.7.1.6.2 for piles end bearing on rock downdrag shall be considered at the strength and extreme limit state.

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 2 (Old Abutment 1) - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

3. Use APile to estimate shaft resistance and total downdrag load.

Downdrag Load Factors:
(Ref. 7, Tables 3.4.1-1 and 3.4.1-2)

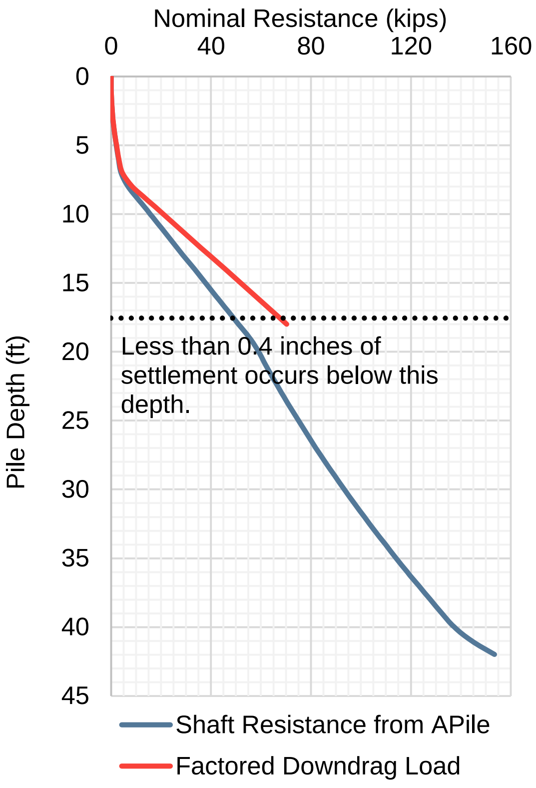
Soil Type	Strength I
Sand	1.05
Clay	1.40

Abutment Loads:

	Strength I		
Per Pile	117	kips	(Ref. 8)

Shaft resistance and downdrag load for each foot of pile length:

Pile Depth below Base of Abutment (feet)	Soil Type from APile	Shaft Resistance from APile (kips)	Factored Downdrag Load (kips)	Total Factored Axial Load* (kips)
0	Sand	0.0	0.0	117.0
1	Sand	0.1	0.1	117.1
2	Sand	0.3	0.3	117.4
3	Sand	0.7	0.7	117.9
4	Sand	1.3	1.4	118.5
5	Sand	2.1	2.2	119.4
6	Sand	3.0	3.2	120.4
7	Clay	4.0	4.6	121.8
8	Clay	6.8	8.5	125.8
9	Clay	11.2	14.6	132.0
10	Clay	15.7	20.9	138.4
11	Clay	20.1	27.1	144.6
12	Clay	24.5	33.3	150.8
13	Clay	28.9	39.4	157.0
14	Clay	33.4	45.7	163.3
15	Clay	37.8	51.9	169.5
16	Clay	42.2	58.0	175.7
17	Clay	46.6	64.2	181.9
18	**Clay**	**51.0**	**70.4**	**188.1**
19	Clay	55.5		117.8
20	Sand	59.1		117.8
21	Sand	62.0		117.9
22	Sand	65.1		117.9
23	Sand	68.3		118.0
24	Sand	71.6		118.0
25	Sand	75.0		118.1
26	Sand	78.5		118.1
27	Sand	82.0		118.1
28	Sand	85.7		118.2
29	Sand	89.5		118.2
30	Sand	93.3		118.3
31	Sand	97.3		118.3



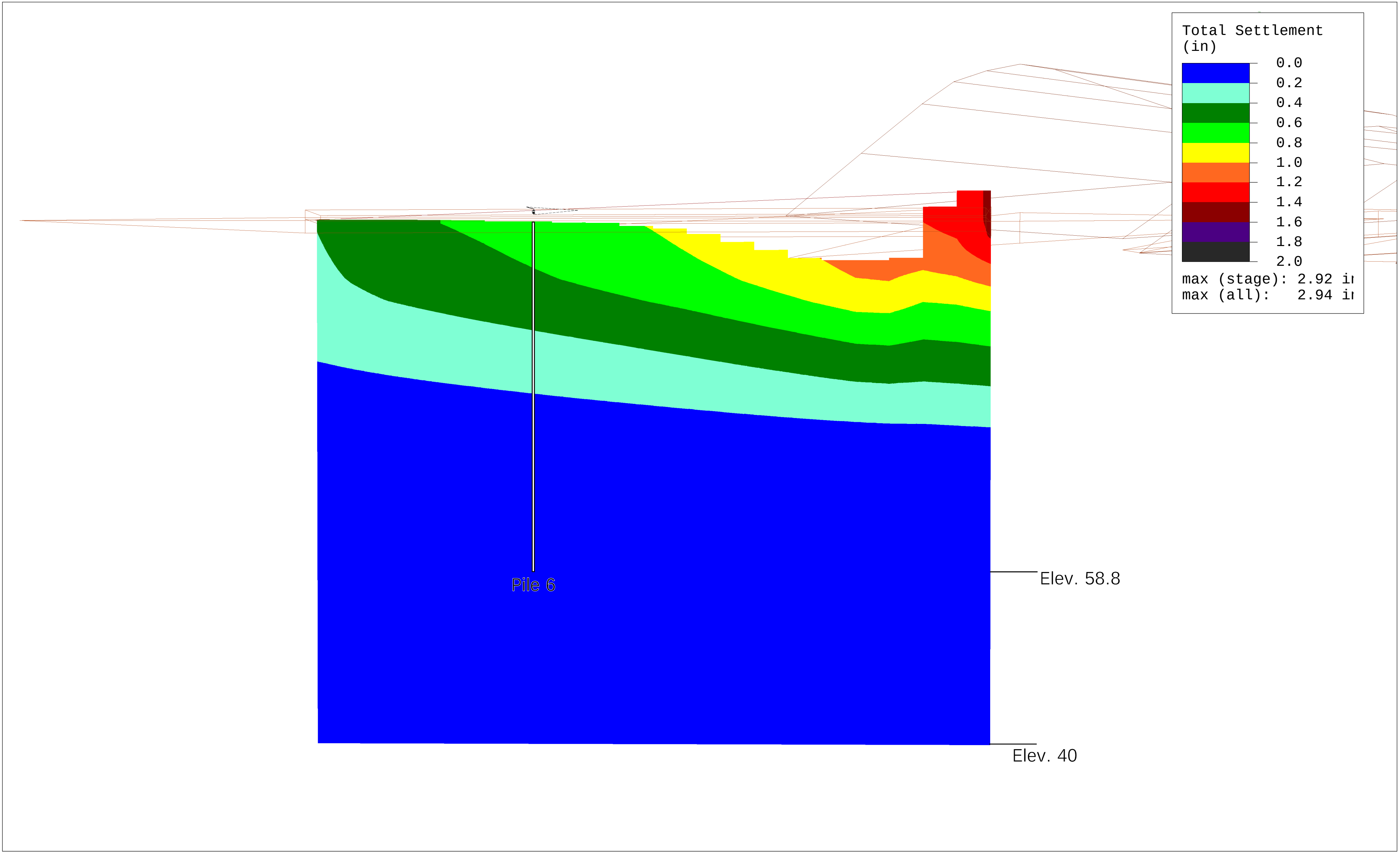
32	Sand	101.4		118.3
33	Sand	105.5		118.4
34	Sand	109.8		118.4
35	Sand	114.1		118.5
36	Sand	118.5		118.5
37	Sand	123.1		118.6
38	Sand	127.7		118.6
39	Sand	132.4		118.6
40	Sand	137.3		118.7
41	Sand	144.2		118.7
42	Sand	153.4		118.8

*Strength I, including Substructure Loading, Downdrag, and Pile Weight

Strength I factored downdrag load = 70.4 kips per pile

CONCLUSIONS

Based on the analysis of settlement between 1983 and present-day, downdrag is estimated to have developed along the upper 17.6 feet of pile 6 below the base of the Abutment 2 (Old Abutment 1). A factored downdrag load of 70.4 kips per pile for the Strength I limit state was calculated.



Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.004
Analysis Description	Total Settlement along Abutment 2 (Old Abutment 1) (Sta. 113+91) at Present-Day (41 years after embankment widening)

Drawn By LMP	*Checked By* KAR	*Reviewed By* CCB	*Scale* As shown	Attachment 1
Date 11/1/2024	*File Name* Hogan Rd Highway Interchange 1958 surface 1983Fill 10 24 -v3.s3z			

SETTLE3 5.023

Attachment 2

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

                      APILE for Windows, Version 2019.9.11

                       Serial Number :  264781183

                       A Program for Analyzing the Axial Capacity
                       and Short-term Settlement of Driven Piles
                                 under Axial Loading.
                     (c) Copyright ENSOFT, Inc., 1987-2019
                           All Rights Reserved

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


     This program is licensed to :

     WSP
     APILE Global, Global License

     Path to file locations      : C:\Users\USLP710461\WSP O365\MaineDOT Bangor
Hogan Road Bridge & DDI - Project Files\5 Technical Work\GDR Pile & Abutment
Design\Existing Bridge - Downdrag\APile Models\Abutement 2 Pile 6 - Max loading\
     Name of input data file     : HP10x42_Abutment-2 Pile 6MaxL.ap9d
     Name of output file         : HP10x42_Abutment-2 Pile 6MaxL.ap9o
     Name of plot output file    : HP10x42_Abutment-2 Pile 6MaxL.ap9p

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

               Date:  October 24, 2024     Time:  12:34:06



1
          *********************
          * INPUT INFORMATION *
          *********************



          Existing Pile Downdrag at Existing Abutment 1

          DESIGNER   : LMP

          JOB NUMBER : 31404817.004


          METHOD FOR UNIT LOAD TRANSFERS :
```

```
- FHWA (Federal Highway Administration)
  Unfactored Unit Side Friction and Unit Side Resistance are used.



COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)


TYPE OF LOADING :
- COMPRESSION


PILE TYPE :

H-Pile/Steel Pile


DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY     = 0.290E+08 PSI
- CROSS SECTION AREA        =     12.40 IN2


NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL     =     42.20 FT.
- BATTER ANGLE              =      0.00 DEG
- PILE STICKUP LENGTH, PSL  =      0.00 FT.
- ZERO FRICTION LENGTH, ZFL =      0.00 FT.
- PERIMETER OF PILE         =     39.60 IN.
- TIP AREA OF PILE          =     12.40 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION       =      1.00 FT.



