Structural Geotechnical Report

Bridge SN: 016-0927
Oak Park Avenue over Interstate I-80
IDOT PTB 205-003, Work Order #5
Cook County, Illinois

Prepared for:

Illinois Department of Transportation
Contract Number: 62W20

Project Design Engineer Team:
Ardmore Roderick

Geotechnical Consultant:
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October 30, 2025

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Oak Park Avenue over Interstate I-80
Cook County, Illinois
PTB 205-003, Work Order #5

Dear Mr. Brodaski:

Attached is a copy of the Structural Geotechnical Report for the above referenced project. The report provides a description of the site investigation, site conditions and foundation and construction recommendations. The site investigation for the proposed bridge construction included advancing two (2) bridge structural soil borings for the abutments to depths that achieved Nominal Driven Bearing Capacity of 500 kips for a HP 14 Pile per the IDOT Geotechnical Manual Chapter 3, Article 3.4.3.2, three (3) soil borings for the bridge piers to depths of 40 feet each, and four (4) subgrade borings to depths of 10 feet each.

Should you have any questions or require additional information, please call us at 630-994-2600.

Sincerely,

Matthew J Heron, P.E.
Project Engineer

Dawn Edgell, P.E.
Sr. Project Engineer

Ala E Sassila, Ph.D., P.E.
Principal

TABLE OF CONTENTS

Exhibits

Tables

Appendices

Structural Geotechnical Report
Oak Park Avenue over Interstate I-80
IDOT PTB 205-003, Work Order #5
Cook County, Illinois
SN: 016-0927

1.0 INTRODUCTION

GSG Consultants, Inc. (GSG) completed a geotechnical investigation for the proposed expansion to the Oak Park Avenue bridge over interstate I-80 in the village of Tinley Park in Cook County, Illinois. The purpose of the investigation was to explore the subsurface conditions, to determine engineering properties of the subsurface soil, and develop design and construction recommendations for the reconstruction of the bridge. **Exhibit 1** shows the general project location.

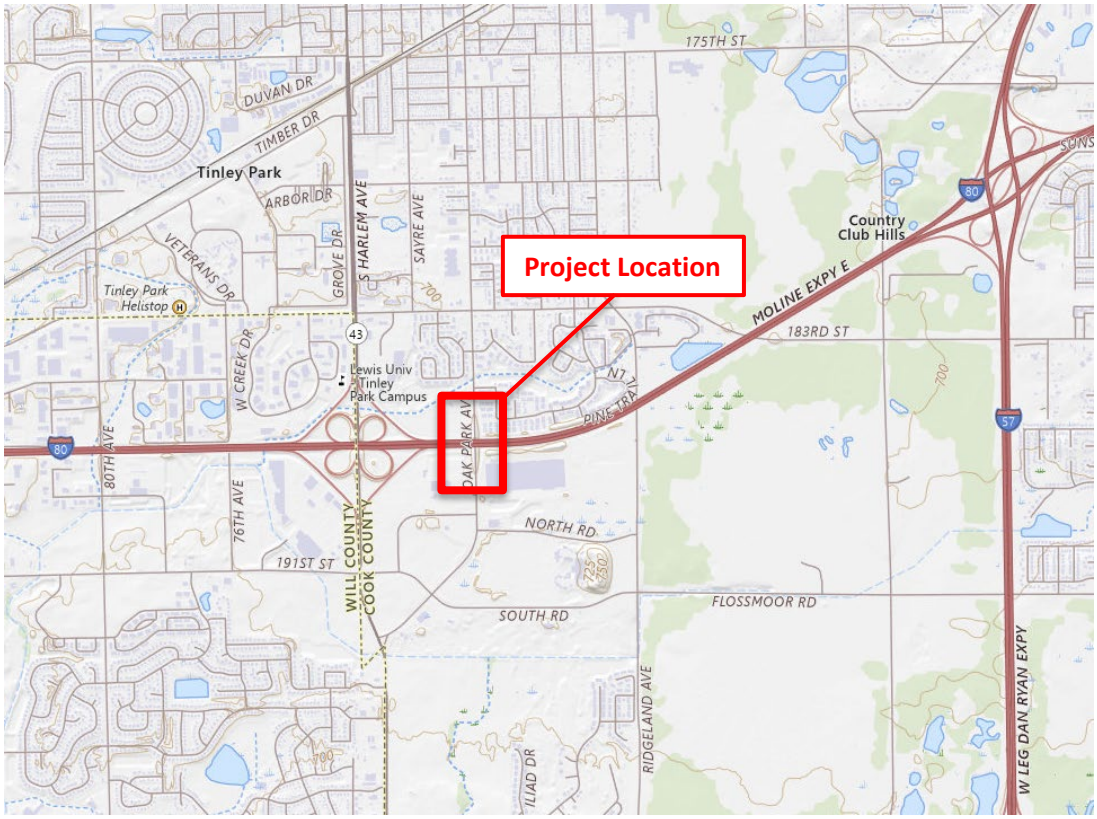


Exhibit 1 – Project Location Map
(Source: USGS Topographic Maps, usgs.gov)

1.1 Existing Bridge Information

Based on the General Plan and Elevation (GPE) dated 8/1/25 (**Appendix A**) and provided by Ardmore Roderick, the proposed project is to replace the existing bridge deck for the Oak Park Avenue bridge over Interstate I-80 (Structure Number 016-0927). The existing bridge will be

expanded approximately 9 feet to the west in order to accommodate a new 10-foot-wide shared-use path. The Oak Park Avenue bridge is currently a four-span structure with a bridge above the interstate has two lanes of traffic with two five-foot shoulders. **Exhibits 2a though 2c** show the existing conditions of the bridge which will be reconstructed.

Exhibit 2a – Existing Bridge, Looking East

Exhibit 2b – Existing Bridge, Looking West

Exhibit 2c – Existing Bridge Deck, Looking North

1.2 Proposed Bridge Information

The existing bridge, SN: 016-0927 at Station 300+00 will be widened in order to add a shared-use path for Oak Park Avenue over Interstate I-80. Based on the GPE (dated 8/1/25), the bridge is planned to remain a four-span structure with a total length of 226'-11" back-to-back between the abutments. The out-to-out distance of the bridge deck is proposed to be 51'-2", expanding the bridge deck by 9'2" to the west. The expanded semi-integral abutments and piers are anticipated to be supported by metal piles. The GPE shows slopes of 1:2 (V:H) below the abutments to Interstate I-80. Based on the proposed plans, the existing superstructure will be completely removed, and traffic will be detoured during construction. As part of the bridge expansion, new wingwalls will be constructed below each abutment on the west side of the bridge. It is assumed that the walls will be soldier pile walls.

2.0 SITE SUBSURFACE CONDITIONS

This section describes the subsurface exploration program and laboratory testing program completed as part of this project. The proposed locations and depths of the soil borings were selected in accordance with IDOT requirements. The borings were completed in the field based on field conditions and accessibility.

2.1 Subsurface Exploration

The site subsurface exploration for the proposed bridge reconstruction was conducted between December 2, 2024, and March 5, 2025. The investigation included advancing five (5) borings to depths between 40 and 113 feet for the bridge structure, and four (4) subgrade borings to a depth of 10 feet each for the roadway approaches. The bridge borings for the abutments were completed per IDOT requirements, to meet Nominal Driven Bearing Capacity of 500 kips for a HP 14 Pile per the IDOT Geotechnical Manual Chapter 3, Article 3.4.3.2. The pier borings were drilled to depths of 40 feet as the anticipated footings for the widened bridge are expected to match the existing bridge shallow foundation design.

The locations of these soil borings were reviewed and approved by Ardmore Roderick and adjusted in the field as necessary based on utilities and access. Elevations and as-drilled locations for the borings were gathered by GSG's field crew using GPS surveying equipment. The approximate as-drilled locations of the soil borings are shown on the Soil Boring Location Plan & Subsurface Profiles **(Appendix B)**. **Table 1** presents a summary of the borings.

Table 1 – Summary of Subsurface Exploration Borings

Boring ID	Abutment/Pier Location	Northing (ft)	Easting (ft)	Depth (ft)	Surface Elevation (ft)
BSB-1	North Abutment	1,779,645.454	1,134,724.966	98.5*	717.79
BSB-2	North Pier	1,779,570.117	1,134,701.650	40.0	699.00
BSB-3	Middle Pier	1,779,523.614	1,134,708.050	40.0	698.50
BSB-4	South Pier	1,779,436.592	1,134,714.675	40.0	699.00
BSB-5	South Abutment	1,779,342.531	1,134,728.199	113.0*	718.42
SGB-1	Subgrade	1,780,279.113	1,134,746.632	10.0	694.72
SGB-2	Subgrade	1,779,977.095	1,134,708.217	10.0	705.93
SGB-3	Subgrade	1,779,114.737	1,134,734.934	10.0	708.09
SGB-4	Subgrade	1,778,810.513	1,134,792.932	10.0	698.06

*borings were terminated prior to encountering bedrock; borings met IDOT capacity requirement for termination

The soil borings were drilled using truck mounted Diedrich D-50 (hammer efficiency 97.7%) and Mobile B-57 (hammer efficiency 92.1%) drill rigs, each equipped with 3¼-inch I.D. hollow stem augers and an automatic hammer. Soil sampling was performed according to AASHTO T 206, "Penetration Test and Split Barrel Sampling of Soils." Soil samples were obtained at 2.5-foot intervals to a depth of 10 feet for the shallow subgrade borings and 30 feet for the remaining locations; then at 5-foot intervals to the boring termination depths. The bridge abutment borings (B-1 and B-5) were advanced using blind drilling (no sampling) to determine if bedrock depth could be encountered; the borings were terminated prior to encountering refusal on bedrock. Water level measurements were made in each boring when evidence of free groundwater was detected on the drill rods or in the samples. The boreholes were also checked for free water immediately after auger removal, and before filling the open boreholes with soil cuttings and surface patching with asphalt where necessary to match the existing pavement. Copies of the Soil Boring Logs are provided in **Appendix C**.

GSG's field representative inspected, visually classified and logged the soil samples during the subsurface exploration activities and performed unconfined compressive strength tests on cohesive soil samples using a calibrated Rimac compression tester and a calibrated hand penetrometer in accordance with IDOT procedures and requirements. Representative soil

samples were collected from each sample interval and were placed in jars and returned to the laboratory for further testing and evaluation.

2.2 Laboratory Testing Program

All samples were inspected in the laboratory to verify the field classifications. A laboratory testing program was undertaken to characterize and determine engineering properties of the subsurface soils encountered in the area of the proposed bridge improvements. Moisture content tests (AASHTO T-265) were performed on representative soil samples.

The laboratory tests were performed in accordance with test procedures outlined in the most current IDOT Geotechnical Manual, and per ASTM and AASHTO requirements. Based on the laboratory test results, the soils encountered were classified according to the AASHTO and the Illinois Division of Highways (IDH) classification systems. The results of the laboratory testing program are shown along with the field test results in the Soil Boring Logs **(Appendix C)**.

2.3 Subsurface Soil Conditions

This section provides a brief description of the soils encountered in the borings performed in the vicinity of the proposed bridge. Variations in the general subsurface soil profile were noted during the drilling activities. Detailed descriptions of the subsurface soils are provided in the soil boring logs and are shown graphically in the Boring Location Plan & Subsurface Profiles. The soil boring logs provide specific conditions encountered at each boring location and include soil descriptions, stratifications, penetration resistance, elevations, location of the samples, and laboratory test data. Unless otherwise noted, soil descriptions indicated on boring logs are visual identifications. The stratifications shown on the boring logs represent the conditions only at the actual boring locations and represent the approximate boundary between subsurface materials; however, the actual transition may be gradual.

Bridge Abutments

Boring BSB-1 was drilled on the shoulder of Oak Park Avenue near the existing north abutment and had an initial elevation of 717.8 feet, while boring BSB-5 was drilled on the shoulder of Oak Park Avenue near the existing south abutment and had an initial elevation of 718.4 feet. The borings initially noted 6 to 10 inches asphalt over 10 to 12 inches of reinforced concrete. Beneath the pavement, boring BSB-1 noted brown sand and gravel fill materials to a depth of 4 feet. Both borings then encountered brown and gray silty clay and sandy clay fill soils to depths ranging from 26 to 28.5 feet (El. 689.3 to 692.4 feet). The clay fill materials had unconfined compressive

strengths ranging from 1.7 to 6.0 tsf. A sand seam was noted in boring BSB-1 at 12 feet and both borings encountered a layer of asphalt at depths of 21.5 to 22 feet, approximately the same elevation as the existing roadway beyond the bridge embankment.

Underlying the fill layers, the borings encountered brown and gray, medium stiff to hard silty clay to depths of 33.5 to 38.5 feet (El. 679.3 to 684.9 feet). The brown silty clay had unconfined compressive strength values ranging between 0.8 and 7.3 tsf, with an average value of 2.5 tsf. The borings then encountered layers of gray, stiff to hard silty clay and loose to dense sand and gravel until sampling was terminated at depths of 80 to 100 feet. The borings were then blind drilled and terminated at depths of 98.5 and 113 feet (El. 605.4 and 619.3 feet) without finding the top of bedrock. The gray silty clay unconfined compressive strength values ranging between 1.3 and 5.0 tsf, with an average value of 3.2 tsf. The granular soil had SPT blow count 'N' values between 9 and 40 blows per foot (bpf), with an average 'N' value of 23 bpf.

Bridge Piers

Boring BSB-2 was drilled on the outside shoulder of westbound I-80 near the existing north pier at an initial elevation of 699.0 feet; boring BSB-3 was drilled on the median shoulder of westbound I-80 near the existing middle pier at an elevation of 698.5 feet; and boring BSB-4 was drilled on the outside shoulder of eastbound I-80 near the existing south pier at an initial elevation of 699.0 feet. Borings BSB-2 and BSB-4 initially noted 1 to 6 inches asphalt over 10 inches of concrete, while boring BSB-3 noted 14 inches of asphalt. Beneath the pavement, the borings noted gray sand and gravel fill materials to depths of 2.5 to 4 feet (El. 695.0 to 696.5 feet). The granular fill materials had SPT blow count 'N' values between 9 and 40 blows per foot (bpf).

Underlying the fill layers, the borings encountered brown and gray, very stiff to hard silty clay to depths of 11 to 13.5 feet (El. 685.5 to 687.5 feet). The brown silty clay had unconfined compressive strength values ranging between 2.3 and 4.5 tsf, with an average value of 3.5 tsf. The borings then generally encountered gray stiff to hard silty clay until the boring termination depth of 40 feet (El. 659.0 feet). The gray silty clay unconfined compressive strength values ranging between 1.7 and 5.2 tsf, with an average value of 2.6 tsf.

Subgrade

Borings SGB-1 and SGB-2 were drilled off the shoulders of Oak Park Avenue north of Interstate I-80, while borings SGB-3 and SGB-4 were drilled off the shoulders of Oak Park Avenue south of

Interstate I-80. The borings had initial elevations between 694.7 to 708.1 feet, with elevations rising along the embankment approaching the bridge. The borings initially noted 12 inches topsoil and gravel. Beneath the surficial soils, the borings noted brown and gray silty clay fill materials to the boring termination depths of 10 feet (El. 684.7 to 698.1 feet). The clay fill materials had unconfined compressive strengths ranging from 1.5 to 7.1 tsf. Cobbles were noted in boring SGB-1 at 3.5 feet, while a sand seam was noted in boring SGB-4 at 6.5 feet.

2.4 Groundwater Conditions

Water levels were checked in each boring to determine the general groundwater conditions present at the site and were measured while drilling and after each boring was completed. Groundwater was not encountered during or immediately after drilling at the boring locations. None of the borings were left open after leaving the site due to safety concerns.

Based on the soil color change from brown to gray, it is anticipated that the long-term groundwater level is approximately between elevations of 680 to 685 feet. Perched water may also be present within the existing fill materials or confined sand layers observed in the borings. Water level readings were made in the boreholes at times and under conditions shown on the boring logs and stated in the text of this report. However, it should be noted that fluctuations in groundwater level may occur due to variations in the rainfall, other climatic conditions, or other factors not evident at the time measurements were made and reported herein.

3.0 GEOTECHNICAL ANALYSES

This section provides GSG's geotechnical analysis and recommendations for the design of the proposed bridge widening based on the results of the field exploration, laboratory testing, and geotechnical analysis. Subsurface conditions between borings may vary from those encountered at the boring locations. If structure locations, loadings, or elevations are changed, we request that GSG be contacted so that we may re-evaluate our recommendations.

3.1 Scour

The proposed bridge structure will carry Oak Park Avenue over interstate I-80 and have no waterways in the vicinity; therefore, scour will not be a concern for this project.

3.2 Embankment Settlement

It is understood that the new embankment will be expanded west by 10 feet and the embankment will have a 2H:1V slope. Therefore, it is anticipated that up to 5 feet of new fill may be required to construct the new embankment.

An analysis was performed to evaluate the anticipated total settlement due to the new embankment construction for the roadway expansion. Immediate settlement for cohesionless soils can typically occur during the filling operations, while the consolidation settlement for cohesive soils generally occurs over a longer period of time. The maximum estimated total settlements within the existing fill soils and native soils were calculated as shown in **Table 2** where 90% of the total settlement is estimated to be completed within 12 months. The settlement values provided in **Table 2** do not include any potential settlement of the newly constructed embankment materials as it is assumed the new embankment will be compacted and constructed per the IDOT specifications. Settlement estimates were calculated for both the north and south abutment, in the area of the proposed fill.

Table 2 – Anticipated Abutment Fill Settlement – Preliminary Calculations

Location	Nearest Boring	Roadway Fill Area		Assumed New Fill Height (feet)	Anticipated Total Settlement (inches)
		Assumed Width (feet)	Assumed Length (feet)		
North Abutment	BSB-1	10	50	5.0	0.20
South Abutment	BSB-5	10	50	5.0	0.20

Based on the general nature of the cohesive soils, encountered in the area of the proposed abutments, the estimated settlement of the existing soils from the new fill could be approximately 0.20 inches. Accordingly, downdrag will not be anticipated to be an issue in areas where pile foundations are constructed.

3.3 Roadway Fill Settlement Treatment and Recommendations

If the anticipated settlement is excessive for the proposed improvement, special design recommendations may be considered to mitigate the impact to the bridge construction. Some areas of the subgrade soil beneath the new roadway fill may require in-situ ground improvement in order to mitigate the anticipated settlement after the anticipated filling operations. The recommended ground improvement techniques are discussed below. The treatment alternative that is selected must also consider the proposed bridge foundation construction schedule.

Proper instrumentation, as outlined in IDOT Geotechnical Manual in Section 6.4.4.6-Instrumentation and Control of Embankment Construction, will be required to monitor the state of stress in the soil during the loading period, to ensure that loading does not proceed so rapidly as to cause a shear failure.

A maintenance program will likely be necessary throughout the construction stage to account for settlement of the new fill. This will require additional quantities of fill materials to be placed during construction, which should be accounted for when estimating earthwork quantities.

3.4 Slope Stability

Global slope stability for the embankment side slopes at both abutments were performed using data from the soil borings. GSG utilized Slide2, which is a comprehensive slope stability analysis software to evaluate global slope stability based on the limit equilibrium method. The proposed embankment slopes were analyzed based on the information provided by Ardmore Roderick and the soil boring data at the abutment locations. Circular failure analyses were evaluated using the simplified Bishop analysis method. The analyses were performed using the soil parameters provided in **Appendix E**.

A circular analysis was evaluated for both short term (undrained) and long term (drained) conditions for the slopes at each abutment. The slope geometries were based on the GPE dated 8/1/2025. The slopes at the abutments were evaluated at 1V:2H sloping condition. The results of the analyses are summarized in **Table 3**.

Table 3 – Slope Stability Analyses Results

Analysis Exhibit	Location	Structure		Failure Type	Factor of Safety
Exhibit 1	North Abutment	1V:2H	Slope to I-80	Circular – Short Term	5.7
Exhibit 2				Circular – Long Term	2.4
Exhibit 3	South Abutment	1V:2H	Slope to I-80	Circular – Short Term	5.4
Exhibit 4				Circular – Long Term	2.3

Based on the analyses performed, the proposed embankment design meets the minimum factor of safety of 1.5 for fill embankments per IDOT. Copies of the slope stability analyses are included in the Slope Stability Analyses Exhibits (**Appendix F**).

