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PROFESSIONAL
ENGINEER
OF
ILLINOIS

Robert Chantome
September 8, 2014
Exp 11/2015

Geotechnical Design Memorandum

F.A.I. Route 74
Section 81-1HB-1
Rock Island County
Job No. P-92-032-01
Contract No. 64C08
PTB No. N/A
I-74 Over 12th Avenue Bridges
Structure Nos. 081-0182 (WB) and 081-0183 (EB)

September 2014

Prepared for:
Illinois Department of Transportation, District 2
819 Depot Avenue
Dixon, Illinois 61021

Structure Designer:
Modjeski and Masters, Inc.
#4 Sunset Hills Professional Center
Edwardsville, Illinois 62088
(618) 659-9102

Prepared By:
Hanson Professional Services Inc.
1525 South Sixth Street
Springfield, Illinois 62703
(217) 788-2450

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Table of Contents

Tables

1. Project Description

This memorandum provides additional geotechnical data and recommendations for the proposed I-74 Over 12th Avenue Bridges, which are part of the Central Section of the I-74 over the Mississippi River Project.

This memorandum was prepared to address changes to the overall project staging and to IDOT pile design policies that have occurred since the Structure Geotechnical Report (SGR) was prepared. This memorandum supplements the SGR prepared by Hanson Professional Services Inc. in April 2012. Geotechnical evaluations and design recommendations in the SGR still should be considered valid, except as specifically referenced herein.

2. Proposed Structures

The proposed bridges will now be constructed in two stages instead of three. The stage line will be located between the WB and EB structures. The east side (WB I-74) will be constructed first, then the west side (EB I-74) will be constructed in the following year. The MSE walls beneath the bridges will follow a similar sequence.

3. Site Investigation

IDOT District 2 drilled three borings in August 2010, February 2011, and April 2011. Hanson was provided logs of these borings in May 2014.

All known borings are shown on the Boring Location Plan included in the Appendix. Logs of the three additional borings are included in the Appendix so that they may be added to the plans.

4. Subsurface Profile

The three additional borings encountered a profile similar to that found in the other nearby borings used to prepare the SGR.

5. Geotechnical Evaluations

Differential settlement is still anticipated near the proposed stage line. The current, single stage line at the joint between the two independent structures is preferable to the previous, multiple stage lines.

6. Design Recommendations

The IDOT Bureau of Bridges and Structures has requested that a project-specific pile design procedure be used for all bridges in the I-74 over the Mississippi River Project. This pile design procedure is expected to be adopted as official policy prior to construction of this project. Copies of the documents provided by IDOT are included in the Appendix.

Table 6.1 lists revised design parameters for the piles. Since final design of the structures has already been completed, design parameters are provided only for the actual pile loads and sizes being used. Piles should be driven through oversized sleeves as currently shown on the plans.

Table 6.1 Pile Design Parameters

Location	Cutoff Elevation (ft)	Pile Type	Factored Resistance Available, R_F (kips)	Geotech Losses, R_{Sdd} (kips)	Nominal Required Bearing, R_N (kips)	Estimated Pile Length (ft)	Soil Setup Pile Length (ft)
081-0182 (WB) North Abutment	670.1	HP 10x42	60	0	100	41	59
	670.1	HP 14x73	188	0	313	68	97
081-0182 (WB) South Abutment	671.9	HP 10x42	60	0	100	38	56
	671.9	HP 14x73	182	0	303	61	92
081-0183 (EB) North Abutment	677.1	HP 10x42	60	0	100	39	66
	676.3	HP 14x73	183	0	305	68	113
081-0183 (EB) South Abutment	677.9	HP 10x42	60	0	100	43	68
	677.9	HP 14x73	183	0	305	77	127

7. Construction Considerations

The first stage of construction will require top-down shoring for near-vertical cuts in the I-74 median. The height of this shoring exceeds the maximum values in the Bridge Manual's Design Guide 3.13.1 – Temporary Sheet Piling Design. The existing retaining wall between the bridge abutments will have a significant impact on the design of the shoring. A contractor-designed temporary wall is recommended. Guide Bridge Special Provision No. 44, Temporary Soil Retention System (Revised: May 11, 2009), should be included in the construction documents.

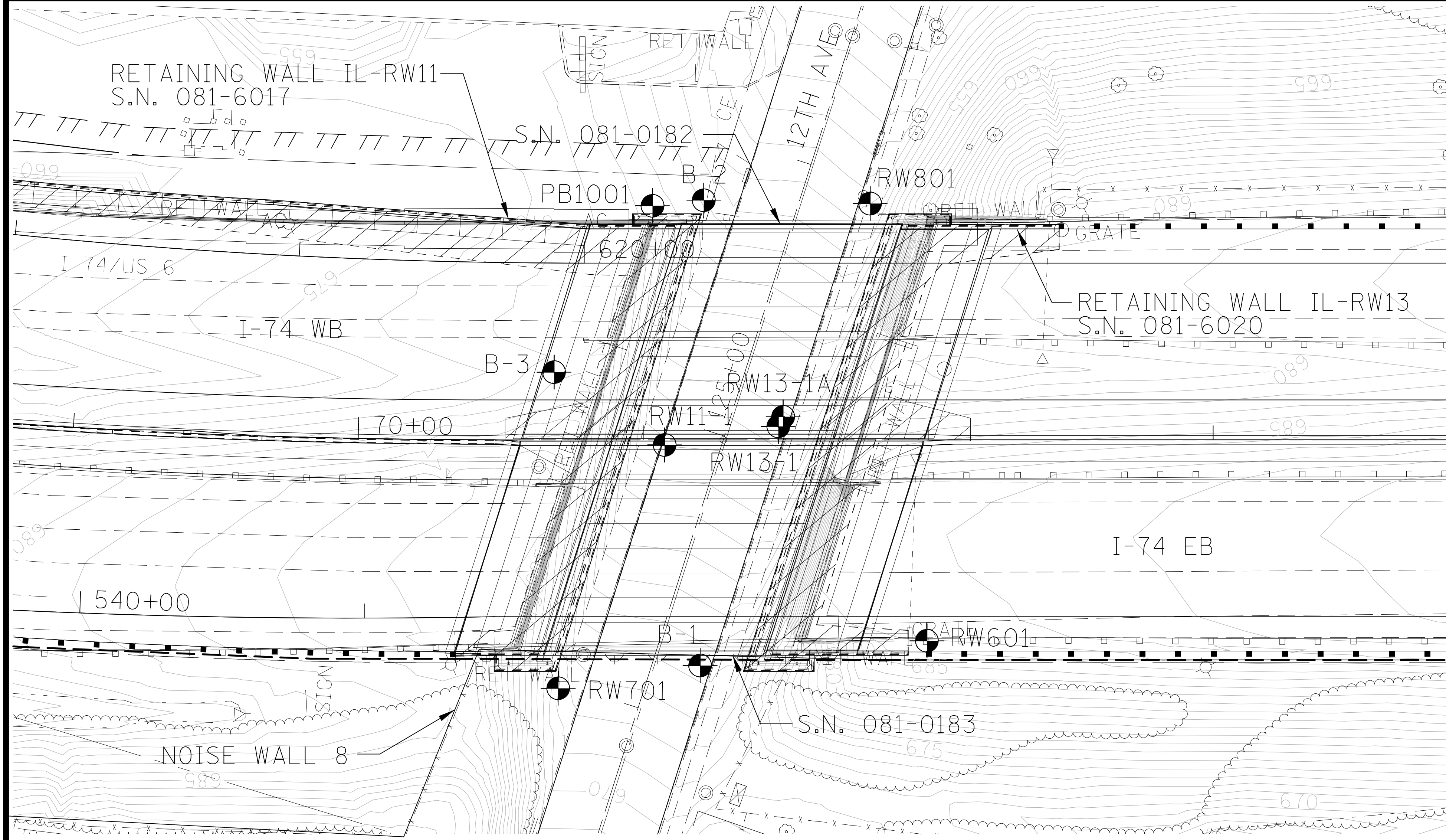
A piling special provision is required for structures that use the project-specific pile design procedure. A draft copy of this special provision is included in the Appendix.

References

Hanson Professional Services Inc. (2012, April). Structure Geotechnical Report, I-74 Over 12th Avenue Bridges, Structure Nos. 081-0182 (WB) and 081-0183 (EB).

Appendix

Boring Location Plan
Additional Boring Logs
I-74 over the Mississippi River Project Pile Design Procedure
Special Provisions



LEGEND

RW600 BORING LOCATION

0 50 100
SCALE IN FEET

N

HANSON

BORING LOCATION PLAN

I-74 OVER 12TH AVENUE
S.N. 081-0182 & 081-0183
ROCK ISLAND COUNTY, ILLINOIS

08H0120E

REV. 9/03/14

Illinois Department of Transportation

Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Date 8/30/10

ROUTE I-74 DESCRIPTION 081-0101, 0102 I-74 Bridge over 12th Avenue in Moline LOGGED BY W. Garza

SECTION ______ LOCATION Moline Twp. - 4NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island DRILLING METHOD Hollow Stem Auger HAMMER TYPE B-53 Diedrich Automatic

STRUCT. NO. 081-0101, 0102
Station ______

BORING NO. B-1
Station 121+88
Offset 32.00ft Lt CL
Ground Surface Elev. 663.1 ft

Surface Water Elev. ______ ft
Stream Bed Elev. ______ ft

Groundwater Elev.:
First Encounter ______ ft
Upon Completion ______ ft
After ______ **Hrs.** ______ ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Asphalt parking lot					
	660.60				
STIFF tan SILT	659.10		8 12 13	1.3 S	20
		-5			
MEDIUM tan SILT with SAND lens	656.10		10 11 16	0.7 B	21
HARD gray SILTY CLAY LOAM TILL	654.10		6 9 13	6.2 B	11
		-10			
VERY STIFF gray SILTY CLAY LOAM TILL	651.60		6 8 11	3.3 B	13
VERY STIFF gray SILTY CLAY LOAM TILL	649.10		3 5 8	3.5 B	13
		-15			
VERY STIFF gray SILTY CLAY LOAM TILL	646.60		3 6 8	2.7 B	14
VERY STIFF gray SILTY CLAY LOAM TILL	644.10		2 5 7	3.1 B	13
		-20			

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray SILTY CLAY LOAM TILL	641.60		3 5 9	2.7 B	14
VERY STIFF gray SILTY CLAY LOAM TILL	639.10		3 5 8	2.1 B	13
		-25			
VERY STIFF gray SILTY CLAY LOAM TILL	636.60		3 5 9	2.5 B	14
VERY STIFF gray SILTY CLAY LOAM TILL	634.10		2 5 10	3.3 B	13
		-30			
VERY STIFF olive-green CLAY LOAM TILL with SAND lens	631.60		2 5 8	2.9 B	17
VERY STIFF gray CLAY LOAM TILL	629.10		3 7 10	3.5 B	14
		-35			
VERY STIFF gray CLAY LOAM TILL	626.60		3 5 9	2.9 B	13
VERY STIFF gray CLAY LOAM TILL	624.10		3 6 11	2.9 B	14
		-40			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206)

Illinois Department of Transportation

Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Date 8/30/10

ROUTE I-74 DESCRIPTION 081-0101, 0102 I-74 Bridge over 12th Avenue in Moline LOGGED BY W. Garza

