Soils &
Engineering
Services, Inc.

April 10, 2019 Project 12939.3 R08

Mr. Jeff Hanson, Project Manager
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Subject: Geotechnical Exploration and Analyses Report
Structure B-13-732
(IH-39/90 Northbound Bridge over Femrite Drive)
Project ID 1007-10-02/05
Illinois State Line - Madison
USH-12/18 Interchange
IH-39
Dane County, Wisconsin

Dear Mr. Hanson

We have completed the requested exploration consisting of the performance of two standard soil borings at the subject site, and the associated laboratory testing and geotechnical engineering analyses. The purpose of these soil borings was to obtain information about the soil, bedrock, and groundwater conditions at the soil boring locations. We present our findings, comments, recommendations, and analyses results in the enclosed *Geotechnical Exploration and Analyses Report* for the subject bridge.

Respectfully submitted,

SOILS & ENGINEERING SERVICES, INC.

Craig M Bower

Craig M. Bower, P.E.

CMB:DER:cmb

Enclosure

GEOTECHNICAL EXPLORATION AND ANALYSES REPORT

STRUCTURE B-13-732
(IH-39/90 NORTHBOUND BRIDGE OVER FEMRITE DRIVE)
PROJECT ID 1007-10-02/05
ILLINOIS STATE LINE - MADISON
USH-12/18 INTERCHANGE
IH-39
DANE COUNTY, WISCONSIN

SES Project Number 12939.3

Prepared By

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Mr. Jeff Hanson, Project Manager

April 10, 2019

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I. INTRODUCTION

This *Geotechnical Exploration and Analyses Report* for Structure B-13-732 summarizes the findings of the geotechnical exploration, laboratory tests, and geotechnical engineering analyses performed for the design and construction of a new three-span IH-39/90 Northbound Bridge over Femrite Drive for the northern section of the IH-39/90 reconstruction project in the Town of Blooming Grove, Dane County, Wisconsin. We completed this work under the general direction of Dane Partners, LLC who established the general scope of the work.

The intent of this report is to convey the information obtained from the soil borings, to present the results of laboratory and field tests, to provide the results of our engineering analyses, and to present our comments and recommendations for the design and construction of the proposed structure. Variations in soil conditions should be expected between or beyond the borings, or between sample intervals, the nature and extent of which might not become evident until construction is undertaken. The project geotechnical engineer, or their designated representative, should make observations at the time of construction of the structure to determine if the subsurface conditions are as indicated by the exploration performed, and to validate our comments, analyses, and recommendations presented in this report for the subject structure.

II. PROJECT INFORMATION

The subject bridge described herein is a part of the planned widening and reconstruction of Interstate Highway 39 (IH-39) in Dane County. The planned IH-39 improvement consists of the design and construction of bridges, bridge widening, retaining walls, sign structures, sound barrier walls, roadway widening, and roadway reconstruction. The portion of the IH-39 improvement in Dane County is considered the North Segment for the proposed work. Separate reports address each of the improvements involved with this project.

The subject of this report is a new IH-39/90 Northbound Bridge over Femrite Drive bridge with Structure ID B-13-732. Ayres Associates, Inc. (AAI) provided preliminary drawings dated February 13 and March 12, 2019, for use in preparing this report. Please refer to enclosed Exhibits A through C in Appendix A which present the drawings provided by AAI that we utilized in our analyses.

Based on the drawings provided, the proposed three-span bridge will have a total length of approximately 149 feet and a total width of approximately 63 feet. Two abutments and two piers will provide foundation support for the three-span bridge structure. The piers will be located north and south of the edges of Femrite Drive. The southern and northern bridge spans are approximately 42 feet long and the center bridge span is approximately 60 feet long. New north and south approach embankments will be constructed diagonally across the open area between the existing Northbound and Southbound IH-39/90 roadway embankments.

III. FIELD EXPLORATION

We performed two standard soil borings, designated Borings SB-9 and SB-10 at or near the staked locations found in the field and as presented on the Location Sketches, Drawings 12939.3-8A and 12939.3-8B, enclosed in Appendix B. We adjusted the location of Boring SB-9 approximately 25 feet south of the staked location and the location of Boring SB-10 approximately 32 feet north of the staked location due to access restrictions.

We drilled and sampled borings for Structure B-13-732 to the following depths below ground surface and corresponding elevation:

Soil Boring	Substructure Unit	Ground Surface Elevation (feet)	Bottom of Boring	
			Depth (feet-inch)	Elevation (feet)
SB-9	South Abutment	881.2	48'-9"	832.5
SB-10	North Abutment	885.0	54'-3"	830.8

We used 2¼-inch-inside-diameter hollow-stem augers to maintain an open borehole as we advanced the borehole of Boring SB-9 to the last sample depth. We used 3¼-inch-inside-diameter hollow-stem augers to maintain an open borehole as we advanced the borehole of Boring SB-10 to the last sample depth. We added water to the inside of the HSA to counteract the water pressure present during the drilling of these borings.

As we advanced the boreholes of these borings, we obtained soil samples at 2½-foot intervals starting at a depth of 1-foot below the ground surface and continued to a depth of 10 feet. We increased the sampling interval to 5 feet from a depth of 10 feet to the boring termination depth. We performed this sampling using a 2-inch-outside-diameter split-barrel sampler according to AASHTO Designation T206. We visually identified the recovered soils in general compliance with the Unified Soil Classification System (USCS).

After obtaining the last split-barrel sample at Boring SB-10, we extended the borehole using rotary drilling and rock coring methods. We performed the rotary drilling using a tungsten carbide insert tri-cone roller bit. We performed the rock coring using a high-speed-rotary, double-tube, diamond-tipped NQ2 core barrel in hard bedrock. We used water to cool the tri-cone and core barrel bit and to flush the rock cuttings from the borehole. The rock core sample recovered was placed sequentially into a core box and transported to our laboratory. Our analysis of the recovered rock core is presented on Page 5 and we include a picture of the rock core in Appendix B.

We depict the subsoil stratification at the locations of the borings on the WisDOT Boring Logs enclosed in Appendix B. The WisDOT Boring Logs are drafted with a depth scale and the depth and respective elevation of each stratum change is denoted.

AAI staked the requested boring locations in the field. Due to drilling rig access restrictions and existing roadway embankments, and/or the presence of utility lines, we drilled Borings SB-9 and SB-10 at an offset from the requested locations. AAI determined the coordinates and ground surface elevation of each of the soil borings after completion of the drilling and sampling and provided this information to us. The "northing" and "easting" grid coordinates, stationing, and ground surface elevations at the locations of the borings are provided on the WisDOT Boring Logs.

IV. SOIL AND BEDROCK STRATIGRAPHY

The soil stratigraphy encountered at Borings SB-9 and SB-10 can generally be characterized as topsoil and fill material overlying native soil strata. Both of the borings encountered bedrock within the depths drilled.

We found the surficial soils at Boring SB-10 to be frozen on the day of the sampling as follows:

Boring	Ground Surface Elevation (feet)	Estimated Frost Top		Estimated Frost Bottom	
		Depth (feet-inch)	Elevation (feet)	Depth (feet-inch)	Elevation (feet)
SB-10	885.0	0'-0"	885.0	3'-0"	882.0

The borings encountered variable fill material and topsoil strata. We describe the fill material and topsoil strata encountered at the borings as follows:

- Boring SB-9 encountered 12 inches of very dark brown LEAN CLAY WITH SAND (CL) FILL TOPSOIL over 24 inches of brown fine to coarse SILTY SAND WITH GRAVEL (SM) FILL mixed with LEAN CLAY (CL) TOPSOIL and LEAN CLAY (CL) over 18 inches of dark gray SANDY SILT WITH GRAVEL (ML) FILL.

- Boring SB-10 encountered 8 inches of very dark brown LEAN CLAY WITH SAND (CL) FILL TOPSOIL over 28 inches of brown SANDY LEAN CLAY WITH GRAVEL

(CL) FILL over 5 feet of brown fine to coarse SILTY SAND WITH GRAVEL, COBBLES, AND BOULDERS (SM) FILL.

Below the topsoil and fill material, the borings encountered a variable native soil strata. We describe the native soil strata encountered at the borings as follows:

- Boring SB-9 encountered brown LEAN CLAY (CL) over brown fine to medium SILTY SAND WITH GRAVEL (SM) GLACIAL TILL over brown fine to medium SILTY SAND WITH GRAVEL AND COBBLES (SM) GLACIAL TILL over brown fine POORLY-GRADED SAND WITH GRAVEL (SP) over brown fine to medium SILTY SAND WITH GRAVEL (SM) GLACIAL TILL.

- Boring SB-10 encountered brown fine to medium POORLY-GRADED SAND (SP) over brown SILT (ML) over brown fine to medium SILTY SAND WITH GRAVEL AND COBBLES (SM) GLACIAL TILL.

We note that the gravel content varied from trace to some and the cobble content varied from none to few in the GLACIAL TILL strata.

Below the native soil stratification, Borings SB-9 and SB-10 encountered white to light brown slightly- to moderately-weathered SANDSTONE bedrock. We characterize the cored SANDSTONE bedrock to be white to light brown unweathered poorly-cemented SANDSTONE bedrock with cross bedding and annealed vertical fractures.

Please refer to the enclosed WisDOT Boring Logs for a further description of the topsoil, fill material, native soil strata, and bedrock encountered at the locations of Borings SB-9 and SB-10.

V. GROUNDWATER

Our drilling crew found the boreholes of the borings to have a water level and were caved at or after the completion of the drilling and sampling at the borings.

We summarize the water and caved level depths and respective elevations for the borings as follows:

Boring	Ground Surface Elevation (feet)	Water Level			Caved Level		
		Depth (feet-inch)	Elevation (feet)	Comments	Depth (feet-inch)	Elevation (feet)	Comments
SB-9	881.2	12'-7"	868.6	At completion	14'-3"	867.0	At completion

Boring	Ground Surface Elevation (feet)	Water Level			Caved Level		
		Depth (feet-inch)	Elevation (feet)	Comments	Depth (feet-inch)	Elevation (feet)	Comments
SB-10	885.0	17'-6"	867.5	At completion	20'-2"	864.8	At completion

We expect the groundwater level to fluctuate as influenced by precipitation, snowmelt, surface water runoff, and other hydrological and hydrogeological factors. The groundwater level at the time of construction of the subject bridge may be higher or lower than the groundwater levels encountered on the days that we performed the borings.

VI. LABORATORY AND FIELD TESTS

We performed laboratory tests on selected split-barrel soil samples and a portion of the bedrock core sample obtained from the borings to determine the physical properties of the fill material, native soil, and bedrock encountered at the boring locations. For the soil samples, the laboratory tests consisted of natural moisture content (MC), percentage of soil particles passing the No. 200-mesh sieve (Fines), and particle size distribution analysis. For the bedrock core sample, the laboratory tests consisted of density of rock (Wet Wt) and unconfined compressive strength of rock (r_u). In addition to the above tests, we tested some of the cohesive soils for approximate unconfined compressive strength (q_p) using a spring penetrometer.

The field test consisted of the standard penetration test (SPT). We performed the SPT during the sampling procedure at the boring locations. The SPT is the number of blows per foot ("N" value) needed to drive the split-barrel sampler using a 140-pound hammer free-falling for 30 inches. We determined the corrected standard penetration resistance {$(N_1)_{60}$-value} which is the N-value corrected for hammer efficiency and normalized to an effective overburden pressure at 1-atmosphere based on estimated densities for the soils encountered by the borings.

We include the laboratory and field test results obtained for this report on the WisDOT Boring Logs and/or Laboratory Test Result Record, Figure 1, enclosed in Appendix B. We used the results from the Fines and particle size distribution tests to confirm or modify the USCS soil identifications in general compliance with USCS classification procedures as defined in ASTM Designation D2487.

We observed and evaluated the core sample in our laboratory for rock identification, Core Recovery (CR), and Rock Quality Designation (RQD). The CR and RQD values are indexes of the quality of the rock mass. We show the CR and RQD values assigned to the rock core on the WisDOT Boring Log for Boring SB-10 and in the following table:

Boring Number	Ground Surface Elevation (feet)	Core Information					
		Depth (feet-inches)	Elevation (feet)	Length (feet)	CR (%)	RQD	
						(%)	Description
SB-10	885.0	49'-0" to 54'-3"	— to —	5.3	52	7	very poor

In summary, we performed a total of 5.3 feet of rock coring. Of this, 5.3 feet (100.0%) consisted of very poor quality rock. A picture of the rock core is included in Appendix B.

The laboratory and field test results for the borings suggest the following:

- The granular fill material is in a moist relative moisture condition. At Boring SB-9, the granular fill material is in a loose to medium dense state of relative density. At Boring SB-10, the granular fill material is in a loose to very dense state of relative density.

- The condition of the cohesive fill material is unknown as it was frozen at the time of our field work.

- The granular native soil strata are in a moist relative moisture condition to approximately elevation 864 feet below which they are in a wet relative moisture condition

- At Boring SB-9, the granular native soil strata are in a medium dense state of relative density with loose layers to approximately elevation 849.5 feet and in a dense state of relative density to approximately elevation 844 feet below which they are in a dense to very dense state of relative density.

- At Boring SB-10, the granular native soil strata are in a medium dense state of relative density to approximately elevation 873 feet and in a medium dense state of relative density with dense layers to approximately elevation 858 feet below which they are in a very dense state of relative density.

We used the laboratory and field test results in our evaluation of the topsoil and native soil strata encountered at the boring locations to determine soil parameters to use in the design of the foundation system for the subject IH-39/90 Northbound Bridge over Femrite Drive.

VII. ANALYSES PROCEDURES, ASSUMPTIONS, & GIVEN INFORMATION

Using the information obtained from Borings SB-9 and SB-10 performed in the vicinity of the abutments of the proposed bridge structure, we performed geotechnical analyses for shallow foundations and deep driven pile foundations for the substructure units of the proposed bridge using Load and Resistance Factor Design (LRFD) procedures presented in Chapter 11 of the WisDOT *Bridge Manual* (WBM) dated January 2019, and related chapters in the AASHTO *LRFD Bridge Design Specifications for Highway Bridges, 8th Edition*.

A. Shallow Spread Footing Foundations

Per Chapter 12 of the IH-39 Project Manual, the use of a driven pile foundation for the corridor structures is preferred unless shallow spread footing foundations are placed on bedrock or other high bearing capacity material. Therefore, shallow spread footing foundations for the abutments are not being considered since bedrock was not encountered by the borings performed.

B. Deep Driven Pile Foundations

1. Existing IH-39/90 Southbound Structure B-13-462

We reviewed the WisDOT *Site Investigation Report* dated August 30, 1995, and the pile driving data for the 1997 construction of the existing IH-39/90 Southbound Bridge (Structure B-13-462) over Femrite Drive. Our review of this information is as follows:

- The WisDOT *Site Investigation Report* contained the following information:
    - The 1995 borings encountered low to moderate strength granular soil strata over moderate to high strength granular soil strata. Bedrock was encountered within the depth explored by these borings. This is similar to the soil stratification encountered by Borings SB-9 and SB-10 for the current work.
    - Recommended 10x42 H-piles with a design bearing capacity of 55 tons or 110 kips. The report recommends a factor of safety of 2, which equates to a Rn_{dyn} of 110 tons or 220 kips for these piles
    - The recommended H-pile tip elevations for the substructure units were as follows:
        - North Abutment – 835 feet
        - Pier 1 (north) – 830 feet
        - Pier 2 (south) – 830 feet
        - South Abutment – 832 feet
    - The report states that dewatering for the construction of the piers for this bridge may be needed.
    - The report states that boring completed in 1960 in the vicinity of Pier 2 encountered boulders, but that the piles installed during the

construction of the previous bridge did not encounter any boulders during driving.

- The pile driving records for existing Southbound IH-39/90 Bridge (Structure B-13-462) over Femrite Drive shows that 10x42 H-piles were driven to support the substructure units. These records indicated the following:
  - The 10x42 H-piles were designed for a bearing capacity of 55 tons or 110 kips, which equates to a Rn_{dyn} of 110 tons or 220 kips using the recommended factor of safety of 2.
  - For the North Abutment:
    - The plan length was 50 feet.
    - The cut-off lengths ranged from 41.8 to 58.8 feet with an average of 49.0 feet.
    - The estimated tip elevations ranged from 841.0 to 834.0 feet with an average of 833.7 feet. This indicates the piles drove approximately 9 shallower to 8 feet deeper than anticipated.
    - The Rn_{dyn} ranged from 236 to 314 kips with an average of 262 kips.
  - For the Pier 1 (north):
    - The plan length was 40 feet.
    - The cut-off lengths ranged from 26.3 to 42.3 feet with an average of 34.0 feet.
    - The estimated tip elevations ranged from 840.0 to 824.0 feet with an average of 832.3 feet. This indicates the piles drove approximately 2 feet shallower to 14 feet deeper than anticipated.
    - The Rn_{dyn} ranged from 224 to 383 kips with an average of 275 kips.
  - For the Pier 2 (south):
    - The plan length was 40 feet.
    - The cut-off lengths ranged from 21.2 to 38.4 feet with an average of 29.9 feet.
    - The estimated tip elevations ranged from 844.7 to 827.5 feet with an average of 836.0 feet. This indicates the piles drove approximately 2 to 19 feet deeper than anticipated.
    - The Rn_{dyn} ranged from 235 to 335 kips with an average of 277 kips.
  - For the South Abutment:
    - The plan length was 50 feet.
    - The cut-off lengths ranged from 33.9 to 54.0 feet with an average of 44.8 feet.