3.5 Seismic Parameters

The seismic hazard for the site was evaluated per the IDOT Geotechnical Manual, IDOT Bridge Design Manual, and AASHTO LRFD Bridge Design Specifications. The Seismic Soil Site Class was determined per the requirements of All Geotechnical Manual Users (AGMU) Memo 9.1, Design Guide for Seismic Site Class Determination, and the "Seismic Site Class Determination" Excel spreadsheet provided by IDOT, which is provided in **Appendix G**. A global Site Class Definition was determined for this project, and was found to be Soil Site Class D. The Seismic Performance Zone (SPZ) was determined using Figure 2.3.10-2 in the IDOT Bridge Manual and was found to be Seismic Performance Zone 1.

The AASHTO Seismic Design Parameters program was used to determine the peak ground acceleration coefficient (PGA), and the short (S_{DS}) and long (S_{D1}) period design spectral acceleration coefficients for each of the proposed structures. For this section of the project, the S_{DS} and the S_{D1} were determined using 2020 AASHTO Guide Specifications as shown in **Table 4**. Given the site location and materials encountered, the potential for liquefaction is minimal.

Table 4 – Seismic Parameters

Building Code Reference	PGA	S_{DS}	S_{D1}
2020 AASHTO Guide for LRFD Seismic Bridge Design	0.046g	0.160g	0.093g

4.0 GEOTECHNICAL BRIDGE DESIGN RECOMMENDATIONS

The foundations for the proposed bridge must provide sufficient support to resist the dead and live loads, as well as seismic loading. The foundation design recommendations presented within this section were completed per the AASHTO LRFD 9th Edition (2020). The anticipated loads provided by Ardmore Roderick are in **Table 5**.

Table 5 – Summary of Substructure Loads

Substructure ID	Combined Service Load (Kips)	Total Factored Load (Kips)	Combined Service Load per foot of Abutment (Kips/ft)	Total Factored Load per foot of Abutment (Kips/ft)
North Abutment	756.1	1,053.4	14.48	20.17
North Pier	1,114.5	1,559.0	21.36	29.88
Central Pier	1,351.9	1,894.9	25.92	36.32
South Pier	1,192.8	1,674.0	22.86	32.09
South Abutment	756.1	1,053.4	14.48	20.17

4.1 Bridge Foundation Recommendations

GSG evaluated potential foundation systems for the proposed bridge. GSG's evaluation included shallow foundations, drilled shafts, and driven piles. The results of the evaluation are presented below.

4.1.1 Shallow Foundations

Based on the anticipated span length and the anticipated loads, shallow foundations could be a feasible option for the proposed bridge piers to match the existing foundations. Based on preliminary design information, shallow foundations for the piers could bear near elevations 687.5 to 690.0 feet, to match the existing foundations. Design recommendations for shallow foundations are provided in *Section 4.2*. However, there will be differential settlement between the new and existing foundations. For this reason, it is anticipate that the piers will be supported on driven pile foundations.

4.1.2 Drilled Shaft Foundations

Drilled shafts are generally not recommended for semi-integral abutments because they do not have the lateral flexibility necessary to accommodate the thermal movements for integral abutments. Therefore, drilled shafts will not be further discussed in this report.

4.1.3 Driven Pile Foundations

Driven piles could be considered to support the bridge abutments and piers. Piles considered for this site include metal shell piles and H-piles. Concrete piles are not recommended for this site because the pile lengths cannot be readily adjusted to accommodate variability in soil conditions. Metal shell piles and H-piles are a feasible option for the construction of the abutments and piers for the proposed bridge structure.

Driving shoes for the piles, in accordance with Section 1006.05 (e) of the IDOT Standard Specifications for Road and Bridge Construction (SSRBC), should be considered to guard against the dense granular soils and cobbles below depths of 75 feet. For metal shell piles, a wall thickness of 0.25" or greater is recommended to minimize potential damage during driving. Design recommendations for driven piles are provided in Section 4.3 of this report.

4.2 Shallow Foundations Recommendations

Bearing resistance for the pier spread footings shall be evaluated at the strength limit state using load factors, and factored bearing resistance. The bearing resistance factor, ϕ_b, for clay is 0.50 per AASHTO Table 10.5.5.2.2-1. Bearing on the very stiff silty clay, the spread footings could be designed using nominal bearing resistance provided in **Table 6**. The nominal bearing resistance of the footings should not be greater than the compressive resistance of the footing concrete. The shallow footings should be designed such that the eccentricity of loading at the strength limit state should not exceed 1/3 of the corresponding footing dimensions (0.33 of footing width or length) AASHTO 10.6.3.3, Eccentric Load Limitations. The footing bearing elevations for the piers are anticipated to be between 687.5 and 690.0 feet.

Table 6 – Recommended Bearing Resistance for Bridge Piers

Boring IDs	Elevation* (feet)	Nominal Resistance (ksf)	Factored Bearing Resistance (ksf)	Bearing Resistance for 1-inch Settlement Service Limit (ksf)	Anticipated Bearing Soil
BSB-2 thru BSB-4	687.5 to 690.0	16.6	8.3	5.1	Very Stiff to Hard Silty Clay

*Assumed properties of native silty clay at bearing: cohesion = 2.7 ksf, unit weight = 138 pcf, resistance factor = 0.50

The minimum depth of the shallow foundations should be 3.5 feet below ground level to alleviate the effects of frost. The subgrade soils encountered at the bearing elevation should be cleared of any unsuitable material, such as topsoil.

Due to the new foundations bearing on unloaded soil, there will be differential settlement with the existing foundations should shallow foundations be used for the bridge piers. The existing foundations are not expected to experience additional loading and are therefore not expected to experience additional settlement. Large differential settlement could cause uneven pavement and cracking of the superstructure. It is understood that the widened portion of the bridge will be supported on independent foundations and an additional beam line, eliminating the connection points at the existing foundation and pier caps.

4.2.1 Shallow Foundation Lateral Resistance

The shallow foundations should be designed to resist sliding and overturning lateral and/or eccentric bridge loading. Resistance to lateral loads can be provided by a combination of friction at the foundation base, and by passive resistance acting against the vertical faces of foundation elements. A coefficient of friction of 0.35 may be used for footings. For passive resistance, an equivalent fluid pressure of 275 pounds per cubic foot (pcf) acting against the footing may be used. Passive resistance in the upper one foot of soil should be neglected unless the area is covered by concrete or pavement. The friction and passive resistance may be used concurrently provided the passive resistance is reduced by 50%.

4.3 Driven Pile Foundation Design Recommendations

It is our understanding that the expanded roadway embankment will be constructed concurrently with the bridge construction. According to AASHTO Section 3.11.8-Downdrag, the pile should be designed to resist the downdrag if the ground settlement is 0.4 inches or greater. Based on the anticipated embankment settlement, no downdrag is anticipated to occur.

GSG utilized the Modified IDOT static method-excel spreadsheet to estimate the pile lengths at various axial geotechnical resistances for driven piles per IDOT AGMU Memo 10.2. No geotechnical losses due to downdrag or liquefaction were included in the axial pile resistance calculations.

Tables 7a through 7e summarize the estimated maximum pile lengths for representative pile sections along with the factored resistance available for piles that are feasible for the proposed

substructures. Metal shell piles and H-piles are a feasible option for the construction of the abutments and piers for the proposed bridge structure.

The estimated pile lengths shown in **Tables 7a through 7e** and in **Appendix D** are based on the pile cutoff elevations shown on the GPE and noted below each table. The actual pile length and resistance should be evaluated based on test piles installed in accordance with the specifications provided in Section 512.15 of IDOT Standard Specifications for Road and Bridge Construction. Per section 3.10.1.11 of the IDOT Bridge Manual (2023), the minimum pile spacing should be 3 pile diameters, and the maximum pile spacing should not be more than 3.5 times the effective footing thickness plus one foot, not to exceed a total of 8 feet.

Table 7a – North Abutment Pile Design (BSB-1)

Pile Section	Nominal Required Bearing (kips)	Factored Resistance Available (kips)	Estimated Pile Length (feet)*
Metal Shell 12"Φ w/.25" walls (Max. R_N = 392 Kips)	336	185	47.0
	392	215	52.0
Metal Shell 14"Φ w/.25" walls (Max. R_N = 459 Kips)	396	218	47.0
	459	252	51.0
Metal Shell 14"Φ w/.312" walls (Max. R_N = 570 Kips)	511	281	63.0
	570	313	68.0
Metal Shell 16"Φ w/.312" walls (Max. R_N = 654 Kips)	588	323	63.0
	654	360	68.0

* Estimated pile length is based on assuming the pile cut off elevation: 711.9 ft., and ground elevation at beginning of pile driving: 710.9 ft.

Table 7b – Pier 3 Pile Design (BSB-2)

Pile Section	Nominal Required Bearing (kips)	Factored Resistance Available (kips)	Estimated Pile Length (feet)*
Metal Shell 12"Φ w/.25" walls (Max. R_N = 392 Kips)	168	93	22.0
	219	121	27.0
Metal Shell 14"Φ w/.25" walls (Max. R_N = 459 Kips)	199	110	22.0
	263	145	27.0
Metal Shell 16"Φ w/.312" walls (Max. R_N = 654 Kips)	231	127	22.0
	310	170	27.0

* Estimated pile length is based on assuming the pile cut off elevation: 691.0 ft., and ground elevation at beginning of pile driving: 690.0 ft.

Table 7c – Pier 2 Pile Design (BSB-3)

Pile Section	Nominal Required Bearing (kips)	Factored Resistance Available (kips)	Estimated Pile Length (feet)*
Metal Shell 12"Φ w/.25" walls (Max. R_N = 392 Kips)	151	83	20.0
	181	99	25.0
Metal Shell 14"Φ w/.25" walls (Max. R_N = 459 Kips)	181	99	20.0
	214	118	25.0
Metal Shell 16"Φ w/.312" walls (Max. R_N = 654 Kips)	212	117	20.0
	249	137	25.0

* Estimated pile length is based on assuming the pile cut off elevation: 688.5 ft., and ground elevation at beginning of pile driving: 687.5 ft.

Table 7d – Pier 1 Pile Design (BSB-4)

Pile Section	Nominal Required Bearing (kips)	Factored Resistance Available (kips)	Estimated Pile Length (feet)*
Metal Shell 12"Φ w/.25" walls (Max. R_N = 392 Kips)	164	90	22.0
	192	106	27.0
Metal Shell 14"Φ w/.25" walls (Max. R_N = 459 Kips)	196	108	22.0
	228	125	27.0
Metal Shell 16"Φ w/.312" walls (Max. R_N = 654 Kips)	229	126	22.0
	264	145	27.0

* Estimated pile length is based on assuming the pile cut off elevation: 691.0 ft., and ground elevation at beginning of pile driving: 690.0 ft.

Table 7e – South Abutment Pile Design (BSB-5)

Pile Section	Nominal Required Bearing (kips)	Factored Resistance Available (kips)	Estimated Pile Length (feet)*
Metal Shell 12"Φ w/.25" walls (Max. R_N = 392 Kips)	337	186	42.0
	392	215	47.0
Metal Shell 14"Φ w/.25" walls (Max. R_N = 459 Kips)	398	219	42.0
	459	252	47.0
Metal Shell 14"Φ w/.312" walls (Max. R_N = 570 Kips)	483	266	52.0
	570	313	57.0
Metal Shell 16"Φ w/.312" walls (Max. R_N = 654 Kips)	561	309	52.0
	654	360	57.0

* Estimated pile length is based on assuming the pile cut off elevation: 711.9 ft., and ground elevation at beginning of pile driving: 710.9 ft.

4.4 Pile Driving Considerations

The subsurface conditions appear to be consistent throughout the soil boring locations. The soil borings were completed at the proposed substructure locations. Therefore, the subsurface soil

conditions during the pile driving would be fairly predictable. IDOT standard practice includes driving one (1) indicator pile for each abutment to determine the actual pile length. A test pile shall be performed at each abutment prior to production driving so that actual, on-site, field data can be gathered to determine pile driving requirements for the project.

Driving shoes for the piles, in accordance with Section 1006.05 (e) of the IDOT Standard Specifications for Road and Bridge Construction (SSRBC), should be included to protect piles from damage during driving. For metal shell piles, a wall thickness of 0.25" or greater is recommended to minimize potential damage during driving with a conical tip welded to the pile to avoid abrupt overstress.

Pile setup is a consideration that can contribute to an increase in long-term pile resistance of displacement piles (i.e. driven pile). This increase in resistance is referred to as pile setup which is the gain in pile resistance over time that occurs mainly due to dissipation of pore water pressures and healing of the distorted and remolded soils immediately surrounding the pile. The magnitude of soil setup is function of pile type as well as soil type and consistency. A greater magnitude of soil setup is generally expected for soft clays, dense granular deposits, and displacement type piles than for stiff clays, loose granular deposits, and non-displacement type piles. However, pile setup consideration should not be included in the pile resistance during the design phase of the project, but this may be considered during the construction phase if a pile does not achieve the required bearing during installation. Based on the subsurface soil conditions, GSG does not anticipate any setup for the driven piles.

4.5 Lateral Load Resistance

Lateral loadings applied to deep foundations are typically resisted by the soil/structure interaction, pile flexure, or a combination of these factors. Section 3.10.1.10 of the 2023 IDOT Bridge Manual requires performing detailed structure interaction analysis if the factored lateral loading per pile exceeds 3 kips. The analysis shall determine actual pile moment and deflection to determine the selected pile adequacy for the existing loadings. **Tables 8a through 8c** provide generalized soil parameters for the abutments and includes recommended lateral soil modulus and soil strain parameters that can be used for deep foundation analysis via the p-y curve method based on the encountered subsurface conditions. Additional soil parameters that can be used for design are included in **Tables E1 through E3, Appendix E**. GSG recommends designing the abutments using the drained condition for the long-term, permanent design.

Table 8a – Lateral Load Resistance Parameters – North Abutment (BSB-1)

Depth / Elevation Range (feet)	Soil Description	Lateral Modulus of Subgrade Reaction (pci)	Soil Strain (ε_{50})
	New Engineered Clay Fill	1,000	0.005
	New Engineered Granular Fill	90	N/A
3.5 – 26 (714.3 - 691.8)	Fill: Brown and Gray Silty Clay	1,000	0.005
26 – 38.5 (691.8 - 679.3)	Brown and Gray Medium Stiff to Hard Silty Clay	1,000	0.005
38.5 – 58.5 (679.3 - 659.3)	Gray, Very Stiff to Hard Silty Clay	1,000	0.005
58.5 – 63.5 (659.3 - 654.3)	Gray, Loose Sand with Gravel	20	N/A
63.5 – 73.5 (654.3 - 644.3)	Gray, Stiff to Very Stiff Silty Clay	1,000	0.005
73.5 – 78.5 (644.3 - 639.3)	Gray, Medium Dense Silty Loam	60	N/A
78.5 – 80 (639.3 - 637.8)	Gray, Medium Dense Gravel with Sand	60	N/A

**The initial p-y modulus, E_{py} , varies linearly with depth. To obtain E_{py} use the equation $E_{py} = k_{py} * z$, where k_{py} is the subgrade modulus given in the table and z is the distance from the surface to the center point of the layer in inches.*

Table 8b – Lateral Load Resistance Parameters – Piers (BSB-2 to BSB-4)

Depth / Elevation Range (feet)	Soil Description	Lateral Modulus of Subgrade Reaction (pci)	Soil Strain (ε_{50})
	New Engineered Clay Fill	1,000	0.005
	New Engineered Granular Fill	90	N/A
1 – 2.5 (698.0 – 696.5)	Fill: Brown and Gray Silty Clay / Sandy Clay	90	N/A
2.5 – 13 (696.5 – 686.0)	Brown and Gray Very Stiff, Silty Clay	1,000	0.005
13 – 40 (686.0 – 659.0)	Gray, Stiff to Hard Silty Clay	1,000	0.005

**The initial p-y modulus, E_{py} , varies linearly with depth. To obtain E_{py} use the equation $E_{py} = k_{py} * z$, where k_{py} is the subgrade modulus given in the table and z is the distance from the surface to the center point of the layer in inches.*

Table 8c – Lateral Load Resistance Parameters – South Abutment (BSB-5)

Depth / Elevation Range (feet)	Soil Description	Lateral Modulus of Subgrade Reaction (pci)	Soil Strain (ε_{50})
	New Engineered Clay Fill	1,000	0.005
	New Engineered Granular Fill	90	N/A
1.5 – 26 (716.9 - 692.4)	Fill: Brown and Gray Silty Clay / Sandy Clay	1,000	0.005
26 – 33.5 (692.4 - 684.9)	Brown and Gray Very Stiff, Silty Clay	1,000	0.005
33.5 – 53.5 (684.9 - 664.9)	Gray, Stiff to Hard Silty Clay	1,000	0.005
53.5 – 58.5 (664.9 - 659.9)	Gray, Loose Sand	60	N/A
58.5 – 63.5 (659.9 - 654.9)	Gray, Hard Silty Clay	2,000	0.004
63.5 – 73.5 (654.9 - 644.9)	Gray, Medium Dense to Dense Sand with Gravel	60	N/A
73.5 – 78.5 (644.9 - 639.9)	Gray, Stiff Silty Clay	500	0.007
78.5 – 88.5 (639.9 - 629.9)	Gray, Medium Dense to Dense Sand	125	N/A
88.5 – 100 (629.9 - 618.4)	Gray Medium Dense to Very Dense Silty Clay Loam	125	N/A

**The initial p-y modulus, E_{py} , varies linearly with depth. To obtain E_{py} use the equation $E_{py} = k_{py} * z$, where k_{py} is the subgrade modulus given in the table and z is the distance from the surface to the center point of the layer in inches.*

5.0 GEOTECHNICAL WALL DESIGN RECOMMENDATIONS

This section provides retaining wall design parameters including recommendations on foundation type, bearing capacity, settlement, and lateral earth pressures. The foundations for the proposed wingwalls must provide sufficient support to resist the dead and live loads, as well as seismic loading. The foundation design recommendations presented within this section were completed per the AASHTO LRFD 9th Edition (2020).

5.1 Retaining Wall Type Recommendations

The proposed wingwalls are anticipated to be constructed along the west side of each abutment. It is anticipated that the wingwalls will have a maximum exposed height of approximately 5 feet. It is anticipated that the walls will be in a fill area to build up the widened bridge. The ground elevations adjacent to the walls are anticipated to be about 714.3 feet.

The actual wall width and total height of the walls should be based on structural analysis performed by a Licensed Structural Engineer in the State of Illinois.

GSG evaluated deep foundation systems for the proposed walls. Based on the existing and proposed site conditions and the design information, a soldier pile wall will likely be the best alternative.

5.2 Retaining Wall Design Recommendations

The engineering analyses performed for evaluation of the retaining wall options followed the current AASHTO Load and Resistance Factor Design (LRFD) Methodology as required by IDOT. LRFD methodology incorporates the use of load factors and resistance factors to account for uncertainty in applied loads and load resistance of structure elements separately. The AASHTO LRFD Bridge Design Specifications outline load factors and combinations for various strength, extreme event, service, and fatigue limit states. Section 11, which outlines geotechnical criteria for retaining walls, of the AASHTO Specifications requires the evaluation of bearing resistance failure, lateral sliding, and overturning at the strength limit state and excessive vertical displacement, excessive lateral displacement, and overall stability at the service limit state. **Table 9** outlines the load factors used in the evaluation of the retaining walls in accordance with AASHTO Specification Tables 3.4.1-1 and 3.4.1-2.