SECTION ______ LOCATION Moline Twp. - 4NW, SEC. , TWP. 17N, RNG. 1W

COUNTY Rock Island DRILLING METHOD Hollow Stem Auger HAMMER TYPE B-53 Diedrich Automatic

STRUCT. NO. 081-0101, 0102
Station ______

BORING NO. B-1
Station 121+88
Offset 32.00ft Lt CL
Ground Surface Elev. 663.1 ft

Surface Water Elev. ______ ft
Stream Bed Elev. ______ ft

Groundwater Elev.:
First Encounter ______ ft
Upon Completion ______ ft
After ______ Hrs. ______ ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	621.60		3 7 11	3.3 B	14
VERY STIFF gray CLAY LOAM TILL	619.10		3 7 9	3.1 B	14
		-45			
VERY STIFF gray CLAY LOAM TILL	616.60		4 8 11	3.7 B	16
VERY STIFF gray CLAY LOAM TILL	614.10		3 6 10	3.5 B	14
		-50			
HARD tan/gray CLAY LOAM TILL	611.60		4 10 14	5.2 B	12
VERY STIFF gray CLAY LOAM TILL	609.10		3 5 9	2.7 B	14
		-55			
VERY STIFF gray CLAY LOAM TILL	606.60		2 6 10	2.5 B	15
9/3/10 VERY STIFF gray CLAY LOAM TILL	604.10		3 6 10	2.3 B	13
		-60			
VERY STIFF gray CLAY LOAM TILL	601.60		5 7 10	2.7 B	14
VERY STIFF gray CLAY LOAM TILL	599.10		3 6 10	2.7 B	14
		-65			
VERY STIFF gray CLAY LOAM TILL	596.60		4 7 10	2.9 B	13
End of Boring					
		-70			
		-75			
		-80			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Date 2/16/11

ROUTE I-74 DESCRIPTION 081-0101, 0102 I-74 Bridge over 12th Avenue in Moline LOGGED BY W. Garza

SECTION ______ LOCATION Moline Twp. - 4NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island DRILLING METHOD Hollow Stem Auger HAMMER TYPE B-53 Diedrich Automatic

STRUCT. NO. 081-0101, 0102
Station 121+20

BORING NO. B-2
Station 120+32
Offset 16.00ft Rt CL
Ground Surface Elev. 673.8 ft

Surface Water Elev. ______ ft
Stream Bed Elev. ______ ft

Groundwater Elev.:
First Encounter 626.3 ft
Upon Completion 584.8 ft
After 24 Hrs. 618.8 ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Pavement 9" Concrete, 3" Asphalt	671.30				
STIFF tan SILTY LOAM	669.80		1 2 4	1.1 P	23
STIFF light gray SILTY LOAM	667.30	-5	3 5 5	1.7 S	25
VERY STIFF gray/tan SILTY CLAY LOAM	664.80		2 3 5	2.0 B	25
VERY STIFF gray/tan SILTY CLAY LOAM	662.30	-10	2 3 6	2.0 B	23
SOFT tan SANDY LOAM	659.30		1 3 4	0.3 P	19
STIFF gray LOAM TILL	657.30	-15	1 5 7	1.3 B	16
VERY STIFF gray LOAM TILL	654.30	-20	6 8 12	2.3 P	12
DENSE gray dirty SAND with GRAVEL	651.80		15 18 16		8
VERY STIFF gray CLAY LOAM TILL	649.80		8 7 10	2.3 B	13
VERY STIFF gray CLAY LOAM TILL	647.30	-25	4 6 9	2.1 B	15
VERY STIFF gray CLAY LOAM TILL	644.80		2 4 7	2.1 B	14
VERY STIFF gray CLAY LOAM TILL	642.30	-30	3 6 9	2.5 B	14
VERY STIFF gray CLAY LOAM TILL	639.80		2 5 9	2.3 B	14
VERY STIFF gray CLAY LOAM TILL	637.30	-35	3 6 10	3.1 B	13
VERY STIFF gray CLAY LOAM TILL	634.80	-40	3 5 10	2.3 B	14

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
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STRUCT. NO. 081-0101, 0102
Station 121+20

BORING NO. B-2
Station 120+32
Offset 16.00ft Rt CL
Ground Surface Elev. 673.8 ft

Surface Water Elev. ft
Stream Bed Elev. ft

Groundwater Elev.:
First Encounter 626.3 ft ▼
Upon Completion 584.8 ft ▽
After 24 Hrs. 618.8 ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	632.30		4 7 11	3.3 B	14
VERY STIFF gray CLAY LOAM TILL	629.80	-45	3 6 10	2.5 B	14
VERY STIFF gray CLAY LOAM TILL	627.30		3 5 10	3.5 B	13
VERY STIFF gray CLAY LOAM TILL	624.80	▼ -50	4 7 12	3.5 B	14
VERY STIFF gray CLAY LOAM TILL	622.30		5 9 13	3.1 B	14
VERY STIFF gray CLAY LOAM TILL	619.80	-55	4 6 12	2.1 B	16
VERY STIFF gray CLAY LOAM TILL	617.30		5 8 13	2.9 B	16
VERY STIFF gray CLAY LOAM TILL	614.80	-60	5 7 12	3.1 B	14

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	612.30		5 8 14	3.3 B	15
VERY STIFF gray CLAY LOAM TILL	609.80	-65	4 8 12	3.1 B	14
VERY STIFF gray CLAY LOAM TILL	607.30		5 7 14	3.7 B	14
2/17/11 VERY STIFF gray CLAY LOAM TILL	604.80	-70	7 10 13	2.9 B	15
VERY STIFF gray CLAY LOAM TILL	602.30		7 9 13	3.1 B	15
VERY STIFF gray CLAY LOAM TILL	599.80	-75	6 9 13	2.9 B	15
VERY STIFF gray CLAY LOAM TILL	597.30		9 12 16	2.9 B	15
VERY STIFF gray CLAY LOAM TILL	594.80	-80	7 9 14	2.9 B	15

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206)

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SOIL BORING LOG

Date 2/16/11

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SECTION ______ LOCATION Moline Twp. - 4NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island DRILLING METHOD Hollow Stem Auger HAMMER TYPE B-53 Diedrich Automatic

STRUCT. NO. 081-0101, 0102
Station 121+20

BORING NO. B-2
Station 120+32
Offset 16.00ft Rt CL
Ground Surface Elev. 673.8 ft

Surface Water Elev. ______ ft
Stream Bed Elev. ______ ft

Groundwater Elev.:
First Encounter 626.3 ft ▼
Upon Completion 584.8 ft ▽
After 24 **Hrs.** 618.8 ft ▽

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	592.30		7 10 14	3.1 B	15
VERY STIFF gray CLAY LOAM TILL	589.80	-85	4 8 11	2.9 B	15
VERY STIFF gray CLAY LOAM TILL	587.30		5 9 13	2.9 B	15
VERY STIFF gray CLAY LOAM TILL	584.80 ▽	-90	7 10 15	3.3 B	16
VERY STIFF gray CLAY LOAM TILL	582.30		5 8 12	2.9 B	15
End of Boring		-95 -100			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206)

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SOIL BORING LOG

Date 4/7/11

ROUTE I-74 **DESCRIPTION** 081-0101, 0102 I-74 Bridge over 12th Avenue in Moline **LOGGED BY** W. Garza

SECTION ______ **LOCATION** Moline Twp. - 4NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** B-53 Diedrich Automatic

STRUCT. NO. 081-0101, 0102
Station ______

BORING NO. B-3
Station 299+62
Offset 14.00ft Lt CL
Ground Surface Elev. 685.6 ft

Surface Water Elev. ______ ft
Stream Bed Elev. ______ ft

Groundwater Elev.:
First Encounter None ft
Upon Completion Dry ft
After ____ **Hrs.** ______ ft

Description	Bottom Elev. (ft)	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
STIFF brown SILTY CLAY LOAM	683.10			1.1 P	14
STIFF gray LOAM	681.60		6 7 10	1.5 B	13
STIFF tan SILTY CLAY LOAM	679.10	-5	3 6 7	1.7 B	14
STIFF tan/gray SILTY CLAY LOAM	676.60		1 2 4	1.3 P	15
VERY STIFF tan SILTY CLAY LOAM	674.10	-10	5 5 9	3.1 B	15
VERY STIFF light gray SILTY LOAM	671.60		6 12 11	3.5 P	18
SOFT light gray SILT	669.10	-15	2 2 5	0.3 B	24
VERY STIFF brown CLAY LOAM	666.60		1 4 6	2.1 B	23
		-20			
STIFF light brown SILTY CLAY	664.10		2 4 8	1.6 B	22
MEDIUM light brown SILTY CLAY	661.60		1 3 9	0.7 B	18
STIFF light brown SANDY LOAM	659.10	-25	5 7 10	1.2 S	13
STIFF tan SANDY LOAM	656.10		5 12 18	1.6 S	10
VERY STIFF tan LOAM TILL	654.10	-30	5 11 15	2.9 B	12
VERY STIFF gray CLAY LOAM TILL	651.60		7 10 12	2.3 P	13
No Recovery	649.10	-35	20 23 26		
VERY STIFF gray CLAY LOAM TILL	646.60		5 6 12	2.3 B	13
		-40			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206)

Illinois Department of Transportation
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SOIL BORING LOG

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COUNTY Rock Island DRILLING METHOD Hollow Stem Auger HAMMER TYPE B-53 Diedrich Automatic

STRUCT. NO. 081-0101, 0102
Station ____

BORING NO. B-3
Station 299+62
Offset 14.00ft Lt CL
Ground Surface Elev. 685.6 ft

Surface Water Elev. ____ ft
Stream Bed Elev. ____ ft

Groundwater Elev.:
First Encounter None ft
Upon Completion Dry ft
After ____ Hrs. ____ ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	644.10		2 4 12	2.7 B	13
VERY STIFF gray CLAY LOAM TILL	641.60		4 8 11	3.1 P	13
		-45			
VERY STIFF gray CLAY LOAM TILL	639.10		9 9 14	3.5 B	12
VERY STIFF gray CLAY LOAM TILL	636.60		4 9 11	3.5 B	12
		-50			
VERY STIFF gray CLAY LOAM TILL	634.10		5 9 14	3.5 P	14
HARD gray CLAY LOAM TILL	631.60		13 9 14	4.1 B	13
		-55			
VERY STIFF gray CLAY LOAM TILL	629.10		7 12 18	3.3 B	13
VERY STIFF gray CLAY LOAM TILL	626.60		6 10 14	2.5 P	13
		-60			

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
No Recovery	624.10		8 13 16		
VERY STIFF gray CLAY LOAM TILL	621.60		3 7 12	2.9 B	14
		-65			
VERY STIFF gray CLAY LOAM TILL	618.10		4 9 14	3.5 B	14
HARD gray CLAY LOAM TILL	616.60		3 9 13	4.5 B	12
		-70			
HARD gray CLAY LOAM TILL	614.10		4 8 13	4.3 B	14
HARD gray CLAY LOAM TILL	611.60		8 11 16	4.1 B	14
		-75			
No Recovery	609.10		4 10 12		
VERY STIFF gray CLAY LOAM TILL	606.60		3 7 12	3.7 B	14
		-80			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
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STRUCT. NO. 081-0101, 0102
Station

BORING NO. B-3
Station 299+62
Offset 14.00ft Lt CL
Ground Surface Elev. 685.6 ft

Surface Water Elev. ft
Stream Bed Elev. ft

Groundwater Elev.:
First Encounter None ft
Upon Completion Dry ft
After Hrs. ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL			4 9 14	2.3 B	14
	604.10				
VERY STIFF gray CLAY LOAM TILL			4 9 12	3.3 B	14
	601.60				
		-85			
VERY STIFF gray CLAY LOAM TILL			2 8 13	3.1 B	14
	599.10				
End of Boring					
		-90			
		-95			
		-100			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206)

Tasks: Task 868: Re-evaluation of the Illinois Viaduct Pile Design

The content of this item will be sent as an e-mail message to the person or group assigned to the item.

