STRUCTURE GEOTECHNICAL REPORT
OLD WILLOW SPRINGS ROAD OVER
DES PLAINES RIVER PEDESTRIAN BRIDGE
EXISTING SN 016-0539, PROPOSED SN 016-0539
TR RTE. 9250 SECTION 142A-B
IDOT PTB 172/ ITEM 10, CONTRACT 62B99/WO 9
COOK COUNTY, ILLINOIS

For

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11. Abstract

The existing Old Willow Springs Road Bridge over the Des Plaines River will be replaced with a pedestrian truss double-span superstructure. The South Abutment and Pier 3 will be rehabilitated, whereas Piers 1, 2, and 4 will be removed. The existing North Abutment will be removed and a new spill-through abutment, supported on driven piles, will be constructed behind the existing abutment. This report provides our geotechnical evaluations and recommendations for the design and construction of the bridge foundations.

The foundation soils consists of up to 6.5 feet of very soft to medium stiff clay to clay loam and silty clay loam overlying 7.5 feet of medium dense sandy loam and 5.0 to 9.5 feet of very soft to soft organic silt followed by up to 12 feet of very dense silt to silty loam, sand, and weathered bedrock. Very poor to excellent quality limestone bedrock was encountered at elevations of 560.1 to 566.1 feet (24.5 to 30.5 feet bgs). The site classifies in Seismic Class C. Groundwater was recorded at elevations of 589.1 to 591.1 feet.

A minimal change in grade is anticipated and the long-term settlement at the North Abutment is estimated to be less than 0.4-inch. Additionally, the North Abutment end slope will be protected with stone riprap. Therefore, downdrag load allowances and scour reduction will not be required for the new abutment piles.

The proposed spill-through abutment can be supported on driven steel H-piles. The piles should be fitted with pile shoes. Tables are provided for various H-pile sizes with the estimated pile lengths corresponding to the maximum nominal required bearing of the piles. Geotechnical parameters for pile analysis under lateral loads are also included.

11. Path to archived file

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TABLE OF CONTENTS

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COOK COUNTY, ILLINOIS
FOR
HBM ENGINEERING GROUP, LLC.

1.0 INTRODUCTION

This report presents the results of our geotechnical evaluations and recommendations for the proposed replacement of the Old Willow Springs Road Bridge over Des Plaines River at Station 154+61.17 in Cook County, Illinois. The *TSL Plan*, dated November 16, 2016, and provided by HBM Engineering Group, LLC. (HBM) is presented as Exhibit 1.

1.1 Existing Structure

The existing bridge was constructed in 1924 and has not undergone any major repairs. It is a five-span cast-in-place reinforced concrete T-beam bridge supported by reinforced concrete piers and abutments on spread footings. The bridge has a total length of 300 feet and out-to-out bridge width of 39 feet.

1.2 Proposed Structure

Wang Engineering, Inc. (Wang) understands that the existing structure will be replaced with a prefabricated pedestrian truss superstructure capable of supporting maintenance vehicles. Based on the *TSL Plan* (Exhibit 1), dated November 16, 2016, the bridge will be a double-span structure with an out-to-out width of 16.0 feet and a back-to-back of abutments length of 317.5 feet. The existing Pier 3 and South Abutment will be rehabilitated and reused, whereas the North Abutment will be replaced. The existing Piers 1, 2, and 4 will be removed. The new North Abutment will be constructed behind the existing abutment as a spill-through abutment supported on steel H-piles. The North Abutment wingwalls will be supported on steel H-piles as well. A minimal change in grade is anticipated.

The following sections present the existing subsurface soil and groundwater conditions and our recommendations for the design and construction of the pedestrian bridge North Abutment foundations.

2.0 EXISTING GEOTECHNICAL DATA

Borings WS-08, WS-09, and WS-10 were drilled in 1972 near the existing South Abutment, Pier 3, and North Abutment locations, respectively. The borings were advanced from elevations of 589.6 to 591.1 feet to depths of 34 to 41 feet bgs. The boring logs were provided by HBM and the Illinois Department of Transportation (IDOT). Boring WS-10 was considered for our analysis at the North Abutment. The boring locations are shown on the *TSL Plan* (Exhibit 1) and the *Boring Logs* are presented in Appendix A.

2.1 Soil Conditions

Detailed descriptions of the soil conditions are presented on the *Boring Logs* (Appendix A). In descending order, the general lithologic succession encountered beneath the surface includes: 1) very soft to medium stiff clay to clay loam and silty clay loam; 2) medium dense sandy loam and clay loam; 3) dense to very dense silt to silty loam and sand; and 4) very poor to excellent quality limestone (bedrock).

1) Very soft to medium stiff clay to clay loam and silty clay loam

Borings WS-08 and WS-10, drilled along the existing ground surface, encountered up to 6.5 feet of very soft to medium stiff, brown and gray clay to clay loam and silty clay loam with organic matter. This layer has unconfined compressive strength (Q_u) values of 0.2 to 1.0 tsf with an average of 0.6 tsf and moisture content values of 30 to 77%, averaging 48%.

2) *Medium dense sandy loam*

At an elevation of 583.1 feet (6.5 feet bgs), Boring WS-10 revealed 7.5 feet of medium dense, gray sandy loam with Standard Penetration Test (SPT) N-values of 25 to 27 blows/foot. An intermittent layer of very stiff to hard clay loam was sampled within the sandy loam layer.

At elevations of 573.6 to 583.1 feet, Borings WS-08 and WS-09 augured through a 5.0 to 9.5-foot thick layer of very soft to soft, black, brown, and gray organic silt with shells and fibrous materials. The layer is characterized by Q_u values of 0.1 to 0.6 tsf and moisture content values of 154 to 186%. This layer was not encountered in Boring WS-10 near the North Abutment.

3) Dense to very dense silt to silty loam and sand

At elevations of 572.1 to 575.6 feet (14.0 to 18.5 feet bgs), the borings sampled 6 to 12 feet of dense to very dense, gray silty loam to silt and sand with gravel having N-values of 32 to 100 blows/foot.

Beneath the silt to silty loam, 4.5 to 7.5 feet of very dense, gray, broken limestone (possibly weathered bedrock) with layers of clay loam, sand, and gravel was encountered. The weathered bedrock is characterized by N-values of 100 blows/foot to 100 blows per 7 inches of sampler penetration.

4) *Very poor to excellent quality limestone (bedrock)*
The borings encountered bedrock at elevations of 560.1 to 566.1 feet (24.5 to 30.5 feet bgs). Rock cores were obtained from Borings WS-10. The first three NX-size rock cores have recoveries of 50 to 80% with Rock Quality Designation (RQD) values ranging from 0 to 10% indicating very poor rock quality. These rock cores are interbedded with silt or sand layers and clay loam layers. The fourth NX-size rock core obtained between elevations 548.5 and 553.1 (36.5 to 41.0 feet bgs) has a recovery of 100% and an RQD value of 100% indicating excellent quality bedrock.

2.2 Groundwater Conditions
The borings were drilled along the river streambed and groundwater was encountered in the borings at elevations of 589.1 to 591.1 feet (depths of 0.0 to 0.5 feet bgs) at the time of drilling in 1972. For the purpose of our analysis, we have considered the groundwater to be at the surface.

3.0 FOUNDATION ANALYSIS AND RECOMMENDATIONS

The geotechnical evaluations and recommendations for scour considerations, seismic design, and the North Abutment and wingwall foundations are included in the following sections.

3.1 Scour Considerations
The *TSL Plan* (Exhibit 1) provided by HBM indicates an existing streambed elevation of 582.81 feet, an estimated water surface elevation (ESWE) of 588.74 feet and a design high water elevation (DHWE) of 594.8 feet. The abutments and Pier 3 foundations are shown with class A5 stone riprap for scour protection. As per the IDOT *All Bridge Designers Memorandum 14.2 "Revised Scour Design Policy"*, for spill-through abutments protected with stone riprap, the design and check scour elevations are set at the bottom of the abutment cap. For spread footings, the design and check scour elevations should be located at the bottom of the footing elevation. The design scour elevations for the proposed bridge as shown on the *TSL Plan* (Exhibit 1) are presented in Table 1. Based on the borings, we estimate the proposed design scour elevations are sufficient.

