STRUCTURE GEOTECHNICAL REPORT
WESTERN AVENUE (FAP 370) SOUTH APPROACH OVER THE CAL-SAG CHANNEL
SN 016-0777, SECTION 0103BR-1
IDOT D-91-581-10, CONTRACT 60K72
COOK COUNTY, ILLINOIS

for

Bollinger, Lach &Associates, Inc.
333 Pierce Road, Suite 200
Itasca, IL 60143
(630) 438-6400

submitted by

Wang Engineering, Inc.
1145 North Main Street
Lombard, IL 60148
(630) 953-9928

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9. Prepared by Wang Engineering, Inc. 1145 N Main Street Lombard, IL 60148	**Contributor(s)** Author: Mickey L. Snider, P.E. Project Manager: Liviu Iordache QA/QC: Jerry W.H. Wang, PhD, P.E.	**Contact Phone Number** (630) 953-9928 x 27 MSnider@wangeng.com
10. Prepared for Bollinger, Lach & Associates 333 Pierce Road Itasca, IL 60143	**Design / Structural Engineer** Joel Ihde, P.E., S.E.	**Contact Phone Number** (630) 438-6400

11. Abstract

The existing, eight-span south approach structure of the Western Avenue Bridge over the Cal-Sag Channel will be removed and replaced with a new, five-span structure tying into the existing south main span pier (Pier 9). The bridge will have a stub abutment and multi-column piers with caps. This report provides geotechnical recommendations for the design of proposed bridge foundations and embankments.

The existing embankment material behind the south abutment consists of loose to medium dense silty loam fill. Beneath the surface, borings encountered about 10 feet of loam, sandy loam, and stiff to very stiff silty clay fill overlying 5 to 8 feet of very loose organic silt and fibrous peat. Deeper foundation soils include dense to very dense gravelly silty loam and weathered rock overlying strong, fair to excellent quality dolostone bedrock. The site classifies in the Seismic Class D for the new structure and foundations and in the Soil Profile Type I for the reuse and tie-in to the existing Pier 9.

The profile grade will not be significantly changed; thus, we do not anticipate settlement-related problems. The external stability of the south approach embankment is adequate due to the support provided on both sides by existing concrete retaining walls that will remain in place.

The proposed south abutment and approach piers could be supported on steel H-piles (size HP12x53, HP12x74, or HP14x73) designed for the maximum nominal bearing at the top of bedrock or on 3.0- to 6.0-foot diameter drilled shafts socketed into the bedrock. Piles lengths would be about 40 feet at the abutment and 25 feet at the piers; rock sockets would be about 3 to 9 feet long depending on diameter. Some H-piles will require precoring to the top of bedrock and 1- to 2-foot sockets to avoid relatively high vibration levels at adjacent two-story wooden structures.

Stage construction along the south abutment should be supported by flexible steel sheet piling, designed according to IDOT *Design Guide 3.13.1*. The pier excavations will require support of the very loose, organic soils and should include the pay item, *Temporary Soil Retention System* if the excavations cannot be sloped at 1:2.5 (V:H). Temporary or permanent casing will be required at the shaft excavations to prevent issues related to the presence of groundwater and granular soil conditions.

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Lombard, Illinois 60148
Phone (630) 953-9928
www.wangeng.com

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BOLLINGER, LACH & ASSOCIATES

1.0 INTRODUCTION

This report presents the results of our subsurface investigation, laboratory testing, and geotechnical evaluations for the reconstruction of the south approach of the Western Avenue (FAP 370) Bridge over the Calumet-Saganashkee (Cal-Sag) Channel in Blue Island, Cook County, Illinois. A *Site Location Map* is presented as Exhibit 1.

1.1 Proposed Structure

Wang Engineering, Inc. (Wang) understands Bollinger, Lach & Associates (BLA) envisions a new, five-span structure with pile-supported stub abutment and piers replacing the existing eight span south approach structure. The replacement begins at the south abutment and continues to the existing Pier 9, which serves as the south main span pier of the Cal-Sag truss bridge. The main span, as well as Pier 9, will not be replaced as part of this contract. The General Plan and Elevation (GPE) drawing provided by BLA shows the south approach will have a total length of 517.4 feet with span lengths of 106.5, 109.5, 109.5, 109.5, and 79.3 feet. The out-to-out width will measure 68.0 feet to accommodate two 10.5-foot wide traffic lanes, two 11.0-foot wide traffic lanes, a 3.0-foot wide median, two 5.0-foot wide bicycle lanes, and two 5-foot wide sidewalks. The south abutment has a concrete slope wall in front and is supported on both sides by concrete retaining walls.

The purpose of our investigation was to characterize the site soil and groundwater conditions, perform geotechnical analyses, and provide recommendations for the design and construction of structure foundations and temporary soil retention systems.

1.2 Existing Structure

According to the Bridge Condition Report provided by BLA, the existing south approach was built in

1966 as an eight-span, prestressed, precast concrete structure leading to the through truss main span over the channel. The length from the south abutment to main span Pier 9 is approximately 526 feet and the out-to-out width of the approach is 67.8 feet. The substructures consist of a cast-in-place concrete spill-through abutment supported on steel piles and reinforced concrete bent column piers also supported on steel piles. The south abutment is currently supported by concrete retaining walls with staircases on either side.

2.0 SITE CONDITIONS AND GEOLOGICAL SETTING

The site is located in the City of Blue Island, southern Cook County, Illinois. On the USGS *Blue Island 7.5-minute series* map the project is located in the SE ¼ of Section 36, Tier 37 N, Range 13 E of the 3rd Principal Meridian.

The following review of the published geologic data, with emphasis on factors that might influence the design and construction of the proposed engineering works, is meant to place the project area within a geological framework and, thus, to confirm the dependability and consistency of the present subsurface investigation results. For the study of the regional geologic framework, Wang considered western Illinois area in general and Cook County in particular. Exhibit 2 illustrates the *Site and Regional Geology*.

2.1 Physiography

The project area is in an urban area of Blue Island. The surrounding land use is residential and commercial with some constructed within a few feet of the edge of the bridge. Protection of the structure foundations may impact the selection of the pier type, as adjacent excavations may affect these buildings. The approach along Western Avenue is an elevation of about 610 feet behind the south abutment and flies over Broadway Street and Canal Street, which are at about 595 feet in elevation. As the approach reaches the channel bridge the ground surface beneath declines sharply into the channel, which has a surface elevation of about 577 feet.

2.2 Surficial Cover

Quaternary glacigenic deposits unconformably overlie the Paleozoic bedrock. During the Michigan Subepisode (26,000 to 11,000 B.P.) of the Wisconsin glaciation, a glacial lobe extended over northeastern and north central Illinois (Hansel and Johnson 1996). Cook County was under the influence of an extrusion of this lobe, the Joliet sublobe, ultimately responsible for the formation of a series of arcuate, end moraine ridges, separated by low-relief till plains, lake plains, and outlet valleys (Johnson and Hansel 1999, Willman 1971).

The investigated area was built on outwash deposits of the Henry Formation of the Mason Group. Specifically, the bridge spans stratified sand and gravel with lenses of silt, clay, and occasional organic debris. In the project area, the drift thickness is less than 25 feet (Hansel and Johnson, 1996).

2.3 Bedrock

The uppermost bedrock unit in most of Cook County consists of Silurian-age dolostones of the Racine Formation, which can be as thick as 270 feet in the eastern part of the county and thins out toward the western boundary (Willman, 1971; Willman et al., 1975). The top of bedrock in the project area lies at elevations between 550 and 575 feet (NAVD88), that is, it may be encountered at about 25 to 50 feet below ground surface (bgs). No underground mines are known in the area.

Our subsurface investigation results fit into the local geologic context. The borings drilled in the project area revealed that under man-made fill, the native sediments at the project site consist of gravel and sand mixed with organic deposits of the Henry Formation and gray dolostone of the Racine Formation. The borings encountered the top of the bedrock at about 20 to 25 feet bgs.

3.0 METHODS OF INVESTIGATION

The following sections outline the subsurface and laboratory investigations performed by Wang.

3.1 Subsurface Investigation

The subsurface investigation performed by Wang consisted of five structure borings, designated as BSB-01 through BSB-05, drilled in November and December 2012. The borings were drilled from elevations of 613.6 to 593.1 feet to depths of 35.0 to 67.0 feet bgs. Northings, eastings, and elevations were included in the topographic design drawings provided by BLA; stations and offsets were taken from plan and alignment drawings. The boring location data are shown in the *Boring Logs* (Appendix A), and the as-drilled locations are shown in the *Boring Location Plan* (Exhibit 3).

A truck-mounted drill rig, equipped with hollow stem augers, was used to advance and maintain an open borehole. Soil sampling was performed according to AASHTO T 206, "*Penetration Test and Split Barrel Sampling of Soils*." The soil was sampled at 2.5-foot intervals to 30 feet bgs and at 5.0-

foot intervals to the top of bedrock. Soil samples from each interval were placed in sealed jars for further laboratory testing. Each boring included bedrock coring in 5-foot runs with a NWD4-sized core barrel. Photographs of the bedrock cores are provided in Appendix B.

Field boring logs, prepared and maintained by a Wang geologist include lithological descriptions, visual-manual soil classifications, results of Rimac and pocket penetrometer unconfined compressive strength tests, and results of Standard Penetration Tests (SPT), recorded as blows per 6 inches of penetration.

Groundwater observations were made during and after drilling operations. The borings were backfilled with soil cuttings after completion. The surface along Western Avenue was restored as close as possible to the original condition.

3.2 Laboratory Testing

The soil samples were tested in the laboratory for moisture content (AASHTO T 265). The soils were classified according to the IDH Textural Classification system and field visual-manual descriptions were verified in the laboratory. The laboratory results are shown in the *Boring Logs* (Appendix A).

4.0 RESULTS OF FIELD AND LABORATORY INVESTIGATIONS

Detailed descriptions of the soil conditions encountered during the subsurface investigation are presented in the attached *Boring Logs* (Appendix A) and in the *Soil Profile* (Exhibit 4). Please note that strata contact lines represent approximate boundaries between soil types. The actual transition between soil types in the field may be gradual in horizontal and vertical directions.

4.1 Soil Conditions

In descending order, the general lithologic succession encountered along the south approach includes: 1) man-made ground (fill); 2) very loose organic silt and with some fibrous peat; 3) dense to very dense gravelly silty loam; and 4) fair to excellent rock quality dolostone.

(1) Man-made ground (Fill)

Immediately beneath 3-inch thick asphalt and 9-inch thick concrete pavement along Western Avenue, Boring BSB-01 encountered about 23 feet of loose to medium dense, brown silty loam fill with little to some gravel. The silty loam has N-values of 4 to 23 blows/foot with an average of about 12 blows/foot and moisture content values of 14 to 23% with an average of 18%.

Beneath the Western Avenue embankment fill and beneath the surface along the Broadway and Canal Street elevations, the borings encountered various fill materials. The fill consists of loose to medium dense loam and sandy loam with N-values of 6 to 16 blows/foot overlying stiff to very stiff, brown and gray silty clay with unconfined compressive strength (Q_u) values of 1.0 to 2.3 tsf and moisture content values of 18 to 35%.