SOIL INFORMATIONS :



                  LATERAL    EFFECTIVE    FRICTION      Nq
          SOIL    EARTH      UNIT         ANGLE       FACTOR
 DEPTH    TYPE    PRESSURE   WEIGHT       DEGREES      FHWA
  FT.                        LB/FT^3
   0.00   SAND      0.80*      125.00        34.00     55.60**
   6.30   SAND      0.80*      125.00        34.00     55.60**
   6.30   CLAY      0.80*      122.00         0.00      4.80**
  13.90   CLAY      0.80*      122.00         0.00      4.80**
  13.90   CLAY      0.80*       59.60         0.00      4.80**
```

```
  18.90   CLAY      0.80*       59.60         0.00      4.80**
  18.90   CLAY      0.80*       58.60         0.00      4.80**
  19.40   CLAY      0.80*       58.60         0.00      4.80**
  19.40   SAND      0.80*       67.60        35.00     64.00**
  39.70   SAND      0.80*       67.60        35.00     64.00**
  39.70   SAND      0.80*       95.60        50.00    475.00**
  50.00   SAND      0.80*       95.60        50.00    475.00**

*  VALUE ASSUMED BY THE PROGRAM
** VALUE ESTIMATED BY THE PROGRAM BASED ON FRICTION ANGLE



MAXIMUM   MAXIMUM   UNDISTURB  REMOLDED
UNIT      UNIT      SHEAR      SHEAR     BLOW   UNIT SKIN  UNIT END
FRICTION  BEARING   STRENGTH   STRENGTH  COUNT  FRICTION   BEARING
  KSF       KSF        KSF       KSF                 KSF       KSF
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.89      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.89      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
  WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
  PLAN TO LIMIT THE COMPUTED DATA.



         LRFD FACTOR  LRFD FACTOR
          ON UNIT      ON UNIT
 DEPTH    FRICTION     BEARING
  FT.
   0.00    1.000       1.000
   6.30    1.000       1.000
   6.30    1.000       1.000
  13.90    1.000       1.000
  13.90    1.000       1.000
  18.90    1.000       1.000
  18.90    1.000       1.000
  19.40    1.000       1.000
  19.40    1.000       1.000
  39.70    1.000       1.000
  39.70    1.000       1.000
```

```
          50.00    1.000       1.000



1
        **********************
        * COMPUTATION RESULT *
        **********************



                          **********************
                          *  FED. HWY. METHOD  *
                          **********************

           PILE           SKIN          END        ULTIMATE
        PENETRATION   FRICTION       BEARING     CAPACITY
            FT.           KIP           KIP          KIP
            0.00           0.0           0.2          0.2
            1.00           0.1           0.4          0.4
            2.00           0.3           0.8          1.1
            3.00           0.7           1.2          1.9
            4.00           1.3           1.6          2.9
            5.00           2.1           2.0          4.0
            6.00           3.0           2.0          5.0
            7.00           4.0           1.8          5.8
            8.00           6.8           1.3          8.2
            9.00          11.2           1.0         12.3
           10.00          15.7           1.0         16.7
           11.00          20.1           1.0         21.1
           12.00          24.5           1.0         25.6
           13.00          28.9           1.0         30.0
           14.00          33.4           1.0         34.4
           15.00          37.8           1.0         38.8
           16.00          42.2           1.0         43.2
           17.00          46.6           1.0         47.7
           18.00          51.0           1.0         52.0
           19.00          55.5           2.2         57.6
           20.00          59.1           4.4         63.5
           21.00          62.0           6.8         68.8
           22.00          65.1           8.3         73.4
           23.00          68.3           8.6         76.9
           24.00          71.6           8.8         80.4
           25.00          75.0           9.0         84.0
           26.00          78.5           9.2         87.6
           27.00          82.0           9.3         91.3
           28.00          85.7           9.3         95.0
           29.00          89.5           9.3         98.7
           30.00          93.3           9.3        102.6
           31.00          97.3           9.3        106.6
           32.00         101.4           9.3        110.6
```

```
     33.00         105.5           9.3        114.8
     34.00         109.8           9.3        119.0
     35.00         114.1           9.3        123.4
     36.00         118.5           9.3        127.8
     37.00         123.1           9.3        132.3
     38.00         127.7           9.3        137.0
     39.00         132.4          19.2        151.6
     40.00         137.3          36.4        173.7
     41.00         144.2          53.6        197.8
     42.00         153.4          63.5        216.9


    NOTES:
    - AN ASTERISK IS PLACED IN THE END-BEARING COLUMN
      IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
      OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.



        *************************************************
        * COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
        * CURVES FOR AXIAL LOADING                      *
        *************************************************


T-Z CURVE   NO. OF   DEPTH TO CURVE   LOAD TRANSFER   PILE MOVEMENT
   NO.      POINTS        FT.              PSI             IN.

    1         10      0.4167E-01
                                        0.0000E+00      0.0000E+00
                                        0.4336E-02      0.2017E-01
                                        0.7226E-02      0.3908E-01
                                        0.1084E-01      0.7185E-01
                                        0.1301E-01      0.1008E+00
                                        0.1445E-01      0.1261E+00
                                        0.1445E-01      0.2521E+00
                                        0.1445E-01      0.3782E+00
                                        0.1445E-01      0.6303E+00
                                        0.1445E-01      0.2521E+01
    2         10      0.3150E+01
                                        0.0000E+00      0.0000E+00
                                        0.3278E+00      0.2017E-01
                                        0.5463E+00      0.3908E-01
                                        0.8194E+00      0.7185E-01
                                        0.9833E+00      0.1008E+00
                                        0.1093E+01      0.1261E+00
                                        0.1093E+01      0.2521E+00
                                        0.1093E+01      0.3782E+00
                                        0.1093E+01      0.6303E+00
                                        0.1093E+01      0.2521E+01
    3         10      0.6258E+01
```

```
                                   0.0000E+00      0.0000E+00
                                   0.6512E+00      0.2017E-01
                                   0.1085E+01      0.3908E-01
                                   0.1628E+01      0.7185E-01
                                   0.1954E+01      0.1008E+00
                                   0.2171E+01      0.1261E+00
                                   0.2171E+01      0.2521E+00
                                   0.2171E+01      0.3782E+00
                                   0.2171E+01      0.6303E+00
                                   0.2171E+01      0.2521E+01
4         10      0.6342E+01
                                   0.0000E+00      0.0000E+00
                                   0.6599E+00      0.2017E-01
                                   0.1100E+01      0.3908E-01
                                   0.1650E+01      0.7185E-01
                                   0.1980E+01      0.1008E+00
                                   0.2200E+01      0.1261E+00
                                   0.1980E+01      0.2521E+00
                                   0.1980E+01      0.3782E+00
                                   0.1980E+01      0.6303E+00
                                   0.1980E+01      0.2521E+01
5         10      0.1010E+02
                                   0.0000E+00      0.0000E+00
                                   0.2792E+01      0.2017E-01
                                   0.4653E+01      0.3908E-01
                                   0.6979E+01      0.7185E-01
                                   0.8375E+01      0.1008E+00
                                   0.9306E+01      0.1261E+00
                                   0.8375E+01      0.2521E+00
                                   0.8375E+01      0.3782E+00
                                   0.8375E+01      0.6303E+00
                                   0.8375E+01      0.2521E+01
6         10      0.1386E+02
                                   0.0000E+00      0.0000E+00
                                   0.2792E+01      0.2017E-01
                                   0.4653E+01      0.3908E-01
                                   0.6979E+01      0.7185E-01
                                   0.8375E+01      0.1008E+00
                                   0.9306E+01      0.1261E+00
                                   0.8375E+01      0.2521E+00
                                   0.8375E+01      0.3782E+00
                                   0.8375E+01      0.6303E+00
                                   0.8375E+01      0.2521E+01
7         10      0.1394E+02
                                   0.0000E+00      0.0000E+00
                                   0.2792E+01      0.2017E-01
                                   0.4653E+01      0.3908E-01
                                   0.6979E+01      0.7185E-01
                                   0.8375E+01      0.1008E+00
                                   0.9306E+01      0.1261E+00
                                   0.8375E+01      0.2521E+00
```

```
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 8         10      0.1640E+02
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 9         10      0.1886E+02
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
10         10      0.1894E+02
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
11         10      0.1940E+02
                                    0.0000E+00      0.0000E+00
                                    0.2381E+01      0.2017E-01
                                    0.3969E+01      0.3908E-01
                                    0.5953E+01      0.7185E-01
                                    0.7143E+01      0.1008E+00
                                    0.7937E+01      0.1261E+00
                                    0.7143E+01      0.2521E+00
                                    0.7143E+01      0.3782E+00
                                    0.7143E+01      0.6303E+00
                                    0.7143E+01      0.2521E+01
12         10      0.1986E+02
                                    0.0000E+00      0.0000E+00
                                    0.1911E+01      0.2017E-01
                                    0.3185E+01      0.3908E-01
```

```
                                    0.4777E+01      0.7185E-01
                                    0.5732E+01      0.1008E+00
                                    0.6369E+01      0.1261E+00
                                    0.5732E+01      0.2521E+00
                                    0.5732E+01      0.3782E+00
                                    0.5732E+01      0.6303E+00
                                    0.5732E+01      0.2521E+01
13         10      0.1994E+02
                                    0.0000E+00      0.0000E+00
                                    0.1825E+01      0.2017E-01
                                    0.3042E+01      0.3908E-01
                                    0.4563E+01      0.7185E-01
                                    0.5476E+01      0.1008E+00
                                    0.6084E+01      0.1261E+00
                                    0.6084E+01      0.2521E+00
                                    0.6084E+01      0.3782E+00
                                    0.6084E+01      0.6303E+00
                                    0.6084E+01      0.2521E+01
14         10      0.2980E+02
                                    0.0000E+00      0.0000E+00
                                    0.2458E+01      0.2017E-01
                                    0.4097E+01      0.3908E-01
                                    0.6146E+01      0.7185E-01
                                    0.7375E+01      0.1008E+00
                                    0.8195E+01      0.1261E+00
                                    0.8195E+01      0.2521E+00
                                    0.8195E+01      0.3782E+00
                                    0.8195E+01      0.6303E+00
                                    0.8195E+01      0.2521E+01
15         10      0.3966E+02
                                    0.0000E+00      0.0000E+00
                                    0.3055E+01      0.2017E-01
                                    0.5091E+01      0.3908E-01
                                    0.7637E+01      0.7185E-01
                                    0.9164E+01      0.1008E+00
                                    0.1018E+02      0.1261E+00
                                    0.1018E+02      0.2521E+00
                                    0.1018E+02      0.3782E+00
                                    0.1018E+02      0.6303E+00
                                    0.1018E+02      0.2521E+01
16         10      0.3974E+02
                                    0.0000E+00      0.0000E+00
                                    0.3060E+01      0.2017E-01
                                    0.5099E+01      0.3908E-01
                                    0.7649E+01      0.7185E-01
                                    0.9179E+01      0.1008E+00
                                    0.1020E+02      0.1261E+00
                                    0.1020E+02      0.2521E+00
                                    0.1020E+02      0.3782E+00
                                    0.1020E+02      0.6303E+00
                                    0.1020E+02      0.2521E+01
```