- The estimated tip elevations ranged from 848.3 to 828.2 feet with an average of 837.4 feet. This indicates the piles drove approximately 4 feet shallower to 16 feet deeper than anticipated.
- The Rn_{dyn} ranged from 234 to 296 kips with an average of 258 kips.

2. Groundwater Elevations

Based on the results of the borings completed for the subject bridge structure for the North Segment of the IH-39 Reconstruction project, we recommend a design groundwater elevation of 868.0 feet be used for our pile driving and ultimate capacity computations.

3. Pile-Supported Footing Base Information

AAI provided the following pile-supported footing (pile cap) base and lowest ground surface elevations at the substructure units for the subject bridge structure. Where MSE retaining walls are constructed to retain the approach embankments at substructure units, the berm elevation used in our computations is the proposed ground surface elevation in front of the MSE retaining wall.

We note that the approach embankments for this structure will be constructed on the east side of the existing southbound roadway embankment. For our computations, we used the elevations for the proposed structure at the northbound reference line.

Substructure Unit	Nearest Boring(s)	Berm Elevation (feet)	Pile Cap Base Elevation (feet)
South Abutment	SB-9	883.6	881.1
Pier 1 (south)	SB-9	869.8	861.4
Pier 2 (north)	SB-10	869.8	861.5
North Abutment	SB-10	882.2	879.7

We anticipate the piles for the Piers and Abutments will be driven after the area in the vicinity of the substructure units has been graded.

4. Pre-boring

As presented in WBM Chapter 11.3.1.6, pre-boring is required for displacement (i.e, cast-in-place or CIP) piles driven into new embankments over 10 feet in height. Based on the plans provided, pre-boring is not required for the Piers and Abutments.

5. Allowable Driving Resistances

We used the computer program *APILE* to compute the ultimate skin friction and end bearing parameters and the ultimate and factored capacities of the soils encountered by the borings using static analyses.[1] We present the angle of internal friction (φ), unit weight (γ), and cohesion (c) values used in *APILE* in enclosed Table 8-1 in Appendix C and in Table A pages 11 through 14. The cohesion values are based on the approximate unconfined compressive strength readings obtained using a spring penetrometer and the laboratory unconfined compression strength test results. We estimated the density values based on: (1) wet density (γ_w) test results of selected representative samples of the various strata encountered by the borings performed; (2) the standard penetration results obtained by the borings performed; and, (3) our experience with similar material. We computed the angle of internal friction values using empirical formulas based on the $N1_{60}$-values for each soil stratum. As recommended in a 2013 WisDOT Research Program (WHRP) report written by Dr. James Long, we limited the computed maximum friction angle to 36 or 40 degrees depending upon the $N1_{60}$-values for computing skin friction and end bearing capacities for driven piles.[2] We used the percentage loss of soil strength during driving from Table 11.3-4 of the WBM based on the type of soil for each stratum for cast-in-place (CIP) pipe piles. Due to the reported over-driving of H-piles on other WisDOT bridge projects, we increase the percentage loss of soil strength during driving values from the WBM values for H-piles. We used the Static Nominal Axial Compression Resistance Factors (φ_{stat}) from Table 11.3-1 of the WBM based on the type of soil for each stratum. The percentage loss of soil strength and φ_{stat} factors used are included in enclosed Table 8-2 in Appendix C and in Tables A on pages 11 through 14.

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[1]APILE. Vers. 2014. Austin, TX: Ensoft, Inc., 2014. Computer software.

[2]James H. Long, PhD, PE, Improving Agreement Between Static Method and Dynamic Formula for Driven Cast-In-Place Piles in Wisconsin. (Madison, WI: Wisconsin Department of Transportation, 2013).

Table A: Recommended Soil Design Parameters And Foundation Capacities for WisDOT Structure B-13-732 for Recommended 10x42 Steel H-pile

Elevation (feet)	Material Type	Estimated Soil Parameters			Driven Pile Parameters			
		Moist Density, ϒ (pcf)	Angle of Internal Friction, Ø* (degrees)	Cohesion, c (psf)	Driving SSL** (%)	LRFD Factors***	Skin Friction† (psf)	End Bearing† (psf)
---------- Boring SB-9 located in the vicinity of South Abutment of Bridge Structure B-13-732 ----------								
883.6 to 881.2	Construct new embankment with WisDOT Embankment Fill	135.0	30	0	---	---	0	0
881.2 to 881.1	Replace existing dark brown LEAN CLAY WITH SAND (CL) FILL TOPSOIL with WisDOT Embankment Fill	135.0	30	0	---	---	0	0
881.1 to 880.2	Replace existing dark brown LEAN CLAY WITH SAND (CL) FILL TOPSOIL with WisDOT Embankment Fill	135.0	30	0	67	0.35	120	6,200
880.2 to 878.2	***SILTY SAND WITH GRAVEL (SM)*** *— fine to coarse grained, non-plastic to low plasticity fines, brown, moist, loose relative density,* ***FILL****, mixed with LEAN CLAY (CL) TOPSOIL and LEAN CLAY (CL)*	125.0	31	0	67	0.45	190	12,700
878.2 to 876.7	***SANDY SILT WITH GRAVEL (ML)*** *— non-plastic to low plasticity, dark gray, moist, medium dense relative density,* ***FILL***	125.0	31	0	67	0.45	280	23,800
876.7 to 875.7	***LEAN CLAY (CL)*** *— medium plasticity, grayish-brown, moist, hard consistency, occasional gravel*	132.0	0	4,000	83	0.35	4,000	33,300
875.7 to 869.2	***SILTY SAND WITH GRAVEL (SM)*** *— fine to medium grained, non-plastic to low plasticity fines, brown to reddish-brown, moist, medium dense relative density,* ***GLACIAL TILL****, trace to some gravel*	152.0	34	0	67	0.45	610	56,200
869.2 to 868.0	***SILTY SAND WITH GRAVEL AND COBBLES (SM)*** *— fine to medium grained, non-plastic to low plasticity fines, brown to reddish-brown, moist to wet, medium dense to loose relative density,* ***GLACIAL TILL****, trace to some gravel, few cobbles*	152.0	34	0	67	0.45	860	73,200

Table Notes

The Moist Density, Angle of Internal Friction, Cohesion, Driving SSL, and LRFD Static Resistance Factor values are considered estimates based on the soils encountered at the indicated boring.

* The computed Angle of Internal Friction, Ø, for each soil stratum or sub-stratum is presented. For driven pile computations, computed Øs greater than 36° are reduced to 36° or 40° depending upon the $N1_{60}$-value for the indicated stratum/sub-stratum. The reduced driven pile Øs are shown in brackets ([]). Please refer to geotechnical report for further explanation.

** The Driving SSL values presented in Table 11.3-4 of the WisDOT Bridge Manual were increased for the recommended driven steel H-piles based on feedback from pile driving contractors.

*** LRFD Static Resistance Factors and LRFD Load Factors are presented in this column. The LRFD Static Resistance Factors from the WisDOT Bridge Manual Table 11.3-1 or Table 11.3-8 are presented. For soil strata contributing to downdrag, LRFD Load Factors from the AASHTO LRFD Bridge Design Specifications Table 3.4.1-2 are presented in brackets ([]). Please see report for further definition of the LRFD Factors presented.

† The Skin Friction and End Bearing values presented above are the computed unfactored average capacities for the indicated stratum/substratum.

Table Abbreviations and Symbols

pcf = pounds per cubic foot.
SSL = soil strength loss.
psf = pounds per square foot.
NA = Not applicable.
Top = Cohesion @ top of stratum/sub-stratum.
Refusal = Steel pile will obtain capacity in indicated layer due to very dense soil or bedrock.
Bot = Cohesion @ bottom of stratum/sub-stratum.

Table A: Recommended Soil Design Parameters And Foundation Capacities for WisDOT Structure B-13-732 for Recommended 10x42 Steel H-pile

		Estimated Soil Parameters			Driven Pile Parameters			
Elevation (feet)	Material Type	Moist Density, γ (pcf)	Angle of Internal Friction, Ø* (degrees)	Cohesion, c (psf)	Driving SSL** (%)	LRFD Factors***	Skin Friction† (psf)	End Bearing† (psf)
868.0 to 859.2		152.0	34	0	67	0.45	1,040	70,900
859.2 to 854.2		152.0	30	0	67	0.45	920	24,400
854.2 to 849.2		152.0	35	0	67	0.45	1,580	105,000
849.2 to 844.2	***POORLY-GRADED SAND WITH GRAVEL (SP)*** — *fine grained, light brown and brown, wet, dense relative density*	125.0	38 [36]	0	50	0.45	1,930	148,000
844.2 to 837.7	***SILTY SAND WITH GRAVEL (SM)*** — *fine to medium grained, non-plastic to low plasticity fines, brown to reddish-brown, moist, very dense relative density,* ***GLACIAL TILL****, trace to some gravel*	152.0	42 [36]	0	67	0.45	2,150	175,000
837.7 to 832.5	***SANDSTONE*** — *slightly- to moderately-weathered, white and yellowish-brown, wet, very dense relative density*	137.0	45 [40]	0	0	0.45	NA	Refusal
	---------- End of Boring SB-9 @ Elevation 832.5 feet ----------							

Table Notes

The Moist Density, Angle of Internal Friction, Cohesion, Driving SSL, and LRFD Static Resistance Factor values are considered estimates based on the soils encountered at the indicated boring.

* The computed Angle of Internal Friction, Ø, for each soil stratum or sub-stratum is presented. For driven pile computations, computed Øs greater than 36° are reduced to 36° or 40° depending upon the $N1_{60}$-value for the indicated stratum/sub-stratum. The reduced driven pile Øs are shown in brackets ([]). Please refer to geotechnical report for further explanation.

** The Driving SSL values presented in Table 11.3-4 of the WisDOT Bridge Manual were increased for the recommended driven steel H-piles based on feedback from pile driving contractors.

*** LRFD Static Resistance Factors and LRFD Load Factors are presented in this column. The LRFD Static Resistance Factors from the WisDOT Bridge Manual Table 11.3-1 or Table 11.3-8 are presented. For soil strata contributing to downdrag, LRFD Load Factors from the AASHTO LRFD Bridge Design Specifications Table 3.4.1-2 are presented in brackets ([]). Please see report for further definition of the LRFD Factors presented.

† The Skin Friction and End Bearing values presented above are the computed unfactored average capacities for the indicated stratum/substratum.

Table Abbreviations and Symbols

pcf = pounds per cubic foot.
SSL = soil strength loss.
psf = pounds per square foot.
NA = Not applicable.
Top = Cohesion @ top of stratum/sub-stratum.
Refusal = Steel pile will obtain capacity in indicated layer due to very dense soil or bedrock.
Bot = Cohesion @ bottom of stratum/sub-stratum.

Table A: Recommended Soil Design Parameters And Foundation Capacities for WisDOT Structure B-13-732 for Recommended 10x42 Steel H-pile

		Estimated Soil Parameters			Driven Pile Parameters			
Elevation (feet)	Material Type	Moist Density, γ (pcf)	Angle of Internal Friction, Ø* (degrees)	Cohesion, c (psf)	Driving SSL** (%)	LRFD Factors***	Skin Friction† (psf)	End Bearing† (psf)
---------- Boring SB-10 located in the vicinity of North Abutment of Bridge Structure B-13-732 ----------								
885.0 to 884.3	***LEAN CLAY WITH SAND (CL)*** *— medium plasticity, very dark brown,* ***FILL TOPSOIL***	120.0	0	500	---	---	---	---
884.3 to 882.2	***SANDY LEAN CLAY WITH GRAVEL (CL)*** *— medium plasticity, brown,* ***FILL***	125.0	0	500	---	---	---	---
882.2 to 882.0		125.0	0	500	---	---	0	0
882.0 to 879.7	***SILTY SAND WITH GRAVEL, COBBLES, AND BOULDERS (SM)*** *— fine to coarse grained, non-plastic to low plasticity fines, brown, moist, loose to very dense relative density,* ***FILL***	125.0	31	0	---	---	0	0
879.7 to 879.5		125.0	31	0	67	0.45	260	33,000
879.5 to 877.0		125.0	45 [40]	0	33	0.45	360	52,000
877.0 to 875.5	***POORLY-GRADED SAND (SP)*** *— fine to medium grained, brown, moist, medium dense relative density*	125.0	36	0	50	0.45	360	50,500
875.5 to 873.0	***SILT (ML)*** *— non-plastic to low plasticity, dark grayish-brown, moist, medium dense relative density, few sand*	125.0	36	0	67	0.45	480	54,400
873.0 to 868.0	***SILTY SAND WITH GRAVEL AND COBBLES (SM)*** *— fine to medium grained, non-plastic to low plasticity fines, brown to reddish-brown, moist to wet, medium dense to very dense relative density,* ***GLACIAL TILL****, trace to some gravel, few cobbles*	152.0	38 [36]	0	67	0.45	740	83,600
868.0 to 863.0		152.0	41 [36]	0	67	0.45	1,040	103,000

Table Notes

The Moist Density, Angle of Internal Friction, Cohesion, Driving SSL, and LRFD Static Resistance Factor values are considered estimates based on the soils encountered at the indicated boring.

* The computed Angle of Internal Friction, Ø, for each soil stratum or sub-stratum is presented. For driven pile computations, computed Øs greater than 36° are reduced to 36° or 40° depending upon the $N1_{60}$-value for the indicated stratum/sub-stratum. The reduced driven pile Øs are shown in brackets ([]). Please refer to geotechnical report for further explanation.

** The Driving SSL values presented in Table 11.3-4 of the WisDOT Bridge Manual were increased for the recommended driven steel H-piles based on feedback from pile driving contractors.

*** LRFD Static Resistance Factors and LRFD Load Factors are presented in this column. The LRFD Static Resistance Factors from the WisDOT Bridge Manual Table 11.3-1 or Table 11.3-8 are presented. For soil strata contributing to downdrag, LRFD Load Factors from the AASHTO LRFD Bridge Design Specifications Table 3.4.1-2 are presented in brackets ([]). Please see report for further definition of the LRFD Factors presented.

† The Skin Friction and End Bearing values presented above are the computed unfactored average capacities for the indicated stratum/substratum.

Table Abbreviations and Symbols

pcf = pounds per cubic foot. psf = pounds per square foot. Top = Cohesion @ top of stratum/sub-stratum. Bot = Cohesion @ bottom of stratum/sub-stratum.
SSL = soil strength loss. NA = Not applicable. Refusal = Steel pile will obtain capacity in indicated layer due to very dense soil or bedrock.

Table A: Recommended Soil Design Parameters And Foundation Capacities for WisDOT Structure B-13-732 for Recommended 10x42 Steel H-pile

		Estimated Soil Parameters			Driven Pile Parameters			
Elevation (feet)	Material Type	Moist Density, ɤ (pcf)	Angle of Internal Friction, Ø* (degrees)	Cohesion, c (psf)	Driving SSL** (%)	LRFD Factors***	Skin Friction† (psf)	End Bearing† (psf)
863.0 to 858.0		152.0	31	0	67	0.45	820	32,700
858.0 to 848.0		152.0	44 [36]	0	67	0.45	1,560	157,000
848.0 to 842.0		152.0	45 [40]	0	17	0.45	2,730	431,000
842.0 to 836.0	***SANDSTONE*** *— slightly- to moderately-weathered, white to light yellowish-brown, wet, very dense relative density*	137.0	0	72,000 (Top) 144,000 (Bot)	0	0.45	NA	Refusal
836.0 to 830.8	***SANDSTONE*** *— unweathered, crossbedded, white to light brown, wet, layer RQD = 7%, poorly-cemented, annealed vertical fractures, no mineralizations or fossils*	137.0	0	144,000 (Top) 288,000 (Bot)	0	0.45	NA	NA
	--------- End of Boring SB-10 @ Elevation 830.8 feet ----------							

Table Notes

The Moist Density, Angle of Internal Friction, Cohesion, Driving SSL, and LRFD Static Resistance Factor values are considered estimates based on the soils encountered at the indicated boring.

* The computed Angle of Internal Friction, Ø, for each soil stratum or sub-stratum is presented. For driven pile computations, computed Øs greater than 36° are reduced to 36° or 40° depending upon the $N1_{60}$-value for the indicated stratum/sub-stratum. The reduced driven pile Øs are shown in brackets ([]). Please refer to geotechnical report for further explanation.

** The Driving SSL values presented in Table 11.3-4 of the WisDOT Bridge Manual were increased for the recommended driven steel H-piles based on feedback from pile driving contractors.