(2) Very loose organic silt and fibrous peat

At elevations of 583 to 586 feet, the borings encountered 5 to 8 feet of very loose, brown and gray organic silt with shells underlying a thin layer of brown, fibrous peat. The organic silt has N-values of 3 blows/foot or less and moisture content values of 83 to 243%. The peat has a moisture content of 355%.

(3) Dense to very dense sand and gravelly silty loam

Underlying the organic materials, deeper foundation soils consisted of dense to very dense, gray gravelly silty loam and sand/sandy loam continuing to the top of bedrock. The gravelly silty loam and sand has N-values of 31 to 78 blows/foot, and sampling recorded primarily spoon refusals due to the gravel content. The material contains a noticeably higher percentage of weathered bedrock fragments beginning at about 4 to 8 feet above the top of the bedrock.

(4) Strong, fair to excellent quality dolostone

At elevations of 567 to 568 feet, the borings encountered strong, fair to excellent quality, unweathered and horizontally-bedded dolostone. The dolostone has rock quality designation (RQD) values of 63 to 100%; uniaxial compressive strength tests performed on intact rock samples show compressive strength values of approximately 12,000 to 13,500 psi. The bedrock was encountered at 47 feet below the south approach embankment behind the abutment and at about 25 to 28 feet below the surface along the piers.

4.2 Groundwater Conditions

Groundwater was encountered while drilling between elevations of 573.5 and 579.7 feet (15 to 21 feet bgs along Broadway and Canal Streets) generally associated with the sand and silty loam. The normal water elevation in the Cal-Sag Channel, noted in the bridge condition report, is about 577.5 feet, which correlates fairly well with the water levels encountered during the investigation.

4.3 Seismic Design Considerations

The soils within the top 100 feet have a weighted average N-value of 50 blows/foot (AASHTO 2012; Method C controlling). These results classify the site in Seismic Site Class D in accordance with IDOT *All Geotechnical Manual Users (AGMU) 9.1* (2010); the project location belongs to Seismic Performance Zone 1. The seismic spectral acceleration parameters recommended for design in accordance with the 2012 AASHTO *LRFD Design Specifications* are summarized in Table 1 (AASHTO 2012). The factor of safety (FOS) against liquifacton for the saturated sandy soils along the bridge site is greater than the AASHTO-required value of 1.1 (AASHTO 2012).

Table 1: Seismic Design Parameters

Spectral Acceleration Period (sec)	Spectral Acceleration Coefficient[1)] (% g)	Site Factors	Design Spectrum for Site Class D[2)] (% g)
0.0	PGA= 4.4	F_{pga}= 1.6	**A_s= 7.1**
0.2	S_S= 9.6	F_a= 1.6	**S_{DS}= 15.3**
1.0	S_1= 3.8	F_v= 2.4	**S_{D1}= 9.0**

1) Base spectral acceleration coefficients from AASHTO (2012)

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

We understand the reuse of Pier 9 as the end span of the south approach will require design in accordance with LFD, or Allowable Stress Design (ASD). The seismic data required to be shown on the plans in accordance with the AASHTO *Standard Specifications for Highway Bridges* (2002) is included below.

Soil Profile Type: I
Bedrock Acceleration Coefficient (A): 0.037g
The Site Coefficient (S): 1.0
Seismic Performance Category (SPC): A

5.0 FOUNDATION ANALYSIS AND RECOMMENDATIONS

Geotechnical evaluations and recommendations for the approach embankment, approach slab, and structure foundations are included in the following sections. We estimate the global stability of the structure and foundations is adequate and the approach will not undergo excessive settlements. Wang recommends supporting the proposed abutment and piers on driven piles or drilled shafts socketed into the underlying bedrock. Shallow foundations would undergo excessive deformations due to the presence of organic soils and are not recommended.

The GPE shows the proposed abutment constructed immediately in front of the existing. The existing pile foundations will remain in place and the required portions of the abutment will be cut off below the ground surface. The piers are positioned such that the new caps will not interfere with the existing caps; therefore, the existing piers will also be cut off below the ground surface and left in place. Both the existing abutment and piers have battered piles included in their foundations. The proposed pile or shaft layouts will need to be positioned to miss the existing piles and caps.

5.1 Approach Embankments and Slabs

Wang has performed preliminary evaluations of the settlement and global stability for the south approach embankment behind the abutment and the approach slab based on the soil conditions encountered in Boring BSB-01. We do not anticipate excessive settlements, and the global stability meets the IDOT-required FOS.

5.1.1 Settlement

We understand the profile grade along Western Avenue behind the south abutment will not be significantly changed; therefore we do not anticipate excessive settlements or settlement-related problems. A raise in grade of 1 or 2 feet would result in negligible settlement and provide suitable performance of the approach pavement. The initial 10 to 15 feet of the embankment fill encountered behind the abutment is well compacted and we do not anticipate any issues with the subgrade. If a change in the profile grade of greater than 5 feet is proposed, these evaluations should be revisited.

5.1.2 Global Stability

The sides of the existing approach embankment behind the south abutment are supported by two concrete retaining walls. The exact geometry of these walls is not currently known, but they will remain in place after the reconstruction of the approach structure and maintain support of the embankment. The geometry of the end slope will remain the same. Considering the current condition, we estimate the end slope, as well as the retaining walls, meet the IDOT-required FOS.

5.2 Structure Foundations

Wang recommends supporting the abutments and piers on steel H-piles or drilled shafts socketed into bedrock. The soil conditions along the structure show organic, unsuitable soils overlying dense and very dense granular soils; we do not recommend establishing the pile tip above the organic soils or attempting to drive metal shell piles into the dense gravelly soils beneath. We estimate driven piles will extend to the top of bedrock and can be designed for the maximum nominal bearing of each individual pile. The factored and unfactored loads provided by BLA are summarized in Table 2.

Table 2: Summary of Foundation Loads

Substructure ID	Reference Boring	Factored Load (kips)	Unfactored Load (kips)
South Abutment	BSB-01	2660	1825
Pier 1	BSB-02	4415	3125
Pier 2	BSB-03	4075	2880
Pier 3	BSB-04	4215	2980
Pier 4	BSB-05	3915	2850

5.2.1 Driven Piles

IDOT specifies the maximum nominal required bearing (R_{NMAX}) for each pile and states the factored resistance available (R_F) for steel H-piles should be based on a geotechnical resistance factor (Φ_G) of 0.55 (IDOT, 2012a). Nominal tip and side resistance were estimated using the methods and empirical equations presented in *AGMU Memorandum 10.2 – Geotechnical Pile Design* (IDOT, 2011). Based on the loading provided in Table 2 and the length of the foundations, we estimate the load per pile at the abutment will be about 60 to 160 kips for a 3- and 8-foot pile spacing. The loading at the piers will range from about 100 to 300 kips per pile for the same spacing. We understand the existing piles are likely to remain in place and the pile spacing for the proposed foundations will be set primarily by the locations of these existing piles. The factored bearing resistance of the piles immediately above the bedrock elevation is about 80 to 125 kips; we estimate significantly higher bearing resistances and greater flexibility in pile spacing can be achieved by driving to the top of bedrock elevations and designing for the maximum nominal required bearing (MNB) of the pile. In addition, due to the concern for vibration damage (see discussion below) to adjacent building during pile driving the piles certain piles should be installed through precored holes and filled with concrete and controlled low-strength material. The R_F, R_N, estimated pile tip elevations, and pile lengths for HP12x53, HP12x74, and HP14x73 steel H-piles designed for their MNB are summarized in Table 3. The lengths shown in the tables assume a 1-foot pile embedment into the abutment and pier caps, as per the GPE.

The R_F estimates are governed by the relationship $R_F = \phi_G R_N - \phi_G(DD_R+S_C+L_{iq})I_G - (\gamma_p)(\lambda_{IS})DD_L$ (IDOT, 2012a). There is no significant increase proposed for the profile grade along Western Avenue and we do not anticipate downdrag allowances will be required for the abutment piles. Scour and liquefaction reductions will also not be required.

Table 3: Estimated Pile Lengths and Tip Elevations for Steel H-Piles

Structure Unit	Pile Cap Base Elevations (feet)	Required Nominal Bearing, R_N (kips)	Factored Geotechnical Loss (kips)	Factored Geotechnical Load Loss (kips)	Factored Resistance Available, R_A (kips)	Total Estimated Pile Length (feet)	Estimated Pile Tip Elevation (feet)
South Abutment (BSB-01)	605.6	418 HP12x53	0.0	0.0	230	42	564
		589 HP12x74	0.0	0.0	324	42	564
		578 HP14x73	0.0	0.0	318	42	564
Pier 1 (BSB-02)	591.5	418 HP12x53	0.0	0.0	230	25	567
		589 HP12x74	0.0	0.0	324	25	567
		578 HP14x73	0.0	0.0	318	25	567
Pier 2 (BSB-03)	591.5	418 HP12x53	0.0	0.0	230	24	568
		589 HP12x74	0.0	0.0	324	24	568
		578 HP14x73	0.0	0.0	318	24	568

Pier 3 (BSB-04)	590.1	418 HP12x53	0.0	0.0	230	26	564
		589 HP12x74	0.0	0.0	324	26	564
		578 HP14x73	0.0	0.0	318	26	564
Pier 4 (BSB-05)	588.4	418 HP12x53	0.0	0.0	230	22	566
		589 HP12x74	0.0	0.0	324	22	566
		578 HP14x73	0.0	0.0	318	22	566

Driven piles will create vibrations during installation that may be felt by adjacent residents and commercial properties. At the piers the piles might be installed as close as 10 feet from the adjacent wood structures. We performed an analysis of the pile driving to estimate the level of vibration that could occur at the structures. For above-ground structures exhibiting free response to ground motion, such as the structures in question, the accepted peak particle velocity (PPV) is generally a function of the dominant wave frequency, with the PPV threshold for structural damage typically ranging from 0.5 to 2.0 in/sec assuming the vibration frequency from the pile driving is between 12 and 30 Hz (Dowding 2000). Practice has shown that for most cases structural damage to adjacent structures from vibrations produced by pile driving is not expected at distances greater than one pile length. The level of piling-induced vibrations depends on the both the type of pile and hammer, the hammer transferred energy, the type of soil through which the waves propagate, and the distance from the source.

A wave equation analysis was performed with GRL-WEAP Software on an HP14x73 pile driven to the top of bedrock with a 40 kip-foot hammer. This power rating equates to an energy output of about 10 kip-feet and an estimated maximum PPV of about 0.9 in/sec (Appendix C). This PPV is above the conservative threshold for damage and will certainly be felt by adjacent property owners. To achieve the conservative threshold for damage at a PPV of 0.5 in/sec, we estimate the piles will need to be a minimum of 20 feet from the adjacent structures. However, the residents and property owners will still feel these vibrations and may be concerned about the effect on their structures, regardless of the actual structural response.

We recommend precoring any pile location within 25 feet of the two-story wood frame structure adjacent to Pier 3. The piles should be installed within 24-inch diameter boreholes to an elevation of 566 feet, or a depth of one to two feet into the sound bedrock which begins at an elevation of 568 to 567 feet. The sockets in which these piles are set can then be designed for a nominal unit end bearing resistance of 250 ksf, a factored unit end bearing resistance of 125 ksf, and a maximum factored resistance of 393 kips. The rock socket should be backfilled with concrete and the annular space around the piles above the socket should then be filled with either concrete or IDOT controlled low-strength material (CLSM). Alternatively, the piers could be supported on drilled shafts (Section 5.2.2), which will not cause excessive vibration levels. The abutment piles are far enough away that they will not cause significant vibration levels at the adjacent residences and they can be driven as planned.