```
   17         10      0.4485E+02
                                    0.0000E+00      0.0000E+00
                                    0.5881E+01      0.2017E-01
                                    0.9801E+01      0.3908E-01
                                    0.1470E+02      0.7185E-01
                                    0.1764E+02      0.1008E+00
                                    0.1960E+02      0.1261E+00
                                    0.1960E+02      0.2521E+00
                                    0.1960E+02      0.3782E+00
                                    0.1960E+02      0.6303E+00
                                    0.1960E+02      0.2521E+01
   18         10      0.4996E+02
                                    0.0000E+00      0.0000E+00
                                    0.5881E+01      0.2017E-01
                                    0.9801E+01      0.3908E-01
                                    0.1470E+02      0.7185E-01
                                    0.1764E+02      0.1008E+00
                                    0.1960E+02      0.1261E+00
                                    0.1960E+02      0.2521E+00
                                    0.1960E+02      0.3782E+00
                                    0.1960E+02      0.6303E+00
                                    0.1960E+02      0.2521E+01


 TIP   LOAD        TIP MOVEMENT
     KIP               IN.

 0.0000E+00         0.0000E+00
 0.3970E+01         0.6303E-02
 0.7939E+01         0.1261E-01
 0.1588E+02         0.2521E-01
 0.3176E+02         0.1639E+00
 0.4764E+02         0.5294E+00
 0.5716E+02         0.9202E+00
 0.6352E+02         0.1261E+01
 0.6352E+02         0.1891E+01
 0.6352E+02         0.2521E+01


               LOAD VERSUS SETTLEMENT CURVE
               ****************************


 TOP  LOAD     TOP MOVEMENT       TIP  LOAD      TIP MOVEMENT
    KIP             IN.               KIP             IN.
0.5170E+00      0.4985E-03      0.6299E-01      0.1000E-03
0.5170E+01      0.4985E-02      0.6299E+00      0.1000E-02
0.2631E+02      0.2518E-01      0.3149E+01      0.5000E-02
```