*** LRFD Static Resistance Factors and LRFD Load Factors are presented in this column. The LRFD Static Resistance Factors from the WisDOT Bridge Manual Table 11.3-1 or Table 11.3-8 are presented. For soil strata contributing to downdrag, LRFD Load Factors from the AASHTO LRFD Bridge Design Specifications Table 3.4.1-2 are presented in brackets ([]). Please see report for further definition of the LRFD Factors presented.

† The Skin Friction and End Bearing values presented above are the computed unfactored average capacities for the indicated stratum/substratum.

Table Abbreviations and Symbols

pcf = pounds per cubic foot.
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Top = Cohesion @ top of stratum/sub-stratum.
Bot = Cohesion @ bottom of stratum/sub-stratum.
SSL = soil strength loss.
NA = Not applicable.
Refusal = Steel pile will obtain capacity in indicated layer due to very dense soil or bedrock.

6. Design ‘Required Driving Resistance’ (Rn_{dyn}) and ‘Factored Axial Compression Resistance’ (P_r)

Based on the types of soils encountered by Borings SB-9 and SB-10, we evaluated driven 10¾-inch-outside-diameter, steel CIP-pipe-piles and driven 10x42 steel H-piles in our analyses of the deep foundations for the proposed substructure units for North and South Abutments and Piers 1 and 2. AAI requested that the minimum Rn_{dyn} for the CIP-pipe-piles be 130 tons or 260 kips.

The Design Rn_{dyn} for a selected pile type is determined by using the Design P_r for the selected pile and a dynamic resistance factor (φ_{dyn}) of 0.5 for driven piles where the modified Gates dynamic formula is planned to be used for construction driving criteria. The Design P_r and Rn_{dyn} values for the evaluated pile types, as presented in Table 11.3-5 of the WBM, are summarized as follows:

H-Pile Sizes	Maximum P_r (kips)	Maximum Rn_{dyn} (kips)
10x42	180	360

CIP-Pipe-Pile Sizes		Maximum P_r (kips)	Maximum Rn_{dyn} (kips)
Outside Diameter (inches)	Wall Thickness (inches)		
10¾	0.250	130	260
	0.365 or 0.500	150	300

Per Mr. Jeff Horsfall with WisDOT, all piles are driven to the Design Rn_{dyn} at the time of installation. For the subject bridge structure, the Design Rn_{dyn} equals the Maximum Rn_{dyn} for the selected CIP-pipe-piles or H-piles. Thus, we computed the Design Pile Tip Elevation for the selected CIP-pipe-pile or H-pile type based on the Design Rn_{dyn}, the static resistance factor, and the percentage loss of soil strength during driving for the soil strata encountered at each boring performed for the subject bridge structure.

7. Drivability Evaluation

We used the computer program *GRLWEAP* to perform the drivability evaluation for the selected H-piles and CIP-pipe-piles driven into the soils encountered by the borings.[3] We check drivability using the *APILE*-computed ultimate skin friction and ultimate end bearing parameters and the assigned percentage loss of soil strength for each soil stratum. We used *APILE* to create the preliminary data file for use in *GRLWEAP*.

We perform our drivability evaluation using the Delmag D16-32 and Delmag D30-32 diesel pile driving hammers. We initially determine the drivability of piles using the Delmag D16-32 pile driving hammer. If the specified pile is refused using the Delmag D16-32 above the design pile tip elevation, then we determine drivability of the specified pile using the Delmag D30-32 pile driving hammer. We define driven pile refusal as occurring when the estimated number of blows for a pile driving hammer as computed by *GRLWEAP* is equal to or greater than 120 blows per foot. The allowable steel stress is defined as 90 percent of the yield steel stress for a pile type. Per Mr. Jeff Horsfall with WisDOT, the steel stress at any time during driving should not exceed the allowable steel stress for a pile type. If the 'Driving Stress' (σ_{dr}) as computed by *GRLWEAP* for the specified piles exceeds 90 percent of the steel yield stress (35 ksi) of the shell of CIP-pipe-piles or of the steel yield stress (50 ksi) of H-piles, then the pile is overstressed and we make the following adjustments:

- For CIP-pipe-piles
  - Increase steel yield stress to 45 ksi.[4]
  - Increase shell thickness and restart the analyses process.
  - Decrease the Design Rn_{dyn} and restart the analyses process, OR switch to using H-piles.

- For H-Piles
  - Decrease the Design Rn_{dyn} and restart the analyses process.

We use the following default pile hammer cushion parameters included in *GRLWEAP* for the Delmag D16-32 and Delmag D30-32 diesel pile driving hammers equipped with a 21-inch lead size in our drivability evaluation.

***GRLWEAP* Default Pile Hammer Cushion Parameter**	**CIP-Pipe-Pile**	**H-Pile**
Cushion Material	50% Aluminum and 50% Conbest	50% Aluminum and 50% Conbest

[3]GRLWEAP. Vers. 2010. Cleveland, OH: GRL Engineers, Inc. *GRLWEAP*. 2010. Computer software.

[4]Per Mr. Jeff Horsfall with WisDOT, the use of 45 ksi steel for CIP-pipe-piles is acceptable to WisDOT.

GRLWEAP Default Pile Hammer Cushion Parameter	CIP-Pipe-Pile	H-Pile
Area (inches^2)	227	227
Elastic Modulus (ksi)	530	530
Coefficient of Restitution	0.8	0.8
Helmet Weight (kips)	1.7	1.9
Thickness (inches)	2.0	2.0

8. Downdrag Load

Per WBM Chapter 11.3.1.17.1, downdrag load, or negative shaft resistance, occurs due to settlement of the embankment caused by the additional loading imposed on the existing soils from the raising of the grade at a proposed substructure unit. WisDOT states that factored downdrag loads of 126, 168, and 294 kips are allowed in addition to the Design P_r for 10x42, 12x53, and 14x73 H-piles, respectively, driven to refusal on relatively-sound bedrock using modified Gates driving criteria to determine the Driving Rn_{dyn}. Downdrag loads on displacement piles (e.g., CIP piles) or non-end bearing H-piles reduce the Design P_r of the pile unless countermeasures are undertaken to minimize the downdrag condition.

WisDOT specifies that the provisions specified by AASHTO for downdrag should be used to compute downdrag loads. AASHTO specifies that downdrag loads should be applied if the settlement between the soil and the installed pile is 0.4 inches or greater. Soil settlement below the installed pile is ignored in these computations. We used the preliminary cross-section drawings provided by AAI to determine the maximum height of fill to be placed at each substructure unit. We use the procedures and equations presented in AASHTO Section 10.6.2.4 to estimate settlement between the soil and the installed pile to the computed maximum depth of each pile type/size analyzed.

After determining which soil strata could cause downdrag, we apply the AASHTO specified downdrag (DD) load factor to the computed unfactored skin friction load for the affected soil strata. For the driven portion of a pile, we apply a 1.4 DD load factor to the *APILE*-computed unfactored skin friction load. For the prebored portion of a pile or where embankments are constructed after piles are driven, we apply a 1.25 DD load factor to the unfactored skin friction load computed using AASHTO specified drilled shaft analyses procedures.

C. Lateral Deflection Analyses

We understand that the proposed abutments will be Type A1 (sill) abutments. We understand no lateral force analyses are required for the abutments.

VIII. ANALYSES RESULTS

A. Shallow Spread Footing Foundation Analyses

We did not perform shallow spread footing foundation analyses for the abutments or piers for the subject bridge structure for the reasons presented above in Section VII.A.

B. Deep Driven Pile Foundation Analyses

We performed the static and drivability analyses using the AASHTO and WisDOT procedures and the given and estimated information as outlined above in Section VII.B. We used *APILE* to complete the static analyses and *GRLWEAP* to complete the drivability analyses for the deep driven pile foundation for the abutments and pier for the subject bridge structure. Enclosed Table 8-2 in Appendix C and Table A on pages 11 through 14 present the <u>average</u> unfactored end bearing and <u>average</u> unfactored skin friction without soil driving loss values derived from the *APILE* static computations for the soil strata for each of the borings. Enclosed Table 8-3 in Appendix C presents the static and drivability results including the Design Pile Tip Elevations; Design, Driving, and Ultimate Rn_{dyn}; Design and Factored P_r; and estimated Maximum σ_{dr} from the *APILE* and *GRLWEAP* computations for each of the borings. Using static analyses, we used the Design Rn_{dyn} for steel CIP-pipe-piles or steel H-piles and the estimated soil driving loss for each stratum to determine the design pile tip elevation as presented in Table 8-3 in Appendix C. We used a dynamic resistance factor of 0.5 to compute the Design P_r presented in Table 8-3. Using static analyses, we computed the Factored P_r presented in Table 8-3 using the static resistance factors presented in Table 8-1 in Appendix B. Where the computed Factored P_r is higher than the Design $P_{r,}$, we recommend the Design P_r be used in designing the pile foundation layout.

The analyses' results indicate that the use of driven steel CIP-pipe-piles or steel H-piles would be acceptable to support the substructure units for the South Abutment, Pier 1, Pier 2, and North Abutment for the proposed bridge structure. Our analyses indicate that thicker wall CIP-pipe-piles with an increased steel yield stress should be used due to the presence of the very dense GLACIAL TILL soil strata and the SANDSTONE bedrock. Steel H-piles for this bridge structure would be functioning as end-bearing piles terminating in the very dense slightly- to moderately-weathered

SANDSTONE bedrock to unweathered poorly-cemented SANDSTONE bedrock. Based on the presence of bedrock in close proximity to the computed pile tip elevations for the CIP-piles, we do not recommend that CIP-piles be considered for supporting the substructure units for the subject bridge structure due to the potential for the CIP-piles to be damaged during installation.

Therefore, we recommend 10x42 steel, H-piles provide foundation support for the South Abutment, Pier 1, Pier 2, and North Abutment substructure units. Please refer to Table B on page 20 for our recommended Design Rn_{dyn}, Design P_r, Design Pile Tip Elevation, Driving Rn_{dyn}, and other pertinent design values for each substructure unit. Our drivability analyses indicate a Delmag D16-32 or similar pile driving hammer should be able to drive the recommended steel H-piles.

We computed a settlement of approximately 1¼ inches between the existing soil strata and the total length of an installed pile due to the construction of the south approach embankment. Per AASHTO, downdrag load should be considered when computed settlements between existing soil strata and the total length of an installed pile are greater than or equal to 0.4 inches. We compute elevation 869.2 feet to be the elevation below which the estimated settlement between the soil and the installed pile will be less than 0.4 inches. The soil strata above these elevations contribute to downdrag loading of the installed pile.

Due to the minor changes in the ground surface elevation at the Pier 1, Pier 2, and North Abutment substructure units, downdrag load due to settlement between the existing soil strata and the total length of installed pile is estimated to less than 0.4 inches.

Based on the computed settlement between the soil and the total length of installed pile due to the south approach embankment and per AASHTO and the WBM, we computed the Factored Downdrag Load presented in Table B due to soils above the elevation 869.2 feet at Boring SB-9. This Factored Downdrag Load is less than the WisDOT maximum allowed additional load of 126 kips for the recommended 10x42 steel, H-piles. Therefore, no adjustment of the design capacity for the recommended H-piles is required.

Some of the newer bridges along the I-39 Corridor were constructed with either H-piles or CIP piles. We were informed that the H-piles are typically driving deeper than the computed depths in soils and the CIP piles are typically reaching capacity at shallower than computed depths, although some piles are driving deeper and shallower within the same substructure. For bridge Structure B-53-323 (Avalon Road), H-piles for the abutments drove significantly deeper than computed and CIP were driven in the median to depths 3 to 6 feet shallower than computed.

Table B: Recommended Driven Steel H-Pile Information for Structure B-13-732‡

Substructure Unit	South Abutment	Pier 1 (south)	Pier 2 (north)	North Abutment
Nearest Boring	SB-9	SB-9	SB-10	SB-10
Pile Cap Base Elevation (feet)	881.1	861.4	861.5	879.7
H-pile Size	10x42	10x42	10x42	10x42
H-pile Steel Yield Stress (ksi)	50	50	50	50
Design Rn_{dyn} (kips)	360	360	360	360
Design P_r (kips)	180	180	180	180
Factored Downdrag Load (kips)*	41	0	0	0
Design Pile Tip Elevation (feet)	836.6	838.5	840.8	841.0
Computed Pile Length (feet)	44.5	22.9	20.7	38.7
Recommended Plan Pile Length (feet)†	47	24	22	41
Driving Rn_{dyn} (kips)	360	360	360	360
Hammer Type	Delmag D16-32	Delmag D16-32	Delmag D16-32	Delmag D16-32

‡ See attached Table 8-3 in Appendix B for additional driven pile drivability results.
† Please refer to page 20 for an explanation of the recommended plan pile lengths.
* Per WBM 11.3.1.17.1, the Factored Downdrag Loads do not reduce the Design P_r for the recommended 12x42 H-piles.

Based on the indicated variability in the thickness and type of soil overburden and the variability in the density of the bedrock, we recommend an average of approximately 5 percent be added to the computed pile lengths as presented in Table B on page 20 for the recommended 10x42 steel, H-piles installed at the substructure units for the South and North Abutments and Piers 1 and 2 for the subject bridge structure to allow for the anticipated soil overburden and bedrock variability.

Borings SB-9 and SB-10 did encounter cobbles in the GLACIAL TILL strata. The borings did not encounter any boulders in the GLACIAL TILL strata. It is known that GLACIAL TILL soils can contain a variable quantity of these large particles and that these large particles can be erratically located within the GLACIAL TILL soil matrix. Therefore, we recommend pile points be used for the H-piles due to the potential to encounter cobbles and/or boulders within the very dense GLACIAL TILL soils. The pile points help to evenly distribute the driving stresses to the steel of these piles which will help to reduce the potential of damage to the piles during driving.

C. Lateral Deflection Analyses

We did not perform lateral deflection analyses for any of the substructure units for the subject bridge structure.

IX. BRIDGE CONSTRUCTION

We offer the following comments and recommendations regarding the construction of the three-span IH-39/90 Northbound Bridge over Femrite Drive for the North Segment of the IH-39 Reconstruction Project.

A. Shallow Foundations

Shallow spread footing foundations are not being considered for the abutments and piers for the subject bridge structure for the reasons presented above in Section VII.A.

B. Deep Foundations

We recommend a deep foundation system consisting of 10x42 steel, H-piles driven in accordance with the WisDOT *Standard Specifications for Highway and Structure Construction* and relevant special provisions to support the proposed substructure units for the South and North Abutments for the subject bridge reconstruction.

Using the proposed pile cap base elevations as presented on Exhibit A and the recommended plan pile lengths as presented in Table B on page 20, we estimate the recommended steel H-piles will achieve the Design Rn_{dyn} of 300 kips in very dense SANDSTONE bedrock in the vicinity of the following estimated pile tip elevations:

Substructure Unit	Nearest Boring(s)	Estimated Pile Tip Elevation (feet)
South Abutment	SB-9	834
Pier 1 (south)	SB-9	837
Pier 2 (north)	SB-10	839
North Abutment	SB-10	838

Please refer to Table B on page 20 for a summary of the analyses results for the recommended steel H-piles. Additional static and drivability analyses' results information is presented in Tables 8-2 and 8-3 in Appendix C.

The final driven length of the steel H-piles will depend on: (1) the variability of the soil overburden thickness, (2) the variability of the soil overburden density, and (3) the selected pile hammer type. Therefore, variations in the installed length of steel H-piles should be anticipated.

C. General Comments and Recommendations

We recommend that all work associated with the selected foundation system at the project site be performed in accordance with the WisDOT *Standard Specifications for Highway and Structure Construction* and relevant special provisions.

Slope protection should be provided in the vicinity of and around all substructure foundation units as required by WisDOT standards to reduce the potential for erosion.

X. CLOSING COMMENTS

Soils & Engineering Services, Inc. prepared this report for the exclusive use of Dane Partners, LLC and the Wisconsin Department of Transportation to aid in the design of the proposed IH-39/90 Northbound Bridge over Femrite Drive for the North Segment of the IH-39 Reconstruction Project in the Town of Blooming Grove, in Dane County, Wisconsin. The recommendations in this report are based on the project information provided to our office. Soils & Engineering Services, Inc. should review any changes in the nature, design, or location of the proposed bridge after submittal of this *Geotechnical Exploration and Analyses Report* for Structure B-13-732 to revise the recommendations in the report, if necessary. The nature and extent of soil or groundwater variations between the boring locations may not become evident until the time of excavation or construction of the subject bridge. If soil or groundwater variations are evident at the time of excavation or construction, it will be necessary for Soils & Engineering Services, Inc. to re-evaluate the soil and groundwater, and other site conditions, which may result in the revision of our recommendations in this report.