Table 1: Design Scour Elevations

Event/Limit State	Design Scour Elevation (feet)			Item 113
	South Abutment	Pier 3	North Abutment	
Q100	565.70	570.80	593.65	7
Q500	565.20	569.80	593.65	
Design	574.78	571.46	593.65	
Check	574.78	571.46	593.65	

As per the IDOT *All Bridge Designers Memorandum 14.2 "Revised Scour Design Policy"*, engineered scour countermeasures designed for the Q500 flood are required for the reuse of the Pier and South Abutment spread footing foundations.

3.2 Seismic Design Considerations

The Seismic Site Class was determined in accordance with Section 6.12, *Seismic Analysis* of the 2015 IDOT *Geotechnical Manual* using the IDOT spreadsheet "*Seismic Site Class Determination*" dated December 13, 2010. Based on the subsurface soil profile, the site is in Seismic Site Class C. The project location belongs to the Seismic Performance Zone 1. The results of the *Seismic Site Class Determination* are presented in Appendix B.

The seismic spectral acceleration parameters were determined using the AASHTO computer program "*Seismic Design Parameters*, version 2.10" by specifying the site location by latitude and longitude. The location of the bridge was considered to be at Latitude 41.734623 and Longitude -87.880189. The seismic spectral acceleration parameters recommended for design in accordance with the AASHTO 2015 *LRFD Bridge Design Specifications* are summarized in Table 2.

Table 2: Seismic Design Parameters

Spectral Acceleration Period (sec)	Spectral Acceleration Coefficient[1)] (% g)	Site Factors	Design Spectrum for Site Class C[2)] (% g)
0.0	PGA= 4.6	F_{pga}= 1.2	**A_s= 5.5**
0.2	S_S= 9.8	F_a= 1.2	**S_{DS}= 11.8**

Spectral Acceleration Period (sec)	Spectral Acceleration Coefficient[1)] (% g)	Site Factors	Design Spectrum for Site Class C[2)] (% g)
1.0	S_1= 3.7	F_v= 1.7	**S_{D1}= 6.4**

1) Spectral acceleration coefficients based on Site Class D

2) Site Class C values to be presented on plans (A_s = PGA*F_{pga}; S_{DS}= S_S*F_a; S_{D1}= S_1*F_v)

As per the 2012 IDOT *Bridge Manual*, liquefaction analysis is not required for a site located in Seismic Performance Zone 1.

3.3 North Abutment on Driven H-piles

The North Abutment will have a pile cap base elevation of 593.65 feet. Preliminary service and factored loads for the South Abutment, Pier 3, and North Abutment foundations were provided by HBM and are shown in Table 3.

Table 3: Preliminary Foundation Loads

Substructure	Service Dead Load (kips)	Service Live Load (kips)	Estimated Total Service Load (kips)	Estimated Total Factored Load (kips)
South Abutment	993.0	112.9	1105.9	1438.8
Pier 3	1182.8	197.7	1380.5	1824.5
North Abutment	261.4	84.9	346.2	475.2

The North Abutment could be supported on driven piles as proposed by HBM. Driven H-piles designed as friction piles could be considered, however, by driving a few more feet to the top of bedrock the maximum allowable structural pile capacity, Maximum Nominal Required bearing (R_N), can be achieved. Boring WS-10 recorded the top of bedrock at an elevation of 565.3 feet or approximately 28 feet below the proposed bottom of the abutment pile cap. We recommend considering H-piles with pile shoes if the piles are to be driven to maximum R_N. It should be noted that since Boring WS-10 was performed approximately 50 feet away from the center of the proposed North Abutment, the top of bedrock elevation at the North Abutment could be a few feet different than the elevation encountered in Boring WS-10.

Nominal tip and side resistances were estimated using the methods and empirical equations presented in *AGMU Memorandum 10.2-Geotechnical Pile Design* (IDOT 2011) and the accompanying spreadsheet *Pile Capacity and Length Estimates*. There is a minimal change in grade and we estimate the settlement at the North Abutment approach to be less than 0.4 inches. Additionally, the abutment piles will have a stone riprap protected end slope; therefore, we did not consider downdrag allowances and scour reductions for the pile length analysis. The estimated capacities, pile lengths, and pile tip elevations for pile sizes HP10X42, HP12X53, HP12X63, HP14X73, and HP14X89 are shown in Tables 4 through 8. The estimated pile lengths include two feet of embedment into the pile cap.

Table 4: Estimated Pile Lengths and Tip Elevations for HP10X42

Structure Unit (Reference Boring)	Pile Cap Base Elevation (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss, $(DD+S_c+L_{iq})$	Factored Geotechnical Loss Load, (DD Only) (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment (WS-10)	593.65	73	0	0	40	26	569.7(1)
		145	0	0	80	28	567.7
		218	0	0	120	30	565.7(2)
		335(3)	0	0	184	31	564.7

(1) Top of very dense broken limestone (possible weathered bedrock).

(2) Top of bedrock.

(3) Maximum R_N for HP10X42

Table 5: Estimated Pile Lengths and Tip Elevations for HP12X53

Structure Unit (Reference Boring)	Pile Cap Base Elevation (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss, $(DD+S_c+L_{iq})$	Factored Geotechnical Loss Load, (DD Only) (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment (WS-10)	593.65	73	0	0	40	23	572.7
		145	0	0	80	27	568.7(1)
		218	0	0	120	29	566.7

Structure Unit (Reference Boring)	Pile Cap Base Elevation (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss, $(DD+S_c+L_{iq})$	Factored Geotechnical Loss Load, (DD Only) (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
		291	0	0	160	30	565.7(2)
		419(3)	0	0	230	31	564.7

(1) Very dense broken limestone (possible weathered bedrock).

(2) Top of bedrock.

(3) Maximum R_N for HP12X53

Table 6: Estimated Pile Lengths and Tip Elevations for HP12X63

Structure Unit (Reference Boring)	Pile Cap Base Elevation (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss, $(DD+S_c+L_{iq})$	Factored Geotechnical Loss Load, (DD Only) (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment (WS-10)	593.65	73	0	0	40	23	572.7
		145	0	0	80	27	568.7(1)
		218	0	0	120	29	566.7
		291	0	0	160	30	565.7(2)
		436	0	0	240	31	564.7
		497(3)	0	0	273	32	563.7

(1) Very dense broken limestone (possible weathered bedrock).

(2) Top of bedrock.

(3) Maximum R_N for HP12X63

Table 7: Estimated Pile Lengths and Tip Elevations for HP14X73

Structure Unit (Reference Boring)	Pile Cap Base Elevation (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss, ($DD+S_c+L_{iq}$)	Factored Geotechnical Loss Load, (DD Only) (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment (WS-10)	593.65	73	0	0	40	19	576.7
		145	0	0	80	27	568.7(1)
		218	0	0	120	28	567.7
		291	0	0	160	29	566.7
		364	0	0	200	30	565.7(2)
		509	0	0	280	31	564.7
		578(3)	0	0	318	32	563.7

(1) Very dense broken limestone (possible weathered bedrock).

(2) Top of bedrock.

(3) Maximum R_N for HP14X73

Table 8: Estimated Pile Lengths and Tip Elevations for HP14X89

Structure Unit (Reference Boring)	Pile Cap Base Elevation (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss, ($DD+S_c+L_{iq}$)	Factored Geotechnical Loss Load, (DD Only) (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment (WS-10)	593.65	73	0	0	40	18	577.7
		145	0	0	80	26	569.7(1)
		218	0	0	120	28	567.7
		291	0	0	160	29	566.7
		364	0	0	200	30	565.7(2)
		545	0	0	300	31	564.7
		705(3)	0	0	388	32	563.7

(1) Very dense broken limestone (possible weathered bedrock).

(2) Top of bedrock.

(3) Maximum R_N for HP14X89

3.4 Northeast and Northwest Concrete Wingwalls on Piles

As indicated on the *TSL Plan*, the northeast and northwest wingwalls are proposed as concrete T-type wingwalls supported on driven steel H-piles. The northeast wingwall will have a total height of 8.3 feet and a length of 18.9 feet, whereas the northwest wingwall will be 10 feet long with a total height of 8.3 feet. The proposed bottom of the cap is at an elevation of 593.65 feet. The wingwalls could be supported on driven steel H-piles. Tables 9 to 13 provide the estimated capacities, pile lengths, and tip elevations for pile sizes HP10X42, HP12X53, HP12X63, HP14X73, and HP14X89. The estimated pile lengths include two feet of embedment into the pile cap.