In addition to the required precoring within 25 feet of the adjacent wood structure, we recommend providing vibration monitoring during driving of the remaining piles at Piers 2, 3, and 4. The monitoring should be set to trigger at 0.5 in/sec; any pile exceeding the trigger threshold should be removed and the precore procedure, plus the bedrock socket described above, should be performed.

5.2.2 Drilled Shafts

The foundations for the abutment and piers could be supported on drilled shafts socketed into the underlying bedrock. The rock sockets could be designed for either a side friction resistance factor (ϕ_{stat}) of 0.55 or an end bearing ϕ_{stat} of 0.50 (AASHTO 2012). For the drilled shafts advanced through silt and sand to the bedrock, groundwater conditions will require drilling fluid and permanent casing during installation. These construction techniques will reduce the side friction and render it highly variable and unpredictable; therefore, side friction with the soil should not be considered in the design (AASHTO, 2012). In addition, we recommend designing the sockets based on side friction *or* end bearing, but not a combination of both. Generally, smaller diameter sockets (3.0 feet) should be designed for side friction, whereas larger diameter (4-feet or larger) sockets should be designed for end bearing.

The R_F, R_N, and estimated rock socket thickness required for drilled shafts are summarized in Tables 4 and 5. Table 4 summarizes the capacity of 4, 5, and 6-foot diameter shafts designed in end bearing only; for end bearing shafts we recommend establishing the base of the sockets a minimum of 3 feet into the bedrock and designing for a FAIR to GOOD quality nominal unit end resistance of 250 ksf (AASHTO 2012, 10.4.6.4/10.8.3.5.4c and Brown 2008). Table 5 provides the capacity for 3-foot diameter shafts designed based on variable FAIR to GOOD quality nominal unit side friction

(AASHTO 2012, 10.8.3.5.4b). The diameter of the shaft above the bedrock should be drilled 6 inches larger than the socket. The shafts will require casing to protect against groundwater infiltration (Section 6.6).

Table 4: Estimated Rock Socket Thicknesses and Tip Elevations for End Bearing Rock Socket Shafts

Structure Unit	Top of Rock Elevation (feet)	Nominal Unit End Resistance (ksf)	Nominal Shaft Resistance, R_N (kips)	Factored Resistance Available, R_F (kips)	Estimated Number of Shafts	Total Socket Thickness (feet)	Estimated Shaft Tip Elevation (feet)	Shaft Diameter (feet)
South Abutment (BSB-01)	567	250	3141	1571	2	3.0	564	4
			4909	2455	2	3.0	564	5
			7069	3535	2	3.0	564	6
Pier 1 (BSB-02)	568	250	3141	1571	3	3.0	565	4
			4909	2455	2	3.0	565	5
			7069	3535	2	3.0	565	6
Pier 2 (BSB-03)	568	250	3141	1571	3	3.0	565	4
			4909	2455	2	3.0	565	5
			7069	3535	2	3.0	565	6
Pier 3 (BSB-04)	567	250	3141	1571	3	3.0	564	4
			4909	2455	2	3.0	564	5
			7069	3535	2	3.0	564	6
Pier 4 (BSB-05)	568	250	3141	1571	3	3.0	565	4
			4909	2455	2	3.0	565	5
			7069	3535	2	3.0	565	6

Table 5: Estimated Rock Socket Thicknesses and Tip Elevations for 3-foot Diameter Shafts

Structure Unit	Top of Rock Elevation (feet)	Nominal Unit Shaft Resistance (ksf)	Nominal Shaft Resistance, R_N (kips)	Factored Resistance Available, R_F (kips)	Estimated Number of Shafts	Total Socket Thickness (feet)	Estimated Shaft Tip Elevation (feet)
South Abutment (BSB-01)	567	31.7	1455	800	4	4.9	562
			1818	1000	3	6.1	561
			2182	1200	2	7.3	560
Pier 1 (BSB-02)	568	24.7	1455	800	6	6.2	562
			1818	1000	5	7.8	560
			2182	1200	4	9.4	559
Pier 2 (BSB-03)	568	24.7	1455	800	6	6.2	562
			1818	1000	5	7.8	560
			2182	1200	4	9.4	559
Pier 3 (BSB-04)	567	32.4	1455	800	6	4.8	562
			1818	1000	5	6.0	561
			2182	1200	4	7.1	560
Pier 4 (BSB-05)	568	30.7	1455	800	6	5.0	563
			1818	1000	5	6.3	562
			2182	1200	4	7.5	561

5.2.3 Lateral Loading

Lateral loads on piles and shafts should be analyzed for maximum moments and lateral deflections. Recommended lateral soil modulus and strain parameters required for analysis via the p-y curve method are included in Tables 6 (soil parameters) and 7 (bedrock parameters).

Table 6: Recommended Soil Parameters for Lateral Load Analysis

Soil Type (Layer)	Unit Weight, γ (pcf)	Undrained Shear Strength, c_u (psf)	Estimated Friction Angle, Φ (°)	Estimated Lateral Soil Modulus Parameter, k (pci)	Estimated Soil Strain Parameter, ε_{50} (%)
Loose to M Dense SILTY LOAM FILL (1)	120	0	32	60	--
Loose SILTY and SANDY LOAM (1)	115	0	30	40	--
Stiff to V Stiff SILTY CLAY (1)	125	1500	0	1000	0.7
V Loose and Loose ORGANIC SILT (2)	80	0	18	10	--
V Dense and Hard SILTY LOAM and SAND (3)	125	0	36	110	--

Table 7: Recommended Bedrock Parameters for Lateral Load Analysis

Rock Type (Layer)	Total Unit Weight, γ (pcf)	Young's Modulus (ksi)	Uniaxial Comp. Strength (ksi)	RQD (%)	Lateral Rock Modulus Parameter
V Good to Fair Quality Dolostone (4)	135	1,500	12.5	75	0.0005

5.3 Stage Construction Design Recommendations

The GPE shows the approach structure constructed in two stages. The existing foundations will require partial removal excavations and an excavation into the existing end slope will be required to construct the new abutment. We do not anticipate the need for temporary shoring to cut off the pier foundations below the ground surface. These excavations should be sloped at 1:2 (V:H) for a maximum of 5 feet. The excavation to construct the south abutment may require temporary shoring, depending on the geometry of the cut. We estimate temporary support with steel sheet piling is a feasible and effect method along the abutment and the sheeting should be designed based on the charts included in IDOT *Design Guide 3.13.1* (2012a). If the tight right-of-way on either side of the existing piers affect the feasibility of sloped excavations the soils should be supported by a *Temporary Soil Retention System* (IDOT 2012b) due to the organic and soft soils below 8 to 10 feet.

Driving sheet piling will also produce ground vibrations (see Section 5.2.1). At the abutments we do not anticipate concerns with adjacent buildings; however, if steel sheeting is required to support excavations at the piers, an analysis should be performed to evaluate the vibration levels and propose methods for mitigating them adjacent to the buildings.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Site Preparation

Vegetation, topsoil, existing pavement, and debris should be cleared and stripped where foundations and structural fills will be placed. The exposed subgrade should be proofrolled. To aid in locating unstable and unsuitable materials, the proofrolling should be observed by a qualified engineer. Any unstable or unsuitable materials should be removed and replaced with compacted structural fill as described in Section 6.3.

6.2 Excavation and Dewatering

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.

The shallow excavations required to remove the piers will not encounter groundwater. Any precipitation allowed to enter excavations should be immediately removed via sump pump. Any soil allowed to soften under standing water should be removed and replaced with structural fill as described below in Section 6.3.

6.3 Filling and Backfilling

Fill material required to attain the final design elevations should be structural fill material and should be pre-approved prior to placement. Compacted cohesive or granular soil conforming to IDOT Section 204 would be acceptable as structural fill (2012b). The fill material should be free of organic matter and debris. Structural fill should be placed in lifts and compacted according to IDOT Section 205, *Embankment* (2012b). The onsite fill materials (**Layer 1**) could be considered as new fill material assuming they have an organic content less than 10%; materials within 24-inches of the peat and organic soil (**Layer 2**) should not be reused.

Backfill materials must be pre-approved by the Resident Engineer. To backfill the abutment and piers we recommend the porous granular material conforming to the requirements specified in the IDOT Special Provision, *Granular Backfill for Structures* (2013). Backfill material should be placed and compacted in accordance with the Special Provision. Estimated design parameters for granular structural backfill materials are presented in Table 7.

Table 7: Estimated Granular Backfill Parameters

Soil Description	Porous Granular Material Backfill
Unit Weight	125 lbs/ft^3
Angle of Effective Internal Friction	32 degrees
Active Earth Pressure Coefficient	0.31
Passive Earth Pressure Coefficient	3.26
At-Rest Earth Pressure Coefficient	0.5

6.4 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 to not 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.

6.5 Pile Installation

The driven piles shall be furnished and installed according to the requirements of IDOT Section 512, *Piling* (IDOT 2010). The piles are proposed to be driven to the maximum nominal bearing at the top of bedrock, which was encountered at a relatively uniform elevation; therefore, Wang recommends that two test piles, one at the abutment and a second from below at one of the pier locations, be performed prior to ordering production piles. The test piles shall be driven to 110 percent of the nominal required bearing indicated in Section 5.2.1, Table 3. Since the piles are proposed to be driven to termination depths at the top of bedrock, the piles should be installed with metal shoes. The steel H-piles shall be according to AASHTO M270M, Grade 50.

6.6 Drilled Shafts

The installation of drilled shafts will present groundwater challenges within the gravelly silty loam and sand above the bedrock elevation. The shaft excavations will encounter some groundwater infiltration, and the Contractor must be prepared to drill with fluid and install temporary or permanent casings at each shaft location in order to facilitate the rock coring. Failure to anticipate the challenges posed by the groundwater at this location will result in caving or heaving sand and significant weakening of the foundation soils. The mud cake along the side of the shaft, in addition to the casing, will make evaluation of the mobilized skin friction difficult; therefore, the shafts should be designed for side friction in the bedrock only. Prior to coring the bedrock the casing should be firmly seated into the top of the rock and the drilling fluid removed to prevent caking of the fluid on the sides of the bedrock sockets. The shafts should be designed 6 inches larger in diameter than the proposed sockets. The shafts should be constructed in accordance with FHWA Publication NHI-10-016, *Drilled Shafts: Construction Procedures and LRFD Design Methods* (FHWA, 2010).

7.0 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 logs and in Exhibit 3. 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 can be adjusted accordingly.

It has been a pleasure to assist Bollinger, Lach & Associates and the Illinois Department of Transportation on this project. Please call if there are any questions, or if we can be of further service.

Respectfully Submitted,

WANG ENGINEERING, INC.

Mickey L. Snider, P.E.
Senior Geotechnical Engineer

Jerry W.H. Wang, Ph.D., P.E.
QA/QC Reviewer

MICKEY L. SNIDER
STATE OF ILLINOIS
062-058045
PROFESSIONAL ENGINEER

EXPIRES 11/30/13

REFERENCES

AASHTO (2012) *LRFD Bridge Design Specifications*. American Association of State Highway and Transportation Officials, Washington, D.C.