New Item | Edit Item | Delete Item | Workflows | Alert Me | Version History

Title	Task 868: Re-evaluation of the Illinois Viaduct Pile Design
Priority	(2) Normal
Status	Completed
% Complete	100%
Assigned To	David Morrill
Description	Following the FHWA Geotechnical Review Meeting conducted on September 11, 2013, Bill Kramer provided David an email containing additional discussions regarding the FHWA comment on the pile design and construction for the structures in Illinois to be built using the Illinois IDOT spec book and BBS Bridge Manual. Bill suggested that the Benesch Team recheck the piles using an increased resistance factor of 0.60 for piles in soil, 0.65 for H-piles on shale, and 0.70 for H-piles on rock, rather than using 0.55 for all conditions. In addition, the maximum nominal bearing that can be specified for H-piles would increase from 54% to 65% of the H-pile yield strength times its cross-sectional area. To use these increased design values, Bill provided a Guide Bridge Special provision (GBSP) that would be added to the contract plans to assure the piles are not overdriven. Bill also suggested the Benesch Team run some design phase wave equation analysis to verify the pile can be driven to the rock with the hammer size limitations in the GBSP and not overstress the pile in the process. This task is assigned to Andrew to review Bill Kramer's September 11 e-mail and prepare a disposition of Bill's comments and outline the appropriate steps to be taken for the Illinois viaduct pile design.
Start Date	9/23/2013
Due Date	10/7/2013
Carbon Copy	Hossam A. Abdou; Ahmad Abu-Hawash; Robert Chantome; Chris Cromwell; Timothy Dunlay; Andrew J. Keaschall; John M. Kulicki; Rebecca A. Marruffo; Norm McDonald; Todd B. McMeans; Ron Meyer; David Morrill; Thomas P. Murphy; Kevin Placzek; Andrew Wilson; Bob Stanley; Philip A. Ritchie; Robert J. Tipton; Mark Thomson; Sheila Moynihan; Jerilyn M. Hassard; David W. Petermeier
Comments	10/23/13 David Morrill - per Mark Thomson's post below, the new pile design criteria outlined above will be reflected in the calculations and drawings for the I-74 Illinois Viaduct and associated Ramps C and D and for the I-74 and Ramp 7th A structures over 19th street and the I-74 structures over 12th Avenue. The intent of this task has been fulfilled and it is hereby closed. 10/23/13: Mark Thomson: IDOT BB&S is moving forward with plans to revise

IDOT's piling policies in the near future and these changes should be in place well before construction of this project takes place. BB&S agrees that the design team should incorporate the piling changes on the IL structures for this project. As noted, it is anticipated that the structure plans will be revised along with the work to incorporate revisions for changing to the 3B staging option. Revised plans will require BB&S review and approval. If there are any questions, please contact this office. This task is assigned back to Benesch to incorporate the changes.

10/15/2013: Andrew J. Keaschall: The piers and abutments for the proposed Illinois Viaduct and ramp structures are supported on piles driven to rock. The piles at the abutments range in length from 35 to 45 feet and most of them are driven prior to placement of embankment. The piles at the piers range in length from 10 feet to 25 feet. The strata overlying the bedrock varies over the length of the viaduct from soft clayey silts to loose sandy gravels. Pile installation in this area is likely to be very simple in the early stages and will likely be controlled by the special provision phrase "For piles driven to rock, pile driving shall be stopped, independent of the nominal driven bearing predicted by the formula in Article 512.14, when the minimum penetration rate is ¼ in. over 5 blows (or equivalently a maximum penetration rate of 20 blows per 1 in. for no more than 5 blows)." Based on these parameters, the design phase WEAP analysis is likely not required for this particular situation.

We would like to take advantage of the additional capacity available with the proposed modifications to Illinois DOT's pile capacity and GBSP (documents attached). Typically we have found the most efficient pile configuration is one that reduces the overall number of piles based on geometric constraints and then selects a pile that has adequate capacity for that configuration. The design team followed this methodology (even using HP 14x117 in a few places) and maximized the pile spacing while minimizing the number of different pile sections used. Therefore, potential savings associated with pile reconfiguration are likely to be minimal, however, across the board, the pile size can be reduced (in many cases by two sizes).

There are approximately 12,000 linear feet of pile on the Illinois viaduct and associated Ramp C and D. Incorporating the new pile methodology would result in a savings of about 25 pounds per foot of pile (on average) for a total weight savings of 300,000 lbs. This reduction in weight would result in a cost savings of approximately $150,000 for these structures.

With the Illinois DOT's approval, this change will be incorporated for the viaduct. Final plans will be re-submitted as a result of incorporating the Option 3B construction schedule revision and will reflect the updated pile sections with their associated NRB and FRA values.

The structures over 19th street, 12th Avenue and Ramp 7th A over 19th Street have to be re-designed as a result of the Option 3B MOT modifications. Again, with IDOT approval, the updated pile design procedure will be incorporated into the re-design.

This task is re-assigned to Mark Thomson of IDOT for review and discussion with Bill Kramer to provide direction on implementing the new pile criteria.

Attachments	IDOT Pile Design and Construction changes.docx Piling GBSP (WHKS Rev 9-4-13).docx

Version: 5.0
Created at 9/23/2013 11:34 AM by Diane M. Campione
Last modified at 10/23/2013 7:20 PM by David Morrill

IDOT pile design and construction changes proposed for implementation in 2013

1. **New larger H-pile and Metal Shell (MS)** pile sizes will be allowed to be used in design and specified on plans. The following is a list of our current and new pile sizes which will be available.

New piles to be added:	Current piles to remain available:
	Metal Shell 12"Φ w/.179" walls
Metal Shell 16"Φ w/.312" walls	Metal Shell 12"Φ w/.25" walls
Metal Shell 16"Φ w/.375" walls	Metal Shell 14"Φ w/.25" walls
Steel HP 16 X 88	Metal Shell 14"Φ w/.312" walls
Steel HP 16 X 101	Steel HP 8 X 36
Steel HP 16 X 121	Steel HP 10 X 42
Steel HP 16 X 141	Steel HP 10 X 57
Steel HP 16 X 162	Steel HP 12 X 53
Steel HP 16 X 183	Steel HP 12 X 63
Steel HP 18 X 135	Steel HP 12 X 74
Steel HP 18 X 157	Steel HP 12 X 84
Steel HP 18 X 181	Steel HP 14 X 73
Steel HP 18 X 204	Steel HP 14 X 89
	Steel HP 14 X 102
	Steel HP 14 X 117

2. The yield strength **(fy) of Metal Shell piles will be increased from 45ksi to 50ksi** (ASTM A-252 Grade 3 Modified). This will result in a 10% increase in the maximum nominal bearing that can be specified since it is currently computed by taking 85% of the shell yield strength times its steel cross-sectional area)

3. Piles designed using the WSDOT driving formula as construction bearing acceptance will use an **increased resistance factor of 0.60 for piles in soil, 0.65 for H-piles on shale, and 0.7 for H-piles on other rock, rather than 0.55** for all conditions.

4. The **maximum nominal bearing that can be specified for H-piles will increase from 54% to 65%** of the H-pile yield strength times its cross-sectional area. This will result in a 20% increase in the maximum nominal bearing that can be specified.

5. **A new "Soil Setup Pile Length" will be shown on the plans, in addition to the "Estimated Pile Length" currently shown.** While the Estimated pile length is determined using the IDOT Static Method of estimating pile length with the resistance factor for the WSDOT field verification formula (0.6), the setup length is determined using the resistance factor for the IDOT Static Method (0.3). This longer setup length provides theoretically the depth at which pile driving can be stopped and the pile accepted as having capacity without further verification, even though the WSDOT formula does not show bearing. However, accepting the soil setup length pile capacities independent of field bearing verification requires that quality soils boring data is available within 75' of the substructure. Therefore, until we become confident this length consistently provides capacity, piles within 85% of plan bearing will be allowed to setup for at least 24 hours while others must be left for a minimum of 48 hours and re-tapped to verify bearing. A table with longer recommended waiting times based on soil type has been included in the specification so it is understood that the capacities at minimum 24 or 48 hours do not reflect the full setup possible.

6. The **WSDOT dynamic formula will include a new Cs factor which will equal 0.8 when re-tapping** a pile to check for setup capacity gain after a waiting period and 1.0 at all other times. The WSDOT formula was developed to predict long term pile capacity at the end of initial driving and thus includes the average setup expected. When using this formula to check for the actual setup at a specific site, the average setup must be removed from the formula which is done by reducing its capacity by 20% (multiplying by 0.8).

7. **Reduced hammers energy requirements will be added to the specification for piles driven to rock.** This new range of acceptable hammer sizes is based on the WSDOT formula, plan bearing and penetration rates between 4 and 20 blows/inch. Driving can be stopped when the formula shows bearing or when the penetration rate is < ¼ in. over 5 blows for no more than 5 blows, whichever occurs first. Test piles driven to rock will only be required to be driven to plan bearing, not 110% of plan bearing. The current hammer energy criteria (based on the WSDOT formula, plan bearing and penetration rates between 1 and 10 blows/inch) will be retained but only used for piles driven in soil.

Soil (current)

$$E \geq \frac{32.9\ R_N}{F_{eff}}$$

$$E \leq \frac{65.8\ R_N}{F_{eff}}$$

Rock (new)

$$E \geq \frac{\mathbf{28.6\ R_N}}{\mathbf{F_{eff}}}$$

$$E \leq \frac{\mathbf{41.1\ R_N}}{\mathbf{F_{eff}}}$$

8. **A new Simplified Stress Formula (SSF) has been developed to estimate pile stresses during driving.** Designers will now be able to estimate pile stress, considering the specific soil conditions, and avoid the use of those which indicate possible damage during driving. The SSF can also be used by contractors or inspectors to evaluate various hammers being considered and avoid the use of those which indicate possible pile damage. The SSF has been added to our static method of estimating pile length and the WSDOT Pile Bearing Verification spreadsheets. Unacceptable risk of pile damage is defined as SSF estimated stress levels > 90 % of the pile yield strength.