Table 9: Estimated Pile Lengths and Tip Elevations for HP10X42

Structure Unit (Reference Boring)	Pile Cap Base Elevation (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss, ($DD+S_c+L_{iq}$)	Factored Geotechnical Loss Load, (DD Only) (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment Wingwalls (WS-10)	593.65	36	0	0	20	13	582.7
		73	0	0	40	26	569.7(1)
		109	0	0	60	28	567.7

(1) Top of very dense broken limestone (possible weathered bedrock).

Table 10: Estimated Pile Lengths and Tip Elevations for HP12X53

Structure Unit (Reference Boring)	Pile Cap Base Elevation (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss, ($DD+S_c+L_{iq}$)	Factored Geotechnical Loss Load, (DD Only) (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment Wingwalls (WS-10)	593.65	36	0	0	20	13	582.7
		73	0	0	40	23	572.7
		109	0	0	60	27	568.7(1)

(1) Very dense broken limestone (possible weathered bedrock).

Table 11: Estimated Pile Lengths and Tip Elevations for HP12X63

Structure Unit (Reference Boring)	Pile Cap Base Elevation (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss, $(DD+S_c+L_{iq})$	Factored Geotechnical Loss Load, (DD Only) (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment Wingwalls (WS-10)	593.65	36	0	0	20	13	582.7
		73	0	0	40	23	572.7
		109	0	0	60	26	569.7(1)

(1) Very dense broken limestone (possible weathered bedrock).

Table 12: Estimated Pile Lengths and Tip Elevations for HP14X73

Structure Unit (Reference Boring)	Pile Cap Base Elevation (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss, $(DD+S_c+L_{iq})$	Factored Geotechnical Loss Load, (DD Only) (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment Wingwalls (WS-10)	593.65	36	0	0	20	10	585.7
		73	0	0	40	19	576.7
		109	0	0	60	25	570.7

Table 13: Estimated Pile Lengths and Tip Elevations for HP14X89

Structure Unit (Reference Boring)	Pile Cap Base Elevation (feet)	Nominal Required Bearing, R_N (kips)	Factored Geotechnical Loss, $(DD+S_c+L_{iq})$	Factored Geotechnical Loss Load, (DD Only) (kips)	Factored Resistance Available, R_F (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
North Abutment Wingwalls (WS-10)	593.65	36	0	0	20	10	585.7
		73	0	0	40	18	577.7
		109	0	0	60	24	571.7

3.5 South Abutment and Pier 3 on Spread Footings

The existing South Abutment and Pier 3 are supported on spread footing foundations. The existing foundations should be verified to carry the estimated loads. Table 14 provides the bearing resistances for both LFD and LRFD methods. Normally, the reuse of existing foundations is verified using the LFD method. The allowable bearing resistance of the existing foundation soils was estimated considering a factor of safety (FOS) of 3, whereas the factored bearing resistance was estimated considering a bearing resistance factor of 0.45.

Table 14: Allowable and Factored Bearing Resistances

Substructure	Reference Boring	Bottom of Footing Elevation (feet)	Allowable Bearing Resistance (psf)	Factored Bearing Resistance (psf)
South Abutment	WS-8	574.8	4000	5400
Pier 3	WS-9	571.5	6000	8100

3.6 Lateral Design Pressures

For the design of the North Abutment and wingwalls, we recommend a linearly increasing lateral pressure of 40 psf per foot of depth below the finished grade considering a horizontal backfill. Additional lateral pressure from traffic should include a surcharge of 2 feet of soil considering a unit weight of 125 pcf as per AASHTO Specifications. It is understood that a Geocomposite Wall Drain connected to a 4-inch diameter perforated drain with granular backfill will be provided only behind the concrete abutment. Therefore, we recommend including the water pressure considering the groundwater table to be at an elevation of 593.65 feet (bottom of the North Abutment pile cap). The simplified earth pressure distribution shown in the AASHTO *Standard Specifications for Highway Bridges* or other suitable earth pressure distributions should be used.

3.7 Resistance to Lateral Loads

Lateral loads on all piles should be analyzed for maximum moments and lateral deflections. No allowance should be made for the frictional resistance of the concrete cap on soil. The lateral load capacity analysis can be performed using computer programs such as COM624P, L-pile, LATPILE or any other similar program. The estimated soil parameters that may be used to analyze stresses and deflections of piles under lateral loads are presented in Table 15. Group action should be considered in calculating total lateral load resistance at the North Abutment.

Table 15: Recommended Soil Parameters for Lateral Load Pile Analysis
North Abutment (Boring WS-10)

Layer Elevation Soil Description	Unit Weight, γ (pcf)	Shear Strength Properties		Estimated Lateral Soil Modulus Parameter[1], k (pci)	Estimated Soil Strain Parameter[1], ε_{50}
		Cohesion, C_u (psf)	Friction Angle, ϕ (°)		
593.7[2] to 589.6[3,4]\ Existing Fill (Assumed)	120	1000	0	500	0.007
589.6 to 584.6 Medium Stiff to Stiff Clay	53	800	0	100	0.010
584.6 to 583.1 V. Soft to Medium Stiff Silty Clay Loam	53	350	0	30	0.020
583.1 to 578.6 M. Dense Sandy Loam	53	0	34	60	--
578.6 to 575.6 Hard Clay Loam	63	5000	0	2000	0.004
575.6 to 569.6 Dense Silty Loam to Silt	63	0	30	125	--
569.6 to 565.3[5] V. Dense Limestone with Clay Loam Layers	68	0	40	140	--

(1) Based on L-Pile Technical Manual 2012.
(2) Bottom of proposed pile cap elevation.
(3) Grade elevation at boring location.
(4) Groundwater elevation considered at boring location.
(5) Top of bedrock elevation

3.8 Stage Construction Considerations

The *TSL Plan* indicates that the structure will be closed to vehicular traffic and pedestrians during construction.

4.0 CONSTRUCTION CONSIDERATIONS

4.1 Excavation, Dewatering, and Utilities

Foundation excavations should be performed in accordance with local, state, and federal regulations. The potential effect of ground movements upon nearby utilities should be considered during construction.

Borings performed in 1972 by others encountered groundwater at elevations of 589.1 to 591.1 feet (0.0 to 0.5 feet bgs). Groundwater during construction could be at different depth depending on the river level. The TSL plan indicates the EWSE is at an elevation of 588.74 feet and the existing streambed is at an elevation of 582.81 feet.

At the Pier 3 location, we understand that the riverbed will be excavated and stone rip rap will be dumped along the existing pier. We expect that the excavation can be performed under water and stone riprap can be placed without the use of cofferdam. The excavation method and placement of riprap will be as per contractor's means and method.

At the South Abutment, the EWSE is about 1 foot below the existing ground elevation. Boring WS-08 performed in 1972 drilled about 36 feet away from the South Abutment recorded a groundwater elevation of 591.1 feet. The south abutment will be reconstructed above elevation 591.75 and EWSE is 588.74. We expect the groundwater water level will be below the abutment cap reconstruction elevation. The placement of stone riprap can be done with regular dewatering methods such as the sump-pump method.

At the North Abutment, the existing ground elevation is about 0.5 feet higher than the ESWE and Boring WS-10 encountered water at an elevation of 589.1 feet (about 5 feet below the proposed abutment cap). Therefore, we do not anticipate the need for special dewatering efforts for the placement of riprap and construction of the North Abutment.

4.2 Filling and Backfilling

Fill material required to attain the final design elevations should be in accordance with Section 205, *Embankment* of the IDOT *Standard Specifications* (IDOT 2016). All fill and backfill materials should be pre-approved by the site engineer prior to placement. The fill material should be free of organic matter and debris. The backfill behind the abutment should be in accordance with the IDOT *Standard Specifications* and the 2012 IDOT *Bridge Manual*.

4.3 Earthwork Operations

The required earthwork can be accomplished with conventional construction equipment. Moisture and traffic will cause deterioration of exposed subgrade soils. Precautions should be taken by the contractor to prevent water erosion of the exposed subgrade. A compacted subgrade will minimize water runoff erosion.

Earth moving operations should be scheduled not to coincide with excessive cold or wet weather (early spring, late fall or winter). Any soil allowed to freeze or soften due to the standing water should be removed. Wet weather can cause problems with subgrade compaction.