BROWN, DAN (2008) *Load Testing of Drilled Shaft Foundations in Limestone, Nashville Tennessee.*

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IDOT (2011) *All Geotechnical Manual Users Memorandum 10.2 - Static Method of Estimating Pile Length*

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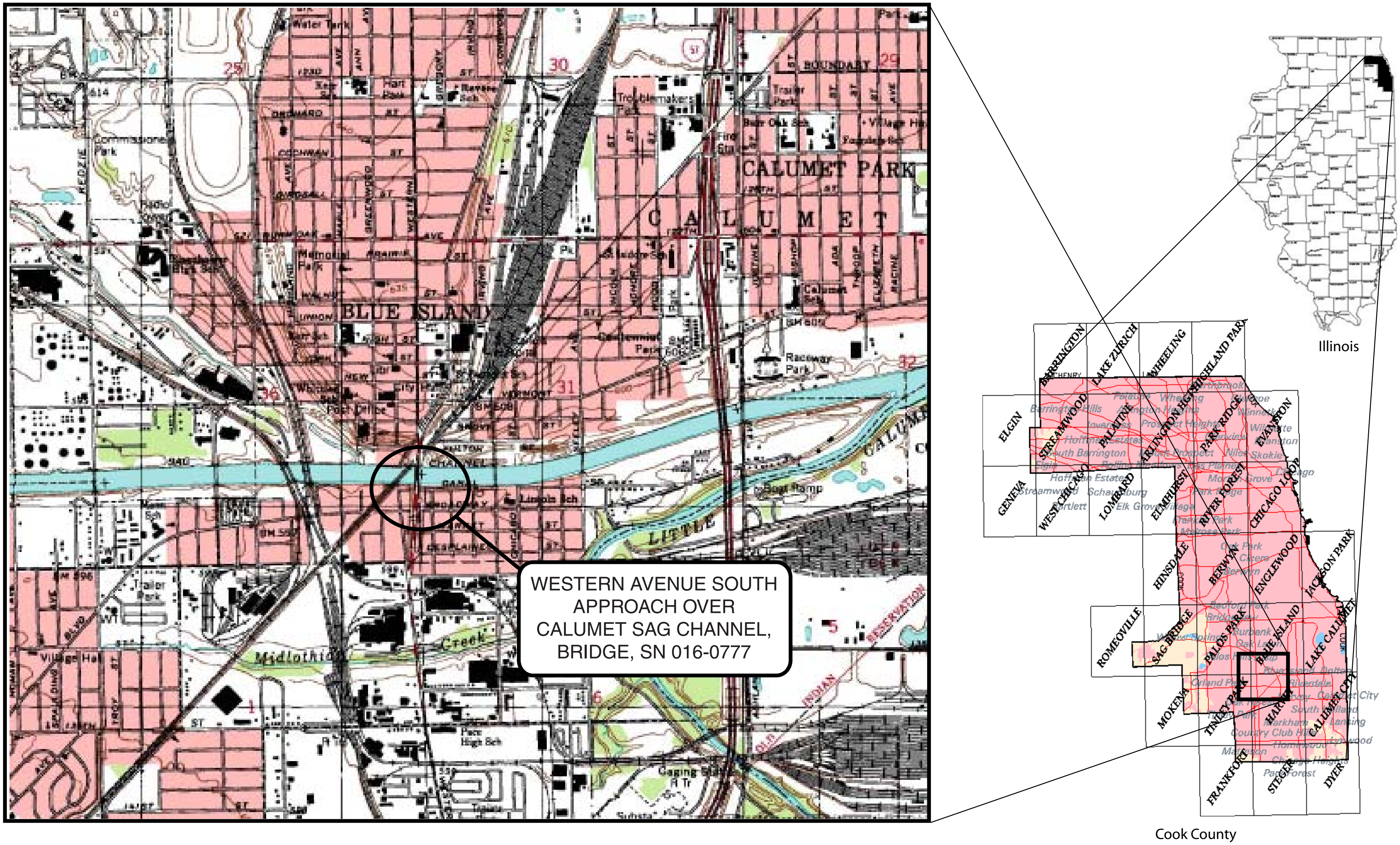
WILLMAN, H.B. (1971) *Summary of the Geology of the Chicago Area*. ISGS Circular C460. Illinois State Geological Survey, Urbana, 77 pp.

WILLMAN, H.B., FRYE, J.C., SIMON, J.A., CLEGG, K.E., SWANN, D.H., ATHERTON, E., COLLINSON, C., LINEBACK, J.A. AND BUSCHBACH, T.C. (1975) *Handbook of Illinois Stratigraphy*. Illinois State Geological Survey, Bulletin 95, Urbana, IL.

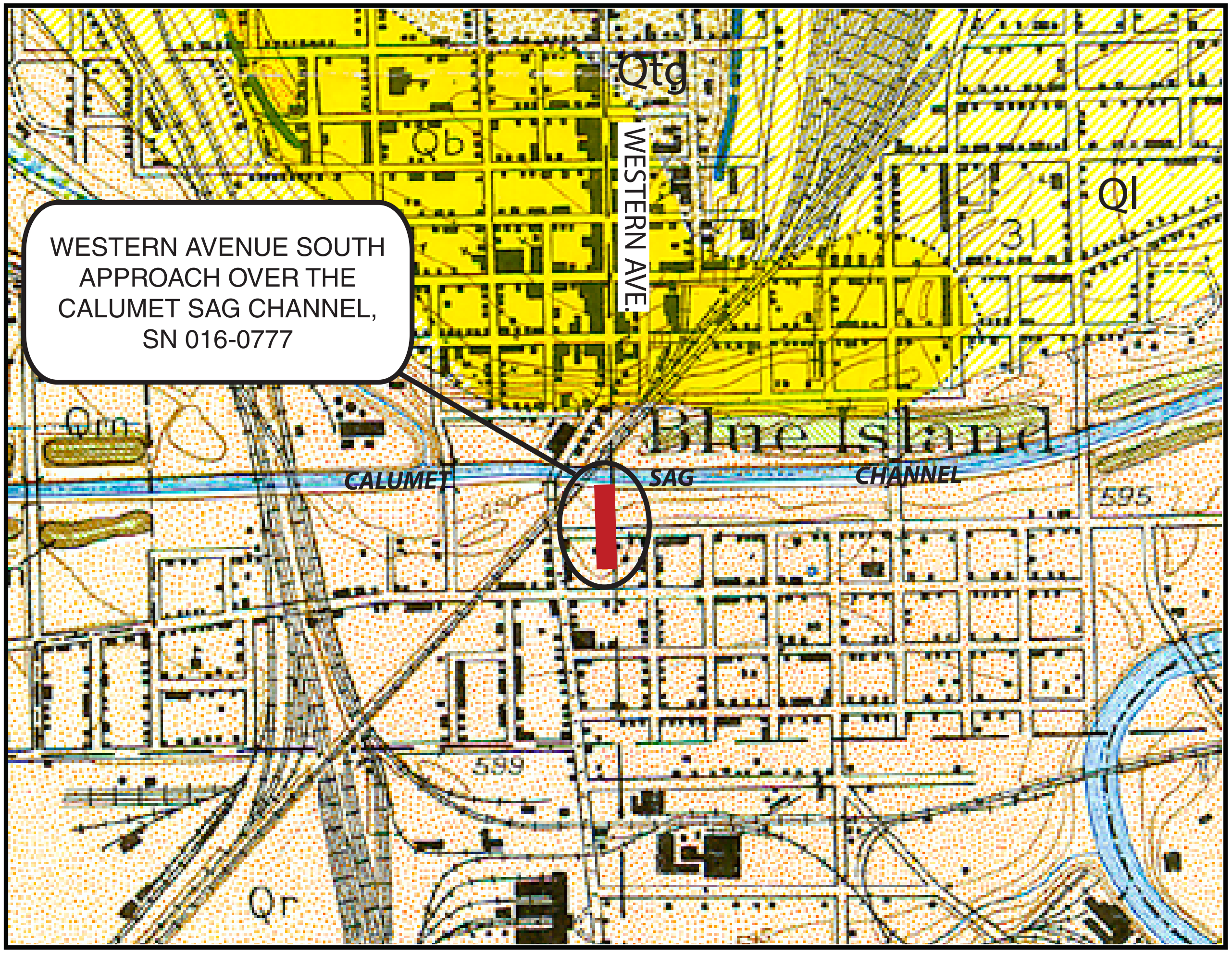
EXHIBITS



0 0.50 1.00 1.50 2.00 2.50 Miles

N

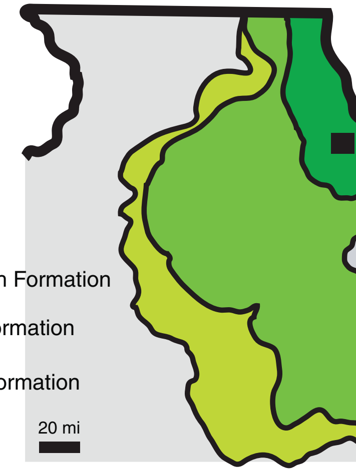
SITE LOCATION MAP: WESTERN AVENUE SOUTH APPROACH TO THE CAL-SAG CHANNEL BRIDGE, SN 016-0777, COOK COUNTY		
SCALE: GRAPHICAL	EXHIBIT 1	DRAWN BY: A. Kurnia CHECKED BY: M. Snider
Wang Engineering		1145 N. Main Street Lombard, IL 60148 www.wangeng.com
FOR BOLLINGER, LACH & ASSOCIATES		165-03-01



0 0.25 0.5 Miles

Modified after Bretz (1926)

REGIONAL GEOLOGY



Wedron Group

- Wadsworth Formation
- Lemont Formation
- Tiskilwa Formation

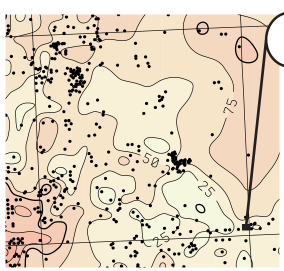
Modified after Hansel and Johnson (1996)

Legend

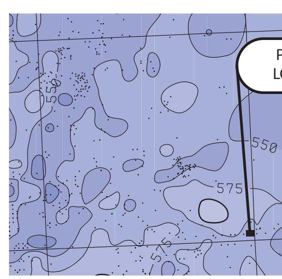
Qm	Made land (Artificial fills, spoil banks, dumps etc.)
Qp	Peat and muck deposits
Qb	Beach ridges and related deposits of sand and gravel (Made during and since various stages of Lake Chicago)
Ql	Glacial lake bottom (Commonly without lacustrine deposits)
Qr	Glacial river bottom (Chiefly erosional, some residual gravel)
Qtg	Tinley Ground Moraine

Modified after Bretz (1926)

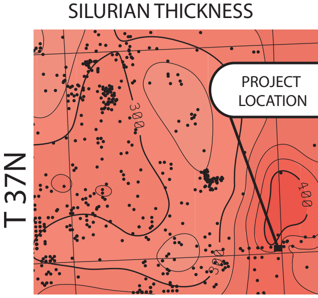
DRIFT THICKNESS



BEDROCK TOPOGRAPHY



SILURIAN THICKNESS



Modified after Leetaru et al. (2004)