```
0.5018E+02      0.4959E-01      0.6299E+01      0.1000E-01
0.8717E+02      0.9220E-01      0.1260E+02      0.2000E-01
0.1396E+03      0.1698E+00      0.1872E+02      0.5000E-01
0.1638E+03      0.2266E+00      0.2215E+02      0.8000E-01
0.1726E+03      0.2583E+00      0.2444E+02      0.1000E+00
0.1830E+03      0.3751E+00      0.3333E+02      0.2000E+00
0.1961E+03      0.6934E+00      0.4636E+02      0.5000E+00
0.2040E+03      0.1005E+01      0.5423E+02      0.8000E+00
0.2084E+03      0.1211E+01      0.5865E+02      0.1000E+01
0.2132E+03      0.2218E+01      0.6352E+02      0.2000E+01
```

CALCULATIONS

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 2 (Old Abutment 1) - Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

OBJECTIVE

Evaluate the downdrag load acting on the HP 10x42 steel pile closest to the maximum settlement point on the Abutment 2 (Old Abutment 1) (Pile 18) of the bridge carrying Hogan Road Eastbound over I-95. This analysis uses estimated settlement from 1983 to present-day.

METHOD

Use APile (Ref. 1) to estimate the shaft resistance contributing to downdrag.

REFERENCES

1. Ensoft, Inc. APILE Version 2019.9.11 - Offshore. Release date October 15, 2021.
2. Maine Department of Transportation Bridge Program. Preliminary Design Report: Hogan Road / I-95 Bridge #5823 over Interstate 95, Exit 187 Diverging Diamond Interchange, Bangor, Maine, Federal Project #1859510, WIN 018595.10. Draft 2-3-2023.
3. WSP calculation titled "Hogan Rd Highway Settlement Embankment 1983 10 28 24.xlsx", dated 10/28/2024.
4. WSP Interpretive Subsurface Profile E-E', included as part of the Geotechnical Design Report.
5. WSP calculations of interpreted soil parameters based on field and laboratory data.
6. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
7. AASHTO LRFD Bridge Design Specifications, 9th Edition. 2020.
8. WSP pile resistance calculation and FB-Multipier model for Abutment 2 (old Abutment 1), dated 10/25/2024
9. VHB computation package titled "1983 Bridge Foundation Loads", dated 4/5/2024.

ATTACHMENTS

1. Profile of estimated settlement beneath Abutment 2 (Old Abutment 1) at 41 years after embankment widening construction at the location of the pile with maximum axial loading
2. Output from APile analysis

ASSUMPTIONS

1. The widened portion of the existing Abutment 2 (Old Abutment 1) is supported on eighteen HP10x42 steel piles, based on the as-built bridge widening plans dated 1983 in Ref. 2, Appendix C. The downdrag is evaluated for the pile closest to the maximum settlement. The estimated pile length at this location as shown in the 1983 as-built plans (Ref. 2, Appendix C, Sheet 13) is 42.2 feet below the base of the abutment. Based on WSP's interpreted subsurface profile along the existing abutment (Ref. 4), the piles are assumed to have been driven into weathered bedrock.
2. The downdrag depth is based on the total settlement estimated to have occurred up to present-day due to the 1983 embankment widening at the Abutment 2 (Old Abutment 1) at the maximum settlement location of the Abutment 2 (Ref. 3).
3. The FHWA automated computation method provided in APile is used for the software computations of unit load transfers and axial pile capacity.
4. Since APile requires soil layers to be defined to a depth of at least the pile tip plus two pile diameters, the bedrock is modeled as "sand" with a friction angle of 50 degrees.
5. Downdrag is calculated with an uncorroded pile section to represent the maximum possible downdrag load.
6. Abutment designations are based on VHB designations in Ref. 9.

CALCULATIONS

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 2 (Old Abutment 1) - Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CALCULATION

1. Establish soil profile and pile properties.

The soil profile used in the downdrag analysis is based on Ref. 4 at the centerline of the Abutment 2 (Old Abutment 1). The soil properties used in the downdrag analysis are taken from Ref. 5 and are based on correlations to field test and laboratory test data.

Base of Abutment 1 elevation =	101.0	feet	(Ref. 2, Appendix C, Sheet 10)
Water table elevation =	87.1	feet	(Ref. 4)

Soil Layer	Elevation (feet)		Depth below Abutment (feet)		Effective Unit Weight (pcf)	Friction Angle (°)	Undrained Shear Strength (psf)	
	Top	Bottom	Top	Bottom			Top	Bottom
Existing Fill	101.0	91.6	0.0	9.4	125	34	-	-
Stiff Clay	91.6	87.1	9.4	13.9	122	-	1340	1340
Stiff Clay	87.1	79.8	13.9	21.2	59.6	-	1340	1340
Medium Clay	79.8	77.1	21.2	23.9	58.6	-	888	888
Glacial Till	77.1	58.2	23.9	42.8	67.6	35	-	-
Bedrock	58.2	-	42.8	-	95.6	50	-	-

Pile steel elastic modulus =	29,000,000	psi	(typical value for steel)
Pile length =	45.3	ft	(Assumption 1)
Pile section box perimeter =	39.6	in	(using pile flange width and depth from Ref. 6)
Pile section tip area =	12.4	in^2	(Ref. 6)
Pile nominal weight =	0.042	kip/ft	(Ref. 6)

2. Identify the depth below the abutment to which settlement is large enough to fully develop downdrag loading.

Per Ref. 7 Article 3.11.8, downdrag can be assumed to fully develop for the length of pile with settlement equal to or greater than 0.4 inches. As calculated in Ref. 3 - Attachment 10, settlement beneath Abutment 2 (Old Abutment 1) up to present-day (41 years after construction) due to construction of the widened bridge abutment and approach embankment is estimated to be greater than or equal to 0.4 inches down to an elevation of 77.62 feet (depth of 23.4 feet below the base of the abutment). Thus, downdrag loading is assumed to be fully developed for the upper 23.4 feet of the pile below the base of the abutment.

Per Ref. 7 Article 10.7.1.6.2 for piles end bearing on rock downdrag shall be considered at the strength and extreme limit state.

CALCULATIONS

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 2 (Old Abutment 1) - Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

3. Use APile to estimate shaft resistance and total downdrag load.

Downdrag Load Factors:
(Ref. 7, Tables 3.4.1-1 and 3.4.1-2)

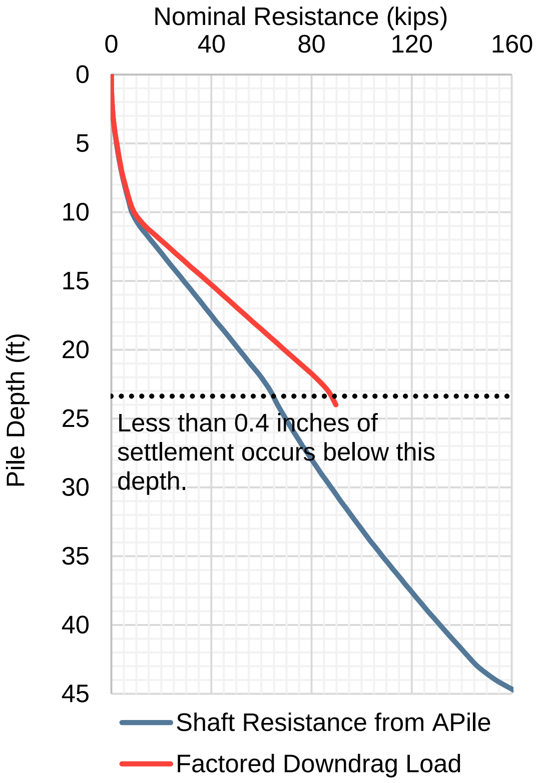
Soil Type	Strength I
Sand	1.05
Clay	1.40

Abutment Loads:

	Strength I		
Per Pile	46.2	kips	(Ref. 8)

Shaft resistance and downdrag load for each foot of pile length:

Pile Depth below Base of Abutment (feet)	Soil Type from APile	Shaft Resistance from APile (kips)	Factored Downdrag Load (kips)	Total Factored Axial Load* (kips)
0	Sand	0.0	0.0	46.2
1	Sand	0.1	0.1	46.3
2	Sand	0.3	0.3	46.6
3	Sand	0.7	0.7	47.1
4	Sand	1.3	1.4	47.7
5	Sand	2.1	2.2	48.6
6	Sand	3.0	3.2	49.6
7	Sand	4.0	4.2	50.7
8	Sand	5.3	5.6	52.1
9	Sand	6.7	7.0	53.6
10	Clay	8.2	9.1	55.8
11	Clay	11.3	13.5	60.1
12	Clay	15.7	19.6	66.3
13	Clay	20.1	25.8	72.5
14	Clay	24.5	32.0	78.7
15	Clay	29.0	38.3	85.1
16	Clay	33.4	44.4	91.3
17	Clay	37.8	50.6	97.5
18	Clay	42.2	56.7	103.7
19	Clay	46.7	63.0	110.0
20	Clay	51.1	69.2	116.2
21	Clay	55.5	75.4	122.4
22	Clay	59.9	81.5	128.6
23	Clay	63.6	86.7	133.9
24	**Sand**	**66.5**	**89.7**	**136.9**
25	Sand	69.6		47.3
26	Sand	73.0		47.3
27	Sand	76.5		47.3
28	Sand	80.2		47.4
29	Sand	83.9		47.4
30	Sand	87.8		47.5
31	Sand	91.7		47.5



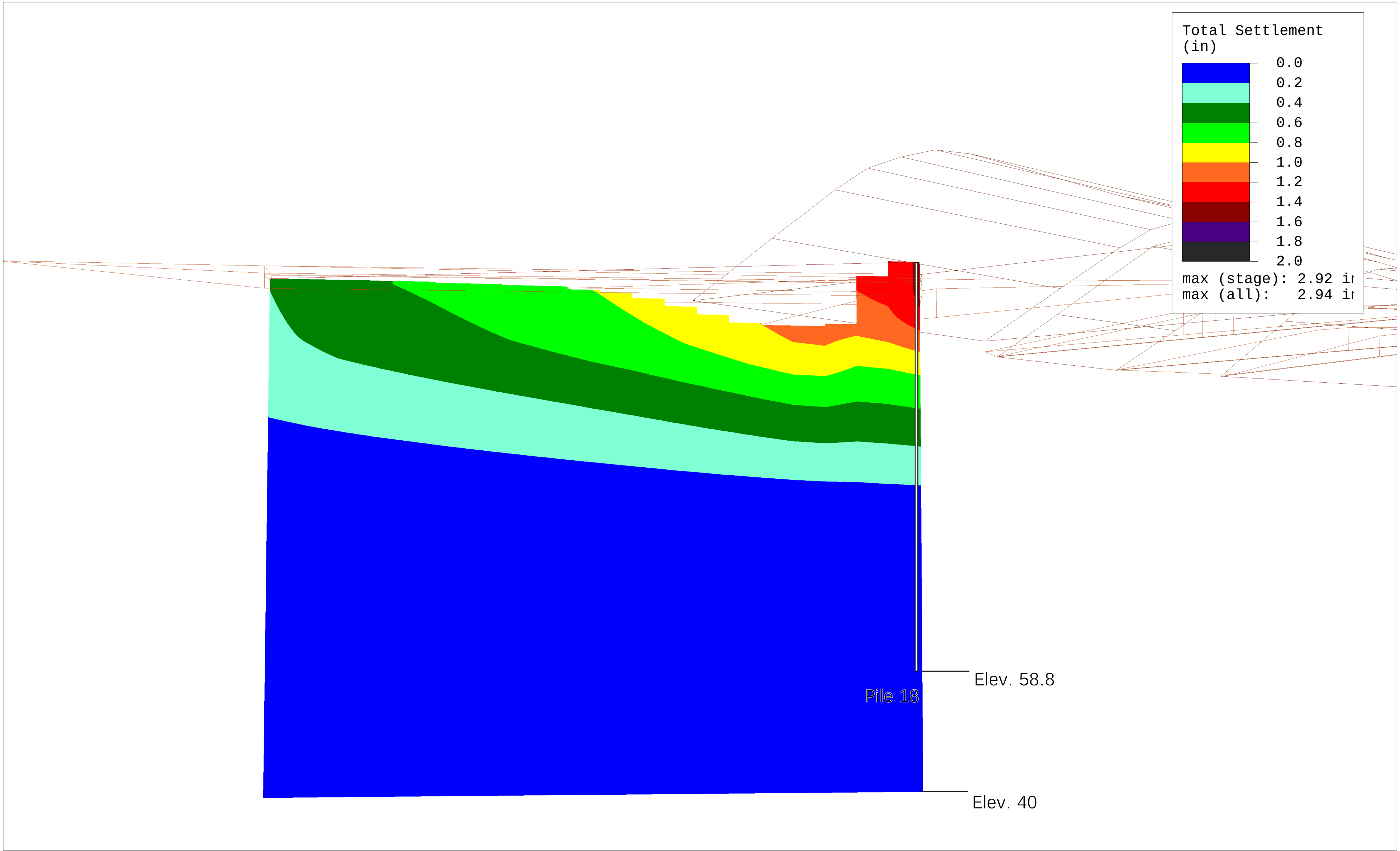
32	Sand	95.7		47.5
33	Sand	99.8		47.6
34	Sand	104.0		47.6
35	Sand	108.3		47.7
36	Sand	112.8		47.7
37	Sand	117.3		47.8
38	Sand	121.9		47.8
39	Sand	126.5		47.8
40	Sand	131.3		47.9
41	Sand	136.2		47.9
42	Sand	141.2		48.0
43	Sand	146.3		48.0
44	Sand	153.6		48.0
45	Sand	163.3		48.1

*Strength I, including Substructure Loading, Downdrag, and Pile Weight

Strength I factored downdrag load = 89.7 kips per pile

CONCLUSIONS

Based on the analysis of settlement between 1983 and present-day, downdrag is estimated to have developed along the upper 23.4 feet of pile 18 below the base of the Abutment 2 (Old Abutment 1). A factored downdrag load of 89.7 kips per pile for the Strength I limit state was calculated.



Project
MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.004

Analysis Description Total Settlement along Abutment 2 (Old Abutment 1) (Sta. 113+91) at Present-Day (41 years after embankment widening)

Drawn By LMP
Checked By KAR
Reviewed By CCB
Scale As shown

Date 11/1/2024
File Name Hogan Rd Highway Interchange 1958 surface 1983Fill 10 24 -v3.s3z

Attachment 1

SETTLE3 5.023

Attachment 2

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

                    APILE for Windows, Version 2019.9.11

                     Serial Number :  264781183

                     A Program for Analyzing the Axial Capacity
                     and Short-term Settlement of Driven Piles
                               under Axial Loading.
                   (c) Copyright ENSOFT, Inc., 1987-2019
                         All Rights Reserved

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


     This program is licensed to :

     WSP
     APILE Global, Global License

     Path to file locations      : C:\Users\USLP710461\WSP O365\MaineDOT Bangor
Hogan Road Bridge & DDI - Project Files\5 Technical Work\GDR Pile & Abutment
Design\Existing Bridge - Downdrag\APile Models\Abutement 2 Pile 18 - Max Settlement\
     Name of input data file     : HP10x42_Abutment-2 Pile 18MaxS.ap9d
     Name of output file         : HP10x42_Abutment-2 Pile 18MaxS.ap9o
     Name of plot output file    : HP10x42_Abutment-2 Pile 18MaxS.ap9p

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

               Date:  October 25, 2024     Time:  16:29:25



1
          *********************
          * INPUT INFORMATION *
          *********************



          Existing Pile Downdrag at Existing Abutment 1

          DESIGNER   : LMP

          JOB NUMBER : 31404817.004


          METHOD FOR UNIT LOAD TRANSFERS :
```

```
- FHWA (Federal Highway Administration)
  Unfactored Unit Side Friction and Unit Side Resistance are used.



COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)


TYPE OF LOADING :
- COMPRESSION


PILE TYPE :

H-Pile/Steel Pile


DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY     = 0.290E+08 PSI
- CROSS SECTION AREA        =     12.40 IN2


NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL     =     45.30 FT.
- BATTER ANGLE              =      0.00 DEG
- PILE STICKUP LENGTH, PSL  =      0.00 FT.
- ZERO FRICTION LENGTH, ZFL =      0.00 FT.
- PERIMETER OF PILE         =     39.60 IN.
- TIP AREA OF PILE          =     12.40 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION       =      1.00 FT.



SOIL INFORMATIONS :



                  LATERAL    EFFECTIVE    FRICTION      Nq
          SOIL    EARTH      UNIT         ANGLE        FACTOR
 DEPTH    TYPE    PRESSURE   WEIGHT       DEGREES       FHWA
  FT.                        LB/FT^3
   0.00   SAND      0.80*      125.00       34.00      55.60**
   9.40   SAND      0.80*      125.00       34.00      55.60**
   9.40   CLAY      0.80*      122.00        0.00       4.80**
  13.90   CLAY      0.80*      122.00        0.00       4.80**
  13.90   CLAY      0.80*       59.60        0.00       4.80**
```

```
  21.20   CLAY      0.80*       59.60         0.00      4.80**
  21.20   CLAY      0.80*       58.60         0.00      4.80**
  23.90   CLAY      0.80*       58.60         0.00      4.80**
  23.90   SAND      0.80*       67.60        35.00     64.00**
  42.80   SAND      0.80*       67.60        35.00     64.00**
  42.80   SAND      0.80*       95.60        50.00    475.00**
  50.00   SAND      0.80*       95.60        50.00    475.00**

*  VALUE ASSUMED BY THE PROGRAM
** VALUE ESTIMATED BY THE PROGRAM BASED ON FRICTION ANGLE



MAXIMUM   MAXIMUM   UNDISTURB  REMOLDED
UNIT      UNIT      SHEAR      SHEAR     BLOW   UNIT SKIN  UNIT END
FRICTION  BEARING   STRENGTH   STRENGTH  COUNT  FRICTION   BEARING
  KSF       KSF        KSF       KSF               KSF       KSF
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.89      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.89      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
  WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
  PLAN TO LIMIT THE COMPUTED DATA.



         LRFD FACTOR  LRFD FACTOR
          ON UNIT      ON UNIT
 DEPTH    FRICTION     BEARING
  FT.
   0.00    1.000       1.000
   9.40    1.000       1.000
   9.40    1.000       1.000
  13.90    1.000       1.000
  13.90    1.000       1.000
  21.20    1.000       1.000
  21.20    1.000       1.000
  23.90    1.000       1.000
  23.90    1.000       1.000
  42.80    1.000       1.000
  42.80    1.000       1.000
```

```
            50.00    1.000       1.000



1
          **********************
          * COMPUTATION RESULT *
          **********************



                            **********************
                            *  FED. HWY. METHOD  *
                            **********************

             PILE          SKIN          END       ULTIMATE
          PENETRATION   FRICTION       BEARING     CAPACITY
              FT.          KIP           KIP          KIP
              0.00           0.0           0.2          0.2
              1.00           0.1           0.4          0.4
              2.00           0.3           0.8          1.1
              3.00           0.7           1.2          1.9
              4.00           1.3           1.6          2.9
              5.00           2.1           2.0          4.0
              6.00           3.0           2.4          5.3
              7.00           4.0           2.8          6.8
              8.00           5.3           3.2          8.4
              9.00           6.7           3.0          9.7
             10.00           8.2           2.3         10.6
             11.00          11.3           1.6         12.8
             12.00          15.7           1.0         16.7