Table 9 – LRFD Load Factors for Wingwall Analyses

	Type of Load	Sliding and Eccentricity Strength Ia	Bearing Resistance Strength Ib	Sliding and Eccentricity Extreme IIa	Bearing Resistance Extreme IIb	Settlement Service I
Load Factors for Vertical Loads	Dead Load of Structural Components (DC)	0.90	1.25	0.90	1.25	1.00
	Vertical Earth Pressure Load (EV)	1.00	1.35	1.00	1.35	1.00
	Earth Surcharge Load (ES)		1.50			
	Live Load Surcharge (LS)		1.75			1.00
Load Factors for Horizontal Loads	Horizontal Earth Pressure Load (EH) Active At-Rest AEP for anchored walls	1.50	 1.50 1.35 1.35	1.00	1.00	1.00
	Earth Surcharge (ES)		1.50			
	Live Load Surcharge (LS)	1.75	1.75			1.00
Load Factor for Vehicular Collision				1.00	1.00	

5.2.1 Lateral Earth Pressures and Loading

The walls shall be designed to withstand earth and live lateral earth pressures. The lateral earth pressures on the abutment walls depend on the type of wall (i.e. restrained or unrestrained), the type of backfill and the method of placement against the wall, and the magnitude of surcharge weight on the ground surface adjacent to the wall. The active earth pressure coefficients (Ka), and the passive earth pressure coefficients (Kp) were determined in accordance with AASHTO Section 3.11.5.3 and 3.11.5.4. **Appendix E** presents the soil design properties for the anticipated soil types at the site.

Traffic and other surcharge loads should be included in the design of the walls. A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall in accordance with AASHTO 3.11.6.4. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height (H_{eq}) as shown in **Table 10** for vehicular loadings parallel to traffic.

Table 10 - Equivalent Height of Soil for Vehicular Loading on Wingwalls Parallel to Traffic

Retaining Wall Height (ft)	Heq Distance from Wall Back face to Edge of Traffic	
	0 feet	1.0 feet or Further
5	5.0 feet	2.0 feet
10	3.5 feet	2.0 feet
≥20	2.0 feet	2.0 feet

Reference: AASHTO LRFD Table 3.11.6.4-2

The wingwalls should be designed with free draining material as reinforced soil mass and the discharging water should be collected within the reinforced fill and drained away from the wall system. This will allow movement of any water behind the wall panel, and no hydrostatic (seepage) pressures will develop in the active soil wedge behind the wall panel. The backfill should be placed in accordance with the IDOT SSRBC. Heavy compaction equipment should not be allowed closer than five (5) feet to the wingwall to prevent inducing high lateral earth pressures and causing wall yielding and/or other damage. The passive lateral earth pressure coefficient (K_p) from the upper 3.5 feet of level backfill at the toe of the wall should be neglected, unless the soil is confined or protected by a concrete slab or well drained pavement. The passive lateral earth pressure coefficient from the upper 3.5 feet of soil for a descending slope at the wall toe should also be neglected, regardless of any surface protection.

5.3 Soldier Pile and Lagging

Soldier pile walls are generally constructed at 8 to 10-foot centers along the alignment into the bearing stratum. The soldier piles could either be driven or drilled. Driving piles is normally less expensive but the designs are limited to H-pile and small W-sections. Drilled soldier piles can utilize larger W-sections, built-up plate sections, or multiple W-sections. For drilled piles, the pile will be placed into the hole and centered, and the annular space around each pile section will be filled with flowable grout. The lagging and piles should be designed based on structural analysis.

Resistance to lateral movement or overturning of the soldier pile is furnished by passive resistance of the soil below the depth of excavation. The design should include a structural evaluation of the pile section to meet applied shear and moment, and an evaluation of overturning to determine embedment depth and other design requirements. The walls shall be designed to withstand earth and live lateral earth pressures. The lateral earth pressures on retaining walls depend on the type of wall (i.e. restrained or unrestrained), the type of backfill and the method of placement against the wall, and the magnitude of surcharge weight on the

ground surface adjacent to the wall. Soldier pile walls are considered flexible; the earth loads may be calculated using active earth pressure for the load above the design grade, and both active and passive earth pressures below the design grade. The active earth pressure coefficient (Ka), and the passive earth pressure coefficient (Kp) are shown in **Appendix E**.

The simplified earth pressure distributions shown in the AASHTO Standard Specifications for Highway Bridges could be used for the wall design. **Appendix E** also provides recommended lateral soil modulus and soil strain parameters that can be used for laterally loaded pile analysis via the p-y curve method based on the encountered subsurface conditions. The passive resistance in front of the wall should be ignored for the upper 3.5 feet due to excavation activities and frost-heave conditions. Construction equipment surcharge loads should be added to the lateral earth pressure.

In order to limit wall deflections and provide additional resistance, the soldier pile and lagging retention system could be restrained with tie-back anchors. The soldier pile and lagging retention system restrained with tie-backs will be subjected to "trapezoidal" lateral soil pressures. For tall retaining walls, the "trapezoidal" pressure will result in greater lateral forces and moments compared to the cantilever design.

5.4 Global Slope Stability

Based on the information provided by Ardmore Roderick, the wingwalls should be designed for external stability of the wall system. The design information in **Table 11** was used to evaluate the overall stability of the walls.

Table 11 – Wingwall Description

Description	Value
Maximum exposed height of the retaining wall (H)*	8'-8.75"
Unit weight of the existing retained soil (embankment)	138 pcf
Estimated embedment length below bottom of concrete facing for soldier pile**	9 feet
Estimated Pile tip elevation – soldier pile wall	705.4 feet

* Based on TSL (dated 8/1/25)
**Additional length may be required for lateral pressures and structural design of the wall system

The actual wall width and total height of the walls should be based on structural analysis performed by a Licensed Structural Engineer in the State of Illinois.

5.5 Global Slope Stability Results

Slide2 is a comprehensive slope stability analysis software used to evaluate the proposed walls for the project based on the limit equilibrium method. Circular failure analyses were evaluated for both a short-term (undrained) and long-term (drained) condition based on the proposed geometry (**Table 11**) for the proposed soldier pile wingwall. The analyses were performed at the sections of the assumed maximum wall heights for the abutments. The results of the analyses are shown in **Table 12**.

Table 12 – Wingwall Global Slope Stability Analyses Results

Analysis Exhibit	Wall Type	Analysis Type	Factor of Safety	Minimum Factor of Safety
Exhibit 5	Soldier Pile	Circular – Short Term	4.6	1.5
Exhibit 6		Circular – Long Term	2.2	1.5

Based on the analyses performed, the proposed wingwalls meet the minimum factor of safety of 1.5. Copies of the Slope Stability analyses exhibits are included in **Appendix F**.

5.6 Drainage Recommendations

The wall design should include drainage system to prevent the buildup of hydrostatic forces behind the wall. This could be accomplished with the installation of drainage blankets, geocomposite drainage panels, or gravel drains behind the facing of the wall with outlet pipes below the facing to collect and remove surface water away from the face of the retaining wall. If weep holes are to be used, it is recommended that a geocomposite wall drain be placed over the interlocks and area of the weep holes. If drainage is not provided, hydrostatic pressure should be included in the wall design and the horizontal earth pressure should be determined in accordance with AASHTO Article 3.11.3.

6.0 GEOTECHNICAL ROADWAY RECOMMENDATIONS

This section provides GSG's geotechnical recommendations for the design of the proposed roadway reconstruction based on the results of the field exploration, laboratory testing, and geotechnical analysis. The proposed pavement section should be designed according to the IDOT Mechanistic Pavement Design (MPD). IDOT policy requires providing a minimum of 12 inches of improved subgrade beneath the pavement section to ensure a stable construction platform. Subgrade improvements including any undercuts or compaction of existing soils should be completed to the proposed elevations in the design plan and in accordance with the Subgrade Treatment and Recommendation Section of this report.

6.1 Subgrade Preparation

It is our understanding that the existing roadway is to be reconstructed to match the proposed grading of the new bridge deck pavement. It is recommended that all existing pavement and base course materials be stripped within the limits of the proposed reconstruction. Based on the asphalt and concrete thicknesses encountered in the borings, it is anticipated that pavement stripping depths will range from 18 to 20 inches. It is recommended to use an average stripping depth of 20 inches for estimating. Aggregate base course materials encountered at the site should be evaluated to determine suitability for reuse as general fill. The contractor should not mix the existing base course materials with existing subgrade soils during the stripping and stockpiling activities. Boring BSB-1 encountered granular base materials to a depth of 3.5 feet. Undercuts of the subgrade soils at the proposed new subgrade elevation should be based on the recommendations provided in this report, and field evaluation of the materials encountered during construction. Any unstable or unsuitable materials encountered during construction activities should be removed and replaced with compacted structural fill.

6.2 Subgrade Treatment and Recommendations

The suitability of the existing subgrade soils for the area of the proposed project was evaluated in terms of frost susceptibly, stability, settlement, and drainage. The evaluation included determining the presence of any unstable, compressible deposits, low-strength soils, high organic content soils, and soils with high-moisture content immediately below the proposed pavement section.

The choice of a specific treatment option depends on several factors, including soil type; required treatment depth; construction variables (cost, availability, and time); project location; and treatment objective. Treatment options for unsuitable subgrade soils include mechanical

stabilization, chemical stabilization, or soil modification. Mechanical stabilization includes removal and replacement with approved structural fill or using geosynthetics (geotextiles and/or geogrids) to stabilize the subgrade. Based on the borings performed in the field exploration program, the subgrade soils are suitable for support of the roadway reconstruction.

GSG recommends mechanical stabilization methods (undercuts) to remediate any unsuitable soils encountered during construction such as subgrade modification or chemical stabilization is not considered economical for the limited area of remediation. If deeper unsuitable soils are encountered during construction, the Engineer should be notified for further remedial treatment options. Approved fill includes IDOT Standard Specifications Section 303: Aggregate Subgrade Improvement, or suitable borrow materials, as specified in the Borrow Material and Compaction Requirements section of this report.

6.3 Drainage Recommendation

The drainage classification of Fair should be used for the project design. The overall groundwater depth is assumed to be deeper than the anticipated frost depth of 45 to 60 inches for the northern Illinois region. However, pavement systems could become saturated following periods of precipitation. The proposed subgrade and pavement should have proper surface grading to prevent water from accumulating and ponding. GSG recommends installing lateral and longitudinal underdrain systems as recommended in Section 6.3.4.2 of the IDOT Geotechnical Manual to maintain the subgrade from deteriorating. These underdrains should be installed at undercut areas and low points in the roadway profile, and along the edge of pavement throughout the roadway improvement.

7.0 CONSTRUCTION CONSIDERATIONS

All work performed for the proposed project should conform to the requirements in the IDOT Standard Specifications for Road and Bridge Construction (2022). Any deviation from the requirements in the manuals above should be approved by the design engineer.

7.1 Existing Utilities and Structures

Before proceeding with construction, all existing underground utility lines or structures that will interfere with construction should be completely relocated from the proposed construction areas. Where possible, existing utility lines that are to be abandoned in place should be removed and/or plugged with cement grout. All excavations resulting from underground utilities or structure removal activities should be cleaned of loose and disturbed materials, including all previously placed backfill, and backfilled with suitable fill materials in accordance with the requirements of this section. During the clearing and stripping operations, positive surface drainage should be maintained to prevent the accumulation of water.

7.2 Site Excavation

If borrow material is to be used for onsite construction, it should conform to Section 204 "Borrow and Furnished Excavation" of the IDOT Construction Manual (2021). The fill material should be free of organic matter and debris. Earth-moving operations should be avoided during excessively cold or wet weather to avoid freezing of softening subgrade soils.

Structural fill shall consist of crushed limestone or recycled concrete consistent with IDOT CA-6 gradation or medium plasticity silty clays. Structural fill should be placed in lifts not to exceed 8 inches in loose thickness and compacted to a minimum of 95% of the material's standard proctor maximum dry density obtained according to the ASTM D698/AASHTO T 99 method.

Materials unsatisfactory for use as structural fill include soils classified as silt or organic silt (ML, MH, PT, OL, and OH) in the Unified Classification System (ASTM D2487). Soils with these classifications may be used for general purpose landscaping and in areas where uncontrolled settlement is acceptable.

Should fill be placed during cool, wet seasons, the use of granular fill may be necessary since weather conditions will make compaction of cohesive soils more difficult. If water seepage while excavating and backfilling procedures, or where wet conditions are encountered such that the water cannot be removed with conventional sump and pump procedures, GSG recommends

placing open grade stone similar to IDOT CA-7 to stabilize the bottom of the excavation. The CA-7 stone should be placed 12 inches above the water level, in 12-inch lifts, and should be compacted with the use of a heavy smooth drum roller or heavy vibratory plate compactor until stable. The remaining portion of the excavation should be backfilled using approved engineered fill.

GSG recommends that foundation excavations, subgrade preparation, and structural fill placement and compaction be inspected by a GSG geotechnical engineer to verify the type and strength of soil materials present at the site and their conformance with the geotechnical recommendations in this report.

7.3 Pile Installation

Based on the variance in soils at the site it is recommended test piles be utilized at the site. One test pile is recommended at each structure. The test-piles are installed based on the preliminary driving criteria in order to evaluate site conditions and are inspected in accordance with the IDOT Standard for Road and Bridge Construction. All test pile installation should be completed in accordance with the IDOT SSRBC Section 512.15. Pile shoes should be used for the abutment piles to facilitate driving into the dense granular soils and protect piles from damage during installation.

7.4 Temporary Earth Retention Systems

Temporary soil retention systems (TSRS) will likely be required for portions of the construction, including the widening of the piers. The design of the TSRS is the responsibility of the contractor.

7.5 Groundwater Management

Based on the overall color change from brown to gray, it is anticipated that the long-term groundwater level is likely between elevations of 680 and 685 feet. However, perched water may be encountered within the existing embankment fill materials. If rainwater run-off or perched water is accumulated at the base of excavation, the contractor should remove accumulated water using conventional sump pit and pump procedures and maintain a dry and stable excavation. The location of the sump should be determined by the contractor based on field conditions. During earthmoving activities at the site, grading should be performed to ensure that drainage is maintained throughout the construction period. Undercut and excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater or surface

run-off. Grades should be sloped away from the excavations to minimize runoff from entering the areas.

If water seepage occurs during excavations or where wet conditions are encountered such that the water cannot be removed with conventional sumping, we recommend placing open grade stone similar to IDOT CA-7 to stabilize the bottom of the excavation below the water table. The CA-7 stone should be placed 12 inches above the water table, in 12-inch lifts, and should be compacted with the use of a heavy smooth drum roller or heavy vibratory plate compactor until stable. The remaining portion of the excavation should be backfilled using approved embankment fill.

8.0 LIMITATIONS

This report has been prepared for the exclusive use of the Illinois Department of Transportation (IDOT) and its Design Section Engineer consultant. The recommendations provided in the report are specific to the project described herein and are based on the information obtained at the soil boring locations within the proposed bridge area. The analyses have been performed and the recommendations provided in this report are based on subsurface conditions determined at the location of the borings. This report may not reflect all variations that may occur between boring locations or at some other time, the nature and extent of which may not become evident until the time of construction. If variations in subsurface conditions become evident after submission of this report, it will be necessary to evaluate their nature and review the recommendations presented herein.

APPENDIX A

GENERAL PLAN AND

ELEVATION

Benchmark: Cut square in end of concrete walk @ NE corner of Oak Park Avenue Bridge over I-80. Elev. 717.94

Existing Structure:
SN 016-0927 was originally constructed in 1967, under Section 1313-811 HB. In 1997, the super structure was raised 1 1/2", the deck was overlaid, and the superstructure was painted. The joints, slopewalls, abutment bearings, and part of the West fascia beam in span 3 were replaced. The structure is a four-span, continuous WF36x135 wide flange steel beam bridge, supported on stub abutments and multi-column and cap piers on spread footings. The overall structure length measures 226'-11" back-to-back of abutments with a 42'-0" out-to-out width.

Traffic to be detoured during bridge reconstruction

No salvage

SEISMIC DATA

Seismic Retrofit Category (SRC) = C
Design Spectral Acceleration at 1.0 sec. (SD1) = 0.093g
Design Spectral Acceleration at 0.2 sec. (SDS) = 0.160g
Soil Site Class = D
Performance Level = 1

HIGHWAY CLASSIFICATION

Oak Park Ave.
Functional Class: Major Collector
ADT: 2700 (2023); 2475 (2046)
ADTT: 25 (2021); 25 (2032)
DHV:305
Two-Way Traffic
Directional Distribution: 50/50
Design Speed: 50 m.p.h.
Posted Speed: 50 m.p.h.

F.A.I. 80
Functional Class: Interstate
ADT: 90,800 (2023); 107,565 (2046)
ADTT: 17,252 (2023); 20,438 (2046)
DHV: N/A
Two-Way Traffic
Directional Distribution: 50/50
Design Speed: 65 m.p.h.
Posted Speed: 70 m.p.h.

DESIGN STRESSES

NEW CONSTRUCTION

f'c = 3,500 psi (Concrete Substructure)
f'c = 4,000 psi (Concrete Superstructure)
fy = 60,000 psi (Reinforcement)
fy = 50,000 psi (Steel, M270 Grade 50)

EXISTING CONSTRUCTION

fc = 1,400 psi (Substructure Concrete)
fs = 20,000 psi (Substructure Reinforcement)

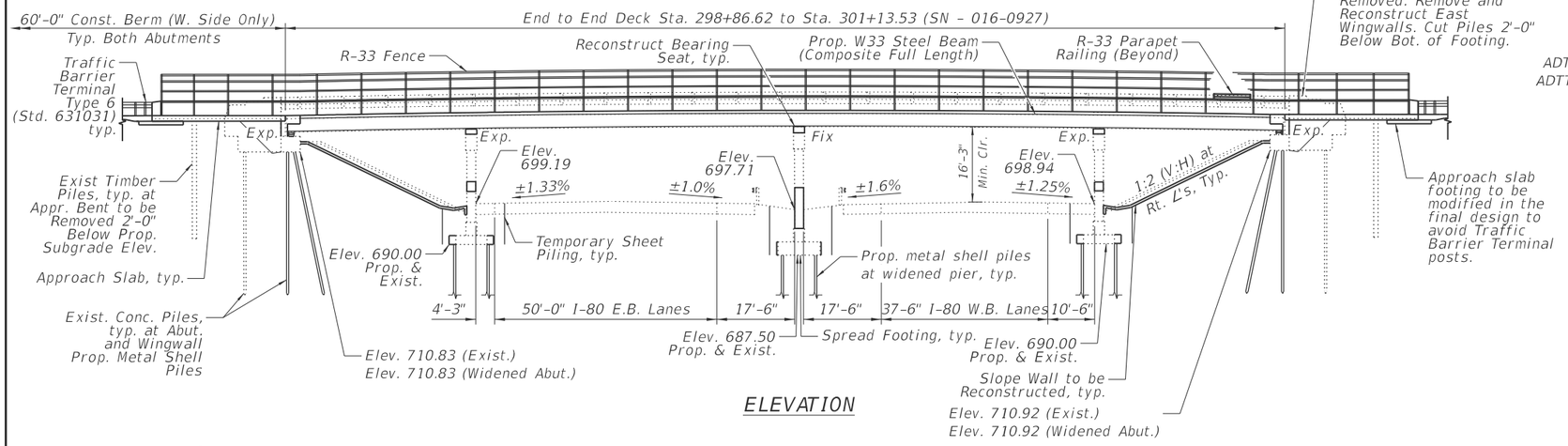
DESIGN SPECIFICATIONS

AASHTO LRFD Bridge Design Specifications, 10th Edition, 2024

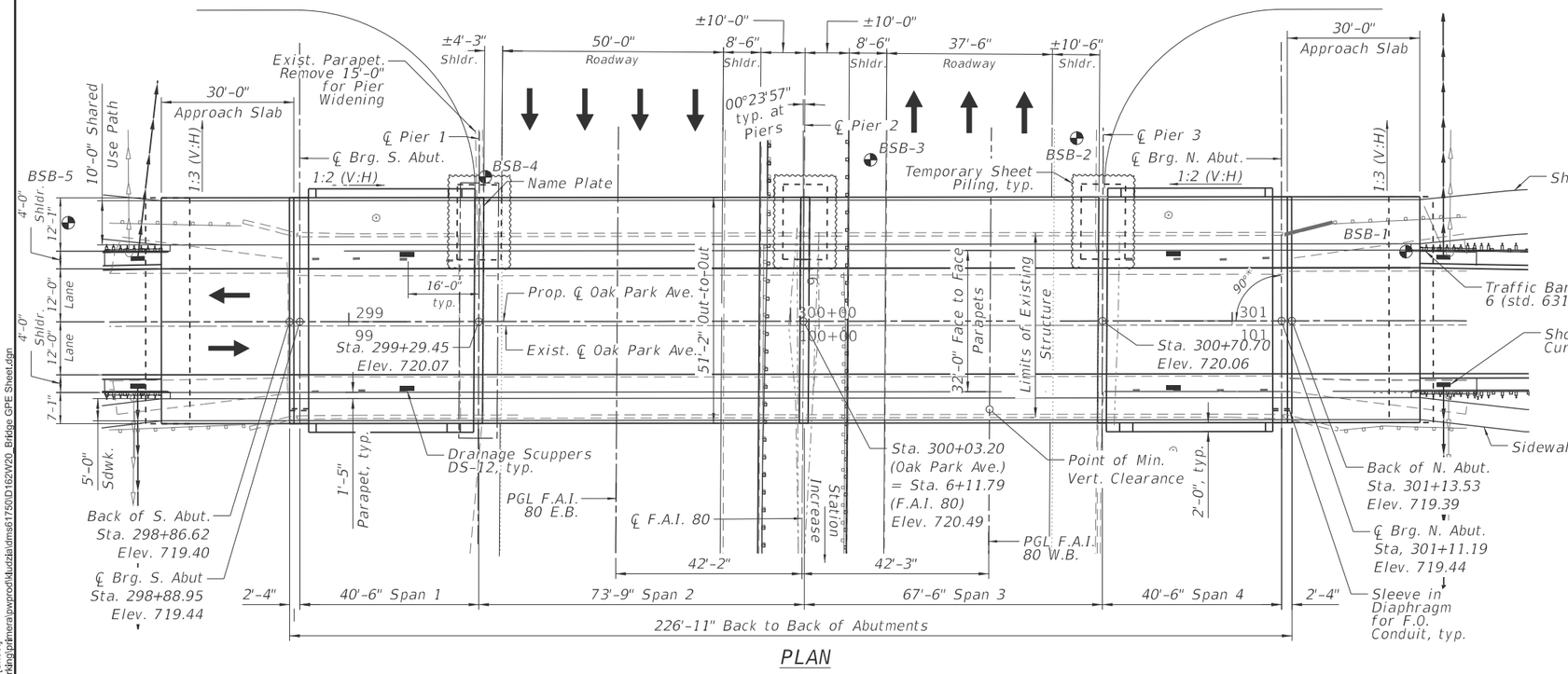
2006 Seismic Retrofitting Manual for Highway Structures: Part I – Bridges (FHWA-HRT-06-032)