0 2 4 8 miles

SITE AND REGIONAL GEOLOGY: WESTERN AVENUE SOUTH APPROACH TO THE CAL-SAG CHANNEL, SN 016-0777, COOK COUNTY		
SCALE: GRAPHICAL	EXHIBIT 2	DRAWN BY: C. Marin CHECKED BY : M. Snider

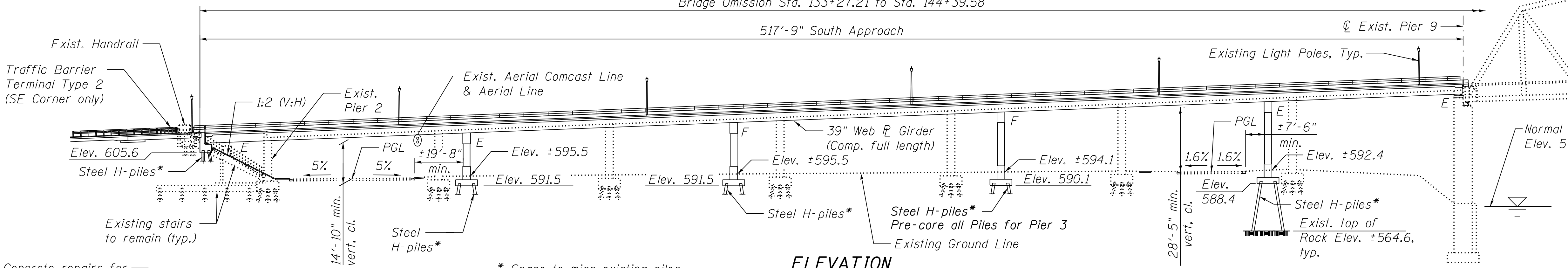
Wang Engineering
1145 N. Main Street
Lombard, IL 60148
www.wangeng.com

FOR BOLLINGER, LACH & ASSOCIATES | 165-03-01

Benchmark: Cut Cross on NW flange bolt of fire hydrant @ NW corner of Canal Street & Redspoon Street. Elev. 596.18.

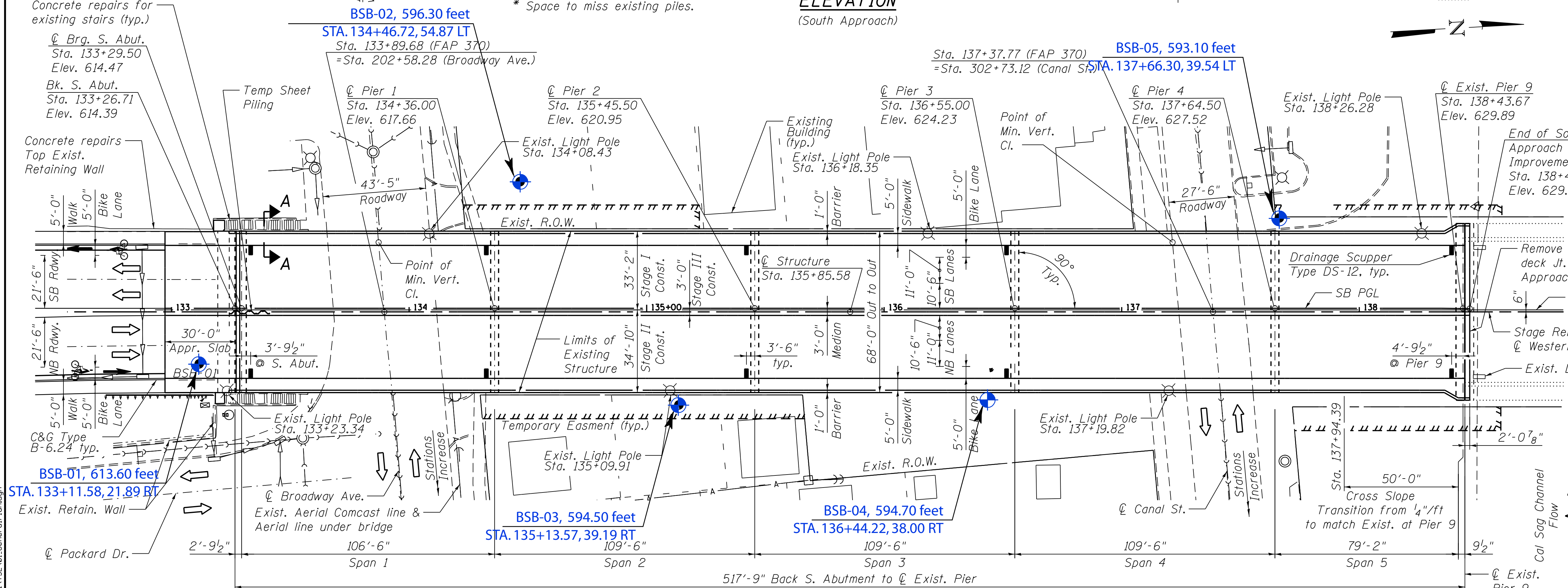
Existing Structure: Structure No. 016-0777 was originally built in 1966 as CH-55A, Section 055A-0103-MFT and repaired in 1986, 1990 and 1998. In 1999 a concrete overlay was installed. The back to back existing abutment length is 1123'-1¼" and the out to out width is 68'-0". The south approach spans (Spans 1 through 8) consist of simple span PPC Box Beams. The substructure consists of a spill through abutment on steel piles and rectangular concrete column bents on steel piles. The existing south approach structure will be removed and replaced. Existing Pier 9 bridge seat for span 5 will be partially removed and rebuilt. One lane of traffic shall be maintained in each direction, and one sidewalk, using staged construction.

Salvage: Existing light poles, aluminum tube railing and stair railings are to be removed and reinstalled.

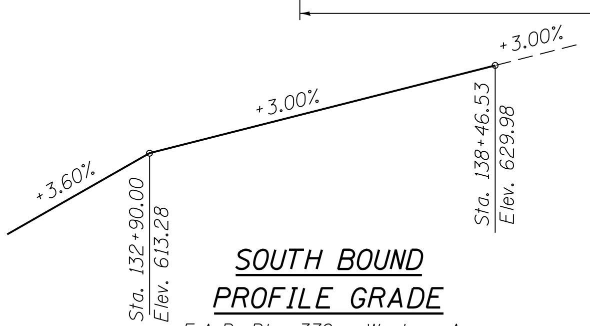


ELEVATION

(South Approach)

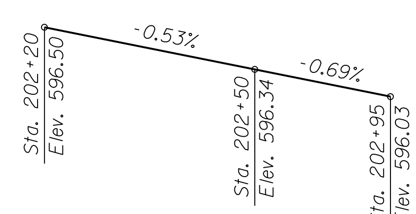


PLAN



SOUTH BOUND PROFILE GRADE

F.A.P. Rte. 370 - Western Ave.



PROFILE GRADE

F.A.U. Rte. 1596 - Broadway Ave.
(Exist. P.G.)

SEISMIC DATA

(New Construction)
Seismic Performance Zone (SPZ) = 1
Design Spectral Acceleration at 1.0 sec. (SD1) = 0.090g
Design Spectral Acceleration at 0.2 sec. (SDS) = 0.153g
Soil Site Class = D

(Existing Pier 9)
Seismic Performance Zone (SPC) = A
Bedrock Acceleration Coefficient (A) = 0.037g
Site Coefficient (S) = 1.0

DESIGN SPECIFICATIONS

2012 AASHTO LRFD Bridge Design Specifications, 6th Edition, with 2012 Interims
2002 AASHTO Standard Specifications for Highway Bridges, 17th Edition (Existing Pier 9)

DESIGN STRESSES

FIELD UNITS (New Construction)
f'c = 3,500 psi
fy = 60,000 psi (Reinforcement)
fy = 50,000 psi (M270 Grade 50)

FIELD UNITS (Existing)
fc = 1,400 psi (Flexure, Except Stressed by Earth)
fs = 20,000 psi (Reinforcement)
fc = 800 psi (Flexure Stressed by Earth)
fc = 75 psi (Ftgs.)

HIGHWAY CLASSIFICATION

FAP Rte. 370 - Western Avenue
Functional Class: Other Principal Arterial (Urban)
ADT: 8200 (2010); 14,000 (2030)
ADTT: 574 (2010); 1064 (2030)
DHV: 820 (2010); 1400 (2030)
Design Speed: 35 m.p.h.
Posted Speed: 35 m.p.h.
Two-Way Traffic
Directional Distribution: 50/50

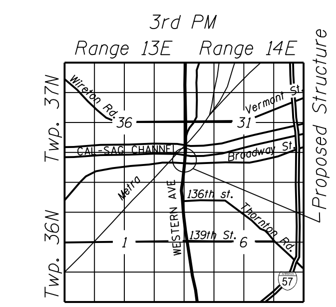
FAU Rte. 1596 - Broadway Avenue
Functional Class: Minor Arterial
ADT: 4300 (2010); 4429 (2032)
ADTT: 86 (2010); 89 (2032)
DHV: 430 (2010); 442 (2032)
Design Speed: 30 m.p.h.
Posted Speed: 30 m.p.h.
Two-Way Traffic
Directional Distribution: 50/50

LOADING HL-93

Allow 50#/sq. ft. for future wearing surface.

LOADING HS20-44

(Exist. Pier 9)
Allow 15#/sq. ft. for future wearing surface.



LOCATION SKETCH

GENERAL PLAN
WESTERN AVE. SOUTH APPROACH
OVER CAL-SAG CHANNEL
F.A.P. RTE 370 - SECTION 0103BR-1
COOK COUNTY
STATION 135+85.58
STRUCTURE NO. 016-0777

BORING LOCATION PLAN: WESTERN AVENUE SOUTH APPROACH TO THE CAL-SAG CHANNEL BRIDGE, SN 016-0777, COOK COUNTY

SCALE: GRAPHICAL	EXHIBIT 3	DRAWN BY: A. Kurnia CHECKED BY: M. Snider

Wang Engineering
1145 N. Main Street
Lombard, IL 60148
www.wangeng.com

FOR BOLLINGER, LACH & ASSOCIATES — 165-03-01

Bollinger, Lach & Associates, Inc.
ITASCA, ILLINOIS

USER NAME =	DESIGNED - JJI	REVISED
	CHECKED - JMT	REVISED
PLOT SCALE =	DRAWN - GM	REVISED
PLOT DATE = 5/1/2013	CHECKED - JMT	REVISED