             13.00          20.1           1.0         21.2
             14.00          24.5           1.0         25.6
             15.00          29.0           1.0         30.0
             16.00          33.4           1.0         34.4
             17.00          37.8           1.0         38.8
             18.00          42.2           1.0         43.3
             19.00          46.7           1.0         47.7
             20.00          51.1           1.0         52.1
             21.00          55.5           1.0         56.5
             22.00          59.9           0.9         60.8
             23.00          63.6           2.2         65.8
             24.00          66.5           4.8         71.3
             25.00          69.6           7.5         77.1
             26.00          73.0           9.2         82.2
             27.00          76.5           9.2         85.8
             28.00          80.2           9.3         89.5
             29.00          83.9           9.3         93.2
             30.00          87.8           9.3         97.0
             31.00          91.7           9.3        100.9
             32.00          95.7           9.3        105.0
```

```
     33.00          99.8          9.3        109.1
     34.00         104.0          9.3        113.3
     35.00         108.3          9.3        117.6
     36.00         112.8          9.3        122.0
     37.00         117.3          9.3        126.5
     38.00         121.9          9.3        131.1
     39.00         126.5          9.3        135.8
     40.00         131.3          9.3        140.6
     41.00         136.2          9.3        145.5
     42.00         141.2         19.2        160.4
     43.00         146.3         36.4        182.7
     44.00         153.6         53.6        207.2
     45.00         163.3         63.5        226.8


    NOTES:
    - AN ASTERISK IS PLACED IN THE END-BEARING COLUMN
      IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
      OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.



        *************************************************
        * COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
        * CURVES FOR AXIAL LOADING                      *
        *************************************************


T-Z CURVE   NO. OF   DEPTH TO CURVE   LOAD TRANSFER   PILE MOVEMENT
   NO.      POINTS         FT.             PSI              IN.

    1         10      0.4167E-01
                                       0.0000E+00      0.0000E+00
                                       0.4336E-02      0.2017E-01
                                       0.7226E-02      0.3908E-01
                                       0.1084E-01      0.7185E-01
                                       0.1301E-01      0.1008E+00
                                       0.1445E-01      0.1261E+00
                                       0.1445E-01      0.2521E+00
                                       0.1445E-01      0.3782E+00
                                       0.1445E-01      0.6303E+00
                                       0.1445E-01      0.2521E+01
    2         10      0.4700E+01
                                       0.0000E+00      0.0000E+00
                                       0.4891E+00      0.2017E-01
                                       0.8151E+00      0.3908E-01
                                       0.1223E+01      0.7185E-01
                                       0.1467E+01      0.1008E+00
                                       0.1630E+01      0.1261E+00
                                       0.1630E+01      0.2521E+00
                                       0.1630E+01      0.3782E+00
```

```
                                   0.1630E+01      0.6303E+00
                                   0.1630E+01      0.2521E+01
3         10      0.9358E+01
                                   0.0000E+00      0.0000E+00
                                   0.9738E+00      0.2017E-01
                                   0.1623E+01      0.3908E-01
                                   0.2434E+01      0.7185E-01
                                   0.2921E+01      0.1008E+00
                                   0.3246E+01      0.1261E+00
                                   0.3246E+01      0.2521E+00
                                   0.3246E+01      0.3782E+00
                                   0.3246E+01      0.6303E+00
                                   0.3246E+01      0.2521E+01
4         10      0.9442E+01
                                   0.0000E+00      0.0000E+00
                                   0.9825E+00      0.2017E-01
                                   0.1637E+01      0.3908E-01
                                   0.2456E+01      0.7185E-01
                                   0.2947E+01      0.1008E+00
                                   0.3275E+01      0.1261E+00
                                   0.2947E+01      0.2521E+00
                                   0.2947E+01      0.3782E+00
                                   0.2947E+01      0.6303E+00
                                   0.2947E+01      0.2521E+01
5         10      0.1165E+02
                                   0.0000E+00      0.0000E+00
                                   0.2792E+01      0.2017E-01
                                   0.4653E+01      0.3908E-01
                                   0.6979E+01      0.7185E-01
                                   0.8375E+01      0.1008E+00
                                   0.9306E+01      0.1261E+00
                                   0.8375E+01      0.2521E+00
                                   0.8375E+01      0.3782E+00
                                   0.8375E+01      0.6303E+00
                                   0.8375E+01      0.2521E+01
6         10      0.1386E+02
                                   0.0000E+00      0.0000E+00
                                   0.2792E+01      0.2017E-01
                                   0.4653E+01      0.3908E-01
                                   0.6979E+01      0.7185E-01
                                   0.8375E+01      0.1008E+00
                                   0.9306E+01      0.1261E+00
                                   0.8375E+01      0.2521E+00
                                   0.8375E+01      0.3782E+00
                                   0.8375E+01      0.6303E+00
                                   0.8375E+01      0.2521E+01
7         10      0.1394E+02
                                   0.0000E+00      0.0000E+00
                                   0.2792E+01      0.2017E-01
                                   0.4653E+01      0.3908E-01
                                   0.6979E+01      0.7185E-01
```

```
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 8         10      0.1755E+02
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 9         10      0.2116E+02
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
10         10      0.2124E+02
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
11         10      0.2255E+02
                                    0.0000E+00      0.0000E+00
                                    0.2274E+01      0.2017E-01
                                    0.3790E+01      0.3908E-01
                                    0.5684E+01      0.7185E-01
                                    0.6821E+01      0.1008E+00
                                    0.7579E+01      0.1261E+00
                                    0.6821E+01      0.2521E+00
                                    0.6821E+01      0.3782E+00
                                    0.6821E+01      0.6303E+00
                                    0.6821E+01      0.2521E+01
12         10      0.2386E+02
```

```
                                    0.0000E+00      0.0000E+00
                                    0.1777E+01      0.2017E-01
                                    0.2962E+01      0.3908E-01
                                    0.4443E+01      0.7185E-01
                                    0.5332E+01      0.1008E+00
                                    0.5924E+01      0.1261E+00
                                    0.5332E+01      0.2521E+00
                                    0.5332E+01      0.3782E+00
                                    0.5332E+01      0.6303E+00
                                    0.5332E+01      0.2521E+01
13         10      0.2394E+02
                                    0.0000E+00      0.0000E+00
                                    0.1770E+01      0.2017E-01
                                    0.2950E+01      0.3908E-01
                                    0.4426E+01      0.7185E-01
                                    0.5311E+01      0.1008E+00
                                    0.5901E+01      0.1261E+00
                                    0.5901E+01      0.2521E+00
                                    0.5901E+01      0.3782E+00
                                    0.5901E+01      0.6303E+00
                                    0.5901E+01      0.2521E+01
14         10      0.3335E+02
                                    0.0000E+00      0.0000E+00
                                    0.2652E+01      0.2017E-01
                                    0.4419E+01      0.3908E-01
                                    0.6629E+01      0.7185E-01
                                    0.7955E+01      0.1008E+00
                                    0.8839E+01      0.1261E+00
                                    0.8839E+01      0.2521E+00
                                    0.8839E+01      0.3782E+00
                                    0.8839E+01      0.6303E+00
                                    0.8839E+01      0.2521E+01
15         10      0.4276E+02
                                    0.0000E+00      0.0000E+00
                                    0.3221E+01      0.2017E-01
                                    0.5368E+01      0.3908E-01
                                    0.8052E+01      0.7185E-01
                                    0.9662E+01      0.1008E+00
                                    0.1074E+02      0.1261E+00
                                    0.1074E+02      0.2521E+00
                                    0.1074E+02      0.3782E+00
                                    0.1074E+02      0.6303E+00
                                    0.1074E+02      0.2521E+01
16         10      0.4284E+02
                                    0.0000E+00      0.0000E+00
                                    0.3226E+01      0.2017E-01
                                    0.5376E+01      0.3908E-01
                                    0.8064E+01      0.7185E-01
                                    0.9677E+01      0.1008E+00
                                    0.1075E+02      0.1261E+00
                                    0.1075E+02      0.2521E+00
```

```
                                       0.1075E+02      0.3782E+00
                                       0.1075E+02      0.6303E+00
                                       0.1075E+02      0.2521E+01
   17         10      0.4640E+02
                                       0.0000E+00      0.0000E+00
                                       0.6170E+01      0.2017E-01
                                       0.1028E+02      0.3908E-01
                                       0.1543E+02      0.7185E-01
                                       0.1851E+02      0.1008E+00
                                       0.2057E+02      0.1261E+00
                                       0.2057E+02      0.2521E+00
                                       0.2057E+02      0.3782E+00
                                       0.2057E+02      0.6303E+00
                                       0.2057E+02      0.2521E+01
   18         10      0.4996E+02
                                       0.0000E+00      0.0000E+00
                                       0.6170E+01      0.2017E-01
                                       0.1028E+02      0.3908E-01
                                       0.1543E+02      0.7185E-01
                                       0.1851E+02      0.1008E+00
                                       0.2057E+02      0.1261E+00
                                       0.2057E+02      0.2521E+00
                                       0.2057E+02      0.3782E+00
                                       0.2057E+02      0.6303E+00
                                       0.2057E+02      0.2521E+01


TIP   LOAD        TIP MOVEMENT
    KIP               IN.

0.0000E+00         0.0000E+00
0.3970E+01         0.6303E-02
0.7939E+01         0.1261E-01
0.1588E+02         0.2521E-01
0.3176E+02         0.1639E+00
0.4764E+02         0.5294E+00
0.5716E+02         0.9202E+00
0.6352E+02         0.1261E+01
0.6352E+02         0.1891E+01
0.6352E+02         0.2521E+01


              LOAD VERSUS SETTLEMENT CURVE
              ****************************


TOP  LOAD     TOP MOVEMENT       TIP  LOAD      TIP MOVEMENT
   KIP             IN.              KIP            IN.
```

```
0.5715E+00       0.5784E-03       0.6299E-01       0.1000E-03
0.5715E+01       0.5784E-02       0.6299E+00       0.1000E-02
0.2905E+02       0.2926E-01       0.3149E+01       0.5000E-02
0.5486E+02       0.5725E-01       0.6299E+01       0.1000E-01
0.9442E+02       0.1053E+00       0.1260E+02       0.2000E-01
0.1493E+03       0.1907E+00       0.1872E+02       0.5000E-01
0.1739E+03       0.2510E+00       0.2215E+02       0.8000E-01
0.1830E+03       0.2841E+00       0.2444E+02       0.1000E+00
0.1936E+03       0.4022E+00       0.3333E+02       0.2000E+00
0.2066E+03       0.7219E+00       0.4636E+02       0.5000E+00
0.2145E+03       0.1034E+01       0.5423E+02       0.8000E+00
0.2189E+03       0.1240E+01       0.5865E+02       0.1000E+01
0.2238E+03       0.2248E+01       0.6352E+02       0.2000E+01
```

