Structural Geotechnical Report

Proposed Retaining Wall #7A along Ramp A
IDOT PTB 198-003
FAI-80 (I-80) over Des Plaines River

Will County, Illinois

Prepared for

Illinois Department of Transportation
Contract Number: D-91-204-19

Project Design Engineer Team
WSP USA

Geotechnical Consultant

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2nd Revision July 21, 2025
Revised June 7, 2024
January 10, 2023

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July 21, 2025

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Structural Geotechnical Report
Proposed Retaining Walls #7A along Ramp A
Will County, IL
PTB 198-003

Dear Mr. Skaleski:

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 retaining wall #7A and embankment included advancing four (4) borings to auger refusal at depths between 8.5 and 13.5 feet and two (2) 10-foot rock cores.

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

Sincerely,

Dawn Edgell.

Brook Geletu, E.I. T
Project Engineer

Dawn Edgell P.E.
Sr. Project Engineer

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1.0 INTRODUCTION

GSG Consultants, Inc. (GSG) completed a geotechnical investigation for the proposed Retaining Wall #7A and associated embankment for the I-80 Reconstruction project in the City of Joliet in Will 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 proposed construction. **Exhibit 1** shows the general project location.

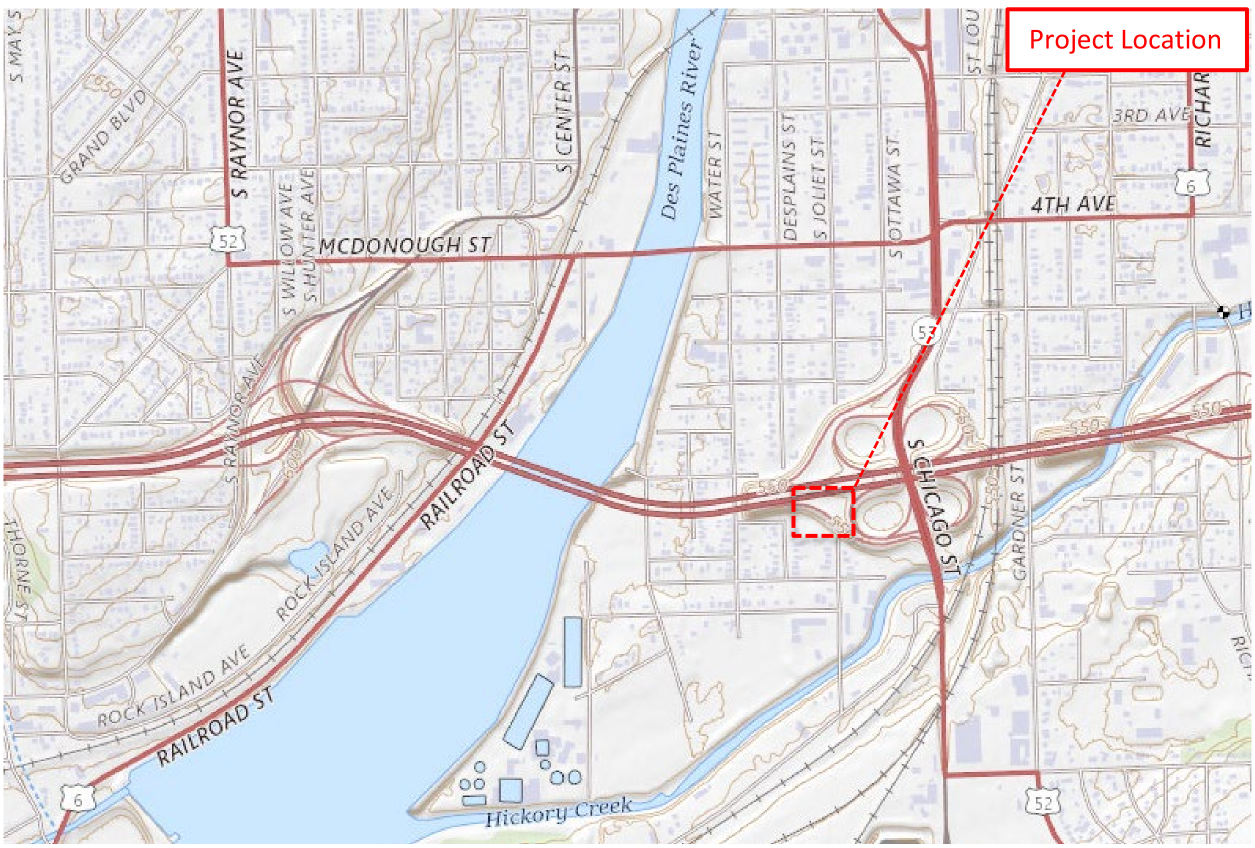


Exhibit 1 – Project Location Map

(Source: USGS Topographic Maps, usgs.gov)

1.1 Existing Site Information

The existing Ramp A will be realigned as part of the I-80 Mainline reconstruction project. There is currently no retaining wall at this location, the existing area slopes from the local City streets to the existing Ramp A. The area is currently overgrown with trees and vegetation and at the end of S. Joliet and S. Des Plaines Streets.

Exhibits 2a through 2c show the existing conditions where the proposed retaining wall and embankment will be constructed.

Exhibit 2a – Existing Boring Location, Looking North from S Joliet St.

Exhibit 2a – Existing Boring Location, Looking East from S Des Plaines St.

Exhibit 2b – Proposed Retaining Wall Location, Looking from Top

1.2 Proposed Structure Information

Based on design information and the approved GPE plan dated May 15, 2024 provided by WSP (see **Appendix A**) and a review of site topography, the proposed wall will be in a fill section along the newly constructed Ramp A embankment. It is anticipated that the proposed wall will have a maximum exposed height of 12.3 feet, for a maximum total height of 15.9 feet. The proposed retaining wall will be approximately 110 feet in length along Ramp A between Sta. 608+76.21 and Sta. 609+94.24. It is anticipated that the proposed structure will be a MSE wall. A new embankment will be constructed along Ramp A between Sta. 608+76.21 and Sta. 609+94.24. It is anticipated that the new embankment will have a maximum height of 25 feet. The new embankment will be sloped away from the wall to the new ramp roadway at a 1V:3H slope. A new noise abatement wall will be constructed at the top of the slope. Recommendations for the proposed noise abatement wall will be included in a separate report. **Table 1** presents a summary of the proposed retaining wall and embankment.

Table 1 – Proposed Retaining Wall and Embankment Summary

<table>
<tr><th>Structure Name</th><th>* Wall Stations</th><th>Approximate Length (ft)</th><th>Maximum Anticipated Exposed Wall Height (ft)</th><th>Maximum Anticipated Embankment Height (ft)</th></tr>
<tr><td>Retaining Wall #7A</td><td rowspan="2">Sta. 608+76.21to Sta. 609+94.24</td><td rowspan="2">110</td><td>12.3</td><td>n/a</td></tr>
<tr><td>Wall Embankment</td><td>n/a</td><td>25</td></tr>
</table>

* Based on proposed Ramp A Stationing

2.0 SITE SUBSURFACE CONDITIONS

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

2.1 Subsurface Exploration and Laboratory Testing

The preliminary site subsurface exploration for the proposed retaining wall structure was conducted on October 28, 2022. The investigation included advancing one (1) boring to a depth of 20 feet including a 10-foot rock core. Additional three (3) borings were completed at the proposed structure location on March 19 and 20, 2025. The investigation included advancing three (3) borings to auger refusal at depths between 8.5 and 9.5 feet and one 10-foot rock core. The locations of the soil borings were reviewed by WSP and adjusted in the field as necessary based on utilities and access. The 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 2** presents a summary of the borings used for the analysis. Copies of the Soil Boring Logs are provided in **Appendix C**.

Table 2 – Summary of Subsurface Exploration Borings

Boring ID	Station **	Offset (ft)	Northing	Easting	Depth (ft)	Surface Elevation (ft)
RWB-56	608+94.47	148.70 RT	1,764,413.85	1,052,161.87	20.0*	522.6
RWB-201	608+39.28	142.44 RT	1,764,434.15	1,052,120.57	9.0	524.5
RWB-202	609+41.29	116.17 RT	1,7644,29.36	1,052,210.49	18.5*	524.0
RWB-203	609+83.82	125.44 RT	1,764,405.61	1,052,239.60	9.5	524.4

* Depth includes Bedrock Core (10 feet), ** Based on proposed Ramp A Stationing

The soil boring was drilled using truck mounted B-57 Mobile (hammer efficiency 89%) 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 the boring termination depths upon encountering auger refusal on bedrock. Water level measurements were made in the boring when evidence of free groundwater was detected on the drill rods or in the samples. The borehole was also checked for free water immediately after auger removal, and before filling the open borehole with soil cuttings and patching the surface with asphalt.

GSG's field representative inspected, visually classified and logged the soil samples during the subsurface exploration activities. 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.

The following laboratory tests were performed on representative soil and rock samples:

- Moisture content ASTM D2216 / AASHTO T-265
- Unconfined Compression Strength on Rock ASTM D2938

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 included in the Laboratory Test Results **(Appendix E)** and are also 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 boring performed in the vicinity of the proposed retaining wall and embankment. 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. 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.

Borings RWB-201 through RWB-203 and RWB-56 were drilled in the grass space and asphalt shoulder along South Des Plaines Street. The surface elevation of the borings ranged from 524.5 to 522.6 feet. Boring RWB-56 initially encountered 4 inches of asphalt underlain by 8 inches of aggregate subbase. Borings RWB-201, 202 and 203 initially noted 3 to 4 inches of topsoil. Below the surficial materials, boring RWB-201 encountered stiff silty clay to a depth of 6.0 feet below grade. Below the clay and from the surface of the remaining borings, medium dense to very dense brown gravel and weathered limestone was encountered to the boring termination depths (auger refusal) at depths of 8.5 to 9.5 feet below grade. Boring RWB-203 noted a layer of silty loam soils at depths of 6.0 to 8.5 feet below grade.

The silty clay had an unconfined compressive strength value of 1.0 tsf. The gravel had SPT N values ranging from 4 to 43 (bpf) blows per foot with an average N value of 20 bpf. The silty loam had SPT N value of 14 bpf. The weathered limestone had SPT N values of 50 blows for 1 inch to 50 blows for 4 inches before refusal.

2.4 Subsurface Bedrock Conditions

When bedrock was encountered, a 10-foot bedrock core was collected at 2 boring locations. The extracted bedrock core was visually inspected, classified and the Rock Quality Designation (RQD) was determined according to ASTM D 6032, "Standard Test Method for Determining Rock Quality Designation (RQD) of Rock Core" and as per the IDOT geotechnical manual by totaling all sections with a length in excess of four inches (4") and dividing it by the total length of the core run. The RQD is given a classification based upon the numeric value as indicated in **Table 3**. Photographs of the rock cores are included with the soil borings in **Appendix C**.

Table 3 - Rock Quality Designation

Rock Quality Designation	Descriptions
< 25%	Very Poor
25 – 50%	Poor
51 – 75%	Fair
76 – 90%	Good
91 – 100%	Excellent

Table 4 provides a summary of the RQD values and unconfined compressive strength value of the rock cores extracted during the site investigation.

Table 4 – Rock Core Summary and Classification

Boring Number	Core Run	Core Depth (feet)	Type of Rock	RQD (%)	RQD Classification	Depth (ft)/ Unconfined Compression Strength (psi)
RWB-56	1	10.0-15.0	Limestone	52.0	Fair	17.0 / 17,041
	2	15.0-20.0	Limestone	75.0	Good	
RWB-202	1	8.5 - 18.5	Limestone	92.5	Excellent	N/A

The soil boring logs provides bedrock conditions encountered at the boring locations. The bedrock cores consisted of limestone that was slightly weathered and moderately fractured. RQD value ranged from 52.0 to 92.5 percent: Fair to Excellent as shown in **Table 4**.

2.5 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. The borings were not left open after leaving the site due to safety concerns.

Based on the general lack of water levels and color change from brown to gray observed in the soil boring, it is anticipated that the long-term groundwater level may be near the bedrock interface. Perched water may be present within the granular soil observed in the borings. Water level readings were made in the borehole 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 for the design of the proposed retaining wall and embankment 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 Embankment Settlement

It is anticipated that new fill soils will be required to construct the proposed wall and embankment. Up to 25 feet of new fill may be required to construct the new sloped embankment.

The proposed new embankment behind the proposed wall was evaluated with respect to settlement. Based on the proposed embankment heights of 25 feet, analyses were performed at the boring locations to evaluate the anticipated amount of total settlement that may be expected. The maximum estimated settlement within the native cohesive and non-cohesive soils were calculated as shown in **Table 5.**

Table 5 – Anticipated Embankment Settlement

Structure Name	Structure Stations *	Embankment Height (ft)	Anticipated Settlement (inches)
New Embankment	Sta. 608+76.21 to Sta. 609+94.24	25	<0.5

* Based on proposed Ramp A Stationing

3.2 Seismic Parameters

The seismic hazard for the site was analyzed 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. A global Site Class Definition was determined for this project, and was found to be Soil Site Class C. 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 the proposed structure. For this section of the project, the S_{DS} and the S_{D1} were determined using 2020 AASHTO Guide Specifications as shown in **Table 6**. Given the site location and materials encountered, the potential for liquefaction is minimal.

Table 6 – Seismic Parameters

Reference/Source	PGA	S_{DS}	S_{D1}
2020 AASHTO Guide for LRFD Seismic Bridge Design	0.049g	0.125g	0.068g

4.0 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 retaining wall must provide sufficient support to resist the dead and live loads, as well as seismic loading.

4.1 Retaining Wall Type Recommendations

It is anticipated that the wall will be constructed in a fill section for the proposed new embankment. There are various types of retaining walls that could be utilized for retaining earth embankments in fill areas. A MSE wall, CIP concrete cantilever wall, or prefabricated modular gravity wall are feasible options for Wall #7A.

Based on the proposed wall height, drawings and location of the wall within a fill area, GSG concurs with the design plan to use an MSE wall for Retaining Wall #7A. Advantages of the MSE wall include a relatively rapid construction schedule that does not require specialized labor or equipment, provided excavation for the reinforcement is not extensive. This type of retaining wall can accommodate relatively large total and differential settlements without distress, and the reinforcement materials are light and easy to handle.

GSG evaluated the global and external stability, and settlement to determine the suitability of the retaining wall for this section of the project. The wall section should be analyzed to determine that adequate factors of safety are achieved relative to sliding and overturning failure.

4.2 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. The selected wall should be also evaluated with respect to the collision load. **Table 7** outlines the load factors used in evaluation of the retaining wall in accordance with AASHTO Specification Tables 3.4.1-1 and 3.4.1-2.

Table 7 - LRFD Load Factors for Retaining Wall Analyses

	Type of Load	Sliding and Eccentricity Strength	Bearing Resistance Strength I	Sliding and Eccentricity Extreme II	Bearing Resistance Extreme II	Settlement Service I
Load Factors for Vertical Loads	Dead Load of Structural Components (DC)	0.90	1.25	1.00	1.00	1.00
	Vertical Earth Pressure Load (EV)	1.00	1.35	1.00	1.00	1.00
	Earth Surcharge Load (ES)		1.50			
	Live Load Surcharge (LS)		1.75		0.50	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	1.50			
	Live Load Surcharge (LS)	1.75	1.75	0.50	0.50	1.00
Load Factor for Vehicular Collision				1.00	1.00	

4.2.1 Lateral Earth Pressures and Loading

The wall should 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. The active earth pressure coefficient (Ka), and the passive earth pressure coefficient (Kp) were determined in accordance with AASHTO Section 3.11.5.3 and 3.11.5.4. **Table 8** presents soil design properties for the retaining wall for the anticipated soil types at the site based on the encountered subsurface conditions. Additional soil parameters for the site are included in **Appendix D**.

Table 8 – Lateral Soil