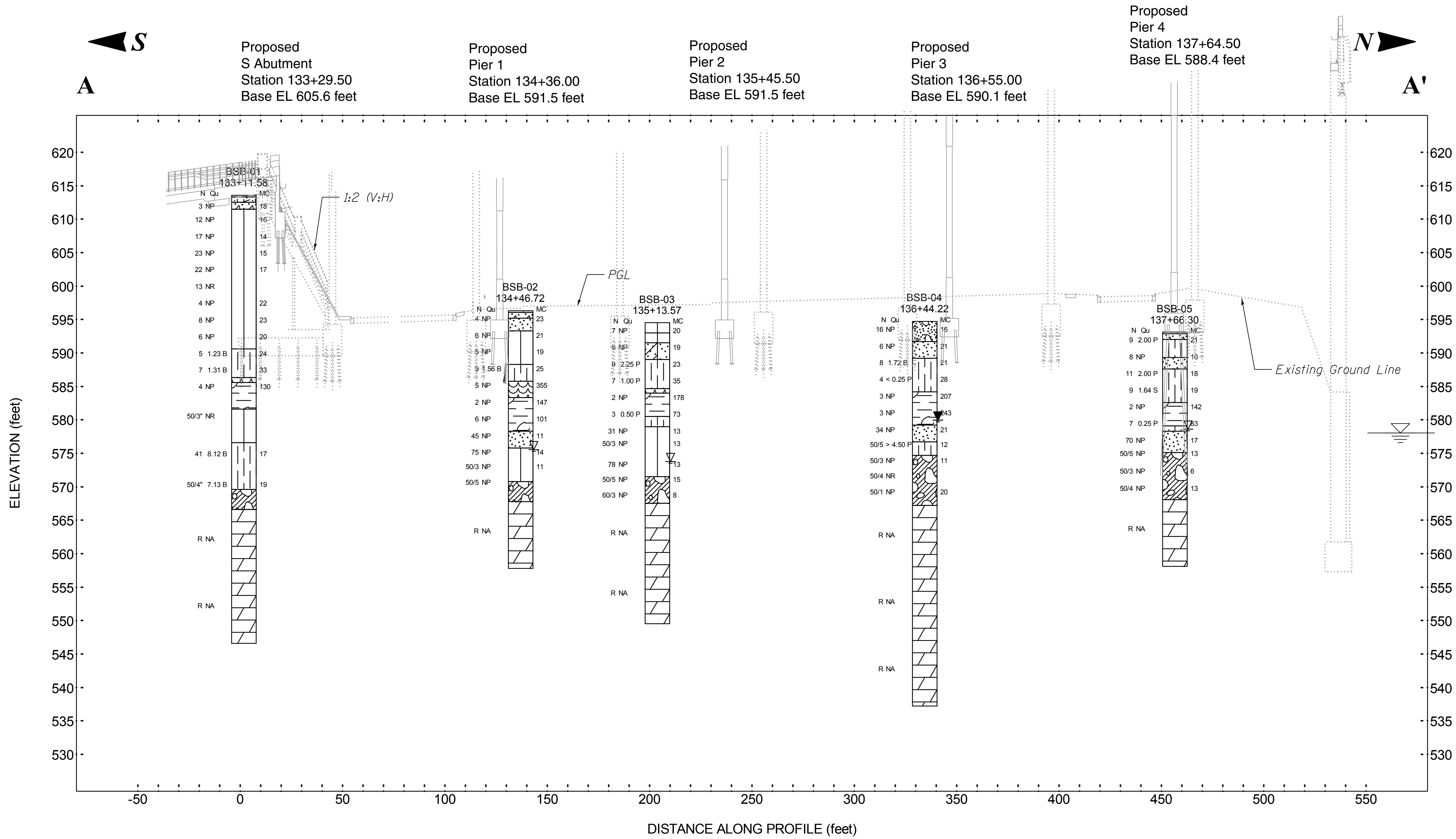
STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

SHEET NO. 1 OF 3 SHEETS

F.A.P. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
370	0103BR-1	Cook	3	1

CONTRACT NO. 60K72
ILLINOIS FED. AID PROJECT

FILE NAME = W:\191-132_IDOT_Western_Ave\CADD Sheets\structural\TSL\01_GeneralPlan.dgn



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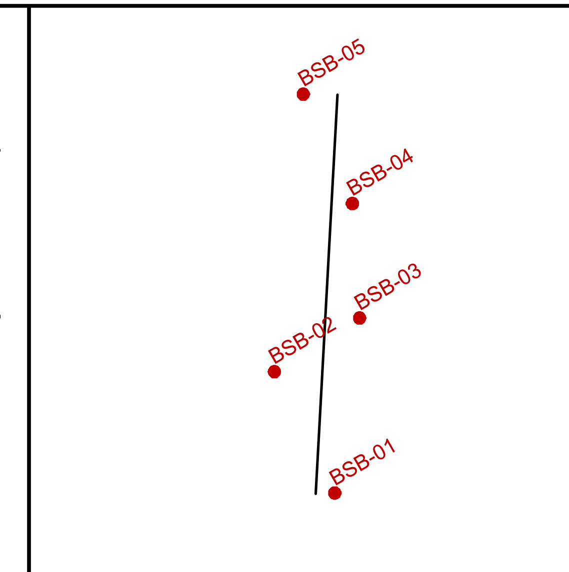
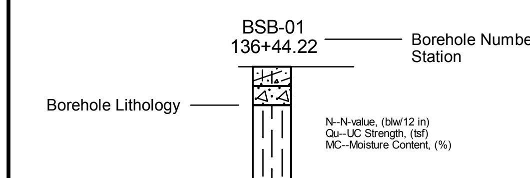


Site Map Scale 1 inch equals 220 feet

Explanation:



Water Level Reading at time of drilling.

Water Level Reading 24-hr after drilling or at end of drilling

Horizontal Scale (feet)

Vertical Exaggeration: 3.5x

Lithology Graphics

Pavement	Crushed stone	IDH Silt, Silty Loam	IDH Silty Clay, Silty Clay Loam
USCS Peat	USCS Low Plasticity Organic silt or clay with shells	Weathered bedrock	Dolomite or Dolomitic Limestone
IDH Sand, Sandy Loam	IDH Loam	Gravelly sand, sandy gravel	

Wang Engineering, Inc.
1145 N Main Street
Lombard, IL 60148

Soil Profile A-A'
SN 016-0777

WEI SINCE 1982

Western Avenue/Cal Sag South Approach
Blue Island, Illinois

JOB NUMBER	PLATE NUMBER
165-03-01	EXHIBIT 4

WEI 11X17 1650301.GPJ WANGENG.GDT 4/24/13

APPENDIX A

Wang Engineering
wangeng@wangeng.com
1145 N Main Street
Lombard, IL 60148
Telephone: 630 953-9928
Fax: 630 953-9938

BORING LOG BSB-01

WEI Job No.: 165-03-01

Client: **Bollinger, Lach, and Associates**
Project: **Western Avenue/Cal Sag South Approach**
Location: **Blue Island, Illinois**

Datum: NGVD
Elevation: 613.60 ft
North: 1815831.48 ft
East: 1162794.69 ft
Station: 133+11.58
Offset: 21.89 RT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample Type / recovery	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
613.4	3-inch thick ASPHALT --PAVEMENT--						
612.5	9-inch thick CONCRETE --PAVEMENT--						
611.5	12-inch thick CRUSHED STONE --AGGREGATE BASE--			1	3, 1, 2	NP	18
	Loose to medium dense, brown SILTY LOAM, little to some gravel and milled asphalt --FILL--	5		2	5, 6, 6	NP	16
				3	5, 7, 10	NP	14
		10		4	13, 8, 15	NP	15
				5	5, 7, 15	NP	17
		15		6	4, 8, 5	NR	
				7	2, 1, 3	NP	22
		20		8	2, 4, 4	NP	23

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample Type / recovery	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
				9	3, 3, 3	NP	20
590.6	Stiff, brown and gray SILTY CLAY --FILL--	25		10	2, 2, 3	1.23 B	24
				11	1, 3, 4	1.31 B	33
586.4	Black PEAT						
585.6	Loose, brown ORGANIC SILT, some shells	30		12	1, 2, 2	NP	130
581.6	Very dense, gray GRAVELLY SILTY LOAM	35		13	50/3"	NR	
576.6	Hard, gray SILTY CLAY	40		14	14, 18, 23	8.12 B	17

GENERAL NOTES

Begin Drilling **12-17-2012** Complete Drilling **12-17-2012**
Drilling Contractor **Wang Testing Service** Drill Rig **D-50 TMR**
Driller **R&N** Logger **B. Wilson** Checked by **M. Snider**
Drilling Method **2.25 ID SSA to 10', mud rotary 10' to rock; boring grouted upon completion**

WATER LEVEL DATA

While Drilling **DRY**
At Completion of Drilling **MUD**
Time After Drilling **NA**
Depth to Water **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

Wang Engineering

wangeng@wangeng.com
1145 N Main Street
Lombard, IL 60148
Telephone: 630 953-9928
Fax: 630 953-9938

BORING LOG BSB-01

WEI Job No.: 165-03-01

Client: **Bollinger, Lach, and Associates**
Project: **Western Avenue/Cal Sag South Approach**
Location: **Blue Island, Illinois**

Datum: NGVD
Elevation: 613.60 ft
North: 1815831.48 ft
East: 1162794.69 ft
Station: 133+11.58
Offset: 21.89 RT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
569.6	DOLOSTONE fragments --WEATHERED BEDROCK-- --HARD DRILLING--	45	15	19 50/4"	7.13 B	19
566.6	Strong, good quality, gray, fresh, horizontally bedded DOLOSTONE interbedded dark gray argillaceous dolomite --Run #1, 47.0' to 57.0' --RECOVERY = 96%-- --RQD = 85%-- --Qu = 13,410 psi--	50	16	CORE		
	--Run #2, 57.0' to 67.0' --RECOVERY = 100%-- --RQD = 88%-- --Qu = 13,150 psi--	55, 60, 65	17	CORE		
546.6	Boring terminated at 67.00 ft	70, 75, 80				

GENERAL NOTES

Begin Drilling: **12-17-2012** Complete Drilling: **12-17-2012**
Drilling Contractor: **Wang Testing Service** Drill Rig: **D-50 TMR**
Driller: **R&N** Logger: **B. Wilson** Checked by: **M. Snider**
Drilling Method: **2.25 ID SSA to 10', mud rotary 10' to rock; boring grouted upon completion**

WATER LEVEL DATA

While Drilling: **DRY**
At Completion of Drilling: **MUD**
Time After Drilling: **NA**
Depth to Water: **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

Wang Engineering

wangeng@wangeng.com
1145 N Main Street
Lombard, IL 60148
Telephone: 630 953-9928
Fax: 630 953-9938

BORING LOG BSB-02

WEI Job No.: 165-03-01

Client **Bollinger, Lach, and Associates**
Project **Western Avenue/Cal Sag South Approach**
Location **Blue Island, Illinois**

Datum: NGVD
Elevation: 596.30 ft
North: 1815970.63 ft
East: 1162725.56 ft
Station: 134+46.72
Offset: 54.87 LT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
596.1	3-inch thick ASPHALT --PAVEMENT--					
595.2	8-inch thick CRUSHED STONE --AGGREGATE BASE--					
	Loose, brown SANDY LOAM --FILL--		1	4 2 2	NP	23
593.3	Loose, brown and gray SILT --FILL--		2	2 2 4	NP	21
		5				
	--Moist--		3	2 2 3	NP	19
588.3	Stiff, brown and gray SILTY CLAY --FILL--		4	3 4 5	1.56 B	25
		10				
585.8	Loose, black and brown PEAT		5	1 2 3	NP	355
583.3	Very loose to loose, gray ORGANIC SILT, some shells		6	0 1 1	NP	147
		15				
			7	1 1 5	NP	101
578.3	Dense, gray SANDY LOAM, trace gravel		8	16 20 25	NP	11
		20				
575.8	Very dense, gray SILTY LOAM, little to some gravel		9	12 32 43	NP	14
			10	26 50/3	NP	11
		25				
570.8	Very dense, gray SILTY LOAM, some weathered dolostone		11	50/5	NP	
567.8	Strong, fair quality, gray, fresh, horizontally bedded DOLOSTONE interbedded dark gray argillaceous dolomite --Run #1, 28.5' to 38.5' --RECOVERY = 100%-- --RQD = 68%-- --Qu = 12,270 psi--	30	1	CORE		
		35				
557.8	Boring terminated at 38.50 ft					
		40				