LOADING HL-93

Allow 50#/sq.ft. For Future Wearing Surface
Live Load Deflection Limit Span/1000



ELEVATION



PLAN

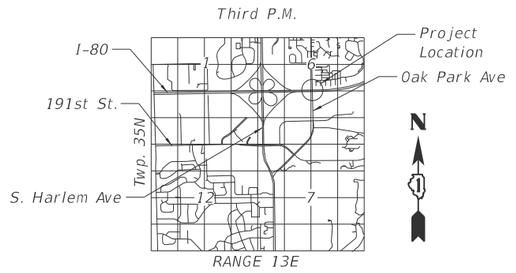
* typ. at Abutments

SCOPE OF WORK

1. Remove existing superstructure and bearings.
2. Construct new pier and abutment cap extensions.
3. Construct new composite superstructure on new bearings.
4. Remove approach slabs and bents and replace with new approach slabs.
5. Remove abutment backwalls and convert to semi-integral abutments with new abutment drainage system behind abutments.
6. Remove west wingwalls below proposed approach slab.
7. Remove east wingwalls and reconstruct with wingwalls square to abutments.
8. Widen existing abutments to the west in kind with converted existing abutments and construct new wingwalls.
9. Widen existing piers to the west.
10. Raise existing pier crash walls to 5'-0" above grade elevation.
11. Repair deteriorated concrete on piers and abutments.
12. Reconstruct slope walls at both ends of bridge.
13. Apply concrete sealer to exposed surfaces of existing substructure.
14. Remove and replace existing approach guardrail terminals.

NOTE:

1. All structural steel shall be galvanized.



LOCATION SKETCH

GENERAL PLAN & ELEVATION
OAK PARK AVENUE OVER I-80
FAU 2774-SECTION 1313-811 HB
COOK COUNTY
STATION 303+03.20
STRUCTURE NO: 016-0927

MODEL: TSL Sheet 1[Sheet]
FILE NAME: c:\ppwork\kingl\pltmera\pwprod\kudzlaldms61750\D162W20_Bridge GPE Sheet.dgn

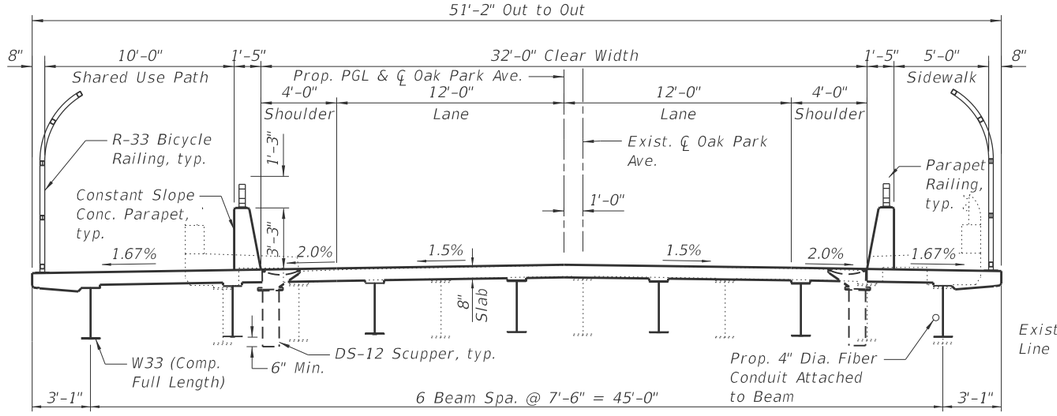
Ardmore Roderick

USER NAME = kingaludzia	DESIGNED - NMB	REVISED -	
	DRAWN - JPK	REVISED -	
	CHECKED - STB	REVISED -	
PLOT DATE = 10/29/2025	DATE - 08/01/2025	REVISED -	

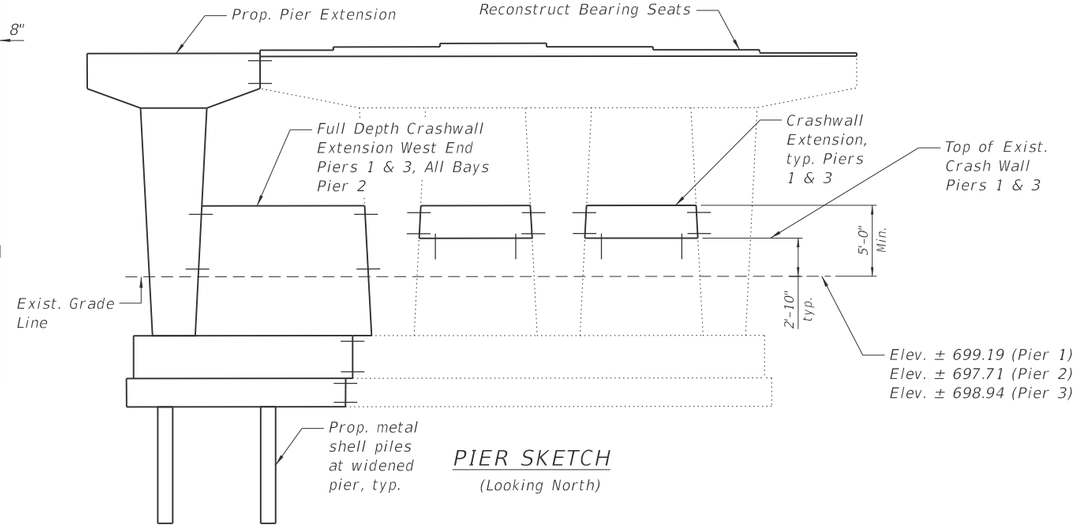
STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

SCALE: | SHEET 1 OF 3 SHEETS | STA. | TO STA.

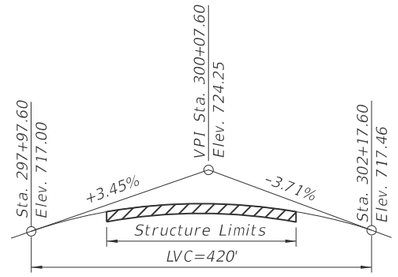
F.A.U RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
2774		COOK		S-01
		CONTRACT NO. 62W20		
ILLINOIS FED. AID PROJECT				



CROSS SECTION
(Looking North)

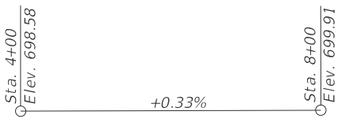


PIER SKETCH
(Looking North)

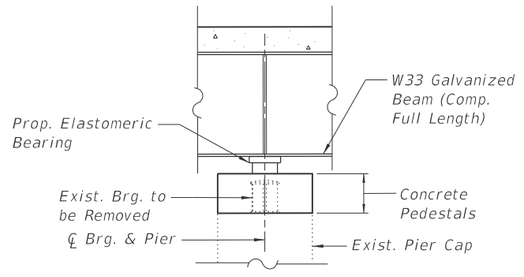


PROFILE GRADE
(Along ℄ Oak Park Ave)

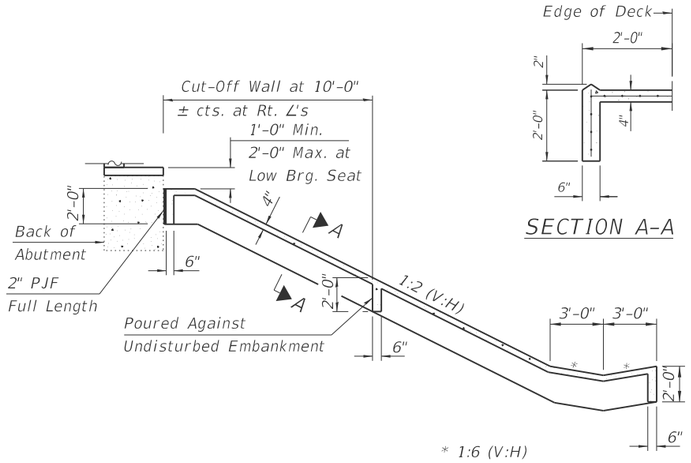
PROFILE GRADE
(Along PGL I-80 East Bound)



PROFILE GRADE
(Along PGL I-80 West Bound)

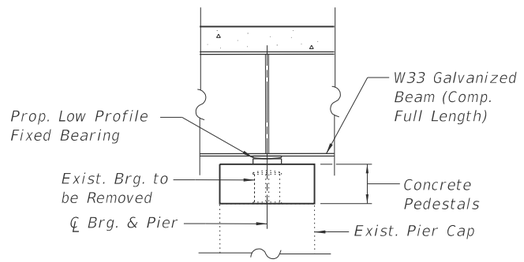


SECTION THROUGH PIERS 1 & 3



SECTION A-A

SECTION THROUGH CONCRETE SLOPEWALL



SECTION THROUGH PIER 2

DETAILS 1 OF 2
OAK PARK AVENUE OVER I-80
FAU 2774-SECTION 1313-811 HB
COOK COUNTY
STATION 300+03.20
STRUCTURE NO: 016-0927

MODEL: TSL Sheet 2[Sheet]
FILE NAME: c:\ww\working\optimera\pwprod\kludzia\dms61750\D162W20_Bridge GPE Sheet 2.dgn

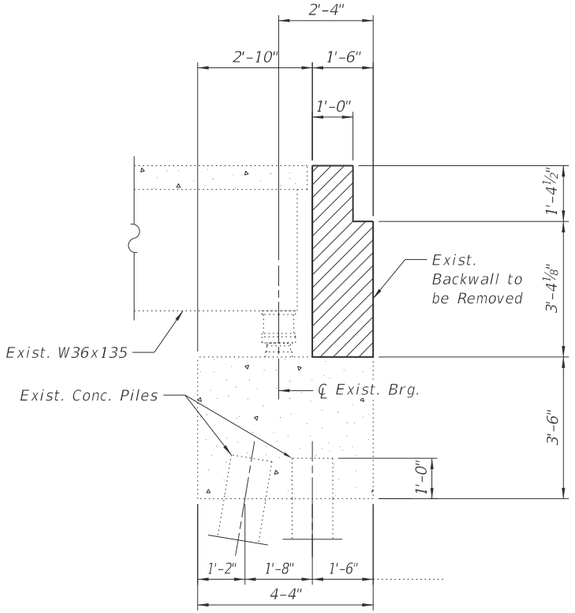
Ardmore Roderick

USER NAME = kingaludzia	DESIGNED - NMB	REVISED -	
	DRAWN - JPK	REVISED -	
	CHECKED - STB	REVISED -	
PLOT DATE = 10/31/2025	DATE - 08/01/2025	REVISED -	

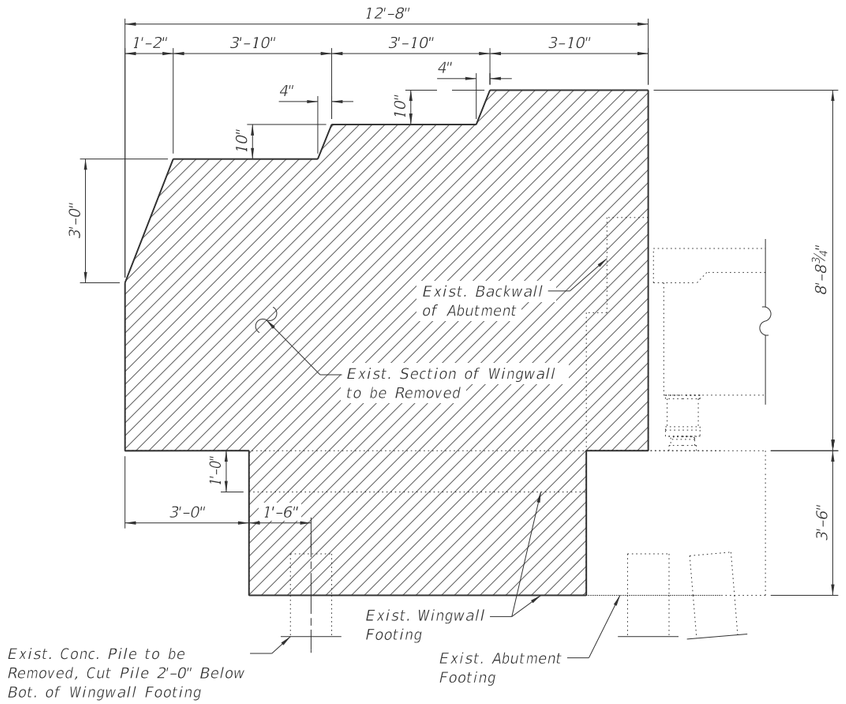
STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

SCALE: | SHEET 2 OF 3 SHEETS | STA. | TO STA.

F.A.U RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
2774		COOK		S-02
		CONTRACT NO. 62W20		
ILLINOIS FED. AID PROJECT				

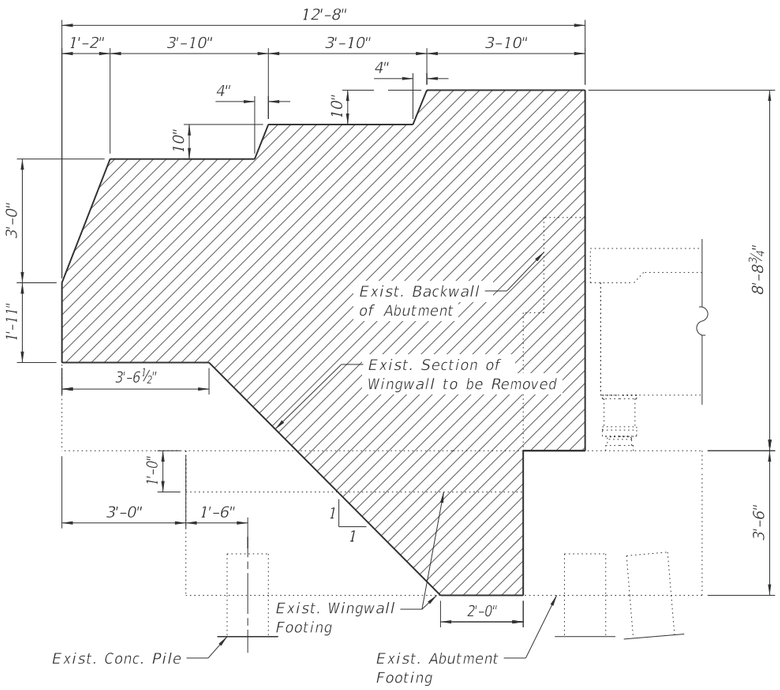


SECTION THRU EXIST. STUB ABUTMENT



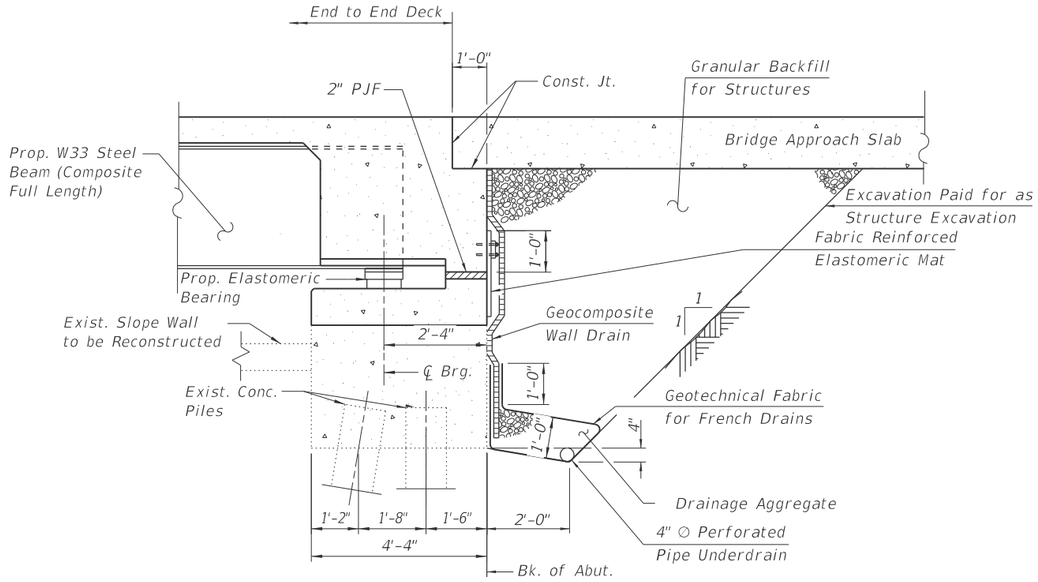
EAST WINGWALL REMOVAL

(Looking East, N.E. Wingwall Shown, S.E. Wingwall Similar)



WEST WINGWALL REMOVAL

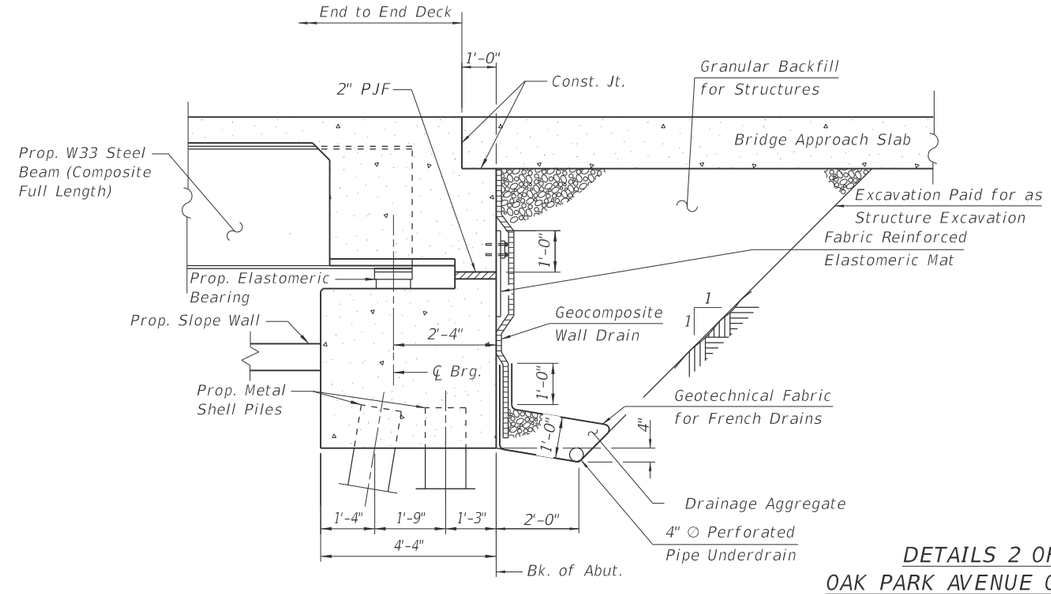
(Looking West, S.W. Wingwall Shown, N.W. Wingwall Similar)



SECTION THRU SEMI-INTEGRAL ABUTMENT

AT EXISTING ABUTMENT SECTION

(Horiz. dim. at Rt. ℄'s)



SECTION THRU SEMI-INTEGRAL ABUTMENT

AT WIDENED ABUTMENT SECTION

(Horiz. dim. at Rt. ℄'s)

DETAILS 2 OF 2
OAK PARK AVENUE OVER I-80
FAU 2774-SECTION 1313-811 HB
COOK COUNTY
STATION 300+03.20
STRUCTURE NO: 016-0927

MODEL: TSL Sheet 3[Sheet]
FILE NAME: c:\ww\work\kingl\pt\mera\pwprod\kludzia\dms617\50\D162W20_Bridge TSL GPE Sheet 3.dgn

Ardmore Roderick

USER NAME = kingaludzia	DESIGNED - NMB	REVISED -	
	DRAWN - KL	REVISED -	
	CHECKED - STB	REVISED -	
PLOT DATE = 8/1/2025	DATE - 08/01/2025	REVISED -	

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

SCALE: | SHEET 3 OF 3 SHEETS | STA. | TO STA.

F.A.U RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
2774		COOK		S-03
		CONTRACT NO. 62W20		
		ILLINOIS FED. AID PROJECT		

APPENDIX B

BORING LOCATION PLAN

AND SUBSURFACE PROFILES

MODEL: Default
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GSG CONSULTANTS, INC.
735 E. REMINGTON RD, SCHAUMBURG, IL 60173
TEL: +1630.994.2600 | WWW.GSG-CONSULTANTS.COM

USER NAME = $USER$	DESIGNED - MH	REVISED -	
	DRAWN - NN	REVISED -	
PLOT SCALE = 160.0000 ' / In.	CHECKED - DE	REVISED -	
PLOT DATE = 2/7/2025	DATE - 01/16/2025	REVISED -	

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

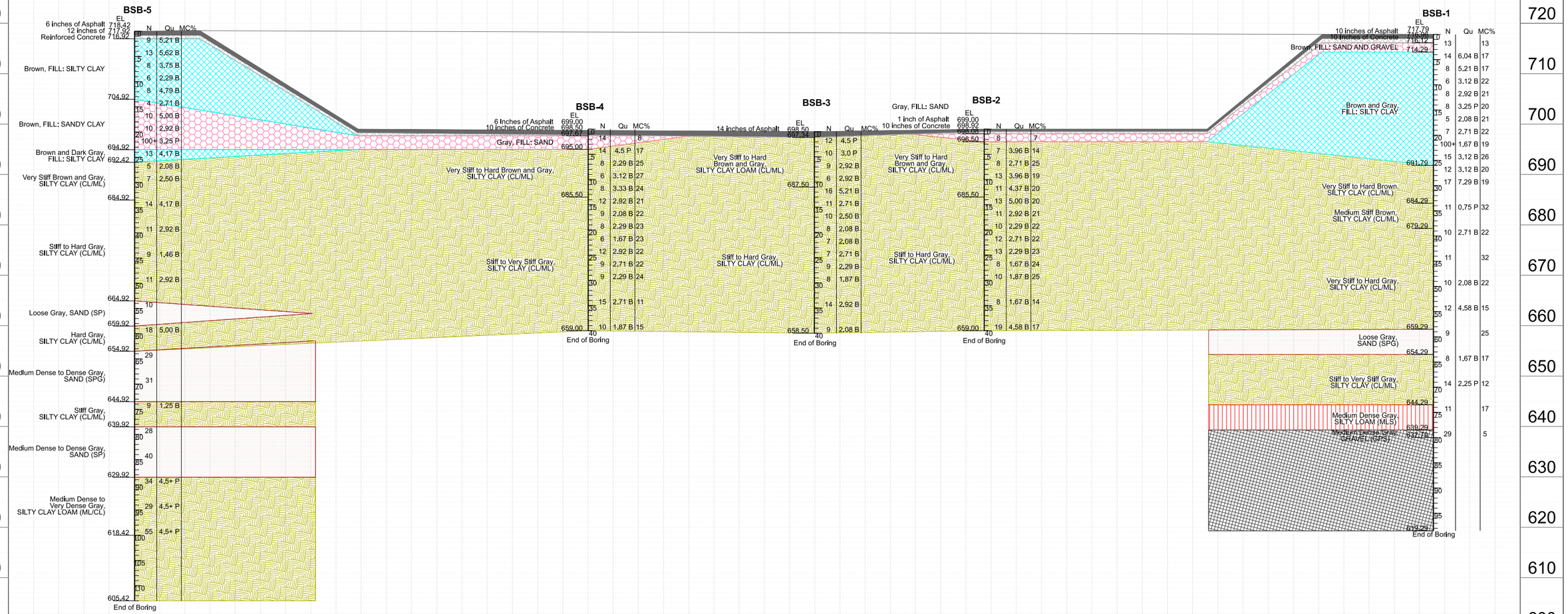
IDOT PTB 205–003
OAK PARK OVER I–80
BORING LOCATION PLAN

SCALE: 1:80 | SHEET 1 OF 1 SHEETS | STA. | TO STA.

F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
		COOK	4	1
		CONTRACT NO. 205-003		
	ILLINOIS	FED. AID PROJECT		

LEGEND

PAVEMENT	FILL: SAND / GRAVEL	SANDY CLAY / LOAM	GRAVEL
BASE COURSE	SILTY CLAY	SILTY SAND / SANDY LOAM	
TOPSOIL	SAND	ORGANIC SILTY CLAY	
FILL: CLAY / SILTY CLAY	SILT / SILTY LOAM	BEDROCK	



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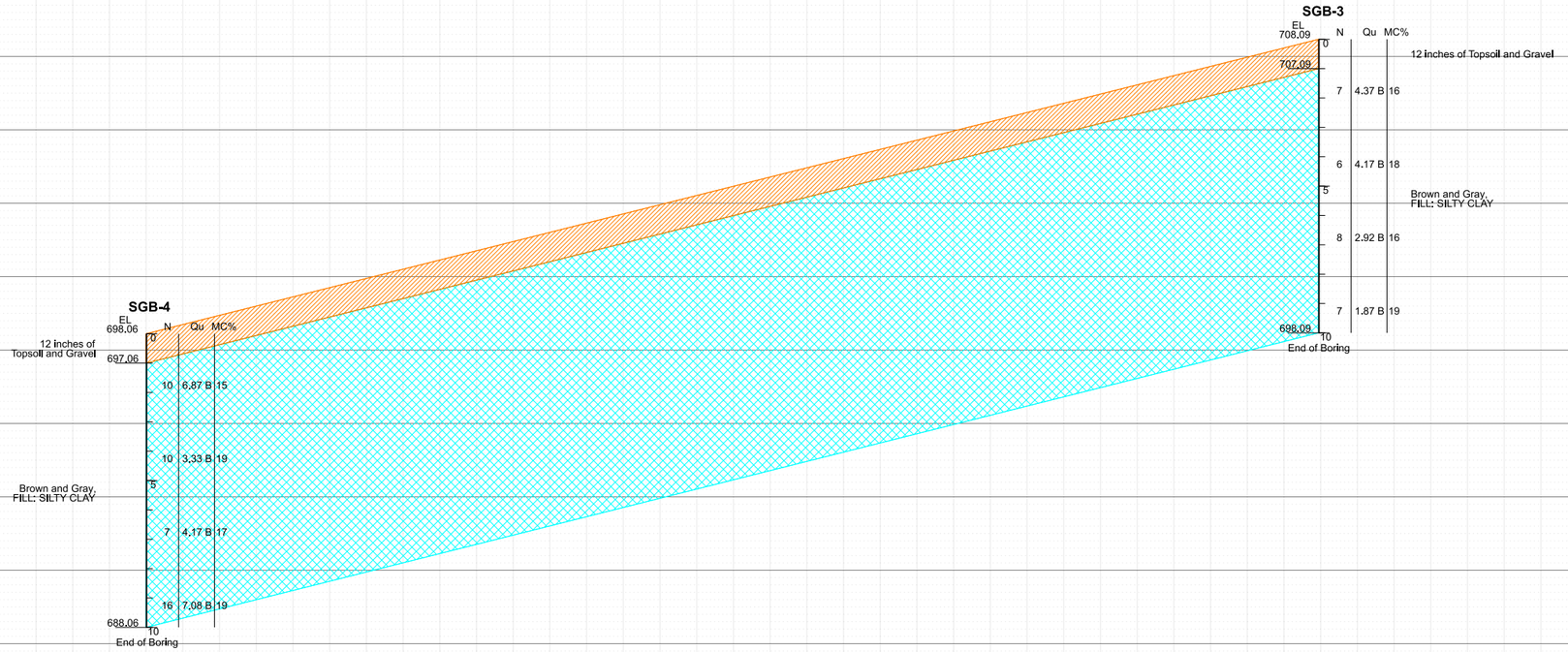
GSG CONSULTANTS, INC.
735 E. REMINGTON RD. SCHAUMBURG, IL 60173
TEL: +1630.994.2600 | WWW.GSG-CONSULTANTS.COM

USER NAME = $USER$	DESIGNED - MH	REVISED -	STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION	IDOT PTB 205–003 OAK PARK OVER I–80 SOIL SUBSURFACE PROFILE	
	DRAWN - NN	REVISED -			
PLOT SCALE = 20.0000 ' / In.	CHECKED - DE	REVISED -			
PLOT DATE = 3/11/2025	DATE - 01/16/2025	REVISED -		SCALE: AS NOTED SHEET 1 OF 3 SHEETS STA. 0+00 TO STA. 275+00	

F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
		COOK	4	2
		CONTRACT NO. 205-003		
	ILLINOIS	FED. AID PROJECT		

LEGEND

PAVEMENT	FILL: SAND / GRAVEL	SANDY CLAY / LOAM	GRAVEL
BASE COURSE	SILTY CLAY	SILTY SAND / SANDY LOAM	
TOPSOIL	SAND	ORGANIC SILTY CLAY	
FILL: CLAY / SILTY CLAY	SILT / SILTY LOAM	BEDROCK	



720 715 710 705 700 695 690 685 680

0+00 25+00 50+00 75+00 100+00 250+00 275+00 300+00 325+00 350+00

MODEL: Default
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GSG CONSULTANTS, INC.
735 E. WASHINGTON RD. SCHAUMBURG, IL 60173
TEL: +1(630).994.2600 | WWW.GSG-CONSULTANTS.COM

USER NAME = $USER$	DESIGNED - MH	REVISED -	
	DRAWN - NN	REVISED -	
PLOT SCALE = 20.0000 ' / In.	CHECKED - DE	REVISED -	
PLOT DATE = 1/22/2025	DATE - 01/16/2025	REVISED -	

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

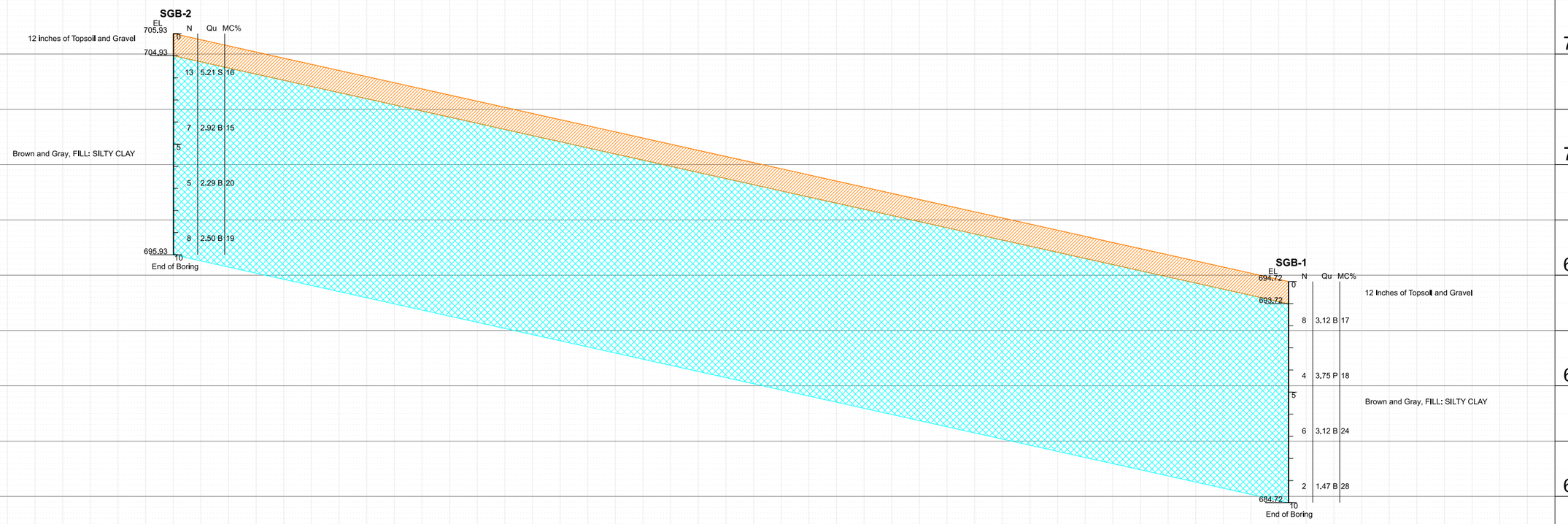
IDOT PTB 207–024
OAK PARK OVER I–80
SOIL SUBSURFACE PROFILE

SCALE: AS NOTED | SHEET 2 OF 3 SHEETS | STA. 0+00 | TO STA. 350+00

F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
		COOK	4	3
		CONTRACT NO. 205-003		
		ILLINOIS	FED. AID PROJECT	

LEGEND

PAVEMENT		FILL: SAND / GRAVEL		SANDY CLAY / LOAM		GRAVEL	
BASE COURSE		SILTY CLAY		SILTY SAND / SANDY LOAM			
TOPSOIL		SAND		ORGANIC SILTY CLAY			
FILL: CLAY / SILTY CLAY		SILT / SILTY LOAM		BEDROCK			



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GSG CONSULTANTS, INC.
735 E. WASHINGTON RD. SCHAUMBURG, IL 60173
TEL: 1630.994.2600 | WWW.GSG-CONSULTANTS.COM

USER NAME = $USER$	DESIGNED - MH	REVISED -	
	DRAWN - NN	REVISED -	
PLOT SCALE = 20.0000 ' / In.	CHECKED - DE	REVISED -	
PLOT DATE = 1/22/2025	DATE - 01/16/2025	REVISED -	

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

IDOT PTB 205–003
OAK PARK OVER I–80
SOIL SUBSURFACE PROFILE

SCALE: AS NOTED | SHEET 3 OF 3 SHEETS | STA. 0+00 | TO STA. 325+00

F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
		COOK	4	4
		CONTRACT NO. 205-003		
		ILLINOIS	FED. AID PROJECT	

APPENDIX C

SOIL BORING LOGS

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 12/3/24

ROUTE Oak Park Avenue **DESCRIPTION** Bridge Abutment Boring **LOGGED BY** TS

SECTION I-80 at Oak Park Avenue **LOCATION** Oak Park Avenue, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.5515147, **Longitude** 87.7835798

COUNTY COOK **DRILLING RIG** Mobile B57 **HAMMER TYPE** AUTO
DRILLING METHOD MUD ROTARY **HAMMER EFF (%)** 92.1

STRUCT. NO. 016-0927
Station 300+00

BORING NO. BSB-1
Station 301+39.39
Offset 15.66ft LT
Ground Surface Elev. 717.79 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter N/A ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)	Description
				10 inches of Asphalt
				10 Inches of Concrete
				716.96
	20			
				716.12
	9		13	Brown, Moist
	4			FILL: SAND AND GRAVEL
				714.29
	3			Brown and Gray, Moist to Very Moist
	6	6.0	17	FILL: SILTY CLAY, trace sand and gravel
-5	8	B		
	4			
	4	5.2	17	
	4	B		
	1			
	3	3.1	22	
-10	3	B		
	3			
	3	2.9	21	
	5	B		Sand seam at 12 feet
	2			
	4	3.3	20	
-15	4	P		
	2			
	2	2.1	21	
	3	B		
	2			
	3	2.7	22	
-20	4	B		
				Brown and Gray, Moist to Very Moist
	2			FILL: SILTY CLAY, trace sand and gravel *(continued)*
	2	1.7	19	2 inches of Asphalt at 22 feet
	50/3"	B		
	4			
	7	3.1	26	
-25	8	B		
				691.79
	5			Very Stiff to Hard
	7	3.1	20	Brown, Moist
	5	B		SILTY CLAY, trace gravel (CL/ML)
	5			
	6	7.3	19	
-30	11	B		
				684.29
	5			Medium Stiff
	4	0.8	32	Brown, Very Moist
-35	7	P		SILTY CLAY, with sand and gravel (CL/ML)
				679.29
	3			
	4	2.7	22	
-40	6	B		

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

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 12/3/24

ROUTE Oak Park Avenue **DESCRIPTION** Bridge Abutment Boring **LOGGED BY** TS

SECTION I-80 at Oak Park Avenue **LOCATION** Oak Park Avenue, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.5515147, **Longitude** 87.7835798

COUNTY COOK **DRILLING RIG** Mobile B57 **HAMMER TYPE** AUTO
DRILLING METHOD MUD ROTARY **HAMMER EFF (%)** 92.1

STRUCT. NO. 016-0927
Station 300+00

BORING NO. BSB-1
Station 301+39.39
Offset 15.66ft LT
Ground Surface Elev. 717.79 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter N/A ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)	Description
				Very Stiff to Hard Gray, Moist to Very Moist SILTY CLAY, trace gravel (CL/ML) *(continued)*
	4			
-45	5 6		32	
	3			
-50	5 5	2.1 B	22	
	7			
-55	6 6	4.6 B	15	
659.29				
	3			Loose Gray, Wet SAND, with gravel and clay (SPG)
-60	3 6		25	
	4			Loose Gray, Wet SAND, with gravel and clay (SPG) *(continued)*
654.29				
	4			Stiff to Very Stiff Gray, Moist SILTY CLAY, trace gravel (CL/ML)
-65	3 5	1.7 B	17	
	9			Sand and gravel seam at 68.5 feet
-70	8 6	2.3 P	12	
644.29				
	8			Medium Dense Gray, Moist SILTY LOAM, with sand, trace gravel (MLS)
-75	3 8		17	
639.29				
	12			Medium Dense Gray, Dry GRAVEL, with sand (GPS)
-80	16 13		5	
637.79				

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

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 12/3/24

ROUTE Oak Park Avenue **DESCRIPTION** Bridge Abutment Boring **LOGGED BY** TS

SECTION I-80 at Oak Park Avenue **LOCATION** Oak Park Avenue, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.5515147, **Longitude** 87.7835798

COUNTY COOK **DRILLING RIG** Mobile B57 **HAMMER TYPE** AUTO
DRILLING METHOD MUD ROTARY **HAMMER EFF (%)** 92.1

STRUCT. NO. 016-0927
Station 300+00

BORING NO. BSB-1
Station 301+39.39
Offset 15.66ft LT
Ground Surface Elev. 717.79 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter N/A ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)	
Blind drilling with no bedrock found				
-85				
-90				
-95				
619.29				
End of Boring				
-100				

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

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 1/7/25

ROUTE Oak Park Avenue **DESCRIPTION** Bridge Pier Boring **LOGGED BY** DV

SECTION I-80 at Oak Park Avenue **LOCATION** Interstate I-80, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.55130833, **Longitude** 87.7836667

COUNTY COOK **DRILLING RIG** Mobile B57 **HAMMER TYPE** AUTO
DRILLING METHOD HSA **HAMMER EFF (%)** 92.1

STRUCT. NO. 016-0927
Station 300+00

BORING NO. BSB-2
Station 300+53.69
Offset 40.19ft LT
Ground Surface Elev. 699.00 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter None ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
1 inch of Asphalt 698.92 10 inches of Concrete 698.08			
Gray, Moist FILL: SAND, with gravel	4		
	3 5		7
696.50			
Very Stiff to Hard Brown and Gray, Moist SILTY CLAY, trace gravel (CL/ML)	2		
	2 5	4.0 B	14
-5			
	2		
	4 4	2.7 B	25
	5		
	5 8	4.0 B	19
-10			
	5		
	5 6	4.4 B	20
685.50			
Stiff to Hard Gray, Moist SILTY CLAY, trace gravel (CL/ML)	4		
	5 8	5.0 B	20
-15			
	4		
	5 6	2.9 B	21
	4		
	4 6	2.3 B	22
-20			

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Stiff to Hard Gray, Moist SILTY CLAY, trace gravel (CL/ML) *(continued)*	5		
	5 7	2.7 B	22
	5		
	6 7	2.3 B	23
-25			
	2		
	4 4	1.7 B	24
	4		
	4 6	1.9 B	25
-30			
	2		
	4 4	1.7 B	14
-35			
	7		
	8 11	4.6 B	17
659.00 -40			

End of Boring

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

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 3/5/25

ROUTE Oak Park Avenue **DESCRIPTION** Bridge Pier Boring **LOGGED BY** DV

SECTION I-80 at Oak Park Avenue **LOCATION** Interstate I-80, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.5511806, **Longitude** 87.7836444

COUNTY COOK **DRILLING RIG** Mobil B57 **HAMMER TYPE** AUTO
DRILLING METHOD HSA **HAMMER EFF (%)** 92.1

STRUCT. NO. 016-0927
Station 300+00

BORING NO. BSB-3
Station 300+18.17
Offset 36.54ft LT
Ground Surface Elev. 698.50 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter None ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
14 inches of Asphalt			
697.34			
Very Stiff to Hard Brown and Gray, Moist SILTY CLAY LOAM, trace gravel (CL/ML)	3		
	4 8	4.5 P	9
	4		
-5	4 6	3.0 P	13
	2		
	4 5	2.9 B	20
	3		
-10	3 3	2.9 B	24
687.50			
Stiff to Hard Gray, Moist SILTY CLAY, trace gravel (CL/ML)	4		
	8 8	5.2 B	19
	3		
-15	5 6	2.7 B	20
	3		
	4 6	2.5 B	21
	2		
-20	4 4	2.1 B	22
Stiff to Hard Gray, Moist SILTY CLAY, trace gravel (CL/ML) *(continued)*	2		
	3 4	2.1 B	24
	2		
-25	3 4	2.7 B	22
	1		
	4 5	2.3 B	22
	3		
-30	4 4	1.9 B	24
	4		
-35	7 7	2.9 B	13
	3		
658.50 -40	4 5	2.1 B	15

End of Boring

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

BBS, form 137 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 1/6/25

ROUTE Oak Park Avenue **DESCRIPTION** Bridge Pier Boring **LOGGED BY** DV

SECTION I-80 at Oak Park Avenue **LOCATION** Interstate I-80, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.5509417, **Longitude** 87.7836222

COUNTY COOK **DRILLING RIG** Mobile B57 **HAMMER TYPE** AUTO
DRILLING METHOD HSA **HAMMER EFF (%)** 92.1

STRUCT. NO. 016-0927
Station 300+00

BORING NO. BSB-4
Station 299+30.98
Offset 32.76ft LT
Ground Surface Elev. 699.00 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter None ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)	Description
698.50				6 inches of Asphalt 10 inches of Concrete
697.67	20			Gray, Moist to Wet FILL: SAND, with gravel
	10 4		8	
695.00	4			Very Stiff to Hard Brown and Gray, Moist to Very Moist SILTY CLAY, trace gravel (CL/ML)
-5	7 7	4.5 P	17	
	2			
	3 5	2.3 B	25	
	3			
-10	3 3	3.1 B	27	
	3			
	3 5	3.3 B	24	
685.50	4			Stiff to Very Stiff Gray, Moist SILTY CLAY, trace gravel (CL/ML)
-15	5 7	2.9 B	21	
	3			
	3 6	2.1 B	22	
	1			
-20	3 5	2.3 B	23	

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)	Description
	2			Stiff to Very Stiff Gray, Moist SILTY CLAY, trace gravel (CL/ML) *(continued)*
	3 3	1.7 B	23	
	4			
-25	5 7	2.9 B	22	
	2			
	4 5	2.7 B	22	
	3			
-30	4 5	2.3 B	24	
	5			
-35	7 8	2.7 B	11	
	4			
659.00 -40	4 6	1.9 B	15	

End of Boring

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

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 12/30/25

ROUTE Oak Park Avenue **DESCRIPTION** Bridge Abutment Boring **LOGGED BY** TS

SECTION I-80 at Oak Park Avenue **LOCATION** Oak Park Avenue, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.5508052, **Longitude** 87.7834389

COUNTY COOK **DRILLING RIG** Diedrich D-50 **HAMMER TYPE** AUTO
DRILLING METHOD MUD ROTARY **HAMMER EFF (%)** 97.7

STRUCT. NO. 016-0927
Station 300+00

BORING NO. BSB-5
Station 298+36.75
Offset 22.45ft LT
Ground Surface Elev. 718.42 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter N/A ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)	Description
				6 inches of Asphalt
				12 inches of Reinforced Concrete
717.92				
	5			
716.92				
	4	5.2	17	Brown, Moist
	5	B		FILL: SILTY CLAY, trace sand and gravel
	5			
	6	5.6	17	
-5	7	B		
	3			
	3	3.8	19	
	5	B		
	2			
	2	2.3	22	
-10	4	B		
	3			
	4	4.8	14	
	4	B		
704.92				
	1			Brown, Moist to Very Moist
	2	2.7	27	FILL: SANDY CLAY, trace gravel
-15	2	B		
	3			
	5	5.0	25	
	5	B		
	3			
	5	2.9	22	
-20	5	B		

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)	Description
				Brown, Moist to Very Moist
				FILL: SANDY CLAY, trace gravel
				(continued)
	50/6"			
		3.3	18	Asphalt at 21.5 feet
		P		
694.92				
	4			Brown and Dark Gray, Very Moist
	7	4.2	31	FILL: SILTY CLAY, trace gravel
-25	6	B		
692.42				
	1			Very Stiff
	2	2.1	31	Brown and Gray, Very Moist
	3	B		SILTY CLAY, trace gravel
				(CL/ML)
	5			
	4	2.5	25	
-30	3	B		
684.92				
	4			Stiff to Hard
	6	4.2	22	Gray, Moist
-35	8	B		SILTY CLAY, trace gravel
				(CL/ML)
	3			
	4	2.9	25	
-40	7	B		

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

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 12/30/25

ROUTE Oak Park Avenue **DESCRIPTION** Bridge Abutment Boring **LOGGED BY** TS

SECTION I-80 at Oak Park Avenue **LOCATION** Oak Park Avenue, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.5508052, **Longitude** 87.7834389

COUNTY COOK **DRILLING RIG** Diedrich D-50 **HAMMER TYPE** AUTO
DRILLING METHOD MUD ROTARY **HAMMER EFF (%)** 97.7

STRUCT. NO. 016-0927
Station 300+00

BORING NO. BSB-5
Station 298+36.75
Offset 22.45ft LT
Ground Surface Elev. 718.42 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter N/A ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)	Description
				Stiff to Hard Gray, Moist SILTY CLAY, trace gravel (CL/ML) *(continued)*
	3			
-45	4 5	1.5 B	25	
	4			
-50	5 6	2.9 B	25	
664.92				
	4			Loose Gray, Wet SAND, trace gravel and clay seams (SP)
-55	4 6		20	
659.92				
	5			
-60	7 11	5.0 B	18	
				Hard Gray, Moist SILTY CLAY, trace gravel (CL/ML) *(continued)*
654.92				
	10			Medium Dense to Dense Gray, Moist to Wet SAND, with gravel, trace clay seams (SPG)
-65	13 16		17	
	7			
-70	15 16		7	
644.92				
	4			Stiff Gray, Moist SILTY CLAY, with sand, trace gravel (CL/ML)
-75	3 6	1.3 B	14	
639.92				
	10			Medium Dense to Dense Gray, Moist to Wet SAND, trace gravel (SP)
-80	12 16		12	

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

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 12/30/25

ROUTE Oak Park Avenue **DESCRIPTION** Bridge Abutment Boring **LOGGED BY** TS

SECTION I-80 at Oak Park Avenue **LOCATION** Oak Park Avenue, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.5508052, **Longitude** 87.7834389

COUNTY COOK **DRILLING RIG** Diedrich D-50 **HAMMER TYPE** AUTO
DRILLING METHOD MUD ROTARY **HAMMER EFF (%)** 97.7

STRUCT. NO. 016-0927
Station 300+00

BORING NO. BSB-5
Station 298+36.75
Offset 22.45ft LT
Ground Surface Elev. 718.42 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter N/A ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)	Description
				Medium Dense to Dense Gray, Moist to Wet SAND, trace gravel (SP) *(continued)*
	13			
-85	17 23		20	
629.92				
	24			Medium Dense to Very Dense Gray, Moist SILTY CLAY LOAM (ML/CL)
-90	18 16	4.5 P	5	
	15			
-95	10 19	4.5 P	11	
	20			
618.42 -100	25 30	4.5 P	11	

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)	Description
				Blind drilling with no bedrock found
-105				
-110				
605.42				End of Boring
-115				
-120				

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

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 12/2/24

ROUTE Oak Park Avenue **DESCRIPTION** Subgrade Boring **LOGGED BY** TS

SECTION I-80 at Oak Park Avenue **LOCATION** Oak Park Avenue, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.5532532, **Longitude** 87.7834859

COUNTY COOK **DRILLING RIG** Mobile B57 **HAMMER TYPE** AUTO
DRILLING METHOD HSA **HAMMER EFF (%)** 92.1

STRUCT. NO. N/A
Station N/A

BORING NO. SGB-1
Station 307+72.02
Offset 25.54ft RT
Ground Surface Elev. 694.72 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter None ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
12 inches of Topsoil and Gravel 693.72			
Brown and Gray, Moist to Very Moist FILL: SILTY CLAY, trace sand and gravel	5		
	5 3	3.1 B	17
Cobbles at 3.5 feet	2		
-5	2 2	3.8 P	18
	2		
	2 4	3.1 B	24
	1		
684.72 -10	1 1	1.5 B	28
End of Boring			
-15			
-20			

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

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 12/2/24

ROUTE Oak Park Avenue **DESCRIPTION** Subgrade Boring **LOGGED BY** TS

SECTION I-80 at Oak Park Avenue **LOCATION** Oak Park Avenue, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.5524251, **Longitude** 87.7836332

COUNTY COOK **DRILLING RIG** Mobile B57 **HAMMER TYPE** AUTO
DRILLING METHOD HSA **HAMMER EFF (%)** 92.1

STRUCT. NO. N/A
Station N/A

BORING NO. SGB-2
Station 304+71.40
Offset 22.65ft LT
Ground Surface Elev. 705.93 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter None ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)	
12 inches of Topsoil and Gravel 704.93				
Brown and Gray, Moist FILL: SILTY CLAY, trace sand and gravel	5			
	6 7	5.2 S	16	
	4			
-5	4 3	2.9 B	15	
	2			
	3 2	2.3 B	20	
	2			
695.93 -10	3 5	2.5 B	19	
End of Boring				
-15				
-20				

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

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 12/2/24

ROUTE Oak Park Avenue **DESCRIPTION** Subgrade Boring **LOGGED BY** TS

SECTION I-80 at Oak Park Avenue **LOCATION** Oak Park Avenue, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.5500581, **Longitude** 87.7835557

COUNTY COOK **DRILLING RIG** Mobile B57 **HAMMER TYPE** AUTO
DRILLING METHOD HSA **HAMMER EFF (%)** 92.1

STRUCT. NO. N/A
Station N/A

BORING NO. SGB-3
Station 296+08.63
Offset 24.07ft LT
Ground Surface Elev. 708.09 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter None ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
12 inches of Topsoil and Gravel 707.09			
Brown and Gray, Moist FILL: SILTY CLAY, trace sand and gravel	4		
	3 4	4.4 B	16
	3		
-5	3 3	4.2 B	18
	3		
	3 5	2.9 B	16
	3		
698.09 -10	4 3	1.9 B	19
End of Boring			
-15			
-20			

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

Illinois Department of Transportation
Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 12/2/24

ROUTE Oak Park Avenue **DESCRIPTION** Subgrade Boring **LOGGED BY** TS

SECTION I-80 at Oak Park Avenue **LOCATION** Oak Park Avenue, **SEC.** 6, **TWP.** 35N, **RNG.** 13E,
Latitude 41.5492222, **Longitude** 87.7833509

COUNTY COOK **DRILLING RIG** Mobile B57 **HAMMER TYPE** AUTO
DRILLING METHOD HSA **HAMMER EFF (%)** 92.1

STRUCT. NO. N/A
Station N/A

BORING NO. SGB-4
Station 293+02.67
Offset 23.90ft RT
Ground Surface Elev. 698.06 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter None ft
Upon Completion N/A ft
After N/A **Hrs.** N/A ft

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
12 inches of Topsoil and Gravel			
697.06			
Brown and Gray, Moist	5		
FILL: SILTY CLAY, trace gravel	5	6.9	15
	5	B	
	3		
	5	3.3	19
-5	5	B	
	3		
Sand seam at 6.5 feet	3	4.2	17
	4	B	
	4		
	6	7.1	19
688.06 -10	10	B	
End of Boring			
-15			
-20			

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

APPENDIX D

IDOT PILE DESIGN TABLES

Pile Design Table for North Abutment utilizing Boring #BSB-1

Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Metal Shell 12"Φ w/.25" walls		
310	170	43
336	185	47
361	198	48
Metal Shell 14"Φ w/.25" walls		
297	163	33
330	182	38
365	201	43
396	218	47
428	236	48
Metal Shell 14"Φ w/.312" walls		
297	163	33
330	182	38
365	201	43
396	218	47
428	236	48
474	260	58
511	281	63
540	297	67
Metal Shell 16"Φ w/.312" walls		
296	163	28
344	189	33
381	210	38
421	231	43
456	251	47
498	274	48
544	299	58
588	323	63
621	342	67
Metal Shell 16"Φ w/.375" walls		
296	163	28
344	189	33
381	210	38
421	231	43
456	251	47
498	274	48
544	299	58
588	323	63
621	342	67
667	367	68
Steel HP 8 X 36		
273	150	73

Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 10 X 42		
305	168	68
Steel HP 10 X 57		
312	171	68
351	193	73
Steel HP 12 X 53		
322	177	53
328	180	58
357	196	63
371	204	68
Steel HP 12 X 63		
325	179	53
331	182	58
360	198	63
374	206	68
429	236	73
Steel HP 12 X 74		
292	161	47
329	181	53
335	184	58
365	201	63
379	208	68
435	239	73

Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 12 X 84		
296	163	47
334	184	53
340	187	58
370	203	63
384	211	68
442	243	73
Steel HP 14 X 73		
319	176	43
345	190	47
387	213	53
393	216	58
430	236	63
443	244	68
519	285	73
Steel HP 14 X 89		
323	178	43
349	192	47
391	215	53
398	219	58
435	239	63
449	247	68
525	289	73
Steel HP 14 X 102		
327	180	43
353	194	47
395	218	53
403	221	58
440	242	63
454	249	68
532	293	73
Steel HP 14 X 117		
302	166	38
331	182	43
357	196	47
400	220	53
407	224	58
445	245	63
459	252	68
539	296	73
Precast 14"x 14"		
257	142	20

Pile Design Table for Pier 3 utilizing Boring #BSB-2

Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Metal Shell 12"Φ w/.25" walls		
219	121	27
Metal Shell 14"Φ w/.25" walls		
263	145	27
Metal Shell 14"Φ w/.312" walls		
263	145	27
Metal Shell 16"Φ w/.312" walls		
231	127	22
310	170	27
Metal Shell 16"Φ w/.375" walls		
231	127	22
310	170	27
Steel HP 8 X 36		
128	70	27

Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 10 X 42		
164	90	27
Steel HP 10 X 57		
169	93	27
Steel HP 12 X 53		
208	115	27
Steel HP 12 X 63		
211	116	27
Steel HP 12 X 74		
214	118	27

Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 12 X 84		
217	120	27
Steel HP 14 X 73		
261	143	27
Steel HP 14 X 89		
264	145	27
Steel HP 14 X 102		
268	147	27
Steel HP 14 X 117		
272	150	27
Precast 14"x 14"		
254	140	22

Pile Design Table for Middle Pier utilizing Boring #BSB-3

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Metal Shell 12"Φ w/.25" walls	181	99	25
Metal Shell 14"Φ w/.25" walls	214	118	25
Metal Shell 14"Φ w/.312" walls	214	118	25
Metal Shell 16"Φ w/.312" walls	249	137	25
Metal Shell 16"Φ w/.375" walls	249	137	25
Steel HP 8 X 36	102	56	25

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 10 X 42	129	71	25
Steel HP 10 X 57	132	73	25
Steel HP 12 X 53	160	88	25
Steel HP 12 X 63	162	89	25
Steel HP 12 X 74	164	90	25

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 12 X 84	166	92	25
Steel HP 14 X 73	196	108	25
Steel HP 14 X 89	198	109	25
Steel HP 14 X 102	201	111	25
Steel HP 14 X 117	204	112	25
Precast 14"x 14"	230	127	20

Pile Design Table for Pier 1 utilizing Boring #BSB-4

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Metal Shell 12"Φ w/.25" walls	192	106	27
Metal Shell 14"Φ w/.25" walls	228	125	27
Metal Shell 14"Φ w/.312" walls	228	125	27
Metal Shell 16"Φ w/.312" walls	264	145	27
Metal Shell 16"Φ w/.375" walls	264	145	27
Steel HP 8 X 36	108	60	27

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 10 X 42	136	75	27
Steel HP 10 X 57	139	77	27
Steel HP 12 X 53	168	92	27
Steel HP 12 X 63	170	93	27
Steel HP 12 X 74	172	95	27

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 12 X 84	175	96	27
Steel HP 14 X 73	205	113	27
Steel HP 14 X 89	207	114	27
Steel HP 14 X 102	210	116	27
Steel HP 14 X 117	213	117	27
Precast 14"x 14"	250	137	22

Pile Design Table for North Abutment utilizing Boring #BSB-5

Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Metal Shell 12"Φ w/.25" walls		
240	132	27
277	152	32
302	166	37
337	186	42
370	204	46
Metal Shell 14"Φ w/.25" walls		
228	126	22
287	158	27
328	180	32
354	195	37
398	219	42
437	240	46
Metal Shell 14"Φ w/.312" walls		
228	126	22
287	158	27
328	180	32
354	195	37
398	219	42
437	240	46
475	261	47
483	266	52
531	292	56
Metal Shell 16"Φ w/.312" walls		
266	146	22
336	185	27
380	209	32
408	224	37
461	254	42
505	278	46
553	304	47
561	309	52
616	339	56
Metal Shell 16"Φ w/.375" walls		
266	146	22
336	185	27
380	209	32
408	224	37
461	254	42
505	278	46
553	304	47
561	309	52
616	339	56
Steel HP 8 X 36		
248	136	67
283	156	72

Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 10 X 42		
251	138	47
285	157	52
305	168	67
Steel HP 10 X 57		
257	141	47
292	160	52
312	171	67
363	200	72
386	212	82
392	216	87
Steel HP 12 X 53		
254	139	37
292	161	42
306	168	47
354	195	52
370	204	67
Steel HP 12 X 63		
256	141	37
295	162	42
309	170	47
358	197	52
373	205	67
442	243	72
469	258	82
476	262	87
Steel HP 12 X 74		
259	143	37
299	164	42
313	172	47
363	200	52
378	208	67
449	247	72
476	262	82
483	266	87
566	311	92

Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 12 X 84		
263	145	37
303	167	42
317	174	47
368	203	52
383	211	67
455	250	72
482	265	82
490	269	87
574	316	92
Steel HP 14 X 73		
211	116	22
276	152	27
297	163	32
305	168	37
355	195	42
368	202	47
435	239	52
443	243	67
534	294	72
Steel HP 14 X 89		
214	118	22
280	154	27
301	165	32
309	170	37
359	197	42
372	205	47
440	242	52
448	246	67
541	297	72
571	314	82
579	319	87
685	377	92
Steel HP 14 X 102		
217	119	22
283	156	27
304	167	32
312	172	37
363	200	42
377	207	47
446	245	52
453	249	67
548	301	72
578	318	82
586	322	87
693	381	92
Steel HP 14 X 117		
220	121	22
287	158	27
308	170	32
316	174	37
368	202	42
381	210	47
452	248	52
458	252	67
554	305	72
585	322	82
593	326	87
702	386	92
Precast 14"x 14"		
263	144	19

APPENDIX E

SOIL PARAMETER

TABLES

Table E1: Summary of Soil and Rock Parameters – North Abutment (Boring BSB-1)

Depth / Elevation Range (feet)	Soil Description	In situ Unit Weight γ (pcf)	Undrained		Drained		Active Earth Pressure Coefficient (K_a)	Passive Earth Pressure Coefficient (K_p)	At Rest Earth Pressure Coefficient (K_o)	Lateral Modulus of Subgrade Reaction (pci)	Soil Strain (ε_{50})	OSHA Soil Type
			Cohesion c (psf)	Friction Angle ϕ (°)	Cohesion c (psf)	Friction Angle ϕ (°)						
	New Engineered Clay Fill*	125	1,000	0	100	28	0.36	2.77	0.53	1,000	0.005	Type B
	New Engineered Granular Fill*	120	0	32	0	32	0.31	3.25	0.47	90	N/A	Type C
1.5 – 3.5 (716.3 - 714.3)	Fill: Brown Sand and Gravel	125	0	32	0	32	0.31	3.25	0.47	90	N/A	Type C
3.5 – 26 (714.3 - 691.8)	Fill: Brown and Gray Silty Clay	138	3,650	0	365	26	0.39	2.56	0.56	1,000	0.005	Type A
26 – 38.5 (691.8 - 679.3)	Brown and Gray Medium Stiff to Hard Silty Clay	138	3,150	0	315	28	0.36	2.77	0.53	1,000	0.005	Type A
38.5 – 58.5 (679.3 - 659.3)	Gray Very Stiff to Hard Silty Clay	138	2,550	0	255	28	0.36	2.77	0.53	1,000	0.005	Type A
58.5 – 63.5 (659.3 - 654.3)	Gray, Loose Sand with Gravel	117	0	30	0	30	0.33	3.00	0.50	20	N/A	Type C
63.5 – 73.5 (654.3 - 644.3)	Gray Stiff to Very Stiff Silty Clay	138	2,000	0	200	28	0.36	2.77	0.53	1,000	0.005	Type A
73.5 – 78.5 (644.3 - 639.3)	Gray, Medium Dense Silty Loam	119	0	31	0	31	0.32	3.12	0.48	60	N/A	Type B
78.5 – 80 (639.3 - 637.8)	Gray, Medium Dense Gravel with Sand	130	0	38	0	38	0.24	4.20	0.38	60	N/A	Type C

*Assumes material placed in accordance with IDOT SSRBC

Table E2: Summary of Soil Parameters – Piers (Borings BSB-2 to BSB-4)

Depth / Elevation Range (feet)	Soil Description	In situ Unit Weight γ (pcf)	Undrained		Drained		Active Earth Pressure Coefficient (K_a)	Passive Earth Pressure Coefficient (K_p)	At Rest Earth Pressure Coefficient (K_o)	Lateral Modulus of Subgrade Reaction (pci)	Soil Strain (ε_{50})	OSHA Soil Type
			Cohesion c (psf)	Friction Angle ϕ (°)	Cohesion c (psf)	Friction Angle ϕ (°)						
	New Engineered Clay Fill*	125	1,000	0	100	28	0.36	2.77	0.53	1,000	0.005	Type B
	New Engineered Granular Fill*	120	0	32	0	32	0.31	3.25	0.47	90	N/A	Type C
1 – 2.5 (698.0 – 696.5)	Fill: Gray Sand with Gravel	124	0	37	0	37	0.25	4.02	0.40	90	N/A	Type C
2.5 – 13 (696.5 – 686.0)	Brown and Gray Very Stiff to Hard Silty Clay	138	3,400	0	340	28	0.36	2.77	0.53	1,000	0.005	Type A
13 – 40 (686.0 – 659.0)	Gray Stiff to Hard Silty Clay	138	2,600	0	260	28	0.36	2.77	0.53	1,000	0.005	Type A

*Assumes material placed in accordance with IDOT SSRBC

Table E3: Summary of Soil and Rock Parameters – South Abutment (Boring BSB-5)

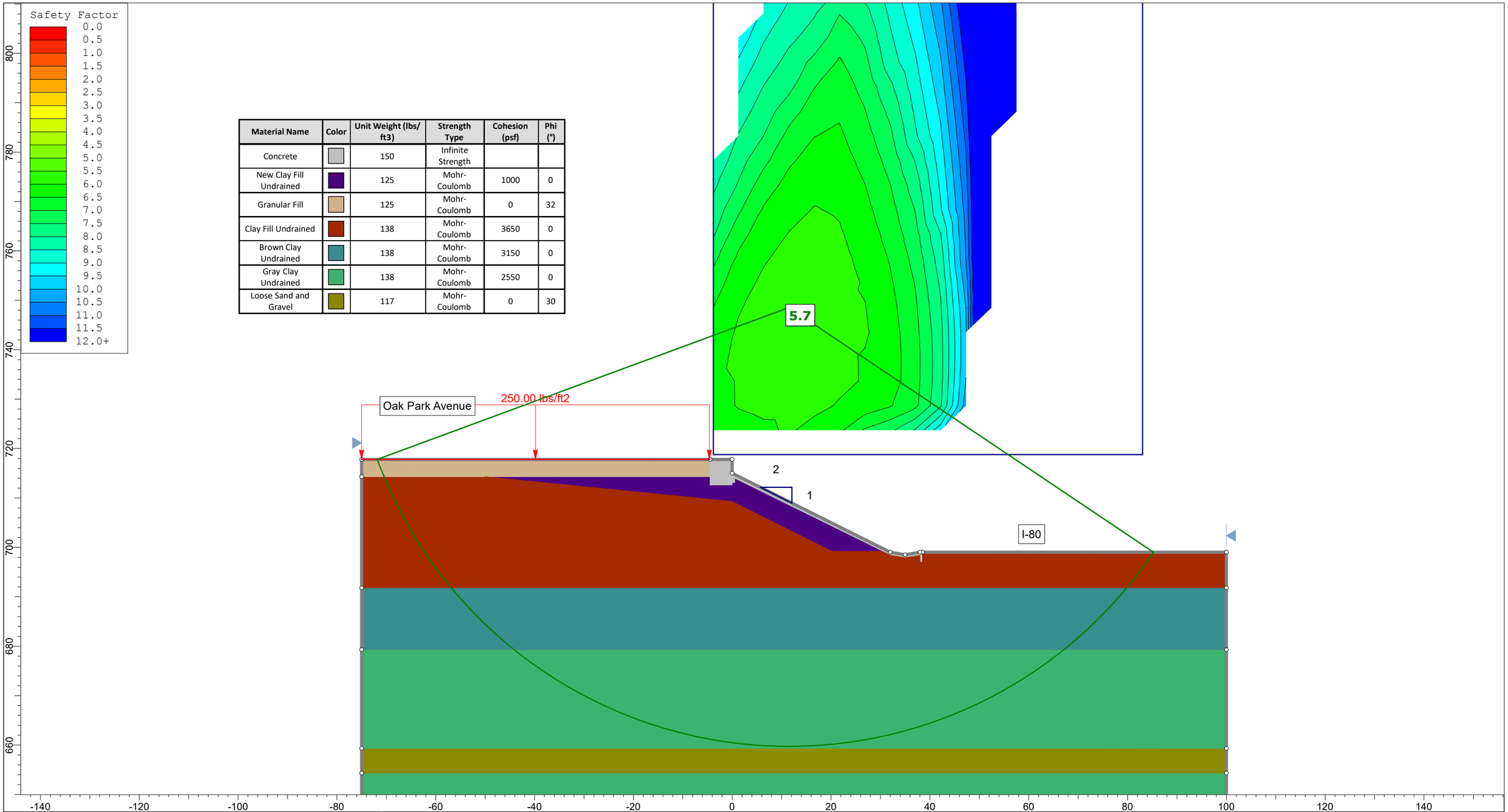
Depth / Elevation Range (feet)	Soil Description	In situ Unit Weight γ (pcf)	Undrained		Drained		Active Earth Pressure Coefficient (K_a)	Passive Earth Pressure Coefficient (K_p)	At Rest Earth Pressure Coefficient (K_o)	Lateral Modulus of Subgrade Reaction (pci)	Soil Strain (ε_{50})	OSHA Soil Type
			Cohesion c (psf)	Friction Angle ϕ (°)	Cohesion c (psf)	Friction Angle ϕ (°)						
	New Engineered Clay Fill*	125	1,000	0	100	28	0.36	2.77	0.53	1,000	0.005	Type B
	New Engineered Granular Fill*	120	0	32	0	32	0.31	3.25	0.47	90	N/A	Type C
1.5 – 26 (716.9 - 692.4)	Fill: Brown and Gray Silty Clay / Sandy Clay	138	3,650	0	365	26	0.39	2.56	0.56	1,000	0.005	Type A
26 – 33.5 (692.4 - 684.9)	Brown and Gray Very Stiff, Silty Clay	138	2,300	0	230	28	0.36	2.77	0.53	1,000	0.005	Type A
33.5 – 53.5 (684.9 - 664.9)	Gray, Stiff to Hard Silty Clay	138	2,550	0	255	28	0.36	2.77	0.53	1,000	0.005	Type A
53.5 – 58.5 (664.9 - 659.9)	Gray, Loose Sand	118	0	31	0	31	0.32	3.12	0.48	60	N/A	Type C
58.5 – 63.5 (659.9 - 654.9)	Gray, Hard Silty Clay	138	5,000	0	500	28	0.36	2.77	0.53	2,000	0.004	Type A
63.5 – 73.5 (654.9 - 644.9)	Gray, Medium Dense to Dense Sand with Gravel	131	0	40	0	40	0.22	4.60	0.36	60	N/A	Type C
73.5 – 78.5 (644.9 - 639.9)	Gray, Stiff Silty Clay	133	1,300	0	130	28	0.36	2.77	0.53	500	0.007	Type B
78.5 – 88.5 (639.9 - 629.9)	Gray Medium Dense to Dense, Sand	132	0	40	0	40	0.22	4.60	0.36	125	N/A	Type C
88.5 – 100 (629.9 - 618.4)	Gray, Medium Dense to Very Dense Silty Clay Loam	134	0	42	0	42	0.20	5.04	0.33	125	N/A	Type B

*Assumes material placed in accordance with IDOT SSRBC

APPENDIX F

SLOPE STABILITY

ANALYSIS EXHIBITS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Concrete		150	Infinite Strength		
New Clay Fill Undrained		125	Mohr-Coulomb	1000	0
Granular Fill		125	Mohr-Coulomb	0	32
Clay Fill Undrained		138	Mohr-Coulomb	3650	0
Brown Clay Undrained		138	Mohr-Coulomb	3150	0
Gray Clay Undrained		138	Mohr-Coulomb	2550	0
Loose Sand and Gravel		117	Mohr-Coulomb	0	30

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SLIDEINTERPRET 9.034

Project	PTB 205-003 Work Order 5 - North Abutment		
Analysis Description	Exhibit 1 - Circular Failure Short Term		
Drawn By	MH	*Company*	GSG Consultants, Inc.
Date	1/20/2025, 3:37:55 PM	*File Name*	North Abutment.slmd

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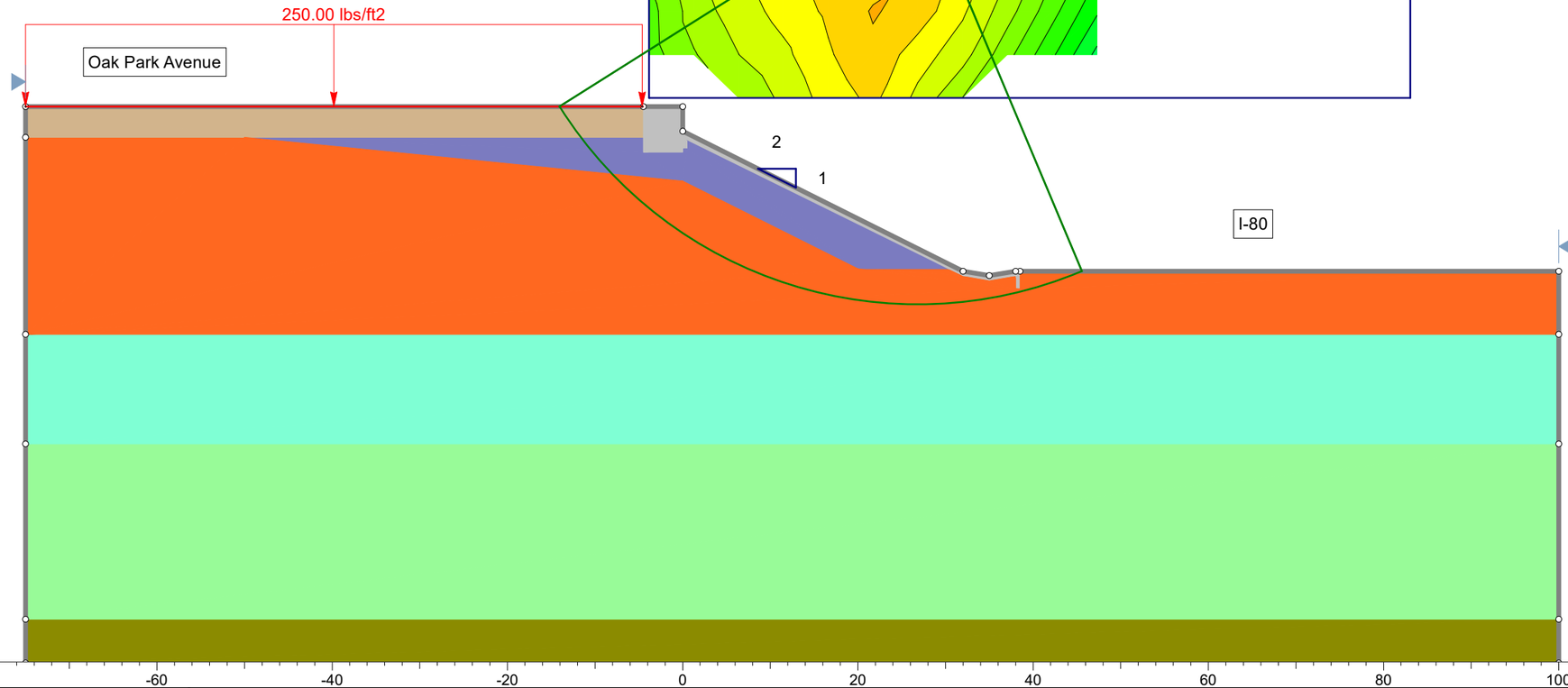
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
7.0
7.5
8.0
8.5
9.0
9.5
10.0
10.5
11.0
11.5
12.0+

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Concrete		150	Infinite Strength		
New Clay Fill Drained		125	Mohr-Coulomb	100	28
Granular Fill		125	Mohr-Coulomb	0	32
Clay Fill Drained		138	Mohr-Coulomb	365	26
Brown Clay Drained		138	Mohr-Coulomb	315	28
Gray Clay Drained		138	Mohr-Coulomb	255	28
Loose Sand and Gravel		117	Mohr-Coulomb	0	30



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Project	PTB 205-003 Work Order 5 - North Abutment		
Analysis Description	Exhibit 2 - Circular Failure Long Term		
Drawn By	MH	Company	GSG Consultants, Inc.
Date	1/20/2025, 3:37:55 PM	File Name	North Abutment.slmd

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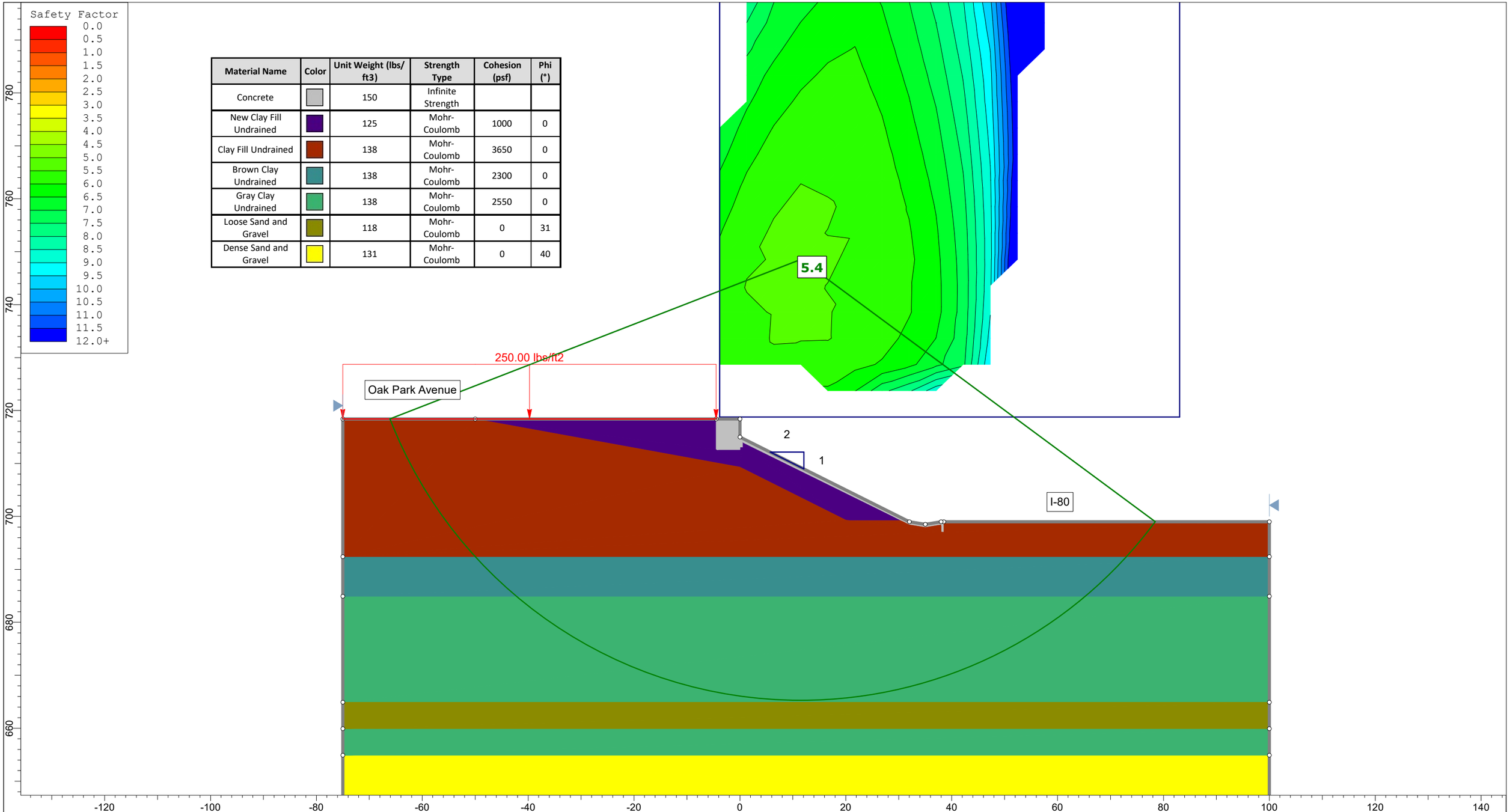

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Concrete		150	Infinite Strength		
New Clay Fill Undrained		125	Mohr-Coulomb	1000	0
Clay Fill Undrained		138	Mohr-Coulomb	3650	0
Brown Clay Undrained		138	Mohr-Coulomb	2300	0
Gray Clay Undrained		138	Mohr-Coulomb	2550	0
Loose Sand and Gravel		118	Mohr-Coulomb	0	31
Dense Sand and Gravel		131	Mohr-Coulomb	0	40

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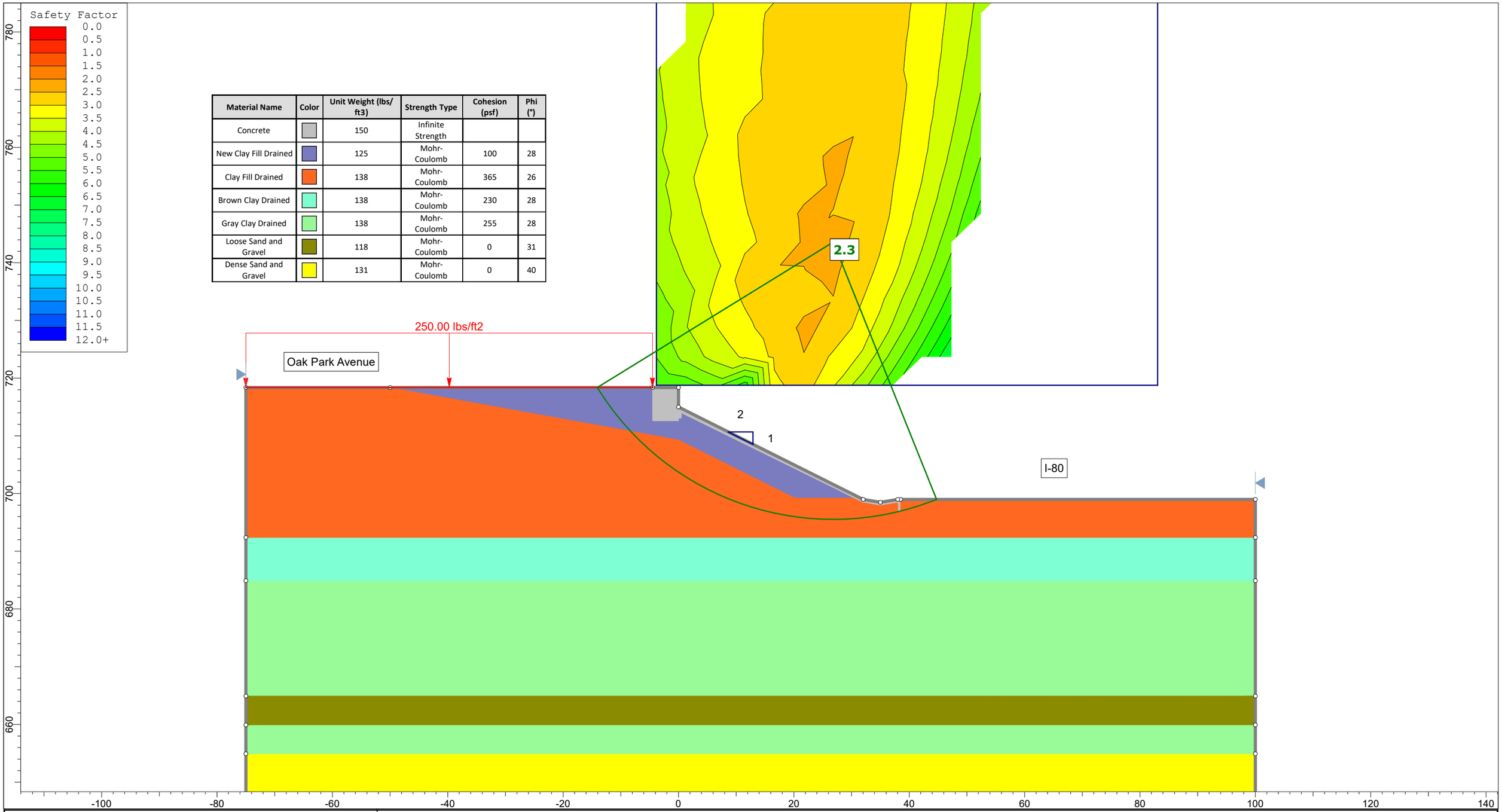
Project	PTB 205-003 Work Order 5 - South Abutment		
Analysis Description	Exhibit 3 - Circular Failure Short Term		
Drawn By	MH	*Company*	GSG Consultants, Inc.
Date	1/20/2025, 3:37:55 PM	*File Name*	South Abutment.slmd



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
Concrete		150	Infinite Strength		
New Clay Fill Drained		125	Mohr-Coulomb	100	28
Clay Fill Drained		138	Mohr-Coulomb	365	26
Brown Clay Drained		138	Mohr-Coulomb	230	28
Gray Clay Drained		138	Mohr-Coulomb	255	28
Loose Sand and Gravel		118	Mohr-Coulomb	0	31
Dense Sand and Gravel		131	Mohr-Coulomb	0	40

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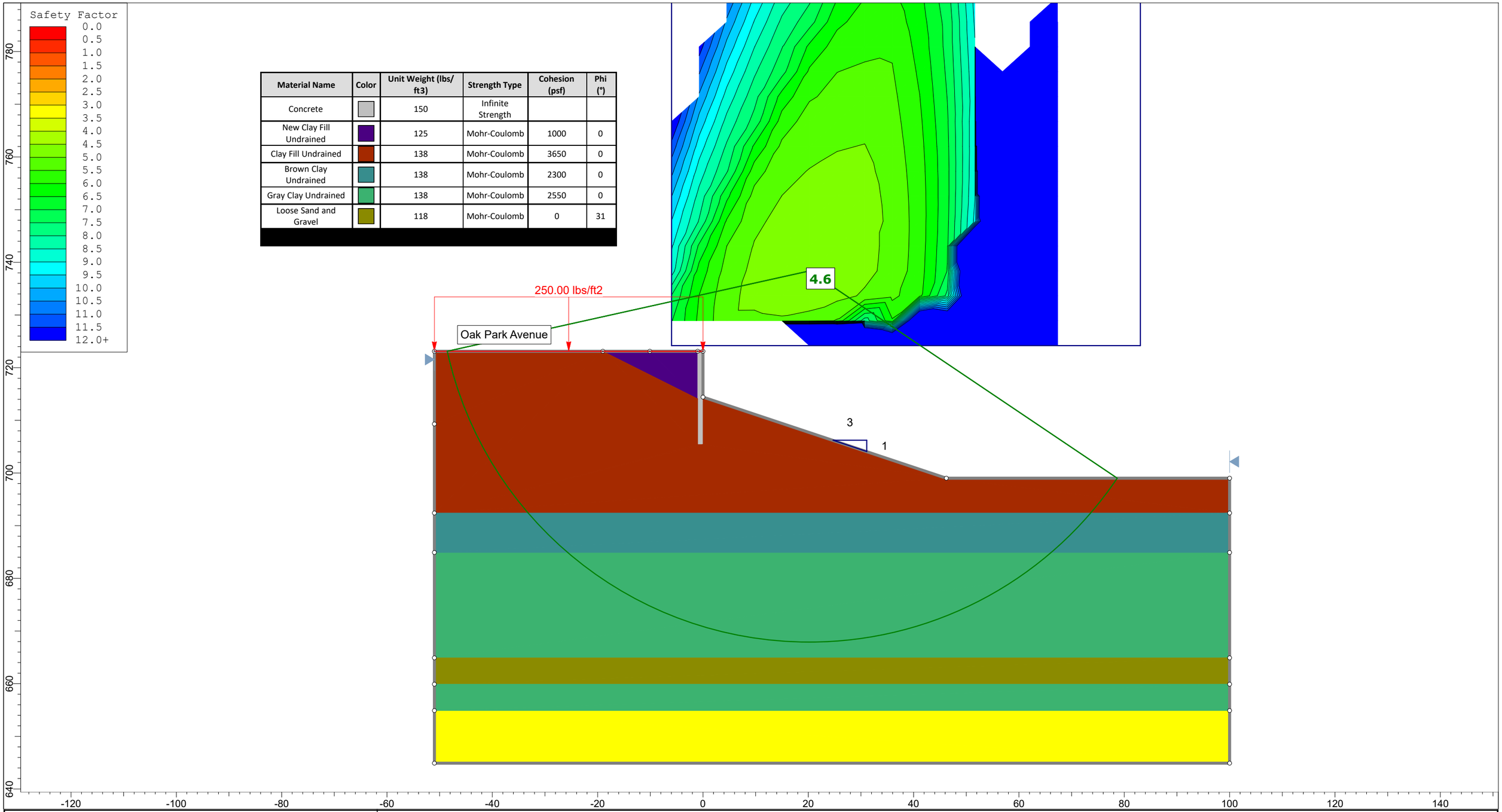
Project	PTB 205-003 Work Order 5 - South Abutment		
Analysis Description	Exhibit 4 - Circular Failure Long Term		
Drawn By	MH	*Company*	GSG Consultants, Inc.
Date	1/20/2025, 3:37:55 PM	*File Name*	South Abutment.slmd



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
Concrete		150	Infinite Strength		
New Clay Fill Undrained		125	Mohr-Coulomb	1000	0
Clay Fill Undrained		138	Mohr-Coulomb	3650	0
Brown Clay Undrained		138	Mohr-Coulomb	2300	0
Gray Clay Undrained		138	Mohr-Coulomb	2550	0
Loose Sand and Gravel		118	Mohr-Coulomb	0	31

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Project	PTB 205-003 Work Order 5 - Soldier Pile Wingwall		
Analysis Description	Exhibit 5 - Circular Failure Short Term		
Drawn By	MH	*Company*	GSG Consultants, Inc.
Date	1/20/2025, 3:37:55 PM	*File Name*	Soldier Pile Wall.slmd

Safety Factor
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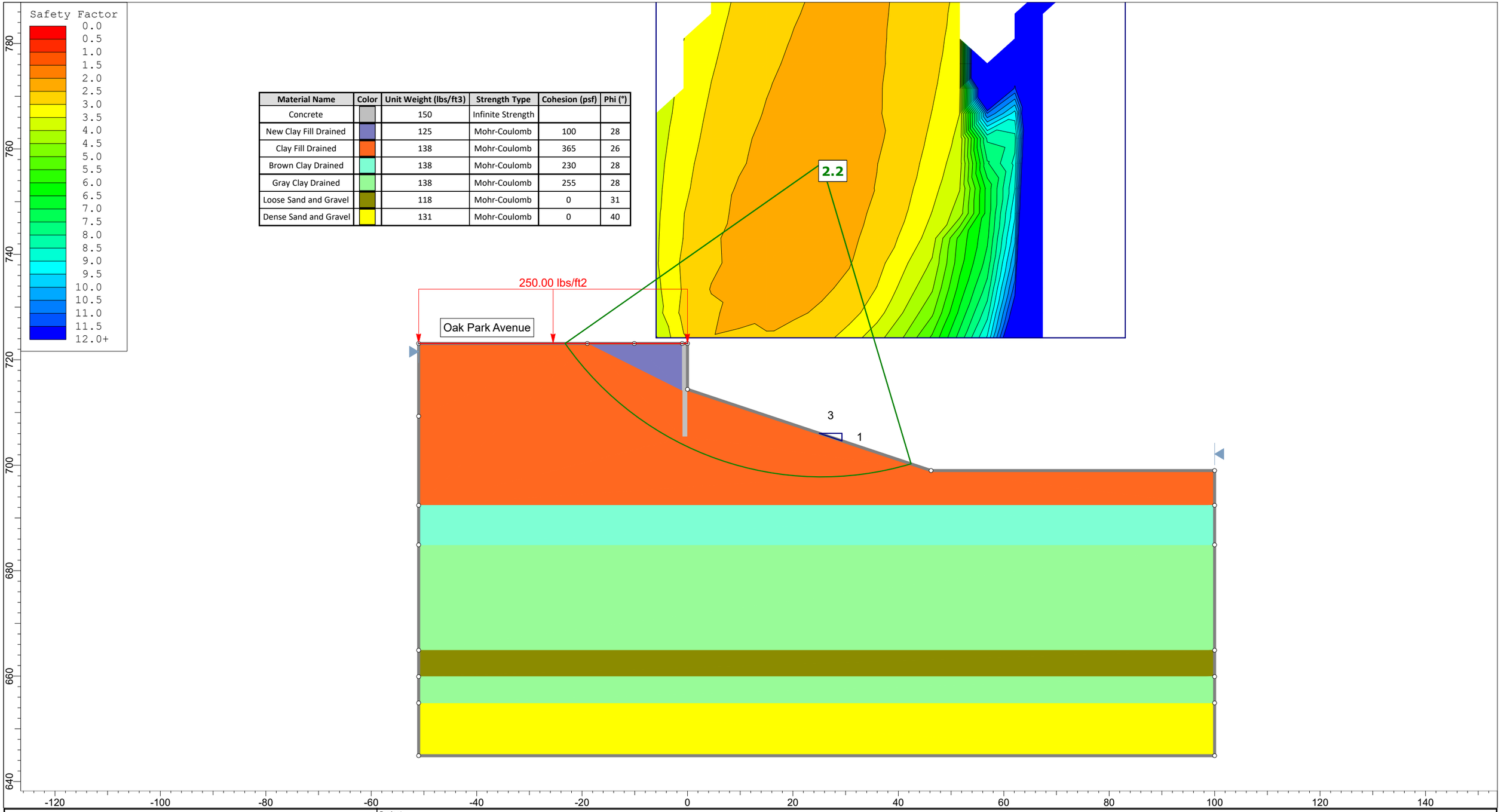
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
7.0
7.5
8.0
8.5
9.0
9.5
10.0
10.5
11.0
11.5
12.0+

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Concrete		150	Infinite Strength		
New Clay Fill Drained		125	Mohr-Coulomb	100	28
Clay Fill Drained		138	Mohr-Coulomb	365	26
Brown Clay Drained		138	Mohr-Coulomb	230	28
Gray Clay Drained		138	Mohr-Coulomb	255	28
Loose Sand and Gravel		118	Mohr-Coulomb	0	31
Dense Sand and Gravel		131	Mohr-Coulomb	0	40



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Project	PTB 205-003 Work Order 5 - Soldier Pile Wingwall		
Analysis Description	Exhibit 6 - Circular Failure Long Term		
Drawn By	MH	*Company*	GSG Consultants, Inc.
Date	1/20/2025, 3:37:55 PM	*File Name*	Soldier Pile Wall.slmd

APPENDIX G

SEISMIC SITE CLASS

DETERMINATION

Illinois Department of Transportation

SEISMIC SITE CLASS DETERMINATION

PROJECT TITLE=====PTB 205-003 WO5 - Oak Park Avenue

Substructure 1

Base of Substruct. Elev. (or ground surf for bents	714.29	ft.
Pile or Shaft Dia.		inches
Boring Number	BSB-1	
Top of Boring Elev.	717.79	ft.
Approximate Fixity Elev.	714.29	ft.

Individual Site Class Definition:

N (bar): 12 (Blows/ft.) Soil Site Class E
N_{ch} (bar): 17 (Blows/ft.) Soil Site Class D <----Controls
s_u (bar): 2.4 (ksf) Soil Site Class C

Seismic Soil Column Depth	Bot. Of Sample Elevation	Sample Thick.	N	Qu	Layer Description Boundary
(ft)		(ft.)		(tsf)	
21.5	692.8	25.00	9	3.34	B
26.5	687.8	5.00	15	5.20	B
31.5	682.8	5.00	11	0.80	B
51.5	662.8	20.00	11	2.88	B
56.5	657.8	5.00	9		B
66.5	647.8	10.00	11	2.00	B
100.0	614.3	33.50	20		B

Substructure 2

Base of Substruct. Elev. (or ground surf for bents	714.92	ft.
Pile or Shaft Dia.		inches
Boring Number	BSB-5	
Top of Boring Elev.	718.42	ft.
Approximate Fixity Elev.	714.92	ft.

Individual Site Class Definition:

N (bar): 15 (Blows/ft.) Soil Site Class D
N_{ch} (bar): 22 (Blows/ft.) Soil Site Class D <----Controls
s_u (bar): 3.06 (ksf) Soil Site Class C

Seismic Soil Column Depth	Bot. Of Sample Elevation	Sample Thick.	N	Qu	Layer Description Boundary
(ft)		(ft.)		(tsf)	
21.5	693.4	25.00	13	3.34	B
26.5	688.4	5.00	6	2.30	B
46.5	668.4	20.00	11	2.88	B
51.5	663.4	5.00	10		B
56.5	658.4	5.00	18	5.00	B
66.5	648.4	10.00	30		B
71.5	643.4	5.00	9	1.30	B
81.5	633.4	10.00	34		B
100.0	614.9	18.50	39	4.50	B

Substructure 3

Base of Substruct. Elev. (or ground surf for bents	695.5	ft.
Pile or Shaft Dia.		inches
Boring Number	BSB-2	
Top of Boring Elev.	699	ft.
Approximate Fixity Elev.	695.5	ft.

Individual Site Class Definition:

N (bar): 12 (Blows/ft.) Soil Site Class E
N_{ch} (bar): (Blows/ft.) NA
s_u (bar): 2.82 (ksf) Soil Site Class C <----Controls

Seismic Soil Column Depth	Bot. Of Sample Elevation	Sample Thick.	N	Qu	Layer Description Boundary
(ft)		(ft.)		(tsf)	
	696.5	2.50	8		B
9.0	686.5	10.00	10	3.78	B
36.5	659.0	27.50	12	2.79	B
100.0	595.5	63.50	12	2.73	B

Substructure 4

Base of Substruct. Elev. (or ground surf for bents	695.5	ft.
Pile or Shaft Dia.		inches
Boring Number	BSB-4	
Top of Boring Elev.	699	ft.
Approximate Fixity Elev.	695.5	ft.

Individual Site Class Definition:

N (bar): 11 (Blows/ft.) Soil Site Class E
N_{ch} (bar): (Blows/ft.) NA
s_u (bar): 2.39 (ksf) Soil Site Class C <----Controls

Seismic Soil Column Depth	Bot. Of Sample Elevation	Sample Thick.	N	Qu	Layer Description Boundary
(ft)		(ft.)		(tsf)	
	696.5	2.50	14		B
9.0	686.5	10.00	9	3.30	B
36.5	659.0	27.50	10	2.39	B
100.0	595.5	63.50	11	2.30	B

Global Site Class Definition: Substructures 1 through 4

N (bar): 13 (Blows/ft.) Soil Site Class E
N_{ch} (bar): 19 (Blows/ft.) Soil Site Class D <----Controls
s_u (bar): 2.67 (ksf) Soil Site Class C