It is recommended that an experienced geotechnical engineer be retained to inspect the exposed subgrade, monitor earthwork operations, and provide material inspection services during the construction phase of this project.

4.4 Pile Installation

Driven piles shall be furnished and installed according to the requirements of Section 512, *Piling* (IDOT 2016) and special provisions. Due to the presence of very dense soils and shallow bedrock, we recommend pile shoes be considered for driven piles. Wang recommends a minimum of one test pile be performed at the North Abutment location.

5.0 QUALIFICATIONS

The analysis and recommendations submitted in this report are based upon the data obtained from the borings drilled at the locations shown on the *Boring Log* (Appendix A) and on the *TSL Plan* (Appendix B). This report does not reflect any variations that may occur between the borings or elsewhere on the site, variations whose nature and extent may not become evident until the course of construction. In the event that any changes in the design and/or location of the bridge are planned, we should be timely informed so that our recommendations may be adjusted accordingly.

It has been a pleasure to assist HBM Engineering Group, LLC. on this phase of the project. Please contact us if there are any questions, or if we can be of further service.

Respectfully Submitted,

WANG ENGINEERING, INC.

2-9-2017

Mohammed (Mike) Kothawala, P.E., D.GE
Sr. Project Manager/Sr. Geotechnical Engineer

Corina T. Farez, P.E., P.G.
Vice President

License expires
11-30-2017

MOHAMMED A KOTHAWALA
062-040483
LICENSED
PROFESSIONAL
ENGINEER
OF
ILLINOIS

REFERENCES

AASHTO (2015) *LRFD Bridge Design Specifications.* American Association of State Highway and Transportation Officials, Washington, D.C.

IDOT (2014) *All Bridge Designers Memorandum 14.2, Revised Scour Design Policy.* Illinois Department of Transportation

IDOT (2011) *All Geotechnical Manual Users Memorandum 10.2, Geotechnical Pile Design*, Illinois Department of Transportation.

IDOT (2012) *Bridge Manual*, Illinois Department of Transportation.

IDOT (2015) *Geotechnical Manual*, Illinois Department of Transportation.

IDOT (2016) *Standard Specifications for Road and Bridge Construction.* Illinois Department of Transportation.

EXHIBITS

Benchmark: USGS BM #31E Standard county bronze section marker Bronze Plus located at center of Wolf Rd. and German Church Rd. (83N). Elev. 621.81

Existing Structure: S.N. 016-0539 was constructed in 1924 under Section 142A-15D, Route 18. This structure has not undergone any major repairs. The structure consists of a five span, cast-in-place reinforced concrete T-beam bridge supported by reinforced concrete piers and abutments on spread footings. The five spans are each 60 feet long for a total length of 300 ft. The out to out bridge width is 39'-3" with a clear roadway width of 27' measured face to face of curbs. The existing bridge will be replaced with a prefabricated Pedestrian Truss Superstructure. Pier 3 will be rehabilitated and Piers 1, 2 and 4 will be removed. The top portion of the existing north abutment will be removed, and new spill thru abutment will be constructed behind the existing abutment. The top unsound portion (to a depth of 4') of the south abutment will be removed and reconstructed to meet new pedestrian bridge geometrics. The structure will be closed to vehicular traffic and pedestrians during construction.

Salvage: No salvage.

DESIGN SCOUR ELEVATION TABLE

Event / Limit State	Design Scour Elevations (ft.) S. Abut.	Pier 3	N. Abut.	Item 113
Q100	565.70	570.80	593.65	7
Q500	565.20	569.80	593.65	
Design	574.78	571.46	593.65	
Check	574.78	571.46	593.65	

DESIGN SPECIFICATIONS

2014 AASHTO LRFD Bridge Design Specifications, 7th Edition with 2015 and 2016 Interim Revisions
2009 AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges with 2015 Interim Revisions

HIGHWAY CLASSIFICATION

Functional Class N/A Bridge to be converted to Pedestrian/Bicycle Bridge Jurisdictional Transfer to Forest Preserve District of Cook County (FPDCC)

DESIGN STRESSES

FIELD UNITS (NEW CONSTRUCTION)

f'c = 3,500 psi (Substructure Concrete)
f'c = 4,500 psi (Superstructure Concrete)
fy = 60,000 psi (Reinforcement)
fy = 50,000 psi (M270 Gr. 50)-Prefab Truss

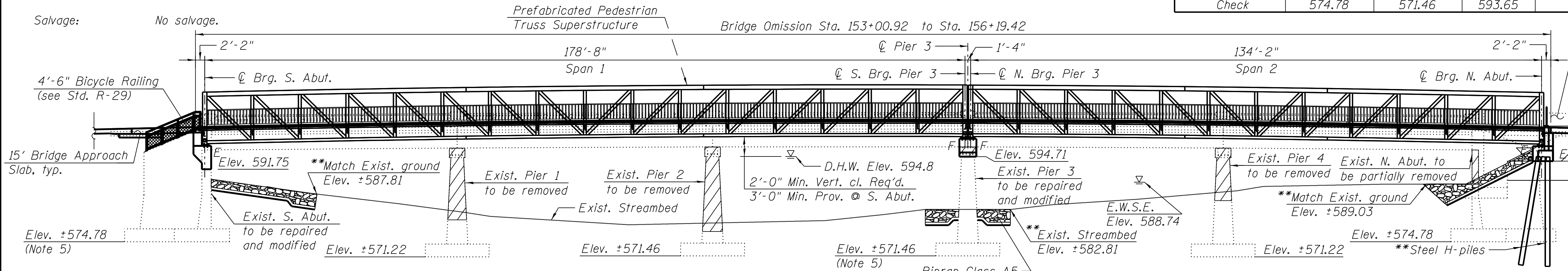
FIELD UNITS (EXISTING CONSTRUCTION)

f'c = 2,500 psi
fy = 33,000 psi (Reinforcement)

LOADING

* H5 (Maintenance vehicle)
Pedestrian Live Load: 90 psf

* H5 loading is in accordance with agreement between Forest Preserve District of Cook County (Owner) and other government agencies.



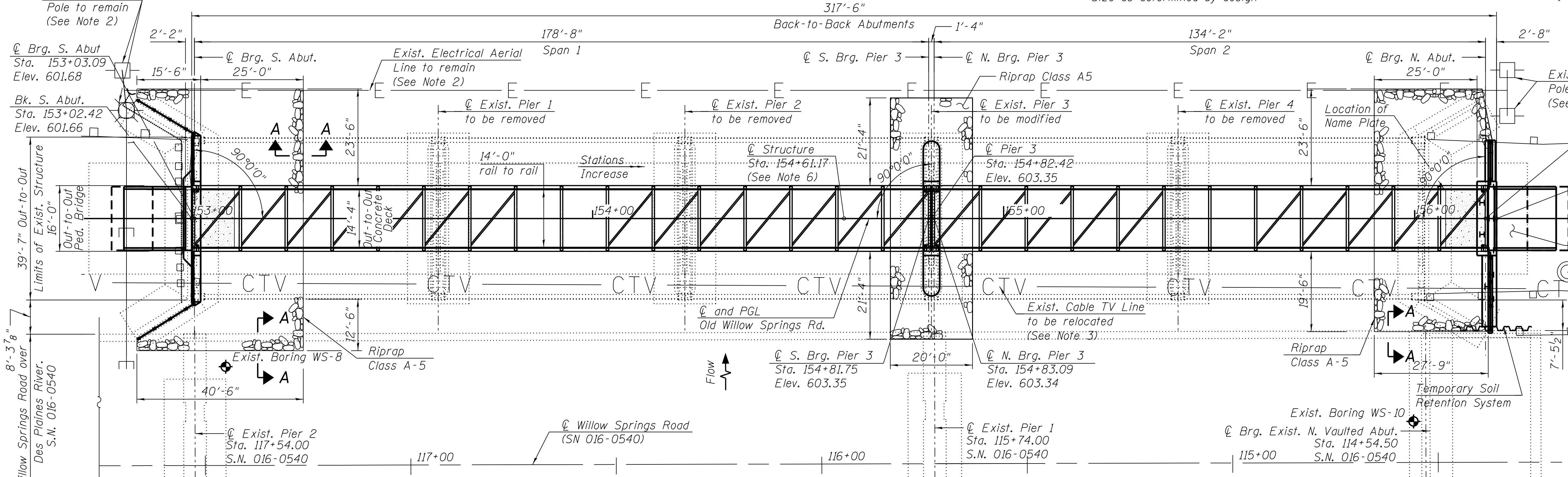
** Existing streambed elevations taken from the hydraulic report and adjusted by subtracting 1.19' from FEMA datum to tie into IDOT datum.