GENERAL NOTES

Begin Drilling **12-13-2012** Complete Drilling **12-13-2012**
Drilling Contractor **Wang Testing Service** Drill Rig **D-50 TMR**
Driller **R&J** Logger **B. Wilson** Checked by **M. Snider**
Drilling Method **2.25 ID SSA to 10', mud rotary 10' to rock; boring grouted upon completion**

WATER LEVEL DATA

While Drilling **21.00 ft**
At Completion of Drilling **MUD**
Time After Drilling **NA**
Depth to Water **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

Wang Engineering

wangeng@wangeng.com
1145 N Main Street
Lombard, IL 60148
Telephone: 630 953-9928
Fax: 630 953-9938

BORING LOG BSB-03

WEI Job No.: 165-03-01

Client: **Bollinger, Lach, and Associates**
Project: **Western Avenue/Cal Sag South Approach**
Location: **Blue Island, Illinois**

Datum: NGVD
Elevation: 594.50 ft
North: 1816032.14 ft
East: 1162823.20 ft
Station: 135+13.57
Offset: 39.19 RT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
	Brown SILTY LOAM, trace wood chips --FILL--					
593.0	Loose, brown SILT --FILL--		1	3 3 4	NP	20
591.5	Loose, brown and gray LOAM --FILL--	5	2	3 3 3	NP	19
589.0	Stiff to very stiff, brown and gray SILTY CLAY --FILL--		3	3 4 5	2.25 P	23
			4	3 3 4	1.00 P	35
584.7	Dark brown PEAT	10				
584.0	Very loose, brown ORGANIC SILT, some shells		5	1 1 1	NP	178
580.5	Medium stiff, gray SILTY CLAY	15	6	0 1 2	0.50 P	73
579.0	Dense to very dense, gray SILTY LOAM, little gravel		7	5 13 18	NP	13
			8	50/3	NP	13
		20	9	16 34 44	NP	13
571.5	Very dense, gray SANDY LOAM, some weathered dolostone fragments --WEATHERED BEDROCK--	25	10	25 50/5	NP	15
			11	15 60/3	NP	8
567.5	Strong, good quality, gray, fresh, horizontally bedded DOLOSTONE interbedded dark gray argillaceous dolomite, trace shale partings --Run #1, 27.0' to 37.0' --RECOVERY = 78%-- --RQD = 63%-- --Qu = 12,140 psi--	30	1	CORE		
	--Run #2, 37.0' to 45.0' --RECOVERY = 94%-- --RQD = 88%--	35, 40		CORE		

GENERAL NOTES

Begin Drilling **12-12-2012** Complete Drilling **12-12-2012**
Drilling Contractor **Wang Testing Service** Drill Rig **D-50 TMR**
Driller **R&J** Logger **B. Wilson** Checked by **M. Snider**
Drilling Method **2.25 ID SSA to 10', mud rotary 10' to rock; boring grouted upon completion**

WATER LEVEL DATA

While Drilling: **21.00 ft**
At Completion of Drilling: **MUD**
Time After Drilling: **NA**
Depth to Water: **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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BORING LOG BSB-03

WEI Job No.: 165-03-01

Client: **Bollinger, Lach, and Associates**
Project: **Western Avenue/Cal Sag South Approach**
Location: **Blue Island, Illinois**

Datum: NGVD
Elevation: 594.50 ft
North: 1816032.14 ft
East: 1162823.20 ft
Station: 135+13.57
Offset: 39.19 RT

Profile	Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample Type *recovery*	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
					2			
	549.5		45					
		Boring terminated at 45.00 ft						
			50					
			55					
			60					

GENERAL NOTES

Begin Drilling **12-12-2012** Complete Drilling **12-12-2012**
Drilling Contractor **Wang Testing Service** Drill Rig **D-50 TMR**
Driller **R&J** Logger **B. Wilson** Checked by **M. Snider**
Drilling Method **2.25 ID SSA to 10', mud rotary 10' to rock; boring grouted upon completion**

WATER LEVEL DATA

While Drilling ▽ **21.00 ft**
At Completion of Drilling ▼ **MUD**
Time After Drilling **NA**
Depth to Water ▼ **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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wangeng@wangeng.com
1145 N Main Street
Lombard, IL 60148
Telephone: 630 953-9928
Fax: 630 953-9938

BORING LOG BSB-04

WEI Job No.: 165-03-01

Client: **Bollinger, Lach, and Associates**
Project: **Western Avenue/Cal Sag South Approach**
Location: **Blue Island, Illinois**

Datum: NGVD
Elevation: 594.70 ft
North: 1816163.45 ft
East: 1162828.93 ft
Station: 136+44.22
Offset: 38.00 RT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
	Medium dense, brown GRAVELLY to SANDY LOAM --FILL--		1	7, 8, 8	NP	16
591.7	Loose, brown SANDY LOAM --FILL--	5	2	2, 3, 3	NP	21
589.2	Very soft to stiff, brown SILTY CLAY, some gravel --FILL--		3	3, 3, 5	1.72 B	21
		10	4	2, 2, 2	< 0.25 P	28
584.2	Very loose, brown ORGANIC SILTY LOAM, some shells		5	1, 2, 1	NP	207
		15	6	1, 1, 2	NP	243
579.2	Dense, gray, fine SAND		7	10, 13, 21	NP	21
576.7	Hard, gray SILT CLAY LOAM, some gravel		8	14, 50/5	> 4.50 P	12
574.7		20				
	Very dense, gray SANDY GRAVEL, weathered dolostone fragments --WEATHERED BEDROCK--		9	39, 50/3	NP	11
			10	50/4	NR	
		25	11	50/1	NP	20
567.2	Strong, excellent quality, gray, fresh, horizontally bedded DOLOSTONE interbedded dark gray argillaceous dolomite --Run #1, 27.5' to 37.5' --RECOVERY = 100%-- --RQD = 100%-- --Qu = 12,380 psi--	30	12	CORE		
		35				
	--Run #2, 37.5' to 47.5' --RECOVERY = 100%-- --RQD = 92%--	40		CORE		

GENERAL NOTES

Begin Drilling	11-30-2012	Complete Drilling	11-30-2012
Drilling Contractor	Wang Testing Service	Drill Rig	D-50 TMR
Driller	R&N	Logger	F. Borzga
Checked by	M. Snider		
Drilling Method	2.25 ID SSA to 10', mud rotary 10' to rock; boring grouted upon completion		

WATER LEVEL DATA

While Drilling	15.00 ft
At Completion of Drilling	15.00 ft
Time After Drilling	NA
Depth to Water	NA

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

Wang Engineering
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1145 N Main Street
Lombard, IL 60148
Telephone: 630 953-9928
Fax: 630 953-9938

BORING LOG BSB-04

WEI Job No.: 165-03-01

Client: **Bollinger, Lach, and Associates**
Project: **Western Avenue/Cal Sag South Approach**
Location: **Blue Island, Illinois**

Datum: NGVD
Elevation: 594.70 ft
North: 1816163.45 ft
East: 1162828.93 ft
Station: 136+44.22
Offset: 38.00 RT

Profile	Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample Type *recovery*	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
					13			
			45					
		--Run #3, 47.5' to 57.5' --RECOVERY = 100%-- --RQD = 93%--				CORE		
			50					
					14			
			55					
	537.2	Boring terminated at 57.50 ft						
			60					

GENERAL NOTES

Begin Drilling **11-30-2012** Complete Drilling **11-30-2012**
Drilling Contractor **Wang Testing Service** Drill Rig **D-50 TMR**
Driller **R&N** Logger **F. Borzga** Checked by **M. Snider**
Drilling Method **2.25 ID SSA to 10', mud rotary 10' to rock; boring grouted upon completion**

WATER LEVEL DATA

While Drilling ▽ **15.00 ft**
At Completion of Drilling ▼ **15.00 ft**
Time After Drilling **NA**
Depth to Water ▼ **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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wangeng@wangeng.com
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Lombard, IL 60148
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BORING LOG BSB-05

WEI Job No.: 165-03-01

Client: **Bollinger, Lach, and Associates**
Project: **Western Avenue/Cal Sag South Approach**
Location: **Blue Island, Illinois**

Datum: NGVD
Elevation: 593.10 ft
North: 1816288.91 ft
East: 1162758.64 ft
Station: 137+66.30
Offset: 39.54 LT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
592.9	3-inch thick ASPHALT --PAVEMENT--					
592.0	8-inch thick CRUSHED STONE --AGGREGATE COURSE--					
	Very stiff, brown SILTY CLAY, trace gravel --FILL--		1	3, 4, 5	2.00 P	21
589.3	Loose, brown SANDY LOAM, trace gravel and brick fragments --FILL--	5	2	3, 4, 4	NP	10
587.6	Stiff to very stiff, brown and gray SILTY CLAY --FILL--		3	4, 5, 6	2.00 P	18
		10	4	6, 4, 5	1.64 S	19
582.6	Very loose, gray ORGANIC SILTY LOAM, trace shells		5	0, 1, 1	NP	142
579.1	Very soft, gray SILTY CLAY		6	0, 1, 6	0.25 P	83
578.3	Very dense, gray SAND	15	7	16, 28, 42	NP	17
575.1	Very dense, gray SILTY LOAM, some shale and weathered dolostone fragments		8	50/5	NP	13
		20	9	10, 50/3	NP	6
			10	22, 50/4	NP	13
568.1	Strong, good quality, gray, fresh, horizontally bedded DOLOSTONE interbedded dark gray argillaceous dolomite, trace shale partings --Run #1, 25.0' to 35.0' --RECOVERY = 97%-- --RQD = 75%-- --Qu = 13,680 psi--	25	1	CORE		
		30				
558.1	Boring terminated at 35.00 ft	35				
		40				

GENERAL NOTES

Begin Drilling **12-13-2012** Complete Drilling **12-13-2012**
Drilling Contractor **Wang Testing Service** Drill Rig **D-50 TMR**
Driller **R&J** Logger **B. Wilson** Checked by **M. Snider**
Drilling Method **2.25 ID SSA to 10', mud rotary 10' to rock; boring grouted upon completion**

WATER LEVEL DATA

While Drilling **14.75 ft**
At Completion of Drilling **MUD**
Time After Drilling **NA**
Depth to Water **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