CALCULATIONS

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 2 (Old Abutment 1) - Pile with minimum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

OBJECTIVE

Evaluate the downdrag load acting on the HP 10x42 steel pile closest to the minimum settlement point on the Abutment 2 (Old Abutment 1) (Pile 2) of the bridge carrying Hogan Road Eastbound over I-95. This analysis uses estimated settlement from 1983 to present-day.

METHOD

Use APile (Ref. 1) to estimate the shaft resistance contributing to downdrag.

REFERENCES

1. Ensoft, Inc. APILE Version 2019.9.11 - Offshore. Release date October 15, 2021.
2. Maine Department of Transportation Bridge Program. Preliminary Design Report: Hogan Road / I-95 Bridge #5823 over Interstate 95, Exit 187 Diverging Diamond Interchange, Bangor, Maine, Federal Project #1859510, WIN 018595.10. Draft 2-3-2023.
3. WSP calculation titled "Hogan Rd Highway Settlement Embankment 1983 10 28 24.xlsx", dated 10/28/2024.
4. WSP Interpretive Subsurface Profile E-E', included as part of the Geotechnical Design Report.
5. WSP calculations of interpreted soil parameters based on field and laboratory data.
6. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
7. AASHTO LRFD Bridge Design Specifications, 9th Edition. 2020.
8. WSP pile resistance calculation and FB-Multipier model for Abutment 2 (old Abutment 1), dated 10/25/2024
9. VHB computation package titled "1983 Bridge Foundation Loads", dated 4/5/2024.

ATTACHMENTS

1. Profile of estimated settlement beneath Abutment 2 (Old Abutment 1) at 41 years after embankment widening construction at the location of the pile with maximum axial loading
2. Output from APile analysis

ASSUMPTIONS

1. The widened portion of the existing Abutment 2 (Old Abutment 1) is supported on eighteen HP10x42 steel piles, based on the as-built bridge widening plans dated 1983 in Ref. 2, Appendix C. The downdrag is evaluated for the pile closest to the minimum settlement. The estimated pile length at this location as shown in the 1983 as-built plans (Ref. 2, Appendix C, Sheet 13) is 42.2 feet below the base of the abutment. Based on WSP's interpreted subsurface profile along the existing abutment (Ref. 4), the piles are assumed to have been driven into weathered bedrock.
2. The downdrag depth is based on the total settlement estimated to have occurred up to present-day due to the 1983 embankment widening at the Abutment 2 (Old Abutment 1) at the minimum settlement location of the Abutment 2 (Ref. 3).
3. The FHWA automated computation method provided in APile is used for the software computations of unit load transfers and axial pile capacity.
4. Since APile requires soil layers to be defined to a depth of at least the pile tip plus two pile diameters, the bedrock is modeled as "sand" with a friction angle of 50 degrees.
5. Downdrag is calculated with an uncorroded pile section to represent the maximum possible downdrag load.
6. Abutment designations are based on VHB designations in Ref. 9.

CALCULATIONS

Date:	11/1/2024	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Existing Pile Downdrag at Abutment 2 (Old Abutment 1) - Pile with minimum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CALCULATION

1. Establish soil profile and pile properties.

The soil profile used in the downdrag analysis is based on Ref. 4 at the centerline of the Abutment 2 (Old Abutment 1). The soil properties used in the downdrag analysis are taken from Ref. 5 and are based on correlations to field test and laboratory test data.

Base of Abutment 1 elevation =	101.0	feet	(Ref. 2, Appendix C, Sheet 10)
Water table elevation =	87.1	feet	(Ref. 4)

Soil Layer	Elevation (feet)		Depth below Abutment (feet)		Effective Unit Weight (pcf)	Friction Angle (°)	Undrained Shear Strength (psf)	
	Top	Bottom	Top	Bottom			Top	Bottom
Existing Fill	101.0	95.5	0.0	5.5	125	34	-	-
Stiff Clay	95.5	87.1	5.5	13.9	122	-	1340	1340
Stiff Clay	87.1	82.6	13.9	18.4	59.6	-	1340	1340
Medium Clay	82.6	82.4	18.4	18.6	58.6	-	888	888
Glacial Till	82.4	62.0	18.6	39.0	67.6	35	-	-
Bedrock	62.0	-	39.0	-	95.6	50	-	-

Pile steel elastic modulus =	29,000,000	psi	(typical value for steel)
Pile length =	41.5	ft	(Assumption 1)
Pile section box perimeter =	39.6	in	(using pile flange width and depth from Ref. 6)
Pile section tip area =	12.4	in^2	(Ref. 6)
Pile nominal weight =	0.042	kip/ft	(Ref. 6)

2. Identify the depth below the abutment to which settlement is large enough to fully develop downdrag loading.

Per Ref. 7 Article 3.11.8, downdrag can be assumed to fully develop for the length of pile with settlement equal to or greater than 0.4 inches. As calculated in Ref. 3 - Attachment 10, settlement beneath Abutment 2 (Old Abutment 1) up to present-day (41 years after construction) due to construction of the widened bridge abutment and approach embankment is estimated to be greater than or equal to 0.4 inches down to an elevation of 93.5 feet (depth of 7.5 feet below the base of the abutment). Thus, downdrag loading is assumed to be fully developed for the upper 7.5 feet of the pile below the base of the abutment.

Per Ref. 7 Article 10.7.1.6.2 for piles end bearing on rock downdrag shall be considered at the strength and extreme limit state.

CALCULATIONS

Date: 11/1/2024
Project No.: 31404817.004
Subject: Existing Pile Downdrag at Abutment 2 (Old Abutment 1) - Pile with minimum settlement
Project Short Title: MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange

Made by: LMP
Checked by: DEB
Reviewed by: CCB

3. Use APile to estimate shaft resistance and total downdrag load.

Downdrag Load Factors:
(Ref. 7, Tables 3.4.1-1 and 3.4.1-2)

Soil Type	Strength I
Sand	1.05
Clay	1.40

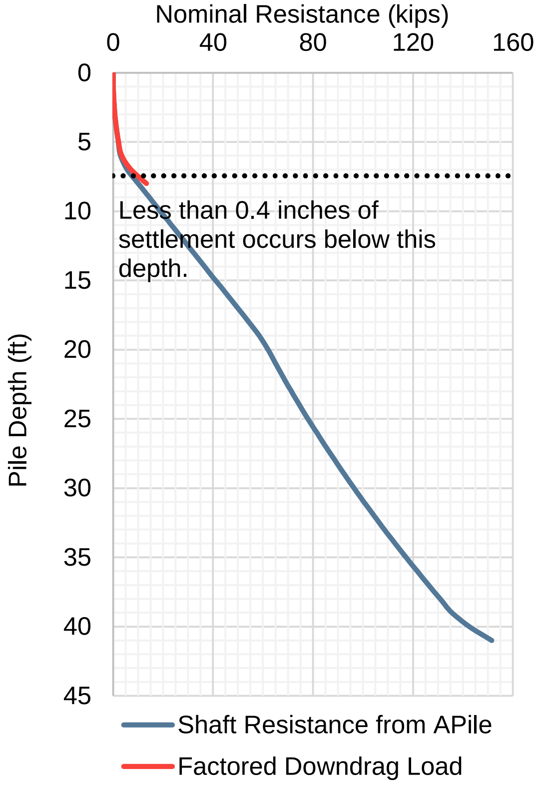
Abutment Loads:

	Strength I	
Per Pile	82.2	kips

(Ref. 8)

Shaft resistance and downdrag load for each foot of pile length:

Pile Depth below Base of Abutment (feet)	Soil Type from APile	Shaft Resistance from APile (kips)	Factored Downdrag Load (kips)	Total Factored Axial Load* (kips)
0	Sand	0.0	0.0	82.2
1	Sand	0.1	0.1	82.3
2	Sand	0.3	0.3	82.6
3	Sand	0.7	0.7	83.1
4	Sand	1.3	1.4	83.7
5	Sand	2.1	2.2	84.6
6	Clay	3.0	3.5	85.9
7	Clay	5.7	7.2	89.7
8	**Clay**	**10.1**	**13.4**	**95.9**
9	Clay	14.5		82.6
10	Clay	18.9		82.6
11	Clay	23.4		82.7
12	Clay	27.8		82.7
13	Clay	32.2		82.7
14	Clay	36.6		82.8
15	Clay	41.0		82.8
16	Clay	45.5		82.9
17	Clay	49.9		82.9
18	Clay	54.3		83.0
19	Clay	58.5		83.0
20	Sand	62.0		83.0
21	Sand	65.0		83.1
22	Sand	68.1		83.1
23	Sand	71.3		83.2
24	Sand	74.6		83.2
25	Sand	78.0		83.3
26	Sand	81.5		83.3
27	Sand	85.1		83.3
28	Sand	88.8		83.4
29	Sand	92.6		83.4
30	Sand	96.4		83.5
31	Sand	100.4		83.5