		SOIL		ROCK	
Pile Type & Size	**Max. Nominal Required Bearing (kips)**	**Maximum Hammer Size (Kip-ft)**	**Minimum Hammer Size (Kip-ft)**	**Maximum Hammer Size (Kip-ft)**	**Minimum Hammer Size (Kip-ft)**
Metal Shell 12"Φ w/.179" walls	283	39568	19784		
Metal Shell 12"Φ w/.25" walls	392	54919	27460		
Metal Shell 14"Φ w/.25" walls	459	64260	32130		
Metal Shell 14"Φ w/.312" walls	570	79849	39925		
Metal Shell 16"Φ w/.312" walls	654	91493	45746		
Metal Shell 16"Φ w/.375" walls	782	109526	54763		
Steel HP 8 X 36	344	48223	24112	30101	20960
Steel HP 10 X 42	403	56413	28207	35214	24520
Steel HP 10 X 57	546	76462	38231	47729	33234
Steel HP 12 X 53	504	70500	35250	44007	30643
Steel HP 12 X 63	598	83734	41867	52269	36395
Steel HP 12 X 74	709	99197	49599	61921	43116
Steel HP 12 X 84	799	111908	55954	69855	48641
Steel HP 14 X 73	695	97363	48682	60775	42319
Steel HP 14 X 89	848	118787	59394	74149	51631
Steel HP 14 X 102	975	136478	68239	85192	59320
Steel HP 14 X 117	1118	156527	78264	97707	68035
Steel HP 16 X 88	839	117390	58695	73277	51024
Steel HP 16 X 101	972	136045	68023	84922	59132
Steel HP 16 X 121	1164	162890	81445	101679	70800
Steel HP 16 X 141	1355	189735	94868	118436	82468
Steel HP 16 X 162	1550	217035	108518	135477	94334
Steel HP 16 X 183	1758	246155	123078	153654	106991
Steel HP 18 X 135	1297	181545	90773	113324	78909
Steel HP 18 X 157	1502	210210	105105	131217	91368
Steel HP 18 X 181	1729	242060	121030	151098	105211
Steel HP 18 X 204	1957	273910	136955	170979	119055

Axial Geotechnical Resistance Design of Driven Piles

This Design Guide has been developed to provide geotechnical and structural engineers with the most recent methods and procedures required by the Department to determine the nominal and factored axial geotechnical resistance of a pile to help ensure cost effective foundation design and construction.

The Geotechnical Engineer must evaluate the subsurface soil/rock profile, develop pile design table(s) for each substructure, and provide them to the structure designer in the Structure Geotechnical Report (SGR). Each table shall contain a series of Nominal Required Bearing (R_N) values, the corresponding Factored Resistances Available (R_F) for design, the Estimated Pile Lengths, and the Soil Setup Pile Lengths, for all feasible pile types. The number of pile types and sizes covered as well as the range of R_N values provided must be large enough to allow the designer sufficient selection to determine the most economical pile type, size and layout such that the factored loading from the LRFD Strength Limit State and Extreme Event Load Combinations is $\leq R_F$. The corresponding R_N provided on the plans will typically be obtained during driving as indicated by dynamic formula or other nominal pile resistance field verification method. To develop the pile design tables, the geotechnical engineer shall use the IDOT Static Method of estimating this nominal pile resistance during driving and provide these values in the SGR as feasible R_N values which can be specified by the designer.

The original IDOT Static Method was developed over 40 years ago to correspond to the allowable pile resistance indicated during driving by the ENR dynamic formula. With the change to LRFD and FHWA Gates formula in 2007, the Department initiated an extensive research study with Dr. James Long of the University of Illinois at Urbana-Champaign to evaluate several static methods and dynamic formulas to determine the most accurate method for estimating pile lengths and resistances for the soils, piles, and hammers common to Illinois. The results of Phase 1 of the research, completed in 2009, indicated that an updated IDOT Static Method (with the new Pile Type Correction Factors) was more accurate than all other static estimating methods studied, including the program “DRIVEN”. It was also found to correspond closest to the most accurate dynamic formula studied which was the WSDOT formula, developed by Tony Allen of the Washington State DOT in 2005. Based on this research, the WSDOT formula was chosen to replace the FHWA Gates formula as the standard method of construction verification with the IDOT Static Method, described below, chosen for use in developing the SGR pile design tables. Phase 2 of the U of I research was completed in 2012 and included the acquisition of additional pile driving analyzer data to further improve correlation of the static and dynamic methods, increase pile capacity, identify potential for pile damage, and provide procedures to prevent piles from running excessively long. The design guide has been subsequently updated to reflect these improvements.

Nominal Required Bearing (R_N) represents the nominal pile resistance expected at any specific length during driving that can be specified by the Designer. It must be calculated at various estimated lengths and is the first step in developing the pile design table.

In the case of displacement piles (such as metal shell, precast, and timber piles), R_N shall be calculated as the sum of the side and tip resistance as follows:

$$R_N = (F_S q_S A_{SA} + F_P q_P A_P)*(I_G)$$

Where the nominal side resistance ($F_S q_S A_{SA}$) is the product of the following:

F_S = The pile type correction factor for side resistance (0.758 for displacement piles in cohesionless soils & 1.174 for displacement piles in cohesive soils)
q_S = The nominal unit side resistance
A_{SA} = The surface area of the pile

And the nominal tip resistance ($F_P q_P A_P$) is the product of the following:

F_P = The pile type correction factor for tip resistance (0.758 for displacement piles in cohesionless soils & 1.174 for displacement piles in cohesive soils)
q_P = The nominal unit tip resistance
A_P = The tip area of the pile

In the case of non-displacement piles (such as steel H-piles), the R_N shall be taken as the lesser of the following:

The fully "plugged" side and tip resistance defined as:

$$R_N = (F_S q_S A_{SAp} + F_P q_P A_{Pp})*(I_G)$$

And the fully "unplugged" side and tip resistance defined as:

$$R_N = (F_S q_S A_{SAu} + F_P q_P A_{Pu})*(I_G)$$

Where:

F_S = The pile type correction factor for side resistance (0.15 for non-displacement piles in cohesionless soils, 0.75 for non-displacement piles in cohesive soils & 1.0 for non-displacement piles in rock)

F_P = The pile type correction factor for tip resistance (0.3 for non-displacement piles in cohesionless soils, 1.5 for non-displacement piles in cohesive soils & 1.0 for non-displacement piles in rock)

A_{SAu} = The unplugged surface area = (4 x flange width + 2 x member depth) x pile length

A_{SAp} = The plugged surface area = (2 x flange width + 2 x member depth) x pile length

A_{Pu} = The cross-sectional area of steel member

A_{Pp} = The flange width x member depth

In the above equations, the term I_G is the bias factor ratio (equal to 0.87 for soil and 1.0 for rock) and is discussed in further detail later in the design guide. The Nominal Unit Side Resistance (q_S) and Nominal Unit Tip Resistance (q_P) shall be calculated as follows:

- *Nominal Unit Side Resistance* (q_S) of **granular soils** is computed using the equations below:

For Hard Till, the equations below are used for the range of N values indicated:

$q_S = 0.07N$ for $N < 30$

$q_S = 0.00136N^2 - 0.00888N + 1.13$ for $N \geq 30$

Very Fine Silty Sand, the equations below are used for the range of N values indicated:

$q_S = 0.1N$ for $N < 30$

$q_S = 42.58e^{\left[\frac{(N-175.05)^2}{-7944}\right]}$ for $30 \leq N < 74$

$q_S = 0.297N - 10.2$ for $N \geq 74$

Fine Sand, the equations below are used for the range of N values indicated:

$q_S = 0.11N$ for $N < 30$

$q_S = 0.3256N + \frac{182}{N} - 12.51$ for $30 \leq N < 66$

$q_S = 0.329N - 9.91$ for $N \geq 66$

Medium Sand, the equations below are used for the range of N values indicated:

$q_S = 0.117N$	for $N < 26$
$q_S = 0.00404N^2 - 0.0697N + 2.13$	for $26 \leq N < 55$
$q_S = 0.356N - 9.1$	for $N \geq 55$

Clean Coarse Sand, the equations below are used for the range of N values indicated:

$q_S = 0.128N$	for $N < 24$
$q_S = 0.00468N^2 - 0.0693N + 2.05$	for $24 \leq N < 50$
$q_S = 0.394N - 9.42$	for $N \geq 50$

Sandy Gravel, the equations below are used for the range of N values indicated:

$q_S = 0.129N$	for $N < 20$
$q_S = 0.0074N^2 - 0.187N + 3.36$	for $20 \leq N < 40$
$q_S = 0.52N - 12.9$	for $N \geq 40$

Where N = Field measured SPT blow count (blows/ft)

- *Nominal Unit Side Resistance* (q_S) of **cohesive soils**, shall be calculated using the equations below for the range of Q_U values indicated:

$q_S = \frac{-1}{2500}Q_u^3 - 0.177Q_u^2 + 1.09Q_u$	for $Q_u \leq 1.5$ tsf
$q_S = 0.0495Q_u^3 - 0.347Q_u^2 + 1.278Q_u - 0.068$	for 1.5 tsf $< Q_u <$ 2 tsf
$q_S = 0.47Q_u + 0.555$	for 2 tsf $\leq Q_u <$ 4.5 tsf
$q_S = 2.67$ ksf	for 4.5 tsf $\leq Q_u$

Where Q_u = Unconfined compression strength of the soil in tsf.

Note that Q_u is input in tsf and q_S is output in ksf.

If $Q_u > 3$ tsf and $N > 30$, treat as granular and use Hard Till equations.

- *Nominal Unit Side Resistance* (q_S) of **rock**, shall be calculated using the equations below for the type of rock encountered:

$q_S = 12.0$ ksf	for Shale
$q_S = 20.0$ ksf	for Sandstone
$q_S = 24.0$ ksf	for Limestone/Dolomite

- *Nominal Unit Tip Resistance* (q_P) of **granular soils,** shall be calculated as follows:

$$q_P = \frac{0.8\,N\,D_b}{D} \le q_\ell$$

Where:

q_ℓ = 8N for sands and gravel

q_ℓ = 6N for fine silty sand and hard till

D = Pile diameter or width (ft)

D_b = Depth of penetration into soil (ft)

N = Field measured SPT blow count (blows/ft)

- *Nominal Unit Tip Resistance* (q_P) of **cohesive soils,** shall be calculated as follows:

$$q_P = 9Q_u$$

Note that Q_u is input in tsf and q_P is output in ksf.

- *Nominal Unit Tip Resistance* (q_P) of **rock,** shall be calculated using the equations below for the type of rock encountered:

q_P = 120.0 ksf	for Shale
q_P = 200.0 ksf	for Sandstone
q_P = 240.0 ksf	for Limestone/Dolomite

Note that actual pile penetration into rock is related to several factors including rock type and strength, degree of weathering, hammer energy, and nominal required bearing. The above empirical side and tip resistance values for rock, when used with the soil side resistance, should provide a conservative, yet practical, estimate of pile penetration into rock and thus total estimated pile length.