APPENDIX B

Run #1

Boring BSB-01
Run #1, 47' to 57', RECOVERY=96%, RQD=85%

Run #2

Boring BSB-01
Run #2, 57' to 67', RECOVERY=100%, RQD=88%

BEDROCK CORE: WESTERN AVENUE SOUTH APPROACH TO THE CAL-SAG CHANNEL BRIDGE, SN 016-0777, COOK COUNTY, ILLINOIS		
SCALE : GRAPHIC	APPENDIX B-1	DRAWN BY: R. Gorlagunta CHECKED BY: C. Marin
Wang Engineering		1145 N. Main Street Lombard, IL 60148 www.wangeng.com
FOR BOLLINGER, LACH & ASSOCIATES, INC.		165-03-01

Run #1

Boring BSB-02
Run #1, 28.5' to 38.5', RECOVERY=100%, RQD=68%

BEDROCK CORE: WESTERN AVENUE SOUTH APPROACH TO THE CAL-SAG CHANNEL BRIDGE, SN 016-0777, COOK COUNTY, ILLINOIS		
SCALE : GRAPHIC	APPENDIX B-2	DRAWN BY: R. Gorlagunta CHECKED BY: C. Marin
Wang Engineering		1145 N. Main Street Lombard, IL 60148 www.wangeng.com
FOR BOLLINGER, LACH & ASSOCIATES, INC.		165-03-01

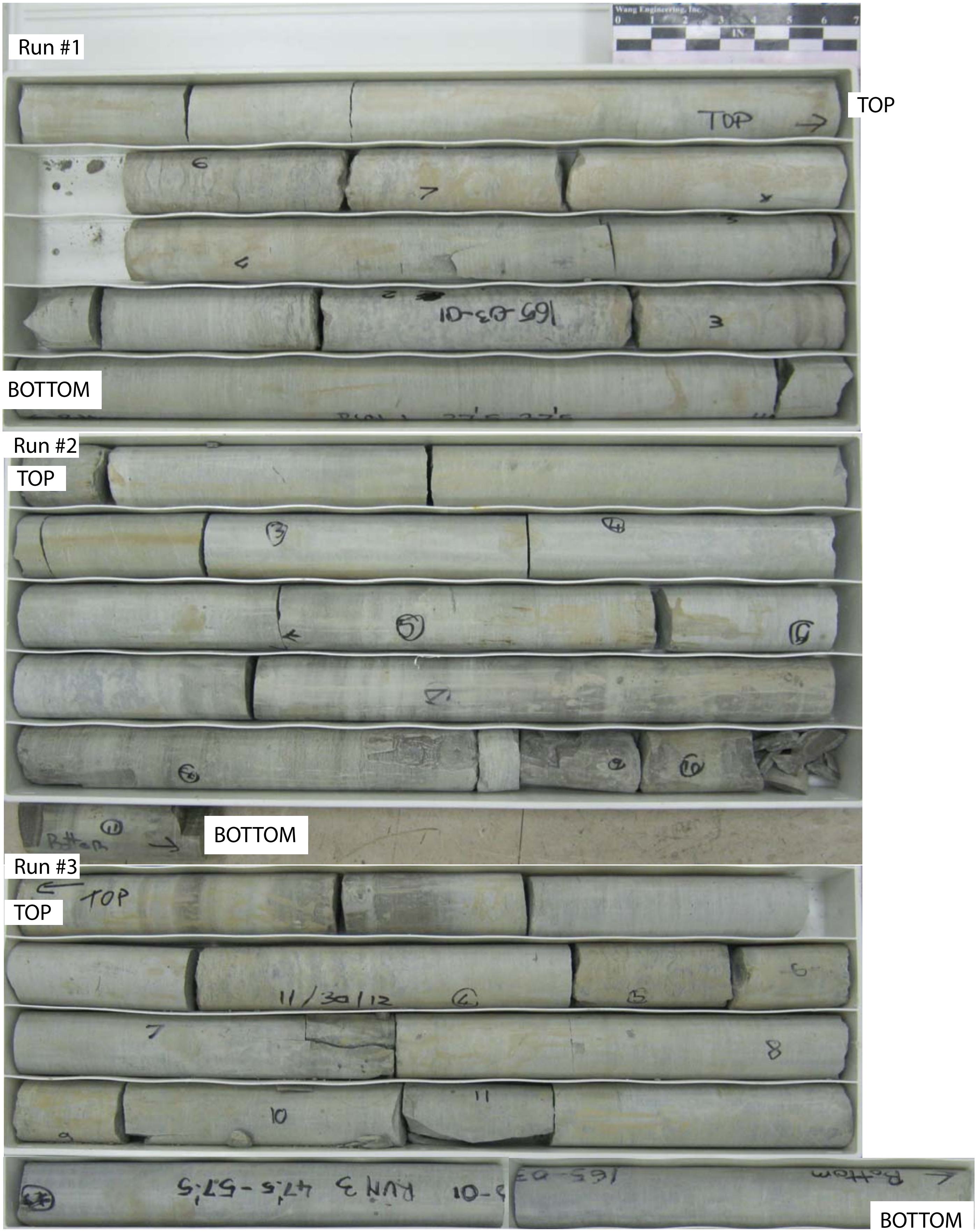
Run #1

Boring BSB-03
Run #1, 27' to 37', RECOVERY=78%, RQD=63%

Run #2

Boring BSB-03
Run #2, 37' to 45', RECOVERY=94%, RQD=88%

BEDROCK CORE: WESTERN AVENUE SOUTH APPROACH TO THE CAL-SAG CHANNEL BRIDGE, SN 016-0777, COOK COUNTY, ILLINOIS		
SCALE : GRAPHIC	APPENDIX B-3	DRAWN BY: R. Gorlagunta CHECKED BY: C. Marin
Wang Engineering		1145 N. Main Street Lombard, IL 60148 www.wangeng.com
FOR BOLLINGER, LACH & ASSOCIATES, INC.		165-03-01



Boring BSB-04

Run #1, 27.5′ to 37.5′, RECOVERY=100%, RQD=100%

Run #2, 37.5′ to 47.5′, RECOVERY=100%, RQD=92%

Run #3, 47.5′ to 57.5′, RECOVERY=100%, RQD=93%

BEDROCK CORE: WESTERN AVENUE SOUTH APPROACH TO THE CAL-SAG CHANNEL BRIDGE, SN 016-0777, COOK COUNTY, ILLINOIS		
SCALE : GRAPHIC	APPENDIX B-4	DRAWN BY: R. Gorlagunta CHECKED BY: C. Marin
Wang Engineering		1145 N. Main Street Lombard, IL 60148 www.wangeng.com
FOR BOLLINGER, LACH & ASSOCIATES, INC.		165-03-01

Run #1

Boring BSB-05
Run #1, 25' to 35', RECOVERY=97%, RQD=75%

BEDROCK CORE: WESTERN AVENUE SOUTH APPROACH TO THE CAL-SAG CHANNEL BRIDGE, SN 016-0777, COOK COUNTY, ILLINOIS

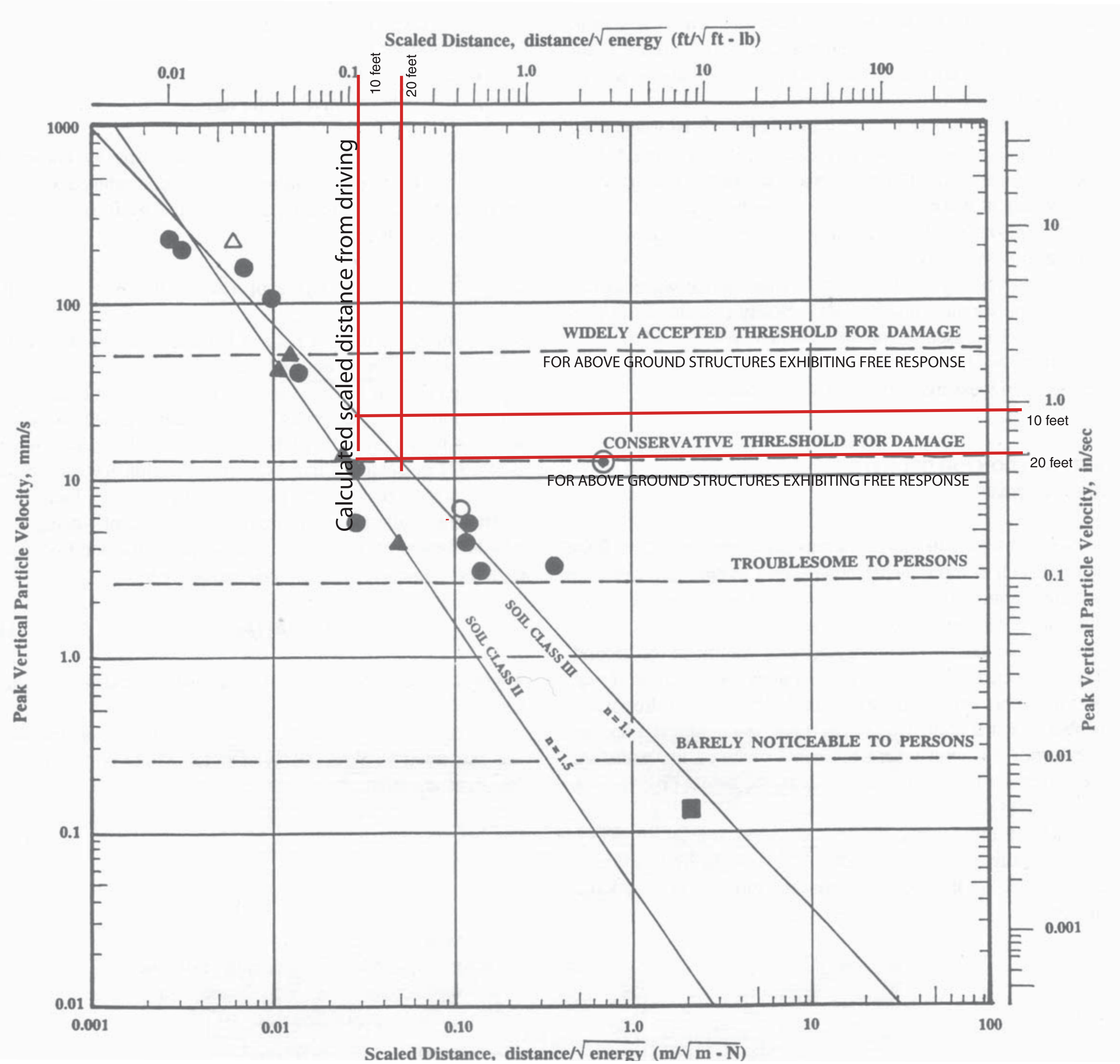
SCALE : GRAPHIC	APPENDIX B-5	DRAWN BY: R. Gorlagunta CHECKED BY: C. Marin

Wang Engineering
1145 N. Main Street
Lombard, IL 60148
www.wangeng.com

FOR BOLLINGER, LACH & ASSOCIATES, INC. 165-03-01

Wang Engineering

APPENDIX C



(After Woods, 1997)

VIBRATION ANALYSIS: WESTERN AVENUE SOUTH APPROACH TO THE CAL-SAG CHANNEL BRIDGE, SN 016-0777, COOK COUNTY		
SCALE: GRAPHICAL	APPENDIX C	DRAWN BY: M. Snider CHECKED BY: L. Iordache
Wang Engineering		1145 N. Main Street Lombard, IL 60148 www.wangeng.com
For Bollinger, Lach, and Associates		165-03-01