ELEVATION

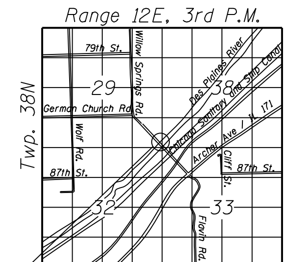
(Looking West)
(Wingwall sheet piling and H-piles are not shown for clarity)

** Size and spacing as determined by design

*** Size as determined by design



PLAN



LOCATION SKETCH

WATERWAY INFORMATION*

Drainage Area = 650 SQ. MI.
Existing Low Grade Elev. = 599.5 ft at Station 152+03
Proposed Low Grade Elev. = 599.5 ft at Station 152+03

Flood	Frequency (YR)	Discharge (CFS)	Waterway Opening (SQ FT) Existing	Waterway Opening (SQ FT) Proposed	Natural H.W.E. (FT)	Head (FT) Existing	Head (FT) Proposed	Headwater Elev. (FT) Existing	Headwater Elev. (FT) Proposed
	10	6000	2663	2755	593.7	0.8	0.8	594.5	594.5
Design	50	7500	2960	3059	594.8	0.8	0.8	595.6	595.6
Base	100	8400	3148	3256	595.6	0.8	0.8	596.4	596.4
Max Calc	500	9300	3310	3422	596.1	0.8	0.8	596.9	596.9

10 Year Velocity through Existing Bridge = 2.3 fps
ALL TIME H.W.E. & DATE= 595.5/July 1957
2 Year Peak Flow (Q) = 4,743 C.F.S.
Estimated Water Surface Elevation = 588.74

10 Year Velocity through Proposed Bridge = 2.2 fps

*All elevations are reduced 1.2' from June, 2000 Hydraulic Report to correlate with April, 2012 Elevation Data.

BORINGS

BORING	LOCATION
WS-8	Sta. 153+10.69, 36.04' Rt.
WS-9	Sta. 154+83.77, 67.61' Rt.
WS-10	Sta. 155+99.73, 49.33' Rt.

SEISMIC DATA

Seismic Performance Zone (SPZ) = 1
Design Spectral Acceleration at 1.0 sec. (S_{D1}) = 0.064 g
Design Spectral Acceleration at 0.2 sec. (S_{DS}) = 0.118 g
Soil Site Class = C

NOTES:

1. For Section A-A, see Sheet 3 of 3.
2. The work should be undertaken in close coordination with ComEd. The Contractor and ComEd will need to verify clearances during construction.
3. The existing Cable TV line at the bottom of deck shall be relocated if coordination determines that the utility line is active.
4. Layout of riprap may be varied to suit the ground in the field as directed by the Engineer.
5. Bottom of existing footing Elev. based on historical documents and survey.
6. Bridge Stations based on survey.

GENERAL PLAN
OLD WILLOW SPRINGS ROAD OVER DES PLAINES RIVER PEDESTRIAN BRIDGE
TR RTE. 9250 SECTION 142A-B
COOK COUNTY
STA. 154+61.17
EXISTING STRUCTURE NO. 016-0539
PROPOSED STRUCTURE NO. 016-0539

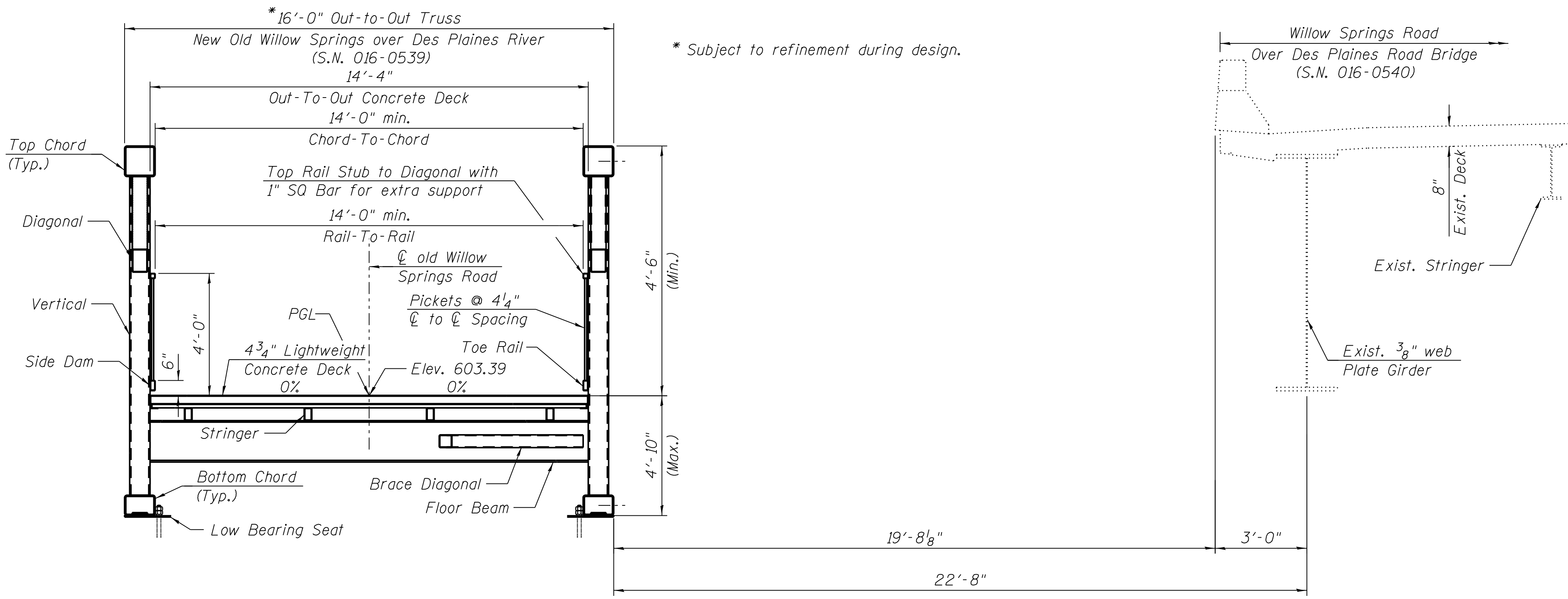
HBM ENGINEERING GROUP, LLC. CONSULTING & DESIGN INSPECTION & RATING RESEARCH & TESTING
4415 WEST HARRISON ST. SUITE 231 HILLSIDE, IL 60162 PHONE: (708) 236-0900 FAX: (708) 236-0901

S1 - GPE.dgn	DESIGNED - MI, JMG, KJD	REVISED -	STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION
USER NAME = robert.boro	DRAWN - KJD	REVISED -	
PLOT SCALE = 28.00 ' / in.	CHECKED - MI, MAI	REVISED -	
PLOT DATE = 11/16/2016	DATE - 11/16/2016	REVISED -	

TR RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
9250	142A-B	COOK	3	1

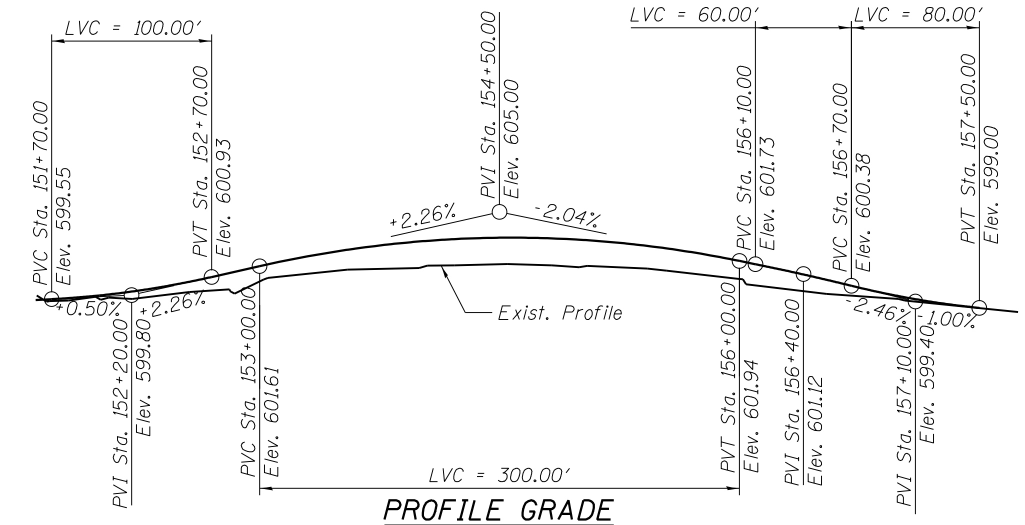
CONTRACT NO. 62B99
ILLINOIS FED. AID PROJECT