Soils & Engineering Services, Inc. should review the final design and specification documents for this project to verify that our recommendations regarding the foundation system are interpreted correctly and implemented in the design of the subject bridge as they are intended. It is further recommended that the project geotechnical engineer be present at the time of pile installation for the proposed bridge structure to observe compliance with the design concept and specifications, and to provide recommendations to modify the design if subsurface conditions differ from those anticipated prior to construction. It is important that the pile load capacity, soil strength, bedrock integrity, degree of compaction, and other soil properties required be confirmed and/or determined at the time of excavation and construction activities for the subject bridge.

The recommendations provided in this report are based on our identification/classification and interpretation of the soils and information given on the WisDOT Boring Logs, and may not be based solely on the contents of the driller's field logs.

Soils & Engineering Services, Inc. prepared this report for the subject bridge in accordance with generally accepted geotechnical engineering practices at this time. Soils & Engineering Services, Inc. offers no other expressed or implied warranty.

Soils & Engineering Services, Inc. will store the soil samples obtained from the soil borings performed for this project for a period of 60 calendar days after the date of this report. Please advise us if we should extend this period.

We recommend that this *Geotechnical Exploration and Analyses Report*, in its entirety, for Proposed Structure B-13-732 be made available to bidding contractors or subcontractors for information purposes. The Appendices, WisDOT Boring Logs, and other attachments referenced in this report should not be separated from the text of this report. This report should be considered invalid if used for purposes other than those described herein.

Safety precautions, such as those required by OSHA and the Wisconsin Department of Safety and Professional Services, should be followed throughout the entire construction of the proposed project. They include, but are not limited to, the proper sloping and/or support of excavation sidewalls and adjacent embankments, roadways, sidewalks, railroad lines, utility lines, and/or buildings.

Soils & Engineering Services, Inc. respectfully submits this *Geotechnical Exploration and Analyses Report*, dated April 10, 2019, for Proposed Structure B-13-732 to **Dane Partners, LLC** and the **Wisconsin Department of Transportation**.

Appendix A

B-13-732 Preliminary Plan Sheet 1 of 4, Exhibit A
60% Plan Cross-Section Sheet PRE-98, Exhibit B
60% Plan Cross-Section Sheet PRE-99, Exhibit C

STATE PROJECT NUMBER

1007-12-78

DESIGN DATA

LIVE LOAD:

DESIGN LOADING: HL-93
INVENTORY RATING FACTOR:
OPERATING RATING FACTOR:
WISCONSIN STANDARD PERMIT VEHICLE (WIS-SPV) = KIPS

STRUCTURE IS DESIGNED FOR A FUTURE WEARING SURFACE OF 20 #/S.F.

RATING BASED ON PLACEMENT OF 5 p.s.f. POLYMER OVERLAY UNDER THIS CONTRACT.

MATERIAL PROPERTIES:

CONCRETE MASONRY	SUPERSTRUCTURE (HPC DECK, PARAPET, DIAPHRAGM, APPROACH SLAB)	f'_c =	4,000 p.s.i.
	ALL OTHER (INCL. APPROACH SLAB FOOTING)	f'_c =	3,500 p.s.i.
HIGH STRENGTH BAR STEEL REINFORCEMENT (GRADE 60)		f_y =	60,000 p.s.i.
STRUCTURAL CARBON STEEL ASTM A709 (GRADE 60)		f_y =	36,000 p.s.i.
36W" PRESTRESSED GIRDER			
CONCRETE MASONRY		f'_c =	8,000 p.s.i.
STRANDS - 0.6" DIA. WITH ULTIMATE TENSILE STRENGTH OF		=	270,000 p.s.i.

FOUNDATION DATA:

SOUTH ABUTMENT TO BE SUPPORTED ON HP 10 x 42 STEEL PILING DRIVEN TO A REQUIRED DRIVING RESISTANCE OF 180 TONS ‡ PER PILE AS DETERMINED BY THE MODIFIED GATES DYNAMIC FORMULA. ESTIMATED LENGTH 45'-0".

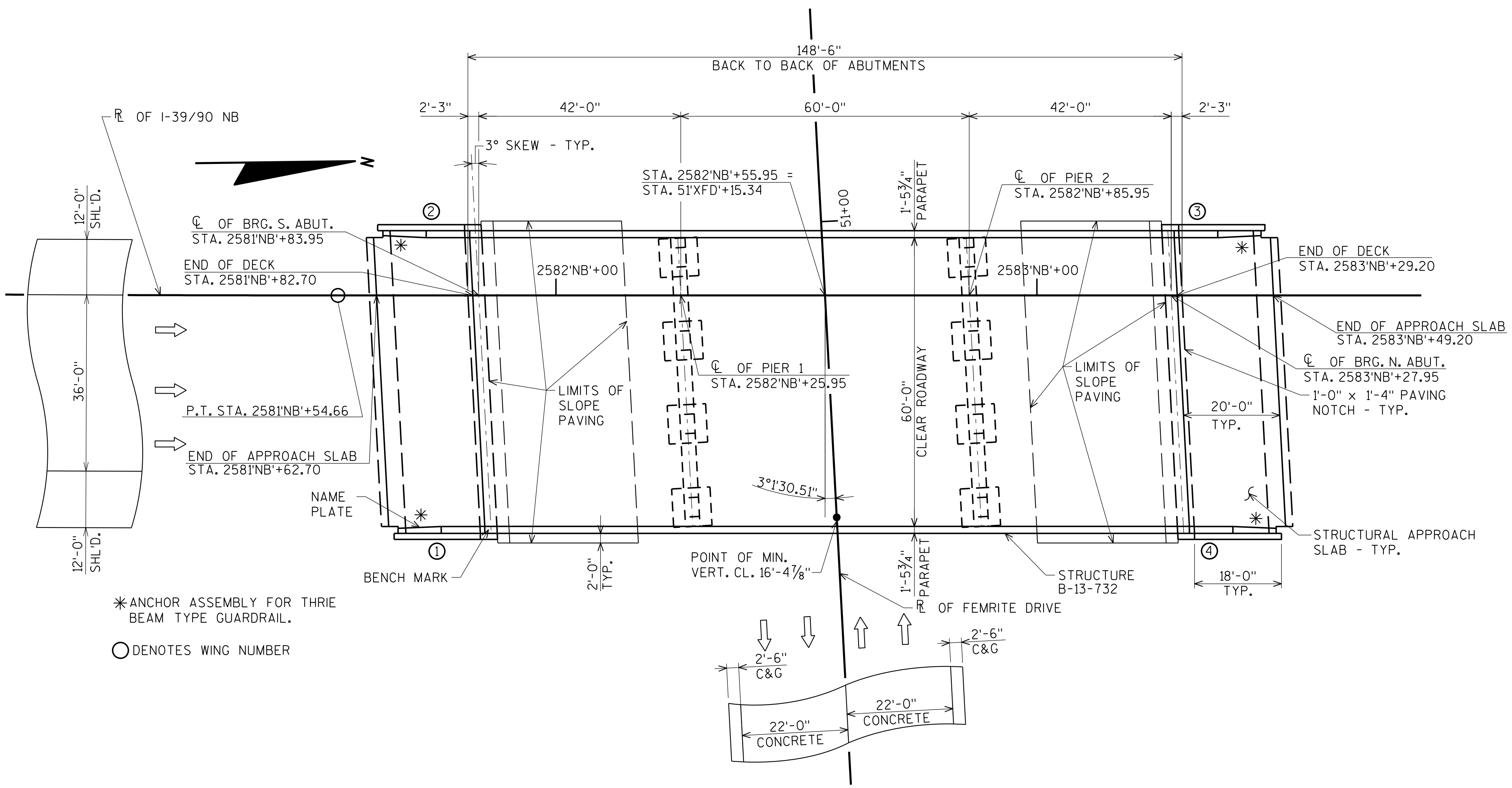
PIERS TO BE SUPPORTED ON HP 10 x 42 STEEL PILING DRIVEN TO A REQUIRED DRIVING RESISTANCE OF 180 TONS ‡ PER PILE AS DETERMINED BY THE MODIFIED GATES DYNAMIC FORMULA. ESTIMATED LENGTH 30'-0" AT PIER 1 AND AT PIER 2.

NORTH ABUTMENT TO BE SUPPORTED ON HP 10 x 42 STEEL PILING DRIVEN TO A REQUIRED DRIVING RESISTANCE OF 180 TONS ‡ PER PILE AS DETERMINED BY THE MODIFIED GATES DYNAMIC FORMULA. ESTIMATED LENGTH 45'-0".

‡ THE FACTORED AXIAL RESISTANCE OF PILES IN COMPRESSION USED FOR DESIGN IS THE REQUIRED DRIVING RESISTANCE MULTIPLIED BY A RESISTANCE FACTOR OF 0.5 USING MODIFIED GATES TO DETERMINE DRIVEN PILE CAPACITY.

TRAFFIC DATA:

I-39/90 NB	FEMRITE DRIVE
A.A.D.T. = 60,400 (2009)	A.A.D.T. = 3,800 (2020)
A.A.D.T. = 72,000 (2040)	A.A.D.T. = 9,800 (2040)
R.D.S. = 70 M.P.H.	R.D.S. = 40 M.P.H.

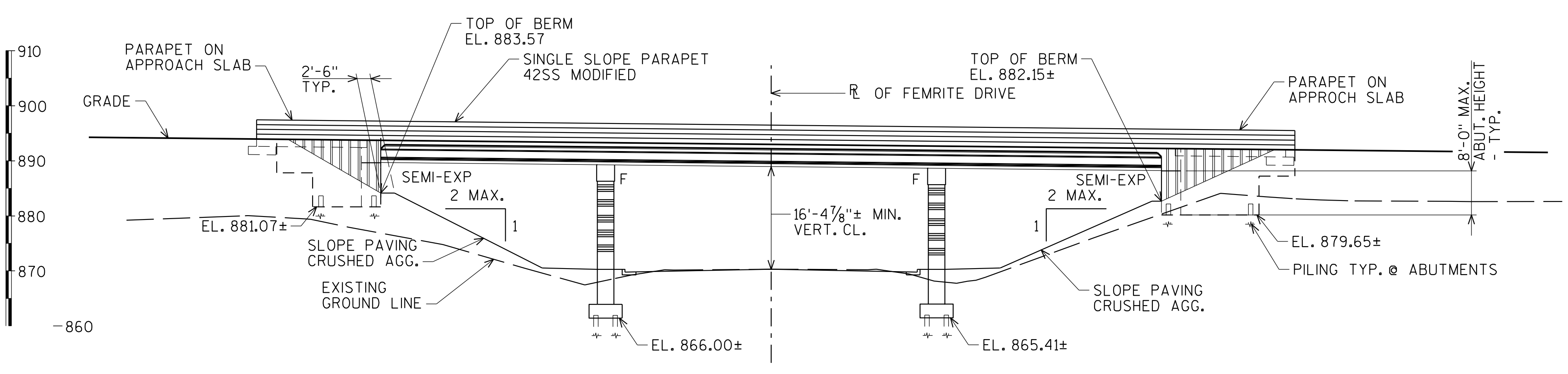


PLAN

THREE SPAN - 36W" PRESTRESSED CONCRETE GIRDER BRIDGE

FOR TYPICAL SECTION SEE SHEET 2

EXHIBIT A



ELEVATION

(NORMAL TO ℄ OF FEMRITE DRIVE)

LIST OF DRAWINGS

1. PRELIMIARY PLAN
2. TYPICAL SECTIONS AND NOTES
3. DETAILS AND QUANTITIES
4. SUBSURFACE EXPLORATION

BRIDGE OFFICE CONTACT:
WILLIAM DREHER
(608)-266-8489

CONSULTANT CONTACT:
CHRIS MCMAHON
(715)-834-3161

NO.	DATE	REVISION	BY

ORIGINAL PLANS PREPARED BY
AYRES ASSOCIATES
3433 Oakwood Hills Parkway
Eau Claire, WI 54701
www.AyresAssociates.com

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION

ACCEPTED ______ CHIEF STRUCTURES DESIGN ENGINEER ______ DATE

STRUCTURE B-13-732

I-39/90 NB OVER FEMRITE DRIVE

COUNTY DANE | TOWN/~~CITY/VILLAGE~~ BLOOMING GROVE

DESIGN SPEC. AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

DESIGNED BY CBM | DESIGN CK'D. | DRAWN BY JLB | PLANS CK'D.

PRELIMINARY PLAN

SHEET 1 OF 4

8

8

$PRFNAME$
U:±41-0660.0X - IH-39 Dane Partners General Bridge Information±BC Struct I39_NB over Femrite±pre

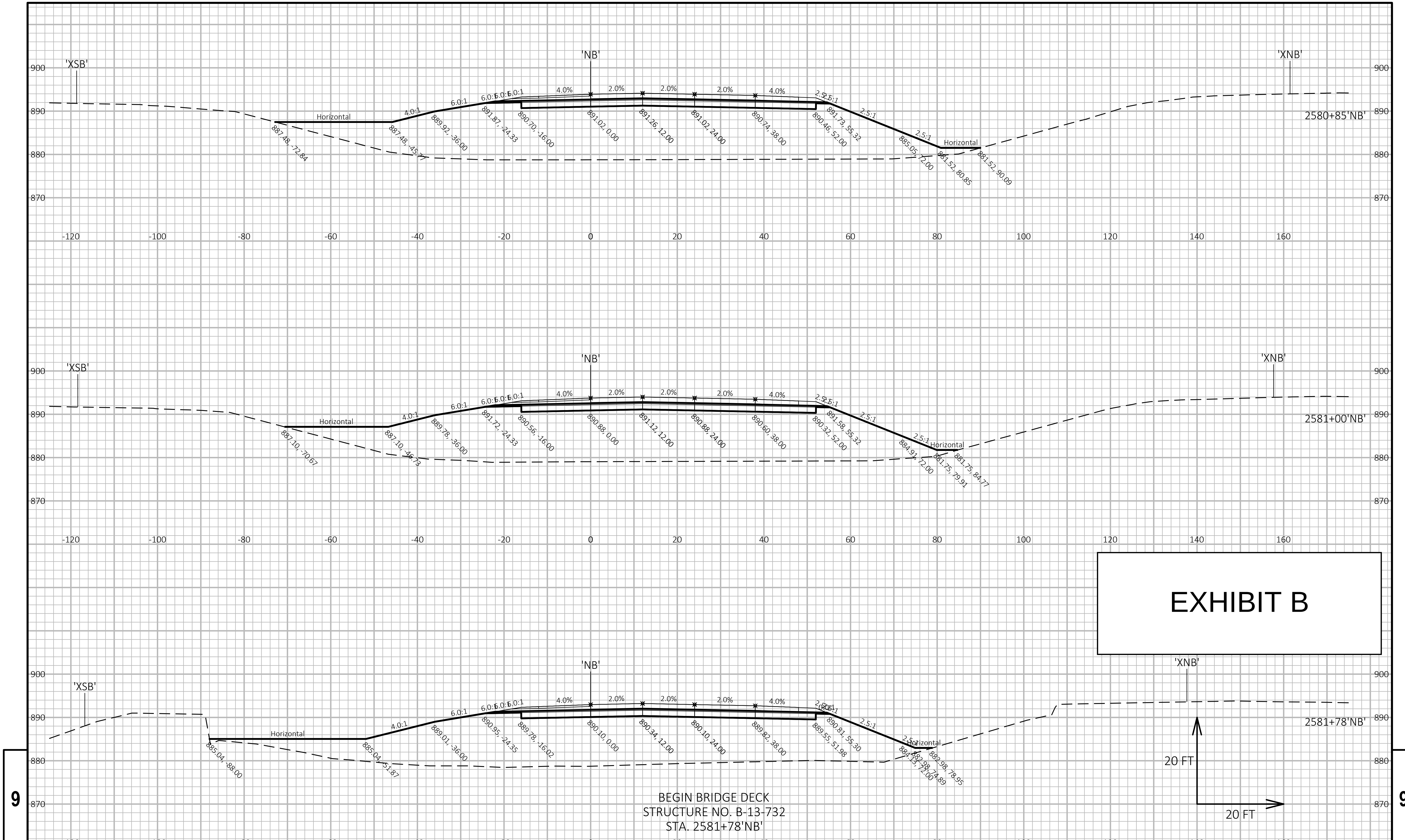
CHECKED BY: DATE:
BACK CHECKED BY: DATE:
CORRECTED BY: DATE:

3/12/2019

PENTABLE:Wisdot_shd.tbl

I.D.