CALCULATIONS

Date: 11/1/2024
Made by: LMP
Project No.: 31404817.004
Checked by: DEB
Subject: Existing Pile Downdrag at Abutment 2 (Old Abutment 1) - Pile with minimum settlement
Reviewed by: CCB
Project Short Title: MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange

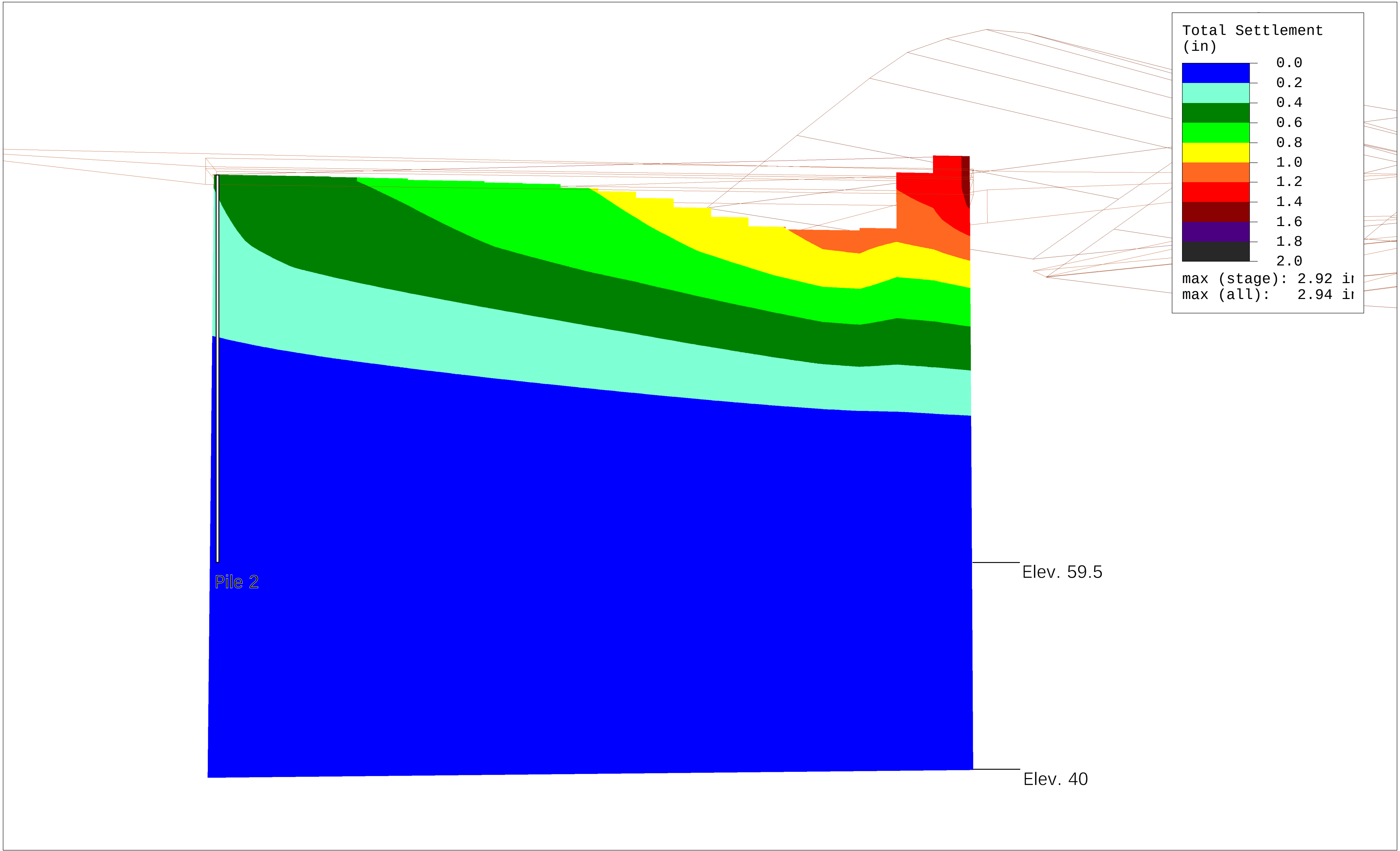
32	Sand	104.5		83.5
33	Sand	108.6		83.6
34	Sand	112.9		83.6
35	Sand	117.2		83.7
36	Sand	121.7		83.7
37	Sand	126.2		83.8
38	Sand	130.9		83.8
39	Sand	135.6		83.8
40	Sand	142.5		83.9
41	Sand	151.5		83.9

*Strength I, including Substructure Loading, Downdrag, and Pile Weight

Strength I factored downdrag load = 13.4 kips per pile

CONCLUSIONS

Based on the analysis of settlement between 1983 and present-day, downdrag is estimated to have developed along the upper 7.5 feet of pile 2 below the base of the Abutment 2 (Old Abutment 1). A factored downdrag load of 13.4 kips per pile for the Strength I limit state was calculated.



Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.004						
Analysis Description	Total Settlement along Abutment 2 (Old Abutment 1) (Sta. 113+91) at Present-Day (41 years after embankment widening)						
Drawn By	LMP	*Checked By*	KAR	*Reviewed By*	CCB	*Scale*	As shown
Date	11/1/2024	*File Name*	Hogan Rd Highway Interchange 1958 surface 1983Fill 10 24 -v3.s3z				

Attachment 1

SETTLE3 5.023

Attachment 2

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

                  APILE for Windows, Version 2019.9.11

                   Serial Number :  264781183

                   A Program for Analyzing the Axial Capacity
                   and Short-term Settlement of Driven Piles
                             under Axial Loading.
                 (c) Copyright ENSOFT, Inc., 1987-2019
                       All Rights Reserved

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


     This program is licensed to :

     WSP
     APILE Global, Global License

     Path to file locations      : C:\Users\USLP710461\WSP O365\MaineDOT Bangor
Hogan Road Bridge & DDI - Project Files\5 Technical Work\GDR Pile & Abutment
Design\Existing Bridge - Downdrag\APile Models\Abutement 2 Pile 2 - Min Settlement\
     Name of input data file     : HP10x42_Abutment-2 Pile 2MinS.ap9d
     Name of output file         : HP10x42_Abutment-2 Pile 2MinS.ap9o
     Name of plot output file    : HP10x42_Abutment-2 Pile 2MinS.ap9p

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

               Date:  October 25, 2024     Time:  17:03:31



1
          *********************
          * INPUT INFORMATION *
          *********************



          Existing Pile Downdrag at Existing Abutment 1

          DESIGNER   : LMP

          JOB NUMBER : 31404817.004


          METHOD FOR UNIT LOAD TRANSFERS :
```

```
- FHWA (Federal Highway Administration)
  Unfactored Unit Side Friction and Unit Side Resistance are used.



COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)


TYPE OF LOADING :
- COMPRESSION


PILE TYPE :

H-Pile/Steel Pile


DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY     = 0.290E+08 PSI
- CROSS SECTION AREA        =     12.40 IN2


NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL     =     41.50 FT.
- BATTER ANGLE              =      0.00 DEG
- PILE STICKUP LENGTH, PSL  =      0.00 FT.
- ZERO FRICTION LENGTH, ZFL =      0.00 FT.
- PERIMETER OF PILE         =     39.60 IN.
- TIP AREA OF PILE          =     12.40 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION       =      1.00 FT.



SOIL INFORMATIONS :



                  LATERAL    EFFECTIVE    FRICTION      Nq
          SOIL    EARTH      UNIT         ANGLE       FACTOR
 DEPTH    TYPE    PRESSURE   WEIGHT       DEGREES      FHWA
  FT.                        LB/FT^3
   0.00   SAND      0.80*      125.00        34.00     55.60**
   5.50   SAND      0.80*      125.00        34.00     55.60**
   5.50   CLAY      0.80*      122.00         0.00      4.80**
  13.90   CLAY      0.80*      122.00         0.00      4.80**
  13.90   CLAY      0.80*       59.60         0.00      4.80**
```

```
  18.40   CLAY      0.80*       59.60         0.00      4.80**
  18.40   CLAY      0.80*       58.60         0.00      4.80**
  18.60   CLAY      0.80*       58.60         0.00      4.80**
  18.60   SAND      0.80*       67.60        35.00     64.00**
  39.00   SAND      0.80*       67.60        35.00     64.00**
  39.00   SAND      0.80*       95.60        50.00    475.00**
  50.00   SAND      0.80*       95.60        50.00    475.00**

*  VALUE ASSUMED BY THE PROGRAM
** VALUE ESTIMATED BY THE PROGRAM BASED ON FRICTION ANGLE



MAXIMUM   MAXIMUM   UNDISTURB  REMOLDED
UNIT      UNIT      SHEAR      SHEAR     BLOW   UNIT SKIN  UNIT END
FRICTION  BEARING   STRENGTH   STRENGTH  COUNT  FRICTION   BEARING
  KSF       KSF        KSF       KSF               KSF       KSF
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.34      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.89      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.89      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
  WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
  PLAN TO LIMIT THE COMPUTED DATA.



         LRFD FACTOR  LRFD FACTOR
          ON UNIT      ON UNIT
 DEPTH    FRICTION     BEARING
  FT.
   0.00    1.000       1.000
   5.50    1.000       1.000
   5.50    1.000       1.000
  13.90    1.000       1.000
  13.90    1.000       1.000
  18.40    1.000       1.000
  18.40    1.000       1.000
  18.60    1.000       1.000
  18.60    1.000       1.000
  39.00    1.000       1.000
  39.00    1.000       1.000
```

```
            50.00    1.000       1.000



1
          **********************
          * COMPUTATION RESULT *
          **********************



                            **********************
                            *  FED. HWY. METHOD  *
                            **********************

             PILE          SKIN           END       ULTIMATE
          PENETRATION   FRICTION       BEARING     CAPACITY
              FT.           KIP           KIP          KIP
              0.00           0.0           0.2          0.2
              1.00           0.1           0.4          0.4
              2.00           0.3           0.8          1.1
              3.00           0.7           1.2          1.9
              4.00           1.3           1.6          2.9
              5.00           2.1           1.7          3.8
              6.00           3.0           1.6          4.5
              7.00           5.7           1.3          6.9
              8.00          10.1           1.0         11.1
              9.00          14.5           1.0         15.6
             10.00          18.9           1.0         20.0
             11.00          23.4           1.0         24.4
             12.00          27.8           1.0         28.8
             13.00          32.2           1.0         33.2
             14.00          36.6           1.0         37.7
             15.00          41.0           1.0         42.1
             16.00          45.5           1.0         46.5
             17.00          49.9           1.0         50.9
             18.00          54.3           2.2         56.6
             19.00          58.5           4.4         62.9
             20.00          62.0           6.7         68.7
             21.00          65.0           8.1         73.1
             22.00          68.1           8.3         76.5
             23.00          71.3           8.6         79.9
             24.00          74.6           8.8         83.5
             25.00          78.0           9.1         87.1
             26.00          81.5           9.2         90.7
             27.00          85.1           9.3         94.4
             28.00          88.8           9.3         98.1
             29.00          92.6           9.3        101.8
             30.00          96.4           9.3        105.7
             31.00         100.4           9.3        109.7
             32.00         104.5           9.3        113.7
```

```
     33.00          108.6            9.3        117.9
     34.00          112.9            9.3        122.2
     35.00          117.2            9.3        126.5
     36.00          121.7            9.3        131.0
     37.00          126.2            9.3        135.5
     38.00          130.9           19.2        150.1
     39.00          135.6           36.4        172.0
     40.00          142.5           53.6        196.1
     41.00          151.5           63.5        215.0


    NOTES:
    - AN ASTERISK IS PLACED IN THE END-BEARING COLUMN
      IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
      OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.