FILE PATH : P:\1111-532 IDOT PTB161 Item 8 (Various-Various)\Work Order #22 Old Willow Springs Rd over Des Plaines River\TS&L Plans\S1 - GPE.dgn



BRIDGE SECTION

(At ℄ of Structure Sta. 154+58.01)

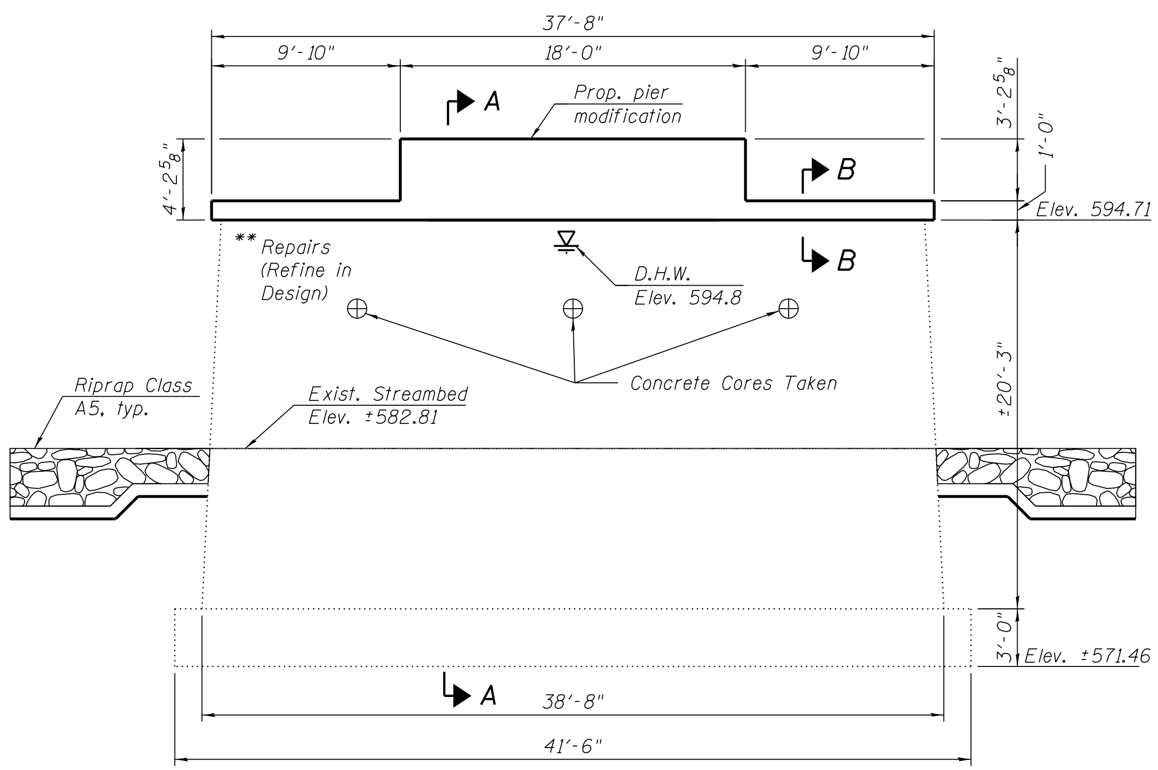


PROFILE GRADE

(Old Willow Spring Road)

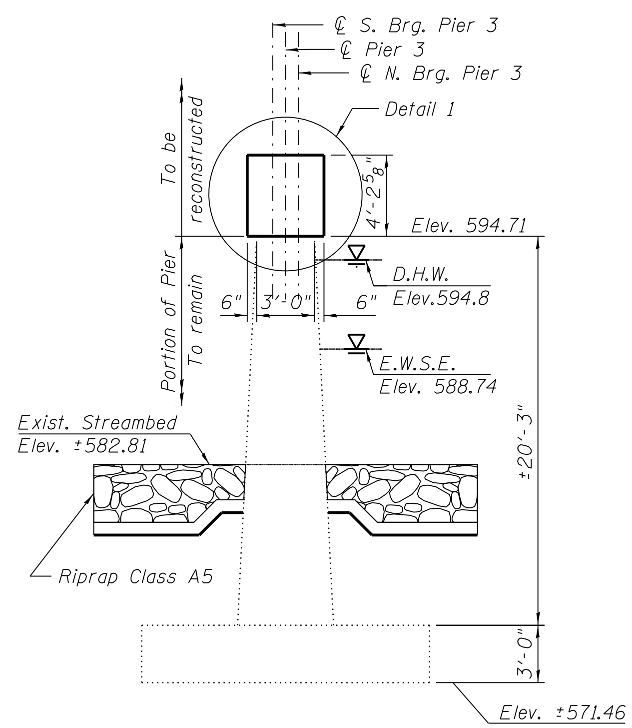
SCOPE OF WORK:

1. Relocate the existing Cable TV line (by others) prior to Letting (Coordinate) at bottom of deck if coordination determines that the utility line is active.
2. Remove the concrete bridge deck, sidewalks, parapets, railings, deck drains, expansion joints and all other superstructure appurtenances.
3. Remove Piers 1, 2 and 4 to a depth 2' below river bed.
4. Remove the concrete cap of Pier 3. Reconstruct cap to meet new bridge seat elevations for a length of 18'-0" (9'-0" on each side of centerline of bridge). Reconstruct the remaining portions of pier to an elevation 1'-0" above removal line.
5. Repair remaining exposed portions of Pier 3 using structural repair of Concrete.
6. Construct new north abutment consisting of a spill-thru abutment supported on HP-piles behind existing abutment.
7. Remove existing north abutment and wingwalls to a depth of 2' below proposed grade.
8. Remove portions of the south abutment (approx. to a depth of 5'-6"') that were found to be unsound. Reconstruct abutment to meet new bridge seat elevations for a length of 18'-0" (9'-0" on each side of centerline of bridge). Reconstruct the remaining portions of abutment to match top of wingwall elevations.
9. Repair remaining, exposed portions of south abutment and wingwalls utilizing epoxy crack injection and structural repair of concrete.
10. Re-grade area in front of abutments and wingwalls and install riprap.
11. Install Prefabricated Pedestrian Truss Superstructure, and fencing.
12. Construct approach pavements and approach roadways.
13. Install the 4'-6" Bicycle Railing above abutments (areas outside bridge limits) and wingwalls.
14. Existing streambed elevations were taken from the hydraulic report and adjusted by subtracting 1.19' from FEMA datum to tie into the IDOT datum.

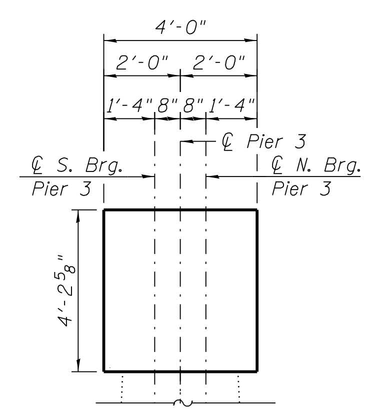


PIER 3 MODIFICATION

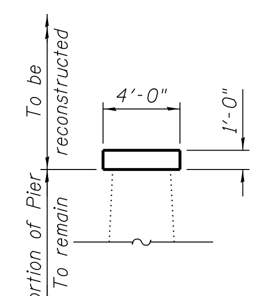
** Concrete Cores Taken



SECTION A-A



DETAIL 1



SECTION B-B

GENERAL PLAN
OLD WILLOW SPRINGS ROAD
OVER DES PLAINES RIVER
PEDESTRIAN BRIDGE
TR RTE. 9250 SECTION 142A-B
COOK COUNTY
STA. 154+58.01
EXISTING STRUCTURE NO. 016-0539
PROPOSED STRUCTURE NO. 016-0539