DATE:



PROJECT NO: 1007-12-78	HWY: IH 39	COUNTY: DANE	CROSS SECTIONS: IH 39 NB	SHEET E

PRE_98

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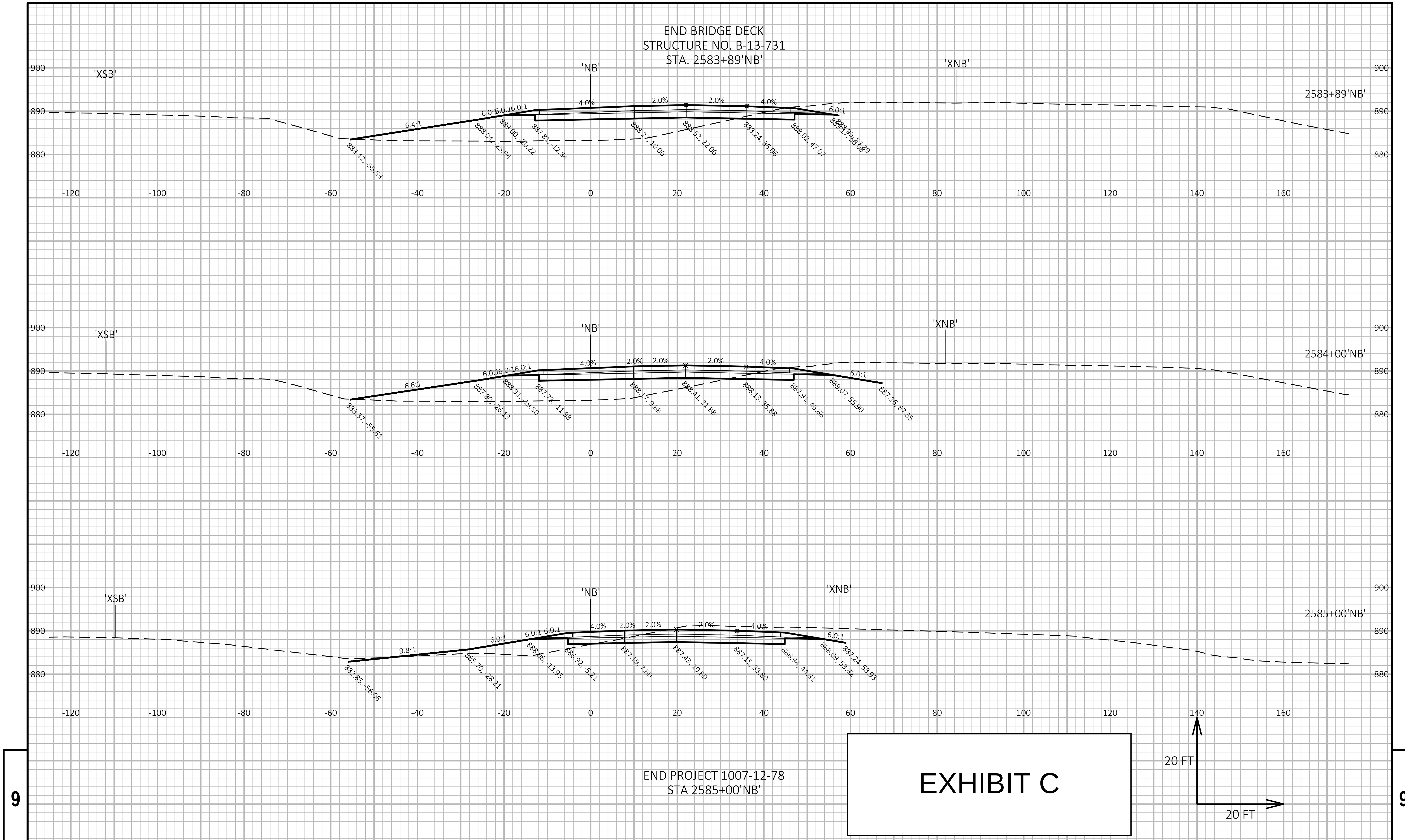
LAYOUT NAME - 26

PLOT DATE : 2/13/2019 4:15 PM PLOT BY : KYLE CORNELIUS PLOT NAME : PLOT SCALE : 1 IN:20 FT HORZ. / 1 IN:20 FT VERT.

WISDOT/CADDS SHEET 49



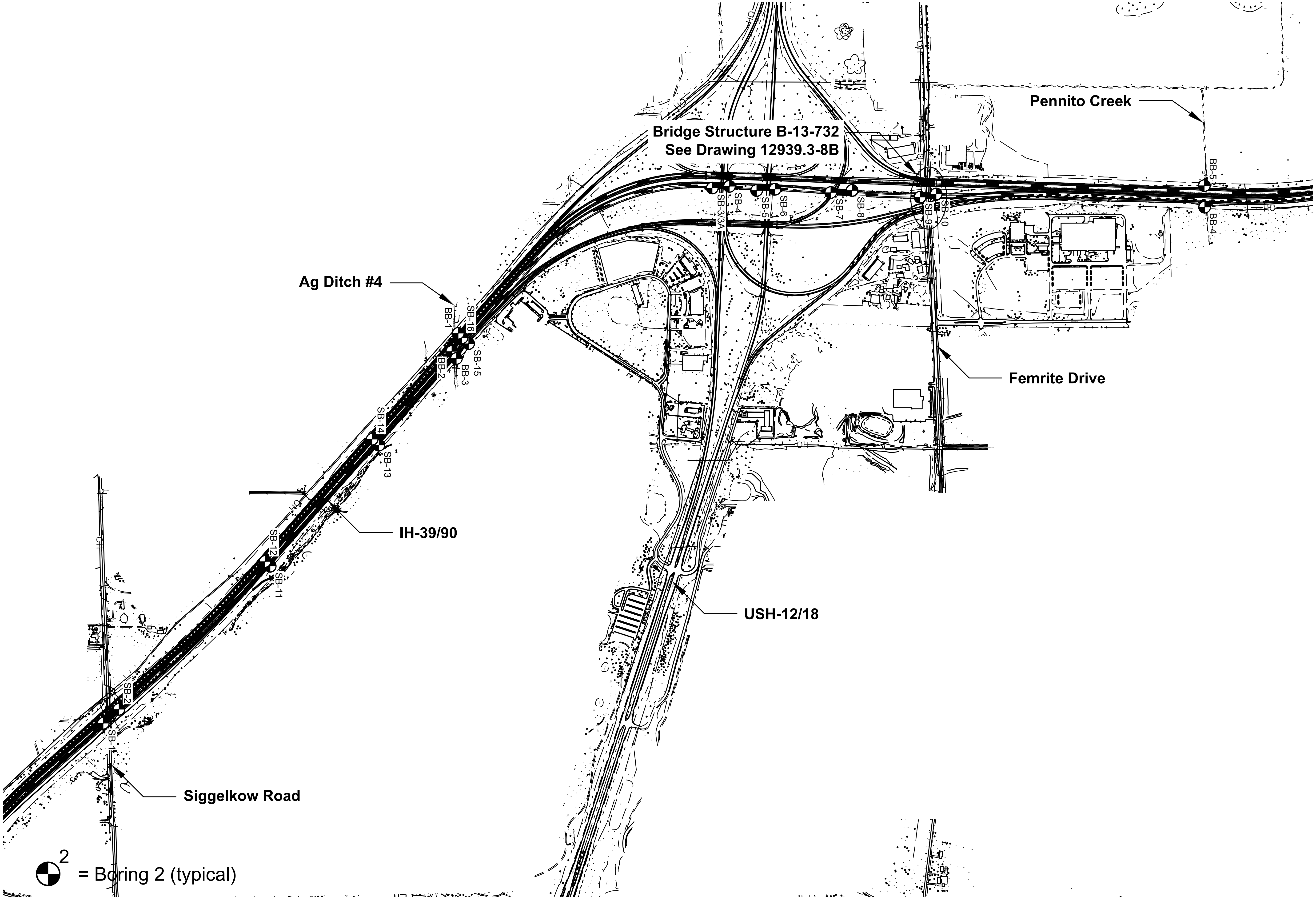
PROJECT NO: 1007-12-78	HWY: IH 39	COUNTY: DANE	CROSS SECTIONS: IH 39 NB	SHEET	E

PRE_99

FILE NAME : P:\2019 PROJECTS\5019001 - BIC\CIVIL3D\SHEETSPLAN\10071278\0902_XS_NB.DWG
LAYOUT NAME - 27
PLOT DATE : 2/13/2019 4:15 PM PLOT BY : KYLE CORNELIUS PLOT NAME : PLOT SCALE : 1 IN:20 FT HORZ. / 1 IN:20 FT VERT.
WISDOT/CADDS SHEET 49

Appendix B

Boring Location Sketch, Drawing 12939.3-8A and 12939.3-8B
WisDOT Boring Logs for Borings SB-9 and SB-10
Laboratory Test Result Record, Figure 1
Boring SB-10 Rock Core Picture



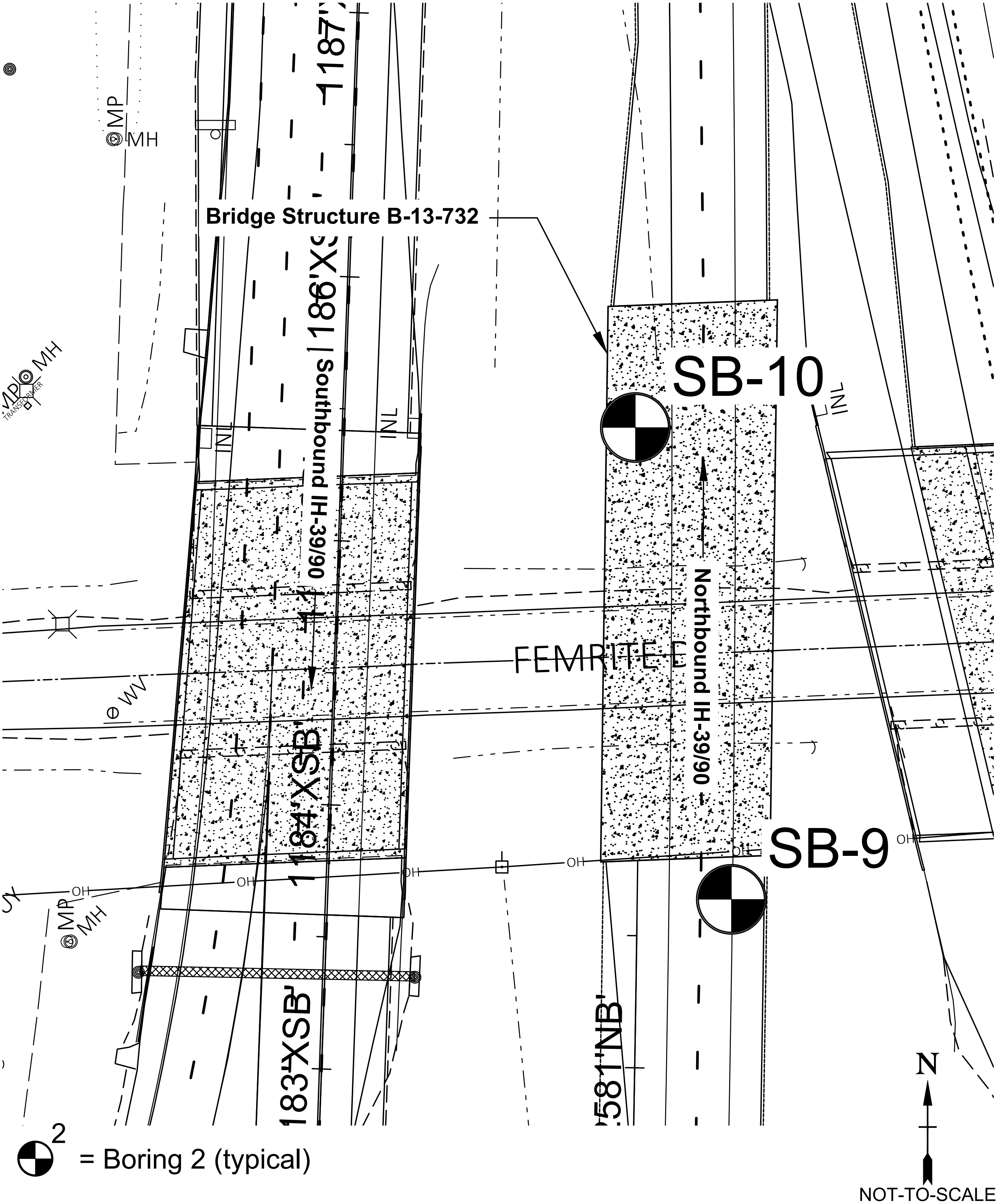
NOT-TO-SCALE

DRAWING
12939.3-8A

LOCATION SKETCH
IH-39 North Segment
IH-39/90 Northbound Bridge Over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

Soils & Engineering Services, Inc.
1102 STEWART STREET • MADISON, WISCONSIN 53713
Phone: 608-274-7600 • 888-866-SOIL (7645)
Fax: 608-274-7511 • Email: soils@soils.ws
CONSULTING CIVIL ENGINEERS SINCE 1966

Created on 04/10/2019 Revised on



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CONSULTING CIVIL ENGINEERS SINCE 1966

LOCATION SKETCH
IH-39 North Segment
IH-39/90 Northbound Bridge Over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

DRAWING
12939.3-8B

Created on 04/10/2019 Revised on

WI Dept. of Transportation 3502 Kinsman Blvd. Madison, WI 53704	WISDOT PROJECT ID: **1007-10-02/05**		**BORING ID: SB-9**
	WISDOT STRUCTURE ID: **B-13-732**		PAGE NO: 1 of 2
WISDOT PROJECT NAME: IH-39 North Segment	CONSULTANT: Dane Partners, LLC	CONSULTANT PROJECT NO:	LATITUDE: / LONGITUDE:
ROADWAY NAME: IH-39/90 NB Bridge over Femrite Drive	DRILLING CONTRACTOR: Soils & Engineering Services, Inc.	DRILLING CONTRACTOR PROJECT NO: 12939.3	NORTHING: 474452.74 / EASTING: 849818.5
DATE STARTED: 1/08/19	CREW CHIEF: S. Hunger	DRILL RIG: CME 550X	COORDINATE SYSTEM: WCCS
DATE COMPLETED: 1/08/19	LOGGED BY: G. Prior	HOLE SIZE: 5.6 in	HORIZONTAL DATUM: WCCS Dane / VERTICAL DATUM: MSL
COUNTY: Dane	LOG QC BY: C. Bower	HAMMER TYPE: Automatic	STREAMBED ELEVATION: NA
STATION: 2581+64.24 'NB (OES)' / OFFSET: 35.598' R	TOWNSHIP: 7N / RANGE: 10E / SECTION: 23	1/4 SECTION: SW / 1/4 1/4 SECTION: SE	SURFACE ELEVATION: 881.18 ft

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	Elev.	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
GS		M		0–1.0	LEAN CLAY WITH SAND, very dark brown, moist, medium plasticity, organic odor, fill topsoil	880.2	CL					HSA	2¼"-inside-diameter continuous flight hollow-stem augers
SPT 1	16	M	5-4-3 (7)	1.0–3.0	SILTY SAND WITH GRAVEL, brown, fine to coarse grained, moist, loose, non-plastic to low plasticity, fill, mixed with LEAN CLAY (CL) TOPSOIL and LEAN CLAY (CL) mixed with LEAN CLAY (CL) TOPSOIL and LEAN CLAY (CL)	878.2	SM						organic odor $N1_{60}$=15
SPT 2	16	M / M	3-6-6 (12)	3.0–4.5	SANDY SILT WITH GRAVEL, dark gray, moist, medium dense, non-plastic to low plasticity, fill	876.7	ML	4.0, 4.4					$N1_{60}$=22
				4.5–5.5	LEAN CLAY, brown, moist, hard, medium plasticity, occasional gravel	875.7	CL						
SPT 3	8	M	6-9-15 (24)	5.5–12.0	SILTY SAND WITH GRAVEL, brown, fine to medium grained, moist, medium dense, non-plastic to low plasticity, trace to some gravel, glacial till	869.2	SM						Drove Stone $N1_{60}$=40
SPT 4	1	M	6-8-9 (17)										Drove Stone $N1_{60}$=26
SPT 5	14	M	8-10-11 (21)	12.0–32.0	SILTY SAND WITH GRAVEL AND COBBLES, brown, fine to medium grained, moist to wet, medium dense to loose, non-plastic to low plasticity, trace to some gravel, few cobbles, glacial till	849.2	SM						$N1_{60}$=28
SPT 6	14	W	8-11-6 (17)										$N1_{60}$=21
SPT 7	16	W 12	3-3-5 (8)										Dry Wt = 128 pcf MC = 12.3% $N1_{60}$=10
SPT 8	12	W	11-14-11 (25)										$N1_{60}$=28
SPT 9	16	W	9-16-21 (37)	32.0–	POORLY GRADED SAND WITH GRAVEL, brown, poorly graded, fine grained, wet, dense		SP						$N1_{60}$=39

WATER LEVEL & CAVE-IN OBSERVATION DATA

WATER ENCOUNTERED DURING DRILLING: NMR	CAVE - IN DEPTH AT COMPLETION: 14.25ft. (WET ☐ DRY ☐)
WATER LEVEL AT COMPLETION: 12.58ft.	CAVE - IN DEPTH AFTER 0 HOURS: NMR (WET ☐ DRY ☐)

NOTES: 1) Stratification lines between soil types represent the approximate boundary; gradual transition between in-situ soil layers should be expected.
2) NE = Not Encountered; NMR = No Measurement Recorded

WI Dept. of Transportation
3502 Kinsman Blvd.
Madison, WI 53704

WISDOT PROJECT ID: 1007-10-02/05
WISDOT STRUCTURE ID: B-13-732

BORING ID: SB-9

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
						POORLY GRADED SAND WITH GRAVEL, brown, poorly graded, fine grained, wet, dense	SP						
				37.0		844.2							
						SILTY SAND WITH GRAVEL, brown, fine to medium grained, moist, very dense, non-plastic to low plasticity, trace to some gravel, glacial till							
SPT 10	18	M 9	17-27-27 (54)									HSA	Dry Wt = 135 pcf 2¼"-inside-diameter continuous flight hollow-stem augers and water MC = 8.5% $N1_{60}$=54
				40			SM						
				43.5		837.7							
SPT 11	5	W	100/6"			SANDSTONE, slightly weathered to moderately weathered, white, wet, very dense							$N1_{60}$=97/6"
				45									
				48.8		832.4							
SPT 12	0		100/3"			End of Boring at 48.8 ft.							$N1_{60}$=97/3"

N:\PROJECTS\12900 PROJECTS\12939.I-39 SEGMENTS\12939.3.I-39 NORTH\12939.3.BRS\732.NB BR.FEMRITE DR\DRAWINGS\12939.3.I39N.NB BR.FEMRITE DR.TN BLOOMG GROVE.DANE CO.1007-10-02.R08.WISDOT LOGS.B-13-732.GPJ IH-39 North Segment 04/10/1

WI Dept. of Transportation 3502 Kinsman Blvd. Madison, WI 53704	WISDOT PROJECT ID: 1007-10-02/05 WISDOT STRUCTURE ID: B-13-732		BORING ID: SB-10 PAGE NO: 1 of 2
WISDOT PROJECT NAME: IH-39 North Segment	CONSULTANT: Dane Partners, LLC	CONSULTANT PROJECT NO:	LATITUDE: / LONGITUDE:
ROADWAY NAME: IH-39/90 NB Bridge over Femrite Drive	DRILLING CONTRACTOR: Soils & Engineering Services, Inc.	DRILLING CONTRACTOR PROJECT NO: 12939.3	NORTHING: 474631.32 / EASTING: 849782.2
DATE STARTED: 1/11/19	CREW CHIEF: S. Hunger	DRILL RIG: CME 550X	COORDINATE SYSTEM: WCCS
DATE COMPLETED: 1/11/19	LOGGED BY: G. Prior	HOLE SIZE: 6.6 in	HORIZONTAL DATUM: WCCS Dane / VERTICAL DATUM: MSL
COUNTY: Dane	LOG QC BY: C. Bower	HAMMER TYPE: Automatic	STREAMBED ELEVATION: NA
STATION: 2583+42.31 'NB (OES)' / OFFSET: 3.053' L	TOWNSHIP: 7N / RANGE: 10E / SECTION: 23	1/4 SECTION: SW / 1/4 1/4 SECTION: NE	SURFACE ELEVATION: 884.97 ft

Sample Type Number	Recovery (in) (RQD)	Moisture	Blow Counts (N Value)	Depth (ft)	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	Elev.	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
GS		F		0.7	LEAN CLAY WITH SAND, very dark brown, frozen, medium plasticity, fill topsoil Frozen, 0' to 3'	884.3	CL					HSA	3¼"-inside-diameter continuous flight hollow-stem augers organic odor
SPT 1	3	F	3-2-2 (4)	3.0	SANDY LEAN CLAY WITH GRAVEL, brown, frozen, medium plasticity, fill	882.0	CL						$N1_{60}$=10
SPT 2	4	M	3-30/5"		SILTY SAND WITH GRAVEL, COBBLES, AND BOULDERS, brown, fine to coarse grained, moist, loose to very dense, non-plastic to low plasticity, fill boulder at 4.42' with bricks, 5.5' to 8'		SM						On Cobble or Boulder $N1_{60}$=6-56/5"
SPT 3	12	M	30-38-17 (55)	8.0		877.0							$N1_{60}$=93
SPT 4	18	M 0	8-12-10 (22)	9.5	POORLY GRADED SAND, brown, poorly graded, fine to medium grained, moist, medium dense	875.5	SP						Fines = 91% MC = 21.8% $N1_{60}$=34
				12.0	SILT, brown, moist, medium dense, non-plastic to low plasticity, few sand	873.0	ML						
SPT 5	16	M	21-14-14 (28)		SILTY SAND WITH GRAVEL AND COBBLES, brown, fine to medium grained, moist to wet, medium dense to dense, non-plastic to low plasticity, trace to some gravel, few cobbles, glacial till		SM						$N1_{60}$=38
SPT 6	0		18-21-19 (40)										$N1_{60}$=49
SPT 7	18	M-W	5-6-7 (13)	27.0		858.0							$N1_{60}$=15
SPT 8	0		20-27-33 (60)		SILTY SAND WITH GRAVEL AND COBBLES, brown, fine to medium grained, moist to wet, very dense, non-plastic to low plasticity, trace to some gravel, few cobbles, glacial till		SM						$N1_{60}$=65
SPT 9	16	M-W	20-21-40 (61)										$N1_{60}$=63

WATER LEVEL & CAVE-IN OBSERVATION DATA

WATER ENCOUNTERED DURING DRILLING: NMR		CAVE - IN DEPTH AT COMPLETION: 20.17ft.	WET ☐ DRY ☐
WATER LEVEL AT COMPLETION: 17.5ft.		CAVE - IN DEPTH AFTER 0 HOURS: NMR	WET ☐ DRY ☐

NOTES: 1) Stratification lines between soil types represent the approximate boundary; gradual transition between in-situ soil layers should be expected.
2) NE = Not Encountered; NMR = No Measurement Recorded

WI Dept. of Transportation
3502 Kinsman Blvd.
Madison, WI 53704

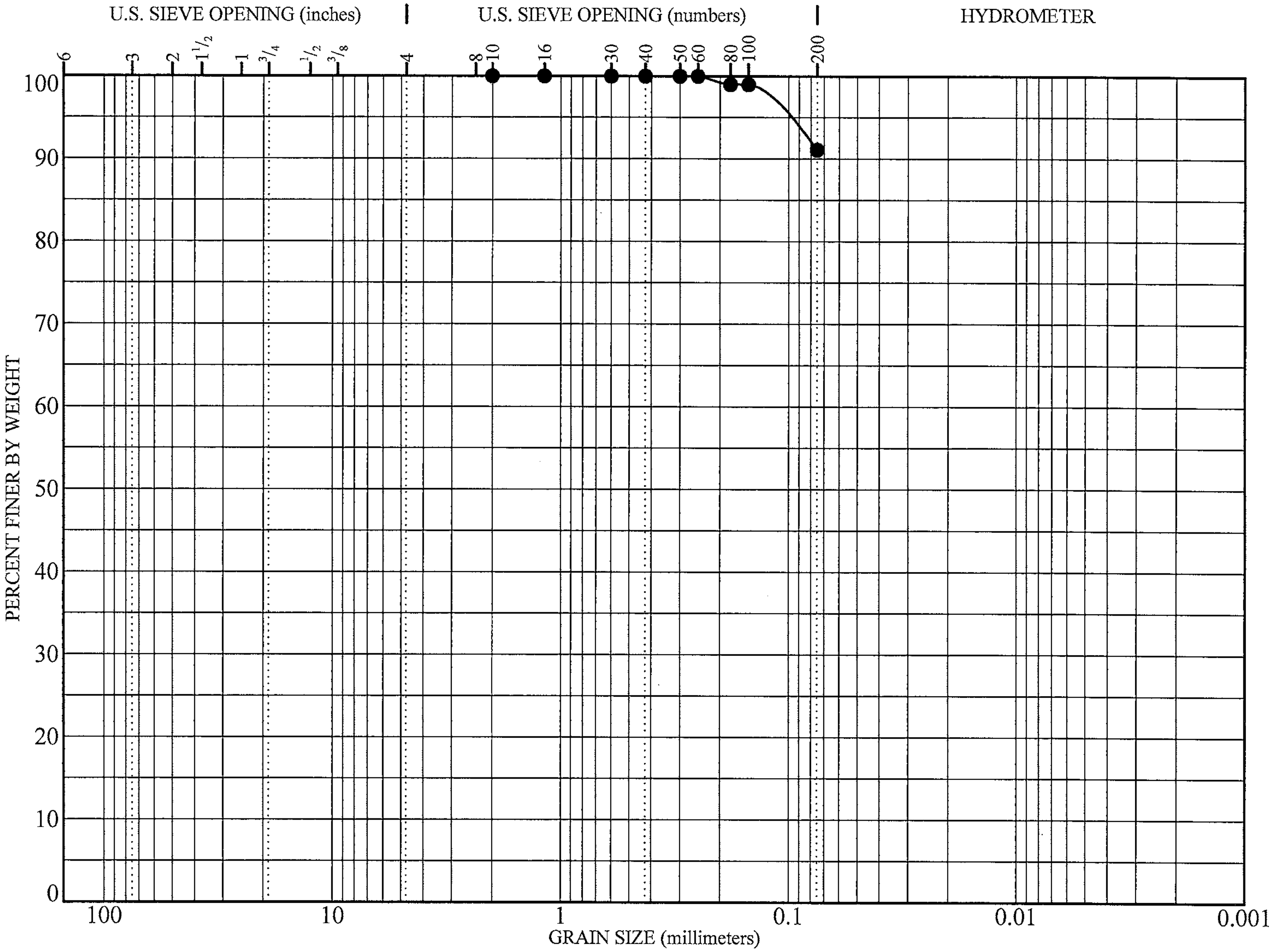
WISDOT PROJECT ID: 1007-10-02/05
WISDOT STRUCTURE ID: B-13-732

BORING ID: SB-10

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
						SILTY SAND WITH GRAVEL AND COBBLES, brown, fine to medium grained, moist to wet, very dense, non-plastic to low plasticity, trace to some gravel, few cobbles, glacial till							
SPT 10	4	M-W	40-60/5"	40			SM						$N1_{60}$=39-59/5"
				43.0		842.0							
						SANDSTONE, slightly weathered to moderately weathered, white to light brown, wet, very dense							
SPT 11	16	W 16	20-21-59 (80)	45									Dry Wt = 118 pcf MC = 16.0% $N1_{60}$=76
SPT 12	2	W	100/2"	49.0		836.0						HSA	3¼"-inside-diameter continuous flight hollow-stem augers and water $N1_{60}$=91/2"
				50		Set HSA at 49'-0" and cleaned out HSA with $2^7/_8$" tri-cone to 49' Switched to NQ2 rock coring at 49'							
RC 13	33 (7)	12				SANDSTONE, unweathered, crossbedded, white to light brown, wet, Layer RQD = 7% poorly-cemented, annealed vertical fractures, no mineralizations or fossils						COR	NQ2 diamond bit rock core and water Wet Wt = 134 pcf CR=52% WL=0% r_u=2360 psi
				54.3		830.7							

End of Boring at 54.3 ft.

PARTICLE SIZE DISTRIBUTION ANALYSIS REPORT



	COBBLES (%)	GRAVEL (%)		SAND (%)			FINES (%)	
		coarse	fine	coarse	medium	fine	SILT (%)	CLAY (%)
●	0	0		9			91.1	

Sieve Size	Percent Finer ●		
#10	100		
#16	100		
#30	100		
#40	100		
#50	100		
#60	100		
#80	99		
#100	99		
#200	91.1		

Sieve Size	Percent Finer ●		

	Grain Size (mm)			Coefficients	
	D_{60}	D_{30}	D_{10}	C_c	C_u
●					

	Sample Information
●	Boring SB-10, 9'-9" Depth: ***SILT (ML)*** *— non-plastic to low plasticity, brown, moist, medium dense relative density, few sand*

Soils & Engineering Services, Inc.
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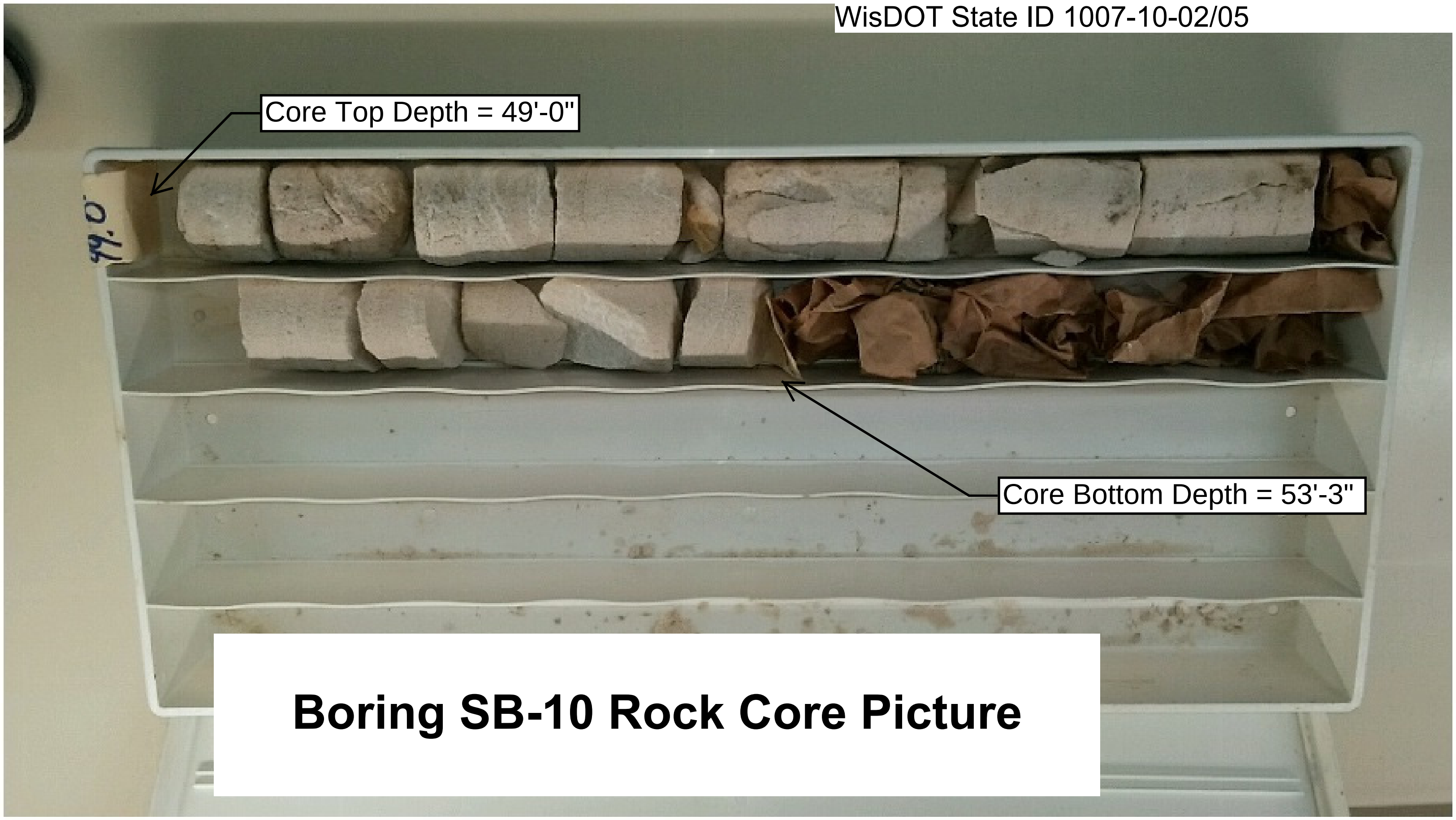
CONSULTING CIVIL ENGINEERS SINCE 1966

LABORATORY TEST RESULT RECORD
IH-39 North Segment
IH-39/90 NB Bridge over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

12939.3

FIGURE 1

IH-39/90 Northbound Bridge over Femrite Drive
Illinois State Line - Madison, USH-12/18 Interchange
City of Madison, Rock County, Wisconsin
WisDOT State ID 1007-10-02/05



Boring SB-10 Rock Core Picture

<u>Appendix C</u>

Table 8-1: Recommended Soil Design Parameters
Table 8-2: Deep Driven Pile Foundation Analyses' Results; *APILE* Static Analyses' Average Unfactored Soil Design Parameters
Table 8-3: Deep Driven Pile Foundation Analyses' Results; *APILE* Static and *GRLWEAP* Drivability Analyses' Results

Table 8-1: RECOMMENDED SOIL DESIGN PARAMETERS
IH-39 North Segment
IH-39/90 NB Bridge over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

Elevation (feet)	Material Type	Estimated Soil Parameters*: Moist Density, γ (pcf)	Estimated Soil Parameters*: Angle of Internal Friction, Ø* (degrees)	Estimated Soil Parameters*: Cohesion, c (psf)	Driven Pile Parameters: Driving SSL** (%)	Driven Pile Parameters: LRFD Factors***
	---------- Boring SB-9 located in the vicinity of South Abutment of Bridge Structure B-13-732 ----------					
883.6 to 881.2	Construct new embankment with WisDOT Embankment Fill	135.0	30	0	---	---
881.2 to 881.1	Replace existing dark brown LEAN CLAY WITH SAND (CL) FILL TOPSOIL with WisDOT Embankment Fill	135.0	30	0	---	---
881.1 to 880.2	Replace existing dark brown LEAN CLAY WITH SAND (CL) FILL TOPSOIL with WisDOT Embankment Fill	135.0	30	0	33 [67]	0.35
880.2 to 878.2	***SILTY SAND WITH GRAVEL (SM)*** *— fine to coarse grained, non-plastic to low plasticity fines, brown, moist, loose relative density,* ***FILL****, mixed with LEAN CLAY (CL) TOPSOIL and LEAN CLAY (CL)*	125.0	31	0	33 [67]	0.45
878.2 to 876.7	***SANDY SILT WITH GRAVEL (ML)*** *— non-plastic to low plasticity, dark gray, moist, medium dense relative density,* ***FILL***	125.0	31	0	33 [67]	0.45
876.7 to 875.7	***LEAN CLAY (CL)*** *— medium plasticity, grayish-brown, moist, hard consistency, occasional gravel*	132.0	0	4,000	50 [83]	0.35
875.7 to 869.2	***SILTY SAND WITH GRAVEL (SM)*** *— fine to medium grained, non-plastic to low plasticity fines, brown to reddish-brown, moist, medium dense relative density,* ***GLACIAL TILL****, trace to some gravel*	152.0	34	0	33 [67]	0.45
869.2 to 868.0	***SILTY SAND WITH GRAVEL AND COBBLES (SM)*** *— fine to medium grained, non-plastic to low plasticity fines, brown to reddish-brown, moist to wet, medium dense to loose relative density,* ***GLACIAL TILL****, trace to some gravel, few cobbles*	152.0	34	0	33 [67]	0.45
868.0 to 859.2		152.0	34	0	33 [67]	0.45
859.2 to 854.2		152.0	30	0	33 [67]	0.45
854.2 to 849.2		152.0	35	0	33 [67]	0.45
849.2 to 844.2	***POORLY-GRADED SAND WITH GRAVEL (SP)*** *— fine grained, light brown and brown, wet, dense relative density*	125.0	38 [36]	0	17 [50]	0.45
844.2 to 837.7	***SILTY SAND WITH GRAVEL (SM)*** *— fine to medium grained, non-plastic to low plasticity fines, brown to reddish-brown, moist, very dense relative density,* ***GLACIAL TILL****, trace to some gravel*	152.0	42 [36]	0	33 [67]	0.45

Table Notes

* The Moist Density, Angle of Internal Friction, Cohesion, Driving SSL, and LRFD Static Resistance Factor values are considered estimates based on the soils encountered at the indicated boring.
* The computed Angle of Internal Friction, Ø, for each soil stratum or sub-stratum is presented. For driven pile computations, computed Øs greater than 36° are reduced to 36° or 40° depending upon the $N1_{60}$-value for the indicated stratum/sub-stratum. The reduced driven pile Øs are shown in brackets ([]). Please refer to geotechnical report for further explanation.
** The presented Driving SSL values from Table 11.3-4 of the WisDOT Bridge Manual are used for driven cast-in-place, steel pipe piles. The increased Driving SSL values shown in brackets ([]) are used for driven steel H-piles based on feedback from pile driving contractors.
*** LRFD Static Resistance Factors and LRFD Load Factors are presented in this column. The LRFD Static Resistance Factors from the WisDOT Bridge Manual Table 11.3-1 or Table 11.3-8 are presented. For soil strata contributing to downdrag, LRFD Load Factors from the AASHTO LRFD Bridge Design Specifications Table 3.4.1-2 are presented in brackets ([]). Please see report for further definition of the LRFD Factors presented.

Table Abbreviations and Symbols

pcf = pounds per cubic foot.
SSL = soil strength loss.
psf = pounds per square foot.
NA = Not applicable.
Top = Cohesion @ top of stratum/sub-stratum.
Bot = Cohesion @ bottom of stratum/sub-stratum.

Printed on 4/9/2019

Table 8-1: RECOMMENDED SOIL DESIGN PARAMETERS
IH-39 North Segment
IH-39/90 NB Bridge over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

		Estimated Soil Parameters*			Driven Pile Parameters	
Elevation (feet)	**Material Type**	**Moist Density, ϒ (pcf)**	**Angle of Internal Friction, Ø* (degrees)**	**Cohesion, c (psf)**	**Driving SSL** (%)**	**LRFD Factors*****
837.7 to 832.5	***SANDSTONE*** *— slightly- to moderately-weathered, white and yellowish-brown, wet, very dense relative density*	137.0	45 [40]	0	0 [0]	0.45
	---------- End of Boring SB-9 @ Elevation 832.5 feet ----------					

Table Notes
* The Moist Density, Angle of Internal Friction, Cohesion, Driving SSL, and LRFD Static Resistance Factor values are considered estimates based on the soils encountered at the indicated boring.
* The computed Angle of Internal Friction, Ø, for each soil stratum or sub-stratum is presented. For driven pile computations, computed Øs greater than 36° are reduced to 36° or 40° depending upon the $N1_{60}$-value for the indicated stratum/sub-stratum. The reduced driven pile Øs are shown in brackets ([]). Please refer to geotechnical report for further explanation.
** The presented Driving SSL values from Table 11.3-4 of the WisDOT Bridge Manual are used for driven cast-in-place, steel pipe piles. The increased Driving SSL values shown in brackets ([]) are used for driven steel H-piles based on feedback from pile driving contractors.
*** LRFD Static Resistance Factors and LRFD Load Factors are presented in this column. The LRFD Static Resistance Factors from the WisDOT Bridge Manual Table 11.3-1 or Table 11.3-8 are presented. For soil strata contributing to downdrag, LRFD Load Factors from the AASHTO LRFD Bridge Design Specifications Table 3.4.1-2 are presented in brackets ([]). Please see report for further definition of the LRFD Factors presented.

Table Abbreviations and Symbols
pcf = pounds per cubic foot.
SSL = soil strength loss.
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NA = Not applicable.
Top = Cohesion @ top of stratum/sub-stratum.
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Table 8-1: RECOMMENDED SOIL DESIGN PARAMETERS
IH-39 North Segment
IH-39/90 NB Bridge over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

Elevation (feet)	Material Type	Estimated Soil Parameters*			Driven Pile Parameters	
		Moist Density, Υ (pcf)	Angle of Internal Friction, Ø* (degrees)	Cohesion, c (psf)	Driving SSL** (%)	LRFD Factors***
	Boring SB-10 located in the vicinity of North Abutment of Bridge Structure B-13-732					
885.0 to 884.3	***LEAN CLAY WITH SAND (CL)*** — *medium plasticity, very dark brown, frozen,* ***FILL TOPSOIL***	120.0	0	500	---	---
884.3 to 882.2	***SANDY LEAN CLAY WITH GRAVEL (CL)*** — *medium plasticity, brown, frozen,* ***FILL***	125.0	0	500	---	---
882.2 to 882.0		125.0	0	500	---	---
882.0 to 879.7	***SILTY SAND WITH GRAVEL, COBBLES, AND BOULDERS (SM)*** — *fine to coarse grained, non-plastic to low plasticity fines, brown, moist, loose to very dense relative density,* ***FILL***	125.0	31	0	---	---
879.7 to 879.5		125.0	31	0	33 [67]	0.45
879.5 to 877.0		125.0	45 [40]	0	33 [33]	0.45
877.0 to 875.5	***POORLY-GRADED SAND (SP)*** — *fine to medium grained, brown, moist, medium dense relative density*	125.0	36	0	17 [50]	0.45
875.5 to 873.0	***SILT (ML)*** — *non-plastic to low plasticity, dark grayish-brown, moist, medium dense relative density, few sand*	125.0	36	0	33 [67]	0.45
873.0 to 868.0	***SILTY SAND WITH GRAVEL AND COBBLES (SM)*** — *fine to medium grained, non-plastic to low plasticity fines, brown to reddish-brown, moist to wet, medium dense to very dense relative density,* ***GLACIAL TILL****, trace to some gravel, few cobbles*	152.0	38 [36]	0	33 [67]	0.45
868.0 to 863.0		152.0	41 [36]	0	33 [67]	0.45
863.0 to 858.0		152.0	31	0	33 [67]	0.45
858.0 to 848.0		152.0	44 [36]	0	33 [67]	0.45
848.0 to 842.0		152.0	45 [40]	0	17 [17]	0.45
842.0 to 836.0	***SANDSTONE*** — *slightly- to moderately-weathered, white to light yellowish-brown, wet, very dense relative density*	137.0	0	72,000 (Top) 144,000 (Bot)	0 [0]	0.45
836.0 to 830.8	***SANDSTONE*** — *unweathered, crossbedded, white to light brown, wet, layer RQD = 7%, poorly-cemented, annealed vertical fractures, no mineralizations or fossils*	137.0	0	144,000 (Top) 288,000 (Bot)	0 [0]	0.45

Table Notes

* The Moist Density, Angle of Internal Friction, Cohesion, Driving SSL, and LRFD Static Resistance Factor values are considered estimates based on the soils encountered at the indicated boring.

* The computed Angle of Internal Friction, Ø, for each soil stratum or sub-stratum is presented. For driven pile computations, computed Øs greater than 36° are reduced to 36° or 40° depending upon the $N1_{60}$-value for the indicated stratum/sub-stratum. The reduced driven pile Øs are shown in brackets ([]). Please refer to geotechnical report for further explanation.

** The presented Driving SSL values from Table 11.3-4 of the WisDOT Bridge Manual are used for driven cast-in-place, steel pipe piles. The increased Driving SSL values shown in brackets ([]) are used for driven steel H-piles based on feedback from pile driving contractors.

*** LRFD Static Resistance Factors and LRFD Load Factors are presented in this column. The LRFD Static Resistance Factors from the WisDOT Bridge Manual Table 11.3-1 or Table 11.3-8 are presented. For soil strata contributing to downdrag, LRFD Load Factors from the AASHTO LRFD Bridge Design Specifications Table 3.4.1-2 are presented in brackets ([]). Please see report for further definition of the LRFD Factors presented.

Table Abbreviations and Symbols

pcf = pounds per cubic foot.
SSL = soil strength loss.
psf = pounds per square foot.
NA = Not applicable.
Top = Cohesion @ top of stratum/sub-stratum.
Bot = Cohesion @ bottom of stratum/sub-stratum.

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Table 8-1: RECOMMENDED SOIL DESIGN PARAMETERS
IH-39 North Segment
IH-39/90 NB Bridge over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

		Estimated Soil Parameters*			Driven Pile Parameters	
Elevation (feet)	Material Type	Moist Density, γ (pcf)	Angle of Internal Friction, Ø* (degrees)	Cohesion, c (psf)	Driving SSL** (%)	LRFD Factors***
	---------- End of Boring SB-10 @ Elevation 830.8 feet ----------					

Table Notes

* The Moist Density, Angle of Internal Friction, Cohesion, Driving SSL, and LRFD Static Resistance Factor values are considered estimates based on the soils encountered at the indicated boring.

* The computed Angle of Internal Friction, Ø, for each soil stratum or sub-stratum is presented. For driven pile computations, computed Øs greater than 36° are reduced to 36° or 40° depending upon the $N1_{60}$-value for the indicated stratum/sub-stratum. The reduced driven pile Øs are shown in brackets ([]). Please refer to geotechnical report for further explanation.

** The presented Driving SSL values from Table 11.3-4 of the WisDOT Bridge Manual are used for driven cast-in-place, steel pipe piles. The increased Driving SSL values shown in brackets ([]) are used for driven steel H-piles based on feedback from pile driving contractors.

*** LRFD Static Resistance Factors and LRFD Load Factors are presented in this column. The LRFD Static Resistance Factors from the WisDOT Bridge Manual Table 11.3-1 or Table 11.3-8 are presented. For soil strata contributing to downdrag, LRFD Load Factors from the AASHTO LRFD Bridge Design Specifications Table 3.4.1-2 are presented in brackets ([]). Please see report for further definition of the LRFD Factors presented.

Table Abbreviations and Symbols

pcf = pounds per cubic foot.
SSL = soil strength loss.
psf = pounds per square foot.
NA = Not applicable.
Top = Cohesion @ top of stratum/sub-stratum.
Bot = Cohesion @ bottom of stratum/sub-stratum.

Table 8-2: DEEP PILE FOUNDATION ANALYSES' RESULTS

Static Analyses' Average Unfactored Soil Design Parameters
IH-39 North Segment
IH-39/90 NB Bridge over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

Boring ==> SB-9 located in the vicinity of South Abutment of Bridge Structure B-13-732

Substructure Unit Foundation Bottom Elev ==>	881.1 feet	**Design Ultimate GWL Elev ==>**	868.0 feet	**Design Contraction Scour Elev ==>**	NA
Design Driving GS Elev ==>	883.6 feet	**Design Driving/Restrike GWL Elev ==>**	868.0 feet	**Design Local Scour Elev ==>**	NA
Design Ultimate GS Elev ==>	883.6 feet	**Design Pre-Boring Elev ==>**	NA		

Pile Type and Size ==>	10¾Ø CIP		HP 10x42											
Elevation (feet)	**Skin Friction (psf)**	**End Bearing (psf)**	**Skin Friction (psf)**	**End Bearing (psf)**	**Skin Friction (psf)**	**End Bearing (psf)**	**Skin Friction (psf)**	**End Bearing (psf)**	**Skin Friction (psf)**	**End Bearing (psf)**	**Skin Friction (psf)**	**End Bearing (psf)**	**Skin Friction (psf)**	**End Bearing (psf)**
883.6 to 881.2	0	0	0	0										
881.2 to 881.1	0	0	0	0										
881.1 to 880.2	90	6,200	120	6,200										
880.2 to 878.2	140	12,600	190	12,700										
878.2 to 876.7	220	23,500	280	23,800										
876.7 to 875.7	4,000	33,600	4,000	33,300										
875.7 to 869.2	480	56,300	610	56,200										
869.2 to 868.0	670	73,300	860	73,200										
868.0 to 859.2	810	71,400	1,040	70,900										
859.2 to 854.2	700	22,500	920	24,400										
854.2 to 849.2	1,230	106,000	1,580	105,000										
849.2 to 844.2	1,540	149,000	1,930	148,000										
844.2 to 837.7	1,710	171,000	2,150	175,000										
837.7 to 832.4	NA	Refusal	NA	Refusal										

Table Abbreviations and Symbols
HP = Steel H-pile.
NA = Not applicable.
NR = Not recommended.
psf = pounds per square foot.
Ø CIP = -inch-outside-diameter, cast-in-place, steel pipe pile.
⊠ PCP = -inch-square, precast concrete pile.
Refusal = Steel pile will obtain capacity in indicated layer due to very dense soil or bedrock.
Elev = Elevation.
GS = Ground Surface.
GWL = Groundwater Level.

The **End Bearing & Skin Friction** values presented above are the computed unfactored average for the indicated stratum.

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Table 8-2: DEEP PILE FOUNDATION ANALYSES' RESULTS
Static Analyses' Average Unfactored Soil Design Parameters
IH-39 North Segment
IH-39/90 NB Bridge over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

Boring ==>	SB-10 located in the vicinity of North Abutment of Bridge Structure B-13-732				
Substructure Unit Foundation Bottom Elev ==>	879.7 feet	**Design Ultimate GWL Elev ==>**	868.0 feet	**Design Contraction Scour Elev ==>**	NA
Design Driving GS Elev ==>	882.2 feet	**Design Driving/Restrike GWL Elev ==>**	868.0 feet	**Design Local Scour Elev ==>**	NA
Design Ultimate GS Elev ==>	882.2 feet	**Design Pre-Boring Elev ==>**	NA		

Pile Type and Size ==>	10¾Ø CIP		HP 10x42											
Elevation (feet)	**Skin Friction (psf)**	**End Bearing (psf)**	**Skin Friction (psf)**	**End Bearing (psf)**	**Skin Friction (psf)**	**End Bearing (psf)**	**Skin Friction (psf)**	**End Bearing (psf)**	**Skin Friction (psf)**	**End Bearing (psf)**	**Skin Friction (psf)**	**End Bearing (psf)**	**Skin Friction (psf)**	**End Bearing (psf)**
885.0 to 882.2	---	---	---	---										
882.2 to 882.0	0	0	0	0										
882.0 to 879.7	0	0	0	0										
879.7 to 879.5	210	32,800	260	33,000										
879.5 to 877.0	290	54,000	360	52,000										
877.0 to 875.5	290	48,900	360	50,500										
875.5 to 873.0	390	54,400	480	54,400										
873.0 to 868.0	590	83,600	740	83,600										
868.0 to 863.0	830	104,000	1,040	103,000										
863.0 to 858.0	620	30,300	820	32,700										
858.0 to 848.0	1,240	157,000	1,560	157,000										
848.0 to 842.0	2,230	429,000	2,730	431,000										
842.0 to 836.0	NA	Refusal	NA	Refusal										
836.0 to 830.7	NA	NA	NA	NA										

Table Abbreviations and Symbols
HP = Steel H-pile.
NA = Not applicable.
NR = Not recommended.
psf = pounds per square foot.
Ø CIP = -inch-outside-diameter, cast-in-place, steel pipe pile.
⊠ PCP = -inch-square, precast concrete pile.
Refusal = Steel pile will obtain capacity in indicated layer due to very dense soil or bedrock.
Elev = Elevation.
GS = Ground Surface.
GWL = Groundwater Level.

The **End Bearing & Skin Friction** values presented above are the computed unfactored average for the indicated stratum.

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Table 8-3: DEEP DRIVEN PILE FOUNDATION ANALYSES' RESULTS
APILE Static and *GRLWEAP* Drivability Analyses' Results
IH-39 North Segment
IH-39/90 NB Bridge over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

Boring ==>	SB-9 located in the vicinity of South Abutment of Bridge Structure B-13-732				
Substructure Unit Foundation Bottom Elev ==>	881.1 feet	**Design Ultimate GWL Elev ==>**	868.0 feet	**Design Contraction Scour Elev ==>**	NA
Design Driving GS Elev ==>	883.6 feet	**Design Driving/Restrike GWL Elev ==>**	868.0 feet	**Design Local Scour Elev ==>**	NA
Design Ultimate GS Elev ==>	883.6 feet	**Design Pre-Boring Elev ==>**	NA		

Pile Type and Size	WisDOT Maximum Rn_{dyn} (kips)	Design Steel Yield Stress (ksi)	Design Rn_{dyn} (kips)	Design P_r (kips)	Design Pile Tip Elevation* (feet)	Vertical Piles Driven to Design Rn_{dyn} At Time of Installation: Pile Driving Hammer	Pile Tip Elevation* (feet)	Pile Refusal** (Yes/No)	Soil Resistance During Driving* (kips)	Max Steel Stress During Driving* (ksi)	Ultimate Static Capacity*** (kips)
Factored Embankment Settlement Downdrag Load = 31 kips for 10¾Ø CIP.											
Per WBM Chapter 11.3.1.17.1, the Design P_r should be reduced by the Factored Embankment Settlement Downdrag Load for 10¾Ø CIP unless countermeasures are taken.											
10¾Ø CIP 0.250Ŧ	260	35	260	130	838.3	D16-32	838.3	No	260	46.8	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.250Ŧ	260	45†	260	130	838.3	D16-32	838.3	No	260	46.8	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.365Ŧ	300	35	300	150	838.0	D16-32	838.0	No	300	36.0	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.365Ŧ	300	45†	300	150	838.0	D16-32	838.0	No	300	36.0	342
10¾Ø CIP 0.500Ŧ	300	35	300	150	838.0	D16-32	838.0	No	300	31.4	342
10¾Ø CIP 0.500Ŧ	300	45†	300	150	838.0	D16-32	838.0	No	300	31.4	342
Factored Embankment Settlement Downdrag Load = 41 kips for 10x42 HP.											
Per WBM Chapter 11.3.1.17.1, Factored Embankment Settlement Downdrag Loads up to 126 kips in addition to the Design P_r are allowed for end-bearing 10x42 HP. See report text.											
10x42 HP	360	50	360	180	836.6	D16-32	836.6	Yes	360	36.2	484

Table Notes
* Elevation, Soil Resistance During Driving, and Max(imum) Steel Stress During Driving values are considered estimates based on the soils encountered at the indicated boring.
** Pile Refusal = Specified pile driving hammer is estimated to require more than 120 blows per foot to drive the indicated steel pile.
*** Ultimate Static Capacity is the estimated capacity using static analyses that the indicated pile could achieve after pile set-up. The Design Rn_{dyn} and Design P_r should be used for design.
‡ Pile driven through upper dense layer(s) to reach design pile tip elevation. Maximum Computed Driving Rn_{dyn} {shown within brackets} for the upper dense layer(s) is less than or equal to the WisDOT Maximum Rn_{dyn} for the indicated pile.
★ Pile Tip Elevation for indicated pile was controlled by recommended 10-foot minimum pile embedment below pre-bore depth.
† Per Mr. Jeff Horsfall with WisDOT, the use of steel with a yield stress of 45 ksi for the shell of CIP pipe piles is acceptable to WisDOT.
✖ Design P_r was reduced due to low strength soil strata below the Design Pile Tip Elevation or due to a Design Ultimate GS Elevation that is less than the Design Driving GS Elevation.

Table Abbreviations and Symbols
NA = Not applicable.
ksi = kips per square inch.
HP = Steel H-pile.
f'_c = ultimate concrete strength in ksi.
Ŧ = -inch wall thickness.
P_r = Factored Axial Compression Resistance.
Ø CIP = -inch-outside-diameter, cast-in-place, steel pipe pile.
⌀ PCP = -inch-square, precast concrete pile.
Rn_{dyn} = Required Driving Resistance.

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Table 8-3: DEEP DRIVEN PILE FOUNDATION ANALYSES' RESULTS
APILE Static and *GRLWEAP* Drivability Analyses' Results
IH-39 North Segment
IH-39/90 NB Bridge over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

Boring ==>	SB-9 located in the vicinity of Pier 1 of Bridge Structure B-13-732				
Substructure Unit Foundation Bottom Elev ==>	861.4 feet	**Design Ultimate GWL Elev ==>**	868.0 feet	**Design Contraction Scour Elev ==>**	NA
Design Driving GS Elev ==>	869.8 feet	**Design Driving/Restrike GWL Elev ==>**	868.0 feet	**Design Local Scour Elev ==>**	NA
Design Ultimate GS Elev ==>	869.8 feet	**Design Pre-Boring Elev ==>**	NA		

Pile Type and Size	WisDOT Maximum Rn_{dyn} (kips)	Design Steel Yield Stress (ksi)	Design Rn_{dyn} (kips)	Design P_r (kips)	Design Pile Tip Elevation* (feet)	Vertical Piles Driven to Design Rn_{dyn} At Time of Installation: Pile Driving Hammer	Pile Tip Elevation* (feet)	Pile Refusal** (Yes/No)	Soil Resistance During Driving* (kips)	Max Steel Stress During Driving* (ksi)	Ultimate Static Capacity*** (kips)
Factored Embankment Settlement Downdrag Load = 0 kips for 10¾Ø CIP.											
No reduction in the Design P_r is recommended due to embankment settlement downdrag loads. See report text.											
10¾Ø CIP 0.250Ŧ	260	35	260	130	839.9	D16-32	839.9	No	260	51.2	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.250Ŧ	260	45†	260	130	839.9	D16-32	839.9	No	260	51.2	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.365Ŧ	300	35	300	150	839.6	D16-32	839.6	No	300	41.8	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.365Ŧ	300	45†	300	150	839.6	D16-32	839.6	No	300	41.8	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.500Ŧ	300	35	300	150	839.6	D16-32	839.6	No	300	35.6	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.500Ŧ	300	45†	300	150	839.6	D16-32	839.6	No	300	35.6	317
Factored Embankment Settlement Downdrag Load = 0 kips for 10x42 HP.											
No reduction in the Design P_r is recommended due to embankment settlement downdrag loads. See report text.											
10x42 HP	360	50	360	180	838.5	D16-32	838.5	No	360	31.6	395

Table Notes
* Elevation, Soil Resistance During Driving, and Max(imum) Steel Stress During Driving values are considered estimates based on the soils encountered at the indicated boring.
** Pile Refusal = Specified pile driving hammer is estimated to require more than 120 blows per foot to drive the indicated steel pile.
*** Ultimate Static Capacity is the estimated capacity using static analyses that the indicated pile could achieve after pile set-up. The Design Rn_{dyn} and Design P_r should be used for design.
‡ Pile driven through upper dense layer(s) to reach design pile tip elevation. Maximum Computed Driving Rn_{dyn} {shown within brackets} for the upper dense layer(s) is less than or equal to the WisDOT Maximum Rn_{dyn} for the indicated pile.
★ Pile Tip Elevation for indicated pile was controlled by recommended 10-foot minimum pile embedment below pre-bore depth.
† Per Mr. Jeff Horsfall with WisDOT, the use of steel with a yield stress of 45 ksi for the shell of CIP pipe piles is acceptable to WisDOT.
✖ Design P_r was reduced due to low strength soil strata below the Design Pile Tip Elevation or due to a Design Ultimate GS Elevation that is less than the Design Driving GS Elevation.

Table Abbreviations and Symbols
NA = Not applicable.
ksi = kips per square inch.
HP = Steel H-pile.
f'_c = ultimate concrete strength in ksi.
Ŧ = -inch wall thickness.
P_r = Factored Axial Compression Resistance.
Ø CIP = -inch-outside-diameter, cast-in-place, steel pipe pile.
⌀ PCP = -inch-square, precast concrete pile.
Rn_{dyn} = Required Driving Resistance.

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Table 8-3: DEEP DRIVEN PILE FOUNDATION ANALYSES' RESULTS
APILE Static and *GRLWEAP* Drivability Analyses' Results
IH-39 North Segment
IH-39/90 NB Bridge over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

Boring ==>	SB-10 located in the vicinity of Pier 2 of Bridge Structure B-13-732				
Substructure Unit Foundation Bottom Elev ==>	861.5 feet	**Design Ultimate GWL Elev ==>**	868.0 feet	**Design Contraction Scour Elev ==>**	NA
Design Driving GS Elev ==>	869.8 feet	**Design Driving/Restrike GWL Elev ==>**	868.0 feet	**Design Local Scour Elev ==>**	NA
Design Ultimate GS Elev ==>	869.8 feet	**Design Pre-Boring Elev ==>**	NA		

Pile Type and Size	WisDOT Maximum Rn_{dyn} (kips)	Design Steel Yield Stress (ksi)	Design Rn_{dyn} (kips)	Design P_r (kips)	Design Pile Tip Elevation* (feet)	Vertical Piles Driven to Design Rn_{dyn} At Time of Installation: Pile Driving Hammer	Pile Tip Elevation* (feet)	Pile Refusal** (Yes/No)	Soil Resistance During Driving* (kips)	Max Steel Stress During Driving* (ksi)	Ultimate Static Capacity*** (kips)
Factored Embankment Settlement Downdrag Load = 0 kips for 10¾Ø CIP.											
No reduction in the Design P_r is recommended due to embankment settlement downdrag loads. See report text.											
10¾Ø CIP 0.250Ŧ	260	35	260	130	842.7	D16-32	842.7	No	260	53.4	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.250Ŧ	260	45†	260	130	842.7	D16-32	842.7	No	260	53.4	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.365Ŧ	300	35	300	150	842.3	D16-32	842.3	No	300	53.9	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.365Ŧ	300	45†	300	150	842.3	D16-32	842.3	No	300	53.9	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.500Ŧ	300	35	300	150	842.3	D16-32	842.3	No	300	37.4	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.500Ŧ	300	45†	300	150	842.3	D16-32	842.3	No	300	37.4	317
Factored Embankment Settlement Downdrag Load = 0 kips for 10x42 HP.											
No reduction in the Design P_r is recommended due to embankment settlement downdrag loads. See report text.											
10x42 HP	360	50	360	180	840.8	D16-32	840.8	No	360	36.2	397

Table Notes
* Elevation, Soil Resistance During Driving, and Max(imum) Steel Stress During Driving values are considered estimates based on the soils encountered at the indicated boring.
** Pile Refusal = Specified pile driving hammer is estimated to require more than 120 blows per foot to drive the indicated steel pile.
*** Ultimate Static Capacity is the estimated capacity using static analyses that the indicated pile could achieve after pile set-up. The Design Rn_{dyn} and Design P_r should be used for design.
‡ Pile driven through upper dense layer(s) to reach design pile tip elevation. Maximum Computed Driving Rn_{dyn} {shown within brackets} for the upper dense layer(s) is less than or equal to the WisDOT Maximum Rn_{dyn} for the indicated pile.
★ Pile Tip Elevation for indicated pile was controlled by recommended 10-foot minimum pile embedment below pre-bore depth.
† Per Mr. Jeff Horsfall with WisDOT, the use of steel with a yield stress of 45 ksi for the shell of CIP pipe piles is acceptable to WisDOT.
✖ Design P_r was reduced due to low strength soil strata below the Design Pile Tip Elevation or due to a Design Ultimate GS Elevation that is less than the Design Driving GS Elevation.

Table Abbreviations and Symbols
NA = Not applicable.
ksi = kips per square inch.
HP = Steel H-pile.
f'_c = ultimate concrete strength in ksi.
Ŧ = -inch wall thickness.
P_r = Factored Axial Compression Resistance.
Ø CIP = -inch-outside-diameter, cast-in-place, steel pipe pile.
⊠ PCP = -inch-square, precast concrete pile.
Rn_{dyn} = Required Driving Resistance.

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Table 8-3: DEEP DRIVEN PILE FOUNDATION ANALYSES' RESULTS
APILE Static and *GRLWEAP* Drivability Analyses' Results
IH-39 North Segment
IH-39/90 NB Bridge over Femrite Drive
Illinois State Line - Madison
USH-12/18 Interchange
Town of Blooming Grove, Dane County, Wisconsin
WisDOT State ID 1007-10-02/05

Boring ==>	SB-10 located in the vicinity of North Abutment of Bridge Structure B-13-732				
Substructure Unit Foundation Bottom Elev ==>	879.7 feet	**Design Ultimate GWL Elev ==>**	868.0 feet	**Design Contraction Scour Elev ==>**	NA
Design Driving GS Elev ==>	882.2 feet	**Design Driving/Restrike GWL Elev ==>**	868.0 feet	**Design Local Scour Elev ==>**	NA
Design Ultimate GS Elev ==>	882.2 feet	**Design Pre-Boring Elev ==>**	NA		

Pile Type and Size	WisDOT Maximum Rn_{dyn} (kips)	Design Steel Yield Stress (ksi)	Design Rn_{dyn} (kips)	Design P_r (kips)	Design Pile Tip Elevation* (feet)	Vertical Piles Driven to Design Rn_{dyn} At Time of Installation: Pile Driving Hammer	Pile Tip Elevation* (feet)	Pile Refusal** (Yes/No)	Soil Resistance During Driving* (kips)	Max Steel Stress During Driving* (ksi)	Ultimate Static Capacity*** (kips)
Factored Embankment Settlement Downdrag Load = 0 kips for 10¾Ø CIP.											
No reduction in the Design P_r is recommended due to embankment settlement downdrag loads. See report text.											
10¾Ø CIP 0.250Ŧ	260	35	260	130	847.5	D16-32	847.5	No	260	55.3	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.250Ŧ	260	45†	260	130	847.5	D16-32	847.5	No	260	55.3	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.365Ŧ	300	35	300	150	846.9	D16-32	846.9	No	300	43.2	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.365Ŧ	300	45†	300	150	846.9	D16-32	846.9	No	300	43.2	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.500Ŧ	300	35	300	150	846.9	D16-32	846.9	No	300	36.4	*Not recommended. Max Stress > 90% Yield Stress*
10¾Ø CIP 0.500Ŧ	300	45†	300	150	846.9	D16-32	846.9	No	300	36.4	323
Factored Embankment Settlement Downdrag Load = 0 kips for 10x42 HP.											
No reduction in the Design P_r is recommended due to embankment settlement downdrag loads. See report text.											
10x42 HP	360	50	360	180	841.0	D16-32	841.0	No	360	33.8	441

Table Notes

* Elevation, Soil Resistance During Driving, and Max(imum) Steel Stress During Driving values are considered estimates based on the soils encountered at the indicated boring.
** Pile Refusal = Specified pile driving hammer is estimated to require more than 120 blows per foot to drive the indicated steel pile.
*** Ultimate Static Capacity is the estimated capacity using static analyses that the indicated pile could achieve after pile set-up. The Design Rn_{dyn} and Design P_r should be used for design.
‡ Pile driven through upper dense layer(s) to reach design pile tip elevation. Maximum Computed Driving Rn_{dyn} {shown within brackets} for the upper dense layer(s) is less than or equal to the WisDOT Maximum Rn_{dyn} for the indicated pile.
★ Pile Tip Elevation for indicated pile was controlled by recommended 10-foot minimum pile embedment below pre-bore depth.
† Per Mr. Jeff Horsfall with WisDOT, the use of steel with a yield stress of 45 ksi for the shell of CIP pipe piles is acceptable to WisDOT.
✖ Design P_r was reduced due to low strength soil strata below the Design Pile Tip Elevation or due to a Design Ultimate GS Elevation that is less than the Design Driving GS Elevation.

Table Abbreviations and Symbols

NA = Not applicable.
ksi = kips per square inch.
HP = Steel H-pile.
f'_c = ultimate concrete strength in ksi.
Ŧ = -inch wall thickness.
P_r = Factored Axial Compression Resistance.
Ø CIP = -inch-outside-diameter, cast-in-place, steel pipe pile.
⌀ PCP = -inch-square, precast concrete pile.
Rn_{dyn} = Required Driving Resistance.

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