<u>Maximum Nominal Required Bearing</u> ($R_{N\,MAX}$) is the maximum R_N value that can typically be specified on the plans to avoid dynamic stresses during driving which would cause damage to the pile. The value may be determined by use of the Simplified Stress Formula (SSF), discussed below, or a wave equation analysis considering the site specific soils and driving equipment to permit more cost effective designs. In the absence of a site specific wave equation drivability

analysis or unless SSF indicates a lesser value should be used, the $R_{N\,MAX}$ may be calculated using the following empirical relationships:

- Metal Shell Piles: $R_{N\,MAX} = 0.85xF_YA_S$

  Where: F_Y = yield strength of the steel shell (50 ksi)
  A_S = the steel shell cross-sectional area (in.^{2})

- Steel H-Piles: $R_{N\,MAX} = 0.65xF_YA_S$

  Where: F_Y = yield strength of the steel (50 ksi)
  A_S = the steel cross-sectional area (in.^{2})

- Precast Piles: $R_{N\,MAX} = 0.3xf'_cxA_g$

  Where: f'_c = compressive strength of concrete (4.5 or 5 ksi)
  A_g = gross concrete cross sectional area of pile (in.^{2})

- Timber Piles: $R_{N\,MAX} = 0.5xF_{co}xA_P$

  Where: F_{co} = resistance in compression parallel to grain (2.7 ksi)
  A_P = cross-sectional timber area at top of pile (in.^{2})

The SSF is a method developed by the U of I to provide a relatively simple and reasonably accurate estimation of the maximum pile stresses during the driving process. The method consists of numerous equations presented near the end of the design guide and has been integrated into the IDOT Static Method of Estimating Pile Length spreadsheet to predict an estimated driving stress for metal shell and steel H-piles.

Use of the SSF requires knowledge of the pile driving system (hammer weight, hammer cushion data, etc.) that is typically unknown during the design phase. To facilitate use of the SSF, a database of open-ended diesel hammers have been incorporated into the IDOT Static Method of Estimating Pile Length spreadsheet to allow driving stresses to be calculated for an array of hammers satisfying the hammer energy requirements for the WSDOT formula. The stresses from the array of hammers have been averaged to indicate an “Average Estimated Driving Stress” as the pile enters each soil or rock layer.

Empirical relationships based solely upon F_Y and cross-sectional pile area can result in poor protection against pile damage during driving. While the $R_{N\ MAX}$ values listed above are generally anticipated to result in acceptable driving stresses, scenarios may be encountered that prevent piles from reaching $R_{N\ MAX}$ prior to exceeding the maximum acceptable driving stress of $0.9*F_Y$. For instance, steel H-piles being driven to shallow rock may become overstressed prior to reaching $R_{N\ MAX}$ and R_N values less than $R_{N\ MAX}$ may need to be chosen to ensure acceptable driving stresses. The SSF is particularly useful during design in identifying soil layers that are considered hard driving conditions for metal shell piles and may result in large driving stresses and potential pile damage. Alternate pile types should be selected when driving stresses are anticipated to exceed $0.9*F_Y$ before an acceptable penetration depth or bearing is achieved. In addition, the SSF has also been incorporated into the WSDOT Pile Bearing Verification spreadsheet to allow Contractors and field inspectors the opportunity to evaluate the estimated driving stresses for the various hammer configurations being considered by the Contractor.

<u>Factored Resistance Available</u> (R_F) represents the net long term axial factored geotechnical resistance available at the top of the pile to support factored structure loadings. It accounts for losses in geotechnical resistance that occurs after driving due to scour, downdrag (DD_R), or liquefaction (Liq.), resistance required to support downdrag loads (DD_L) and reflects the resistance factor used to verify R_N. R_F shall be calculated using the following equation:

$$R_F = R_N(\phi_G) - (DD_R + Scour + Liq.) x(\phi_G)\ x(I_G) - DD_L x(\gamma_p)$$

Where:

- Scour = nominal side resistance (loss) of soil above the design scour elevation.
- Liq. = nominal side resistance (loss) of soil within liquefiable layers.
- DD_R = nominal side resistance (loss) of soil expected to settle > 0.4 in.
- DD_L = nominal side resistance (load) of soil expected to settle > 0.4 in.
- ϕ_G = the Geotechnical Resistance Factor for the construction verification of R_N
- I_G = the Bias Factor Ratio relating the IDOT Static Method to the construction verification method used.
- γ_p = the DD_L Load Factor for the downdrag soil loading on the pile

Applying the geotechnical resistance factor (ϕ_G) to the geotechnical losses may appear unconservative. However, AASHTO LRFD Article 10.7.3.7 requires the factored loads (R_F + $\gamma_p DD_L$) be ≤ the factored resistance below the downdrag layers. Thus, the pile must be driven to a R_N equal to the nominal downdrag resistance (DD_R) to install the pile through the downdrag layer plus (R_F + γ_p DD_L)/ϕ_G which results in both the geotechnical losses and R_N being multiplied by ϕ_G.

The nominal values of the downdrag (DD_R and DD_L), Scour, and Liquefaction (Liq.) shall be calculated using the IDOT Static Method side resistance equations provided above and as described below.

- *Downdrag* is considered twice to represent the loss in side resistance (DD_R) and again to account for the added loading (DD_L) applied to the pile. The LRFD load groups specify that the portion of downdrag which applies a loading to the pile be included with loadings from other applicable sources. However, it is IDOT's policy to require that the downdrag loading (DD_L) and downdrag reduction in resistance (DD_R) for a pile be taken into account by the geotechnical engineer so it can be incorporated in the SGR pile design tables. Thus they should not be included by the structural engineer in calculating the factored loadings.

- *Scour* protection is provided by accounting for the loss in side resistance of soil layers above the design scour elevation in determining the R_F available to designers. The Scour term shall be taken as zero when calculating the R_F to resist Extreme Event I seismic loadings.

- *Liquefaction* is the loss of side resistance in layers expected to liquefy (Liq.) due to the design seismic event. Since liquefied soil of sufficient thickness consolidates, any non-liquefiable layers above such soils will settle and produce downdrag effects which must also be taken into account. Thus, in addition to Liq., losses from DD_R and DD_L for the layers above the liquefied soils shall be calculated and included in the R_F equation. However Liq. and downdrag caused by liquefaction shall only be considered when calculating the R_F to resist Extreme Event I seismic loadings.

The values of geotechnical losses (Scour, DD_R, DD_L, and Liq.) for non-displacement steel H-piles shall be calculated using the surface area assumption, A_{SAp} (representing "plugged" conditions), regardless of whether the controlling value of R_N used "plugged" or "unplugged" side resistance.

Values for the Geotechnical Resistance Factor, Bias Factor Ratio, and DD_L Load Factor, shall be selected as follows:

- *The Geotechnical Resistance Factor* (ϕ_G) shall be selected to represent the reliability of the method used during construction to verify that the R_N has been developed. Statistical calibration from ongoing U of I research using local dynamic pile driving analyzer testing indicates that a ϕ_G of 0.60 should be used to compute R_F for friction piles when the WSDOT formula is specified for construction verification. When more accurate construction verification methods are proposed, such as with static load test or a Pile Driving Analyzer (PDA), the resistance factor used may be increased to the values provided in the AASHTO specifications.

  Research and statistical calibration by U of I has also determined that ϕ_G for the IDOT Static Method for friction piles, without the use of any construction verification methods, should be taken to be 0.3. Comparison of the resistance factors for the WSDOT formula and IDOT Static Method indicates that there is typically a significant advantage to measuring the driven bearing of a pile in the field using a construction verification method. In order to rely on the IDOT Static Method to provide a reliable design pile length without R_N verification, it is critical that the subsurface conditions are adequately characterized at the substructure unit under consideration. To ensure reliable subsurface data, it is recommended that borings be located such that no foundation element is more than 75 ft from a boring location. At such locations, a second pile length will also be provided using the IDOT Static Method ϕ_G of 0.3, in addition to the standard estimated length provided for WSDOT formula. This length should provide the maximum depth the pile should need to be driven to when the formula does not indicate bearing. However, until sufficient confidence is developed, piles reaching this depth will be allowed to setup and re-tapped to verify adequate bearing. This length may be much deeper than the estimated pile length and will be referred to as the Soil Setup Length.

  For end bearing piles being driven to rock, ϕ_G shall equal 0.70 except for piles driven to shale in which case ϕ_G shall equal 0.65. A reduced ϕ_G is specified for shale to account for relaxation that has been reported by some DOT's and continues to be studied by ongoing research with the U of I.

- *The Bias Factor Ratio* (I_G), shall be included in the calculation for the nominal pile resistance (R_N) and also be applied to the geotechnical losses (Scour, DD_R, and Liq.) to account for differences in bias between the method used to estimate these values (using the IDOT static method) and the construction method used to verify the R_N (typically the WSDOT formula). Research by the U of I indicates that I_G should equal 0.87 in soil layers and 1.0 in rock layers when correlating the IDOT Static Method to the WSDOT formula. Since determining the pile

Soil Setup Length at each R_N using the IDOT Static Method is independent of the construction verification method, I_G shall equal 1.0.

- *The DD_L Load Factor* (γ_p) shall be equal to 1.0 for DD_L caused by cohesive or granular soil layers for piles in compression. This load factor has been determined using statistical calibration data for the IDOT Static Method as outlined near the end of the design guide.

  γ_p shall be equal to 0.30 for DD_L caused by cohesive or granular soil layers when the pile is required to provide pullout or uplift resistance.

If it becomes clear during the planning process that earthquake forces may govern the pile design, the SGR pile tables should include both the R_F to support Extreme Event I Limit State loadings by setting the ϕ_G to 1.0, as well as the R_F to support Strength Limit State loadings by setting ϕ_G to the value corresponding to the construction verification method being used (typically 0.6 for the WSDOT formula for friction piles and 0.65 or 0.7 for end bearing piles driven to rock).

In load cases requiring piles to provide uplift resistance, the factored tension or pullout resistance of the pile shall be determined using the nominal side resistance equations provided above and applying a geotechnical resistance factor (ϕ_G) of 0.20 for uplift under Strength Limit State loadings and 0.8 for uplift under Extreme Event I Limit State loadings. For non-displacement steel H-piles, pullout resistance shall be computed using the surface area assumption (A_{SAp}) for a “plugged” condition only. This calculation will provide the minimum tip elevation which must be specified on the plans ensure pullout resistance.

<u>Estimated Pile Lengths</u> shall be provided in the pile design tables corresponding to the R_N and R_F values computed using the equations above. Since calculating these values requires assumption of the pile length, the procedures and guidance provided below shall be used in determining how these lengths should be selected and which should be provided in the pile design tables in the SGR:

- The geotechnical engineer should contact the structural engineer to obtain preliminary substructure locations and their total factored vertical loading as well as the ground surface, pile cutoff, and bottom of footing/substructure excavation elevations.

- The geotechnical engineer shall evaluate the subsurface soil and rock boring data to develop the profile of pile design parameters (N and Qu) at each substructure.

- Compute the relationship between R_N and pile penetration expected as the pile is driven from the footing/substructure excavation elevation through the various soil design profile for each possible pile type at every substructure. This is typically done by breaking up the soil profile into smaller (≈ 2.5' thick) layers and selecting pile lengths corresponding to the bottom of each layer. This provides the R_N consisting of the cumulative side resistance of all layers above the bottom of the layer in question and the tip resistance of the layer just below the bottom of the layer in question.

- Determine the maximum nominal required bearing feasible to specify without causing damage to the pile. This is most often done using the empirical relationships provided above for approximating $R_{N\ MAX}$, but lesser values may need to be considered depending upon the estimated driving stresses determined using the SSF. Wave equations analysis may also be used to determine if higher values of R_N can be provided in the pile design tables.

- Use the total vertical factored substructure loadings divided by the maximum and minimum pile spacing to provide an initial estimate of the range of R_F and determine the corresponding estimated pile lengths to provide in the tables.

- Discuss this initial range of R_F and the corresponding estimated lengths with the structural engineer to help finalize the range to be included in the SGR. It is preferred that the tables contain too many, rather than too few values to allow the designer the most data upon which to determine the most economical pile type and foundation design layout.

- It is important to again verify the preliminary information and adjust the pile design tables if any elevations or loads have changed. The estimated pile length contained in the design tables (and shown on the plans) must include the portions of the pile which will be incorporated in the substructure and footing. Thus, the ground surface adjacent to the pile during driving and proposed pile cutoff elevations must be accurately determined and documented in the SGR.

- In addition, the pile Soil Setup Length (L_{SETUP}) should also be provided for the range of R_F being reported in the SGR. L_{SETUP} is the pile length using the IDOT Static Method ϕ_G of 0.3 which does not require construction verification. L_{SETUP} should be provided in the contract plans to indicate the maximum length that the piles should be driven to in the event that the construction verification method is indicating insufficient R_N and the piles drive significantly longer than the estimated pile length shown on the plans. In this instance, a waiting period shall be endured and the piles re-tapped to check gain in nominal driven bearing due to soil setup.

Construction Verification Methods are typically used in the field to measure the nominal driven bearing (R_{NDB}) of a pile as it is installed, and in some cases afterwards. The benefit of using such methods is that it allows the use of larger design capacities due to the uncertainty in R_N being limited only to the reliability of the construction verification method being used. They also offer the advantage of providing the resistance at each pile which addresses concerns over the soil strength variability across a site and the accuracy of the soils testing. The alternative to relying on construction verification methods is to use a theoretical method (such as the IDOT static pile design procedure), using a bias ratio factor of 1.0 and the methods geotechnical resistance factor (0.3 in the case of the IDOT Static Method). However, since this method is dependent on the soils data and subsequently the assumed soil properties, the quality of soils investigation is critical when not using a construction verification method.

Although there are a number of construction methods available, IDOT has chosen to use the WSDOT formula as the primary means of determining the R_{NDB} of piles considering research completed by the U of I. The WSDOT formula was initially developed to provide a R_{NDB} of a pile, using hammer energy and pile penetration rate at end-of-driving (EOD), that corresponds to the nominal bearing determined using a static load test. The U of I has further studied the correlation between the capacity predicted by the WSDOT formula using EOD data and the capacity measured using dynamic testing at beginning-of-redrive (BOR) conducted days later. Elapsed time between EOD measurements and static load tests or BOR data allows for dissipation of increased pore water pressure that often occurs during pile driving typically resulting in an increase in capacity. This increase in capacity is referred to as soil setup.

The WSDOT formula, in its original form, has been developed to predict a certain amount of setup based upon EOD data. This was also taken into consideration by the U of I in the statistical calibration resulting in the previously discussed 0.60 ϕ_G. As such, using the original form of the WSDOT formula with BOR data to verify soil setup will likely result in an over prediction of pile capacity. As such, IDOT has introduced a soil setup correction factor, C_s, into the WSDOT formula to account for the average assumed setup. Thus, the C_s value shall equal 1.0 during and at the end-of-driving (EOD), but shall be taken as 0.8 after any beginning-of-redrive (BOR) procedure. The modified WSDOT formula including the C_s is shown below and the remaining variables are defined in the IDOT construction specifications.

$$R_{NDB} = \frac{6.6\, C_S\, F_{eff}\, E \ln(10N_b)}{1000}$$

Reliable prediction of the R_{NDB} of a pile bearing in soil, using the WSDOT formula, is partially dependent upon the hammer chosen by the Contractor to drive the pile. An overly robust hammer can suggest very low pile penetration resistance while an undersized hammer may not generate a pile penetration that is sufficient to mobilize the full pile capacity. To address this, IDOT construction specifications requires that pile driving hammers be capable of operating at an energy that results in a pile penetration rate (N_b) between 1 and 10 blows per inch according to the WSDOT formula for EOD and the R_N indicated in the plans. When R_{NDB} is required to be verified using BOR data, an N_b greater than 10 may be experienced depending upon the magnitude of the gain in R_{NDB} due to soil setup. U of I research data suggests that the R_{NDB} predicted by the WSDOT formula remains reliable when compared to R_{NDB} predicted by dynamic testing for a N_b up to approximately 20 when using BOR data and the above mentioned C_s factor. As such, the IDOT construction specifications require that R_N be achieved at an N_b between 1 and 10 for EOD but permits an expanded N_b range of 1 to 20 for BOR.

As an alternative to the WSDOT formula, the field inspector may analyze BOR data using the Wave Equation Analysis of Piles (WEAP) software program. When performing WEAP using the nominal side and tip resistances estimated by the IDOT Static Method, piles will only be required to achieve a R_{NDB} equal to 85% of R_N indicated in the pile data in the contract plans. The reduction in R_{NDB} is a reflection of the statistical bias of the WEAP method compared to dynamic testing and BOR data.

Simplified Stress Formula (SSF) is a method developed by the U of I for estimating stresses during metal shell and steel H-pile driving and is derived from WEAP stress predictions. Equations for estimating driving stresses using the SSF are provided below. Reference is made to research report FHWA-ICT-12-011, "Improved Design for Driven Piles on a Pile Load Test Program in Illinois", for further information regarding development of the SSF method. It is noted that the SSF was developed according to driving data for open-ended diesel hammers as this is the dominant hammer type used on IDOT projects. The Department has extrapolated beyond the research data to include other hammer types, as indicated in some of the formulas found below, and checked the SSF predictions against a limited number of WEAP results.

σ_C = corrected peak compressive stress (ksi)

$$= \frac{\sigma_P \, C_O}{C_S \, C_W \, C_L \, C_R}$$

σ_P = peak compressive stress (ksi)

$$= \frac{F_P}{A_P}$$

F_P = peak force (kips)

$$= C_F \, V_H \, I_H$$

C_F = peak force coefficient

$$= \frac{1}{W_D} e^{(-\xi \, T_X)} \sin (W_D \, T_X) \quad \text{for } I_R > 0.5$$

$$= \frac{1}{e} \quad \text{for } I_R = 0.5$$

$$= \frac{1}{W_D} e^{(-\xi \, T_X)} \sinh (W_D \, T_X) \quad \text{for } I_R < 0.5$$

ξ = damping ratio

$$= \frac{1}{2 \, I_R}$$

W_D $= \sqrt{\xi^2 - 1}$ for $\xi > 1$

$= \sqrt{1 - \xi^2}$ for $\xi < 1$

C_O = overall correction factor

= 0.9 for diesel hammers

= 1.25 for air/steam hammers

A_P = pile cross-sectional area (in.^{2})

I_R = impedance ratio

$$= \frac{I_P}{I_H}$$

I_P = pile impedance (k*s/ft)

$$= \frac{E \, A_P}{c}$$

c = wave speed of pile material (ft/s)

$$= \sqrt{\frac{144 \, E \, g}{\rho}}$$

E = modulus of elasticity of pile material (ksi)

g = acceleration of gravity (ft/s^2)

ρ = density of pile material (kcf)

T_X $= \frac{1}{W_D}$ atan $\left(\frac{W_D}{\xi}\right)$ for $I_R > 0.5$

$= 1$ for $I_R = 0.5$

$= \frac{1}{W_D}$ atanh $\left(\frac{W_D}{\xi}\right)$ for $I_R < 0.5$

C_S = pile set correction factor

$= 0.6281\, s^2 - 0.0058\, s + 0.6956$

s = pile set (in.)

$= \frac{1}{N_b}$

N_b = hammer blows per inch of pile penetration

C_L = pile length correction factor

$= 0.0046\, L + 0.7265$ (for metal shell piles)

$= 0.0011\, L + 0.8953$ (for steel H-piles)

I_H = hammer impedance (k*s/ft)

$= \sqrt{\frac{12\, k_c\, W_H}{g}}$

k_c = hammer cushion axial stiffness (k/in.)

$= \frac{A_C\, E_C}{t}$

V_H = ram impact velocity

$= \sqrt{2\, g\, \text{eff}\, S_T}$

eff = hammer efficiency

= 0.80 for diesel hammers

= 0.67 for single acting air/steam hammers

= 0.50 for double acting air/steam hammers

S_T = hammer stroke (ft)

C_W = hammer ram weight correction factor

$= 1.395 \left(\frac{W_H}{A_P}\right)^2 - 2.869 \left(\frac{W_H}{A_P}\right) + 2.106$

W_H = weight of hammer ram (kips)

L = embedded length of pile in the ground (ft)

A_C = area of hammer cushion (in.^{2})

E_C = composite modulus of elasticity for 2-material hammer cushion (ksi)

$= \frac{E_1 E_2\, t}{(E_1 t_2) + (E_2 t_1)}$

E_1 = modulus of elasticity for hammer cushion material #1 (ksi)

E_2 = modulus of elasticity for hammer cushion material #2 (ksi)

t_1 = thickness of hammer cushion material #1 (in.)

t_2 = thickness of hammer cushion material #2 (in.)

t = total composite thickness for 2-material hammer cushion (in.)

C_R = pile side resistance proportion correction factor

$= -0.5006\, P_S^2 + 0.8226\, P_S + 0.8105$ (for metal shell piles)

$= -0.9767\, P_S^2 + 1.233\, P_S + 0.7044$ (for steel H-piles)

P_S = ratio of cumulative side resistance to total pile resistance

The Downdrag (DD_L) Load Factor (γ_p) has been statistically calibrated for the IDOT Static Method used to estimate the DD_L demand for the Strength Limit State and the WSDOT formula typically used for construction verification of the geotechnical resistance of the pile. An adjusted version of the corrected First Order Second Moment calibration method (used by the U of I in the report FHWA-ICT-12-011, "Improved Design for Driven Piles on a Pile Load Test Program in Illinois") that includes DD_L in addition to dead and live load has been used to generate a load factor consistent with the target reliability index. The adjusted version of the calibration method is indicated below.

ϕ = WSDOT construction verification method geotechnical resistance factor

$$= \frac{\lambda_R Q \sqrt{\frac{1+COV(Q)^2}{1+COV(R)^2}}}{E(Q)e^{\left[\beta\sqrt{\ln\left[(1+COV(R)^2)(1+COV(Q)^2)\right]}\right]}}$$

= 0.6

λ_R = WSDOT construction verification method bias factor

= 0.910

COV(R) = WSDOT construction verification method coefficient of variation

= 0.252

Q = random variable for load

$= \gamma_D Q_D + \gamma_{DD} Q_{DD} + \gamma_L Q_L$

Q_D, Q_{DD}, and Q_L = dead, downdrag, and live loads

γ_D, γ_{DD}, and γ_L = dead, downdrag, and live load factors

$\gamma_D = 1.25$ and $\gamma_L = 1.75$

COV(Q) = load coefficients of variation

$$COV(Q)^2 = \frac{\frac{Q_D^2}{Q_L^2}\lambda_{Q_D}^2 COV(Q_D)^2 + \lambda_{Q_L}^2 COV(Q_L)^2 + \frac{Q_{DD}^2}{Q_L^2}\lambda_{Q_{DD}}^2 COV(Q_{DD})^2}{\frac{Q_D^2}{Q_L^2}\lambda_{Q_D}^2 + 2\frac{Q_D}{Q_L}\lambda_{Q_D}\lambda_{Q_L} + 2\frac{Q_D Q_{DD}}{Q_L^2}\lambda_{Q_D}\lambda_{Q_{DD}} + \lambda_{Q_L}^2 + 2\frac{Q_{DD}}{Q_L}\lambda_{Q_{DD}}\lambda_{Q_L} + \frac{Q_{DD}^2}{Q_L^2}\lambda_{Q_{DD}}^2}$$

λ_{Q_D}, $\lambda_{Q_{DD}}$, and λ_{Q_L} = bias factors for dead, downdrag and live loads

$\lambda_{Q_D}=1.05$ and $\lambda_{Q_L}=1.15$

$COV(Q_D)$, $COV(Q_{DD})$, and $COV(Q_L)$ = dead, downdrag, and live load coefficients of variation

$COV(Q_D) = 0.1$, $COV(Q_{DD}) = COV(KIDOT)$, and $COV(Q_L) = 0.2$

COV(KIDOT) = IDOT Static Method coefficient of variation

= 0.492

$$\mu_{KIDOT} = \text{mean } \frac{\text{Predicted (IDOT Static Method) Resistance}}{\text{Measured (CAPWAP(BOR)) Resistance}}$$

= 1.45

$\lambda_{Q_{DD}}$ = bias for the median 50th percentile of the IDOT Static Method

$$= \frac{\sqrt{1+COV(KIDOT)^2}}{\mu_{KIDOT}}$$

$$= \frac{\sqrt{1+(0.492)^2}}{1.45} = 0.77$$

β = target reliability index

= 2.33

E(Q) = expected load

$$= \lambda_{Q_D} Q_D + \lambda_{Q_{DD}} Q_{DD} + \lambda_{Q_L} Q_L$$

$\frac{Q_D}{Q_L}$ = ratio of dead load to live load

= 2.0 (assumed); $Q_L = 0.5\, Q_D$

$\frac{Q_{DD}}{Q_D}$ = ratio of downdrag load to dead load

= 0.5 (assumed); $Q_{DD} = 0.5\, Q_D$

Substituting all of the above variables into the equation shown for ϕ, trial and error calculations indicate that the downdrag load factor, γ_{DD}, ≈ 1.0.

IDOT STATIC METHOD OF ESTIMATING PILE LENGTH-WSDOT VERIFICATION

I.D.O.T. BBS FOUNDATIONS AND GEOTECHNICAL UNIT

Modified 8/22/2013

SUBSTRUCTURE & REFERENCE BORING=========== North Abutment - Boring PB1001

LRFD, ASD, or EXTREME EVENT ==================== LRFD
PILE CUTOFF ELEV. ============================ 670.11 FT
GROUND SURFACE ELEV. AGAINST PILE =========== 650.98 FT (DURING DRIVING)
GEOTECH. LOSS TYPE (None, Scour, Liquef., DD) ===== None
BOTTOM ELEV. OF SCOUR, LIQUEF., or DD ======== FT
TOP ELEV. OF LIQUEF. (so layers above apply DD) === FT

TOTAL FACTORED SUBSTRUCTURE LOAD ========== 1500 KIPS
TOTAL LENGTH OF SUBSTRUCTURE (along skew)===== 72.00 FT
NUMBER OF ROWS OF PILES PER SUBSTRUCTURE == 2
Approx. Factored Loading Applied per pile at 8 ft. Cts ============= 83.33 KIPS
Approx. Factored Loading Applied per pile at 3 ft. Cts ============= 31.25 KIPS

PILE TYPE AND SIZE =========== Steel HP 14 X 73
Plugged Pile Perimeter====================== 4.700 FT   Unplugged Pile Perimeter=========== 6.975 FT
Plugged Pile End Bearing Area================ 1.379 SQFT   Unplugged Pile End Bearing Area===== 0.149 SQFT

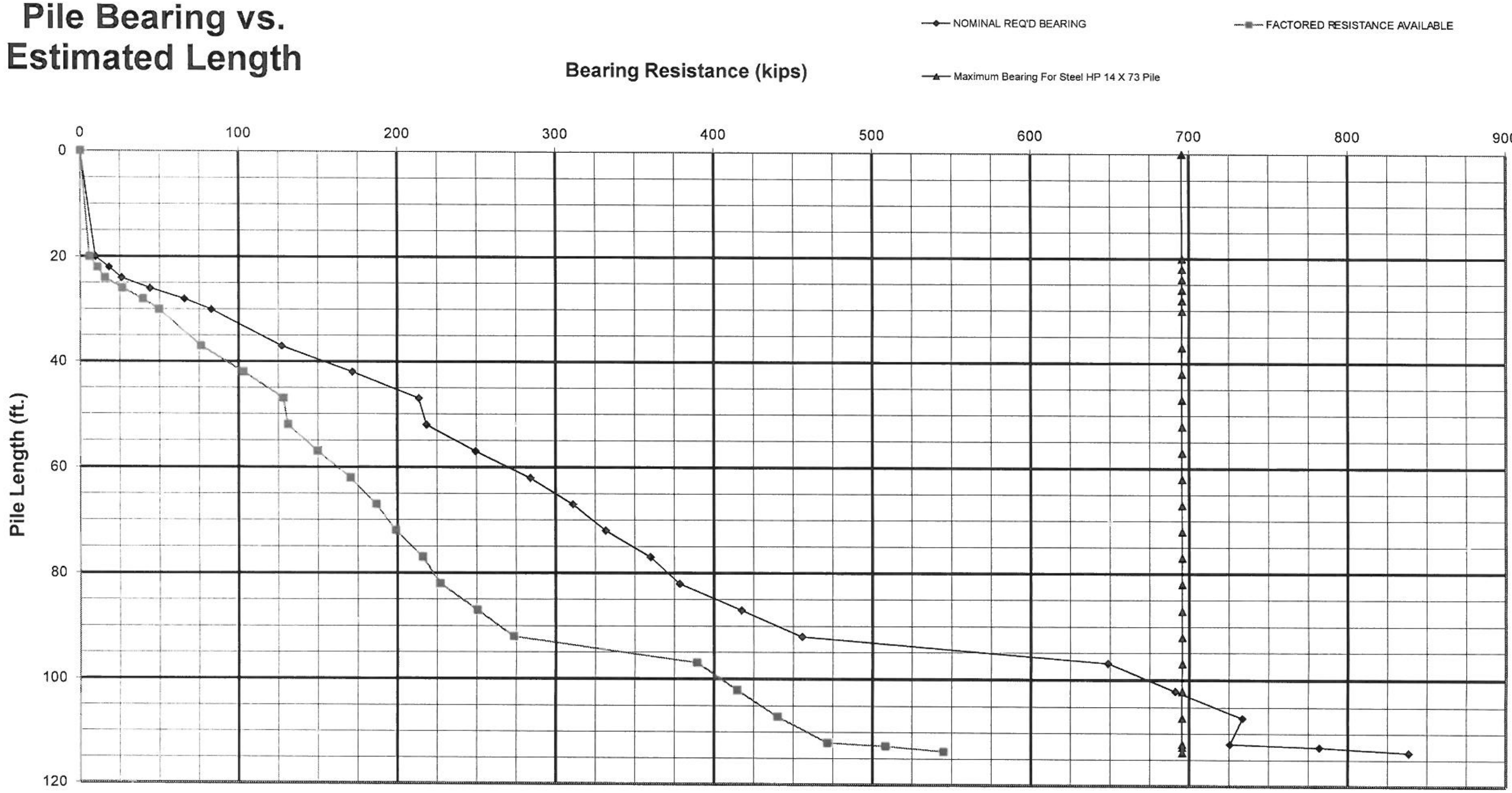
MAX. REQUIRED BEARING & RESISTANCE for Selected Pile, Soil Profile, & Losses

Maximum Nominal Req'd Bearing of Pile	Maximum Nominal Req'd Bearing of Boring	Maximum Factored Resist. Available in Boring	Maximum Pile Driveable Length in Boring
695 KIPS	**695** KIPS	**417** KIPS	**107** FT
	Avg. Est.'d Driving Stress		Soil Setup Pile Length
	29.7 KSI		**N/A - Rock** FT

BOT. OF LAYER ELEV. (FT)	LAYER THICK. (FT)	UNCONF. COMPR. STRENGTH (TSF)	S.P.T. N VALUE (BLOWS)	GRANULAR OR ROCK LAYER DESCRIPTION	NOMINAL PLUGGED SIDE RESIST. (KIPS)	NOMINAL PLUGGED END BRG. RESIST. (KIPS)	NOMINAL PLUGGED TOTAL RESIST. (KIPS)	NOMINAL UNPLUG'D SIDE RESIST. (KIPS)	NOMINAL UNPLUG'D END BRG. RESIST. (KIPS)	NOMINAL UNPLUG'D TOTAL RESIST. (KIPS)	NOMINAL REQ'D BEARING (KIPS)	FACTORED GEOTECH. LOSS FROM SCOUR or DD (KIPS)	FACTORED GEOTECH. LOSS LOAD FROM DD (KIPS)	FACTORED RESISTANCE AVAILABLE (KIPS)	ESTIMATED PILE LENGTH (FT)	SOIL SETUP PILE LENGTH (FT)	AVERAGE ESTIMATED DRIVING STRESS (KSI)
650.49	0.49	4.50	13		4.0		36.4	6.0		***9.4***	9	0	0	6	20	22	-
648.49	2.00	2.00	26		9.2	32.4	***18.0***	13.6	3.5	20.1	18	0	0	11	22	26	-
646.49	2.00	0.30	18		1.9	4.9	47.5	2.8	0.5	***25.9***	26	0	0	16	24	28	-
644.49	2.00	2.00	13		9.2	32.4	97.1	13.6	3.5	***43.9***	44	0	0	26	26	30	-
642.49	2.00	4.50	20		16.4	72.9	89.2	24.3	7.9	***65.5***	66	0	0	39	28	37	-
640.49	2.00	3.00	17		12.1	48.6	93.2	17.9	5.2	***82.6***	83	0	0	50	30	42	-
633.49	7.00	2.50	11		37.1	40.5	***127.1***	55.1	4.4	137.3	127	0	0	76	37	57	16.5
628.49	5.00	2.30	13		25.1	37.2	***171.6***	37.2	4.0	176.6	172	0	0	103	42	67	17.4
623.49	5.00	3.50	16		33.7	56.7	***213.4***	50.1	6.1	227.6	213	0	0	128	47	82	18.8
618.49	5.00	4.00	17		37.3	64.8	***218.4***	55.4	7.0	279.5	218	0	0	131	52	87	19.4
613.49	5.00	2.00	22		22.9	32.4	***249.4***	34.0	3.5	314.4	249	0	0	150	57	92	20.1
608.49	5.00	2.50	22		26.5	40.5	***284.0***	39.4	4.4	354.6	284	0	0	170	62	97	21.1
603.49	5.00	3.00	16		30.1	48.6	***310.9***	44.7	5.2	399.0	311 313	0	0	188 187	68 67	97	21.7
598.49	5.00	2.80	20		28.7	45.3	***331.5***	42.6	4.9	440.7	332	0	0	199	72	97	22.3
593.49	5.00	2.30	20		25.1	37.2	***359.8***	37.2	4.0	478.3	360	0	0	216	77	97	23.1
588.49	5.00	2.50	19		26.5	40.5	***378.3***	39.4	4.4	516.8	378	0	0	227	82	102	23.8
583.49	5.00	2.00	22		22.9	32.4	***417.4***	34.0	3.5	552.5	417	0	0	250	87	112.6	24.5
578.49	5.00	3.00	26		30.1	48.6	***455.6***	44.7	5.2	598.1	456	0	0	273	92	N/A - Rock	25.2
573.49	5.00	3.50	26		33.7	56.7	***648.6***	50.1	6.1	665.4	649	0	0	389	97	N/A - Rock	28.7
568.49	5.00		100	Hard Till	42.4	215.9	***691.0***	63.0	23.3	728.4	691	0	0	415	102	N/A - Rock	29.7
563.49	5.00		100	Hard Till	42.4	215.9	***733.5***	63.0	23.3	791.3	~~733~~	~~0~~	~~0~~	~~440~~	~~107~~	~~N/A - Rock~~	~~30.8~~
558.49	5.00		100	Hard Till	42.4	215.9	***725.5***	63.0	23.3	848.9	~~725~~	~~0~~	~~0~~	~~472~~	~~112~~	~~N/A - Rock~~	~~31.0~~
557.49	1.00			Shale	56.4	165.5	***781.9***	83.7	17.8	932.6	~~782~~	~~0~~	~~0~~	~~508~~	~~112.6~~	~~N/A - Rock~~	~~31.7~~
556.49	1.00			Shale	56.4	165.5	***838.3***	83.7	17.8	1016.3	~~838~~	~~0~~	~~0~~	~~545~~	~~113.6~~	~~N/A - Rock~~	~~33.0~~
555.49	1.00			Shale		165.5			17.8								

ABUTMENT PILE
FRA = 188 kips
Estimated Pile Length = 68 ft
NRB = 313 kips
SOIL SETUP PILE LENGTH = 97 ft

Pile Bearing vs. Estimated Length



PILING
Effective: March __, 2013

This Special Provision amends the following provisions of the Standard Specifications for Road and Bridge Construction.

512.10 Driving Equipment. Revise the first, second and third paragraphs of Article 512.10(a) to read as follows:

(a) Hammers. Piles shall be driven with an impact hammer such as a drop, steam/air, hydraulic, or diesel. The driving system selected by the Contractor shall not result in damage to the pile. The impact hammer shall be capable of being operated at an energy which will maintain a pile penetration rate between 1 and 10 blows per 1 in. (25 mm) when the nominal driven bearing of the pile approaches the nominal required bearing in soil for the end-of-driving condition described in Article 512.14. To avoid potential damage to steel piles driven to rock, the impact hammer shall operate at an energy corresponding to a pile penetration rate between 4 and 20 blows per 1 in. (25 mm) as the pile nears and develops the nominal required bearing in rock.

For hammer selection purposes, the minimum and maximum hammer energy necessary to achieve these penetrations may be estimated as follows.

Soil	Rock
$E \geq \frac{32.9\ R_N}{F_{eff}}$ (English)	$E \geq \frac{28.6\ R_N}{F_{eff}}$ (English)
$E \leq \frac{65.8\ R_N}{F_{eff}}$ (English)	$E \leq \frac{41.1\ R_N}{F_{eff}}$ (English)
$E \geq \frac{10.0\ R_N}{F_{eff}}$ (metric)	$E \geq \frac{8.7\ R_N}{F_{eff}}$ (metric)
$E \leq \frac{20.0\ R_N}{F_{eff}}$ (metric)	$E \leq \frac{12.5\ R_N}{F_{eff}}$ (metric)

Where:

R_N = Nominal required bearing in kips (kN)
E = Energy developed by the hammer per blow in ft-lb (J)
F_{eff} = Hammer efficiency factor according to Article 512.14.

The above hammer options, hammer energy range, and pile penetration rates shall be applicable unless noted otherwise in the construction documents.

512.11 Penetration of Piles. Revise Article 512.11 to read as follows:

Piles shall be installed to a penetration that satisfies all of the following.

(a) The nominal driven bearing, as determined by the formula in Article 512.14, is not less than the nominal required bearing shown on the plans except as permitted below for piles driven to rock.

(b) The pile tip elevation is at or below the minimum tip elevation shown on the plans. In cases where no minimum tip elevation is provided, the piles shall be driven to a penetration of at least 10 ft (3 m) below the bottom of footing or below undisturbed earth, whichever is greater.

Except as required to satisfy minimum tip elevations required in 512.11(b) above, piles not bearing on rock are not required to be driven more than one additional foot (300 mm) after the nominal driven bearing equals or exceeds the nominal required bearing; more than three additional inches (75 mm) after the nominal driven bearing exceeds 110 percent of the nominal required bearing; or more than one additional inch (25 mm) after the nominal driven bearing exceeds 150 percent of the nominal required bearing. For piles driven to rock, pile driving shall be stopped, independent of the nominal driven bearing predicted by the formula in Article 512.14, when the minimum penetration rate is ¼ in. over 5 blows (or equivalently a maximum penetration rate of 20 blows per 1 in. for no more than 5 blows). When piles not bearing or rock fail to achieve nominal driven bearings in excess of the nominal required bearing after driving the full furnished lengths, but are within 85 percent of nominal required bearing, these piles shall be left for a minimum of 24 hours to allow for soil setup and retesting before splicing and driving additional length. After the waiting period has passed, the pile shall be redriven to check the gain in nominal driven bearing upon soil setup. The soil setup nominal driven bearing shall be based on the number of redriving blows necessary to drive the pile an additional 2 in. (75 mm) using a hammer that has been warmed up by applying at least 20 blows to another pile. Within the additional 2 in., the redriving data should be carefully observed and the bearing determined for each ½ in. of pile penetration. In addition to the pile penetration rate, field inspectors are encouraged to carefully monitor the hammer energy during the redrive as increased driving resistance from soil setup may result in greater rebound of the hammer ram and developed hammer energy than experienced during the initial pile driving procedure. The soil setup nominal driven bearing may be taken as the largest value recorded at the ½ in. increments. These piles will be accepted if they exhibit a nominal driven bearing larger than nominal required bearing. In addition, piles within a group, and adjacent to a retested pile that has achieved the nominal required bearing within the additional 2 in. of pile penetration, may be accepted provided the piles exhibited driving behavior similar to the retested pile prior to the setup period. Acceptance of such piles shall be subject to approval of the Engineer and shall require that a minimum of 20 percent of the piles within the group, and no fewer than 2, be retested and achieve the nominal required bearing within the additional 2 in. of pile penetration. Locations of the retested piles should be uniformly scattered across the pile group.

When piles have been driven in excess of the indicated estimated pile length and are not within 85 percent of the nominal required bearing, piles should not be driven longer than the soil setup pile length indicated in the plans. When piles have been driven to this length, they shall be left for a minimum of 48 hours and redriven to check the gain in nominal driven bearing due to soil setup using the above procedure. The Bureau of Bridges and Structures should be contacted for further disposition when piles have not achieved the nominal required bearing upon redrive.

The above mentioned waiting periods for redriving piles to check for gain in nominal driven bearing due to soil setup are minimums and some soil types may exhibit greater soil setup with increased waiting period. When feasible, longer waiting periods that are a function of the soil type at the pile location are encouraged. The following waiting periods are recommended prior to redriving piles to try and maximize the gain in nominal driven bearing due to soil setup:

Recommended Waiting Periods for Redrive Based on Soil Type

Clean Sands	= 1 day
Silty Sands	= 2 days
Sandy Silts	= 4 days
Silts and Clays	= 8 days

512.14 Determination of Nominal Driven Bearing. Revise the first paragraph of Article 512.14 to read as follows:

The nominal driven bearing of each pile shall be determined by the WSDOT formula as follows.

$$R_{NDB} = \frac{6.6\ C_S\ F_{eff}\ E\ Ln\ (10N_b)}{1000} \quad \text{(English)}$$

$$R_{NDB} = \frac{21.7\ C_S\ F_{eff}\ E\ Ln\ (10N_b)}{1000} \quad \text{(metric)}$$

Where:

R_{NDB} = Nominal driven bearing of the pile in kips (kN)
C_s = Soil setup correction factor
1.0 for EOD data
0.8 for BOR data
N_b = Number of hammer blows per inch (25 mm) of pile penetration
E = Energy developed by the hammer per blow in ft lb (J)
F_{eff} = Hammer efficiency factor taken as:
0.55 for air/steam hammers
0.47 for open-ended diesel hammers and steel piles or metal shell piles
0.37 for open-ended diesel hammers and concrete or timber piles
0.35 for closed-ended diesel hammers
0.28 for drop hammers

End-of-driving (EOD) data refers to the information that is collected and analyzed during the initial pile installation procedure. Beginning-of-redrive (BOR) data refers to the redriving information that is collected and analyzed when the pile is driven less than 2 in. following a waiting period to check the gain in nominal driven bearing due to soil setup. When redriving piles, a significant reduction in R_{NDB} is often observed as the pile penetration exceeds 2 in. If the pile does not achieve the required nominal driven bearing within the 2 in. of additional penetration during the redrive, the nominal driven bearing of the pile shall continue to be determined using the WSDOT formula and soil setup correction factor for EOD data after the pile has been driven 4 additional inches.

Per Article 512.10, the hammer chosen by the contractor is required to be capable of developing the nominal required bearing capacity of piles bearing in soil at EOD at an N_b between 1 and 10. When evaluating R_{NDB} of piles bearing in soil for the same hammer using the WSDOT formula and BOR data, the permissible range of N_b is between 1 and 20.

As an alternative to the WSDOT formula, qualified personnel may analyze BOR data using the Wave Equation Analysis of Piles (WEAP) software program. When performing WEAP of BOR data using the Department's geotechnical pile design procedure, piles will only be required to achieve a nominal driven bearing equal to 85% of nominal required bearing indicated in the contract plans.

512.15 Test Piles. Revise the third paragraph of Article 512.15 to read as follows:

Test piles not bearing on rock shall be driven to a nominal driven bearing ten percent greater than the nominal required bearing shown on the plans. The Engineer may stop the driving of any test pile not bearing on rock at tip penetrations exceeding 10 ft (3 m) beyond the estimated length to check for pile setup according to Article 512.11. After any retesting, the Contractor shall recommence test pile driving, providing piling, splices, and any retests until the nominal driven bearing during driving reaches ten percent more than the nominal required bearing or the Engineer stops the driving due to having sufficient data to provide the itemized list of furnished lengths. Test piles bearing on rock shall be driven to the nominal required bearing shown on the plans except pile driving shall be stopped when the pile penetration rate satisfies the criteria indicated in Article 512.11.

1006.05 Metal Piling and Steel Casing. Replace 1006.05(a) and (b) with the following:

(a) Metal Shell Piling. Metal shell piling shall be according to ASTM A 252, Grade 3 except the minimum yield strength shall be 50,000 psi (345,000 kPa).

(b) Steel Piling. Steel piling shall be according to AASHTO M 270, Grade 50 (M 270M, Grade 345).