HBM ENGINEERING GROUP, LLC. CONSULTING & DESIGN INSPECTION & RATING RESEARCH & TESTING
4415 WEST HARRISON ST. SUITE 231 HILLSIDE, IL 60162 PHONE: (708) 236-0900 FAX: (708) 236-0901

S2 - Details.dgn	DESIGNED - MI, JMG, KJD	REVISED -	STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION
USER NAME = robert.boro	DRAWN - KJD	REVISED -	
PLOT SCALE = 10.0 ' / in.	CHECKED - MI, MAI	REVISED -	
PLOT DATE = 11/16/2016	DATE - 11/16/2016	REVISED -	SHEET 2 OF 3 SHEETS

TR RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
9250	142A-B	COOK	3	2
		CONTRACT NO. 62B99		
	ILLINOIS FED. AID PROJECT			

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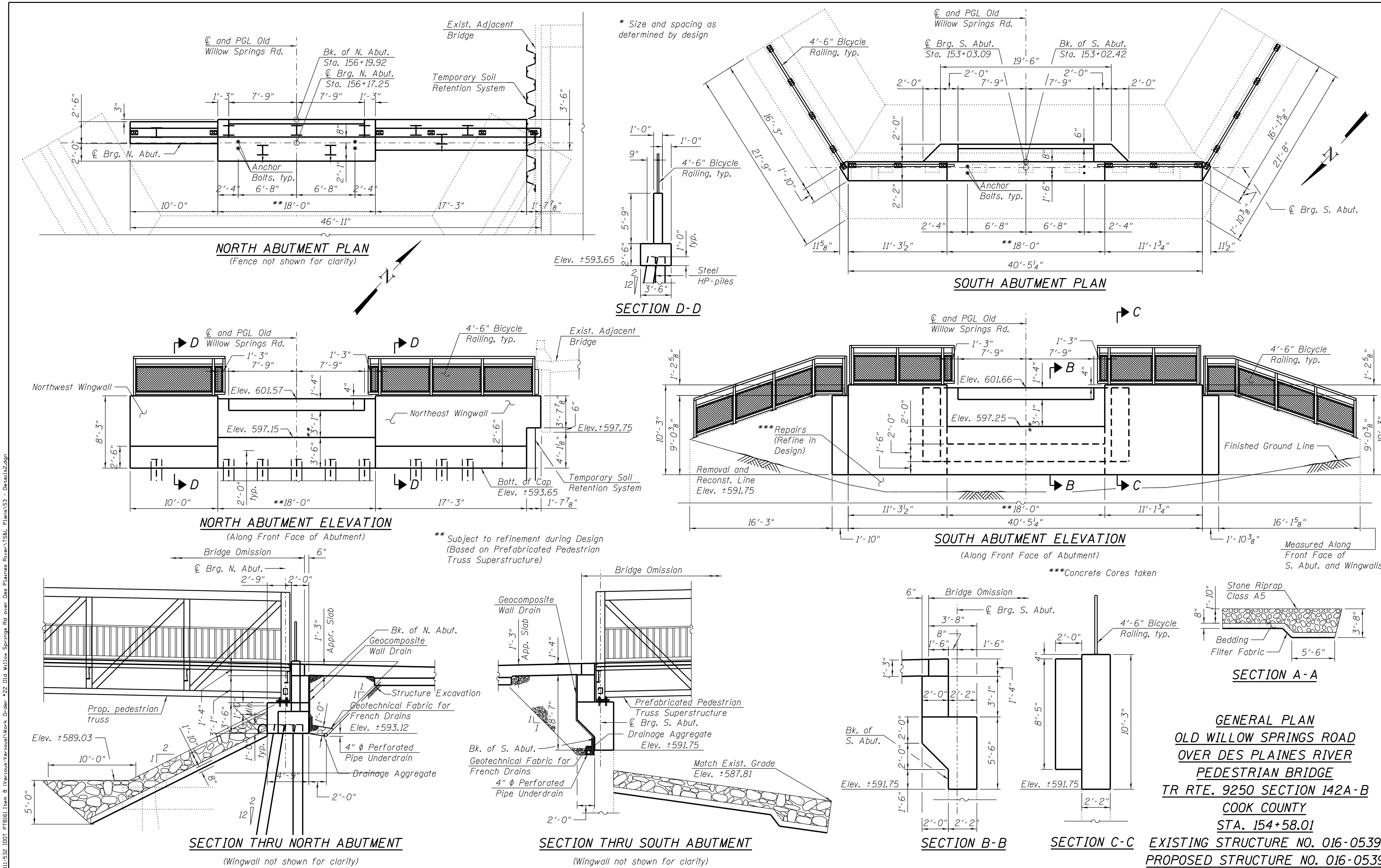

GENERAL PLAN
OLD WILLOW SPRINGS ROAD
OVER DES PLAINES RIVER
PEDESTRIAN BRIDGE
TR RTE. 9250 SECTION 142A-B
COOK COUNTY
STA. 154+58.01
EXISTING STRUCTURE NO. 016-0539
PROPOSED STRUCTURE NO. 016-0539

FILE PATH : P:\1111-532 IDOT PTB161 Item 8 (Various-Various)\Work Order #22 Old Willow Springs Rd over Des Plaines River\TS&L Plans\S3 - Details2.dgn

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S3 - Details2.dgn	DESIGNED - MI, JMG, KJD	REVISED -	
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PLOT DATE = 11/16/2016	DATE - 11/16/2016	REVISED -	

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

SHEET 3 OF 3 SHEETS

TR RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
9250	142A-B	COOK	3	3

CONTRACT NO. 62B99
ILLINOIS FED. AID PROJECT

APPENDIX A

FORM NO. B.D. 137 REV. 9-60

BRIDGE FOUNDATION BORING LOG

Sh. of Sh.

PROJECT PC-91-019-71
BRIDGE Willow Springs Road over Chicago Sanitary & Ship Canal
Date 4/26/72, 4/27/72
ROUTE FAS 119 (SA-18), Section 142B
Bored By Kauzlarich, LW
STA.
Checked By Olsen, HNTB
COUNTY Cook
Boring No. WS-8
Station 117+45
Offset 24' Rt.
Surface Water El. 591.1
Groundwater El. at Completion
After Hours

Description	Elevation / Depth	N	Qu t/sf	w (%)
Ground Surface 591.1	0			
WATER. 589.4				
Gray, soft, CLAY LOAM, trace of gravel, moist.	-5	14	0.4B	29
Brown, soft to firm CLAY, trace of sand, moist. 583.1		5	0.7P	77
Black, very soft to soft, organic SILT, with shells and fiberous material, moist.	-10	3	0.19B 0.2P	164
		2	0.4B	186
577.1				
Gray-brown, soft, organic SILT, trace of shells, moist. 575.1	-15	3	0.6B 0.5P	154
Gray, soft, SILTY CLAY, moist. 573.6		3	0.4B 0.2P	48
Gray, very dense, broken LIMESTONE, with layers of sand and gravel, moist.		100		
	-20			
		100/ 7"		

Description	Elevation / Depth	N	Qu t/sf	w (%)
		100/ 8"		
566.1	-25			
Gray, hard, LIMESTONE, with thin layers of sand or silt (Maximum thickness 10"±)		(Drilled with 2" roller bit)		
	-30			
	-35			
553.1				
Gray, hard, LIMESTONE, solid. 550.6	-40			
Bottom of boring.				
	-45			

N - Standard Penetration Test - Blows per foot to drive 2" O.D. Split Spoon Sampler 12" with 140# hammer falling 30".

Qu - Unconfined Compressive Strength - t/sf

w - Water Content - Percentage of oven dry weight - %

Type Failure:
B - Bulge Failure
S - Shear Failure
E - Estimated Value
P - Penetrometer

FORM NO. B.D. 137 REV. 9-60

BRIDGE FOUNDATION BORING LOG

Sh. of Sh.

PROJECT: PC - 91 - 819 - 71

ROUTE: FAS 119 (SA-18), Section 142B

BRIDGE: Willow Springs Road over Chicago Sanitary & Ship Canal

STA.

Date: 4/28/72, 4/29/72

Bored By: Kauzlarich, LW

Checked By: Ulees, BNTB

COUNTY: Cook

Boring No.: WS-9

Station: 115+72

Offset: 8' Lt.