        *************************************************
        * COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
        * CURVES FOR AXIAL LOADING                      *
        *************************************************


T-Z CURVE   NO. OF   DEPTH TO CURVE   LOAD TRANSFER   PILE MOVEMENT
   NO.      POINTS        FT.              PSI              IN.

    1         10      0.4167E-01
                                       0.0000E+00      0.0000E+00
                                       0.4336E-02      0.2017E-01
                                       0.7226E-02      0.3908E-01
                                       0.1084E-01      0.7185E-01
                                       0.1301E-01      0.1008E+00
                                       0.1445E-01      0.1261E+00
                                       0.1445E-01      0.2521E+00
                                       0.1445E-01      0.3782E+00
                                       0.1445E-01      0.6303E+00
                                       0.1445E-01      0.2521E+01
    2         10      0.2750E+01
                                       0.0000E+00      0.0000E+00
                                       0.2862E+00      0.2017E-01
                                       0.4769E+00      0.3908E-01
                                       0.7154E+00      0.7185E-01
                                       0.8585E+00      0.1008E+00
                                       0.9539E+00      0.1261E+00
                                       0.9539E+00      0.2521E+00
                                       0.9539E+00      0.3782E+00
                                       0.9539E+00      0.6303E+00
                                       0.9539E+00      0.2521E+01
    3         10      0.5458E+01
                                       0.0000E+00      0.0000E+00
```

```
                                   0.5680E+00      0.2017E-01
                                   0.9466E+00      0.3908E-01
                                   0.1420E+01      0.7185E-01
                                   0.1704E+01      0.1008E+00
                                   0.1893E+01      0.1261E+00
                                   0.1893E+01      0.2521E+00
                                   0.1893E+01      0.3782E+00
                                   0.1893E+01      0.6303E+00
                                   0.1893E+01      0.2521E+01
4         10      0.5542E+01
                                   0.0000E+00      0.0000E+00
                                   0.5766E+00      0.2017E-01
                                   0.9611E+00      0.3908E-01
                                   0.1442E+01      0.7185E-01
                                   0.1730E+01      0.1008E+00
                                   0.1922E+01      0.1261E+00
                                   0.1730E+01      0.2521E+00
                                   0.1730E+01      0.3782E+00
                                   0.1730E+01      0.6303E+00
                                   0.1730E+01      0.2521E+01
5         10      0.9700E+01
                                   0.0000E+00      0.0000E+00
                                   0.2792E+01      0.2017E-01
                                   0.4653E+01      0.3908E-01
                                   0.6979E+01      0.7185E-01
                                   0.8375E+01      0.1008E+00
                                   0.9306E+01      0.1261E+00
                                   0.8375E+01      0.2521E+00
                                   0.8375E+01      0.3782E+00
                                   0.8375E+01      0.6303E+00
                                   0.8375E+01      0.2521E+01
6         10      0.1386E+02
                                   0.0000E+00      0.0000E+00
                                   0.2792E+01      0.2017E-01
                                   0.4653E+01      0.3908E-01
                                   0.6979E+01      0.7185E-01
                                   0.8375E+01      0.1008E+00
                                   0.9306E+01      0.1261E+00
                                   0.8375E+01      0.2521E+00
                                   0.8375E+01      0.3782E+00
                                   0.8375E+01      0.6303E+00
                                   0.8375E+01      0.2521E+01
7         10      0.1394E+02
                                   0.0000E+00      0.0000E+00
                                   0.2792E+01      0.2017E-01
                                   0.4653E+01      0.3908E-01
                                   0.6979E+01      0.7185E-01
                                   0.8375E+01      0.1008E+00
                                   0.9306E+01      0.1261E+00
                                   0.8375E+01      0.2521E+00
                                   0.8375E+01      0.3782E+00
```

```
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 8         10      0.1615E+02
                                    0.0000E+00      0.0000E+00
                                    0.2792E+01      0.2017E-01
                                    0.4653E+01      0.3908E-01
                                    0.6979E+01      0.7185E-01
                                    0.8375E+01      0.1008E+00
                                    0.9306E+01      0.1261E+00
                                    0.8375E+01      0.2521E+00
                                    0.8375E+01      0.3782E+00
                                    0.8375E+01      0.6303E+00
                                    0.8375E+01      0.2521E+01
 9         10      0.1836E+02
                                    0.0000E+00      0.0000E+00
                                    0.2698E+01      0.2017E-01
                                    0.4496E+01      0.3908E-01
                                    0.6745E+01      0.7185E-01
                                    0.8093E+01      0.1008E+00
                                    0.8993E+01      0.1261E+00
                                    0.8093E+01      0.2521E+00
                                    0.8093E+01      0.3782E+00
                                    0.8093E+01      0.6303E+00
                                    0.8093E+01      0.2521E+01
10         10      0.1844E+02
                                    0.0000E+00      0.0000E+00
                                    0.2676E+01      0.2017E-01
                                    0.4460E+01      0.3908E-01
                                    0.6690E+01      0.7185E-01
                                    0.8028E+01      0.1008E+00
                                    0.8920E+01      0.1261E+00
                                    0.8920E+01      0.2521E+00
                                    0.8920E+01      0.3782E+00
                                    0.8920E+01      0.6303E+00
                                    0.8920E+01      0.2521E+01
11         10      0.2870E+02
                                    0.0000E+00      0.0000E+00
                                    0.2397E+01      0.2017E-01
                                    0.3995E+01      0.3908E-01
                                    0.5993E+01      0.7185E-01
                                    0.7192E+01      0.1008E+00
                                    0.7991E+01      0.1261E+00
                                    0.7991E+01      0.2521E+00
                                    0.7991E+01      0.3782E+00
                                    0.7991E+01      0.6303E+00
                                    0.7991E+01      0.2521E+01
12         10      0.3896E+02
                                    0.0000E+00      0.0000E+00
                                    0.3018E+01      0.2017E-01
                                    0.5029E+01      0.3908E-01
                                    0.7544E+01      0.7185E-01
```

```
                                       0.9053E+01      0.1008E+00
                                       0.1006E+02      0.1261E+00
                                       0.1006E+02      0.2521E+00
                                       0.1006E+02      0.3782E+00
                                       0.1006E+02      0.6303E+00
                                       0.1006E+02      0.2521E+01
   13         10      0.3904E+02
                                       0.0000E+00      0.0000E+00
                                       0.3129E+01      0.2017E-01
                                       0.5215E+01      0.3908E-01
                                       0.7822E+01      0.7185E-01
                                       0.9386E+01      0.1008E+00
                                       0.1043E+02      0.1261E+00
                                       0.1043E+02      0.2521E+00
                                       0.1043E+02      0.3782E+00
                                       0.1043E+02      0.6303E+00
                                       0.1043E+02      0.2521E+01
   14         10      0.4450E+02
                                       0.0000E+00      0.0000E+00
                                       0.5781E+01      0.2017E-01
                                       0.9634E+01      0.3908E-01
                                       0.1445E+02      0.7185E-01
                                       0.1734E+02      0.1008E+00
                                       0.1927E+02      0.1261E+00
                                       0.1927E+02      0.2521E+00
                                       0.1927E+02      0.3782E+00
                                       0.1927E+02      0.6303E+00
                                       0.1927E+02      0.2521E+01
   15         10      0.4996E+02
                                       0.0000E+00      0.0000E+00
                                       0.5781E+01      0.2017E-01
                                       0.9634E+01      0.3908E-01
                                       0.1445E+02      0.7185E-01
                                       0.1734E+02      0.1008E+00
                                       0.1927E+02      0.1261E+00
                                       0.1927E+02      0.2521E+00
                                       0.1927E+02      0.3782E+00
                                       0.1927E+02      0.6303E+00
                                       0.1927E+02      0.2521E+01


TIP   LOAD        TIP MOVEMENT
    KIP               IN.

0.0000E+00         0.0000E+00
0.3970E+01         0.6303E-02
0.7939E+01         0.1261E-01
0.1588E+02         0.2521E-01
0.3176E+02         0.1639E+00
0.4764E+02         0.5294E+00
```

```
 0.5716E+02         0.9202E+00
 0.6352E+02         0.1261E+01
 0.6352E+02         0.1891E+01
 0.6352E+02         0.2521E+01


               LOAD VERSUS SETTLEMENT CURVE
               ****************************



 TOP  LOAD     TOP MOVEMENT       TIP  LOAD      TIP MOVEMENT
    KIP            IN.               KIP            IN.
0.5197E+00      0.4889E-03      0.6299E-01      0.1000E-03
0.5197E+01      0.4889E-02      0.6299E+00      0.1000E-02
0.2647E+02      0.2470E-01      0.3149E+01      0.5000E-02
0.5046E+02      0.4866E-01      0.6299E+01      0.1000E-01
0.8763E+02      0.9057E-01      0.1260E+02      0.2000E-01
0.1403E+03      0.1672E+00      0.1872E+02      0.5000E-01
0.1645E+03      0.2235E+00      0.2215E+02      0.8000E-01
0.1733E+03      0.2551E+00      0.2444E+02      0.1000E+00
0.1838E+03      0.3717E+00      0.3333E+02      0.2000E+00
0.1968E+03      0.6897E+00      0.4636E+02      0.5000E+00
0.2047E+03      0.1001E+01      0.5423E+02      0.8000E+00
0.2091E+03      0.1207E+01      0.5865E+02      0.1000E+01
0.2140E+03      0.2213E+01      0.6352E+02      0.2000E+01
```