Surface Water El.: 590.6

Groundwater El. at Completion:

After ___ Hours:

Elevation	Description	N	Qu t/s.f.	w (%)
590.6	Ground Surface			
	WATER.			
580.6	Black, very soft, SILT, with organic material & shells.			
575.6	Gray, very dense, fine to coarse SAND, with layers of very stiff, CLAY LOAM, moist.	57		24
572.1	Gray, very dense, fine to coarse SAND & GRAVEL, moist.	[illegible]*		
	Gray, very dense, fine to coarse SAND & GRAVEL, with trace of SILTY LOAM, moist.	89		
		73		
		60		
		100		
560.1	Gray, hard, solid, LIMESTONE.	(Drilled with 2" roller bit)		
556.6	Bottom of boring.			

*Sampler bouncing

N - Standard Penetration Test - Blows per foot to drive 2" O.D. Split Spoon Sampler 12" with 140# hammer falling 30".

Qu - Unconfined Compressive Strength - t/s.f.

w - Water Content - Percentage of oven dry weight - %

Type Failure:
B - Bulge Failure
S - Shear Failure
E - Estimated Value
P - Penetrometer

FORM NO. B.D. 137 REV. 9-60

BRIDGE FOUNDATION BORING LOG

Sh. of Sh.

PROJECT PC-91-819-71

ROUTE FAS 119 (SA-18), Section 142B

COUNTY Cook

BRIDGE Willow Springs Road over Chicago Sanitary & Ship Canal

STA.

Date 5/2/72

Bored By Pagenkopf, LW

Checked By Ulses, HNTB

Boring No. WS-10

Station 114+56

Offset 10' Rt.

Ground Surface 589.6

Surface Water El. 589.6

Groundwater El. at Completion

After ___ Hours

Description	Elevation	N	Qu t/s.f.	w(%)
Ground Surface	589.6 (0)			
WATER.	589.1			
Dark brown to gray-brown, soft to firm, CLAY, trace of organic, moist. A-7-6(20)		2	0.8B 1.0P	60
		*		
	584.6 (-5)			
Gray, very soft, SILTY CLAY LOAM, wet. A-4(8)		U-1	0.2P 0.5S	30
	583.1			
Gray, medium dense, fine to coarse SANDY LOAM, with gravel and layers of clay loam, moist.		27		
	(-10)	25		
	578.6			
Gray, hard, CLAY LOAM, with sand layers, moist.		44	6.8B 4.5+P	12
	575.6			
Gray, dense, SILTY LOAM to SILT, trace of gravel, moist.	(-15)	34		
		32		18
	569.6 (-20)			
Gray, very dense, broken LIMESTONE, with clay loam layers, moist.		107		

Description	Elevation	N	Qu t/s.f.	w(%)
Gray, very dense, broken LIMESTONE.		100/8"		
	565.3			
Hard, blue-gray LIMESTONE, layers interbedded with silt or sand layers.	(-25)	NX R-1	Rec.=30% RQD=10%	
	(-30)	NX R-2	Rec.=50% RQD=0%	
	556.8			
Light gray, hard, LIMESTONE, broken with layers of clay loam.	(-35)	NX R-3	Rec.=30% RQD=0%	
	553.1			
Light gray, hard, LIMESTONE, solid.	(-40)	NX R-4	Rec.=100% RQD=100%	
	548.5			
Bottom of boring.				
	(-45)			

*Sample U-2: 2.0' - 4.0'
3" undisturbed tube
w = 45%
Qu = 0.62B
1.0P

N - Standard Penetration Test - Blows per foot to drive 2" O.D. Split Spoon Sampler 12" with 140# hammer falling 30"

Qu-Unconfined Compressive Strength - t/s.f.

w - Water Content - Percentage of oven dry weight %

Type failure:
B - Bulge Failure
S - Shear Failure
E - Estimated Value
P - Penetrometer

APPENDIX B

S E I S M I C   S I T E   C L A S S   D E T E R M I N A T I O N

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

Modified on 12/10/10

PROJECT TITLE===== Old Willow Springs Road Over Des Plaines River Pedestrian Bridge (SN 016-0539)

Substructure 1

Base of Substruct. Elev. (or ground surf for bents)	574.78	ft.
Pile or Shaft Dia.		inches
Boring Number	WS-08	
Top of Boring Elev.	589.4	ft.
Approximate Fixity Elev.	574.78	ft.

Individual Site Class Definition:

N (bar):	72	(Blows/ft.)	Soil Site Class C
N_{ch} (bar):	100	(Blows/ft.)	Soil Site Class C <----Controls
s_u (bar):		(ksf)	NA, H < 0.1*H (Soil)

Seismic Soil Column Depth (ft)	Bot. Of Sample Elevation	Sample Thick. (ft.)	N	Qu (tsf)	Layer Description Boundary
	588.1	1.30	14	0.40	
	585.6	2.50	14	0.40	
	583.1	2.50	5	0.70	B
	580.6	2.50	3	0.19	B
	578.1	2.50	2	0.40	
	575.6	2.50	3	0.60	
1.2	573.6	2.00	3	0.40	B
3.7	571.1	2.50	100		
6.2	568.6	2.50	100		
9.2	565.6	3.00	100		
11.7	563.1	2.50	100		
14.2	560.6	2.50	100		
16.7	558.1	2.50	100		
19.2	555.6	2.50	100		
22.2	552.6	3.00	100		B
100.0	474.8	77.80	100	5.00	R

Substructure 2

Base of Substruct. Elev. (or ground surf for bents)	571.46	ft.
Pile or Shaft Dia.		inches
Boring Number	WS-09	
Top of Boring Elev.	580.6	ft.
Approximate Fixity Elev.	571.46	ft.

Individual Site Class Definition:

N (bar):	97	(Blows/ft.)	Soil Site Class C <----Controls
N_{ch} (bar):	97	(Blows/ft.)	Soil Site Class C
s_u (bar):		(ksf)	NA, H < 0.1*H (Soil)

Seismic Soil Column Depth (ft)	Bot. Of Sample Elevation	Sample Thick. (ft.)	N	Qu (tsf)	Layer Description Boundary
	578.1	2.50	1	0.20	
	575.6	2.50	1	0.20	B
	573.1	2.50	57		
	572.1	1.00	57		
1.9	569.6	2.50	80		
4.4	567.1	2.50	89		
5.4	566.1	1.00	89		
7.4	564.1	2.00	73		
9.9	561.6	2.50	60		
11.9	559.6	2.00	100		B
100.0	471.5	88.10	100	5.00	R

Substructure 3

Base of Substruct. Elev. (or ground surf for bents)	593.65	ft.
Pile or Shaft Dia.	12	inches
Boring Number	WS-10	
Top of Boring Elev.	589.1	ft.
Approximate Fixity Elev.	587.65	ft.

Individual Site Class Definition:

N (bar):	28	(Blows/ft.)	Soil Site Class D
N_{ch} (bar):	79	(Blows/ft.)	Soil Site Class C <----Controls
s_u (bar):	3.79	(ksf)	Soil Site Class C

Seismic Soil Column Depth (ft)	Bot. Of Sample Elevation	Sample Thick. (ft.)	N	Qu (tsf)	Layer Description Boundary
2.0	585.6	3.50	2	0.80	
4.5	583.1	2.50	1	0.62	B
7.0	580.6	2.50	27		
9.0	578.6	2.00	25		B
12.1	575.6	3.00	44	6.80	B
14.6	573.1	2.50	34		
17.1	570.6	2.50	32		
18.1	569.6	1.00	32		B
20.6	567.1	2.50	107		
21.8	565.8	1.30	100		B
100.1	487.6	78.20	100	5.00	R

Substructure 4

Base of Substruct. Elev. (or ground surf for bents)		ft.
Pile or Shaft Dia.		inches
Boring Number		
Top of Boring Elev.		ft.
Approximate Fixity Elev.		ft.

Individual Site Class Definition:

N (bar):		(Blows/ft.)	NA
N_{ch} (bar):		(Blows/ft.)	NA
s_u (bar):		(ksf)	NA

Seismic Soil Column Depth (ft)	Bot. Of Sample Elevation	Sample Thick. (ft.)	N	Qu (tsf)	Layer Description Boundary

Global Site Class Definition: Substructures 1 through 3

N (bar):	66	(Blows/ft.)	Soil Site Class C
N_{ch} (bar):	92	(Blows/ft.)	Soil Site Class C
s_u (bar):	4.36	(ksf)	Soil Site Class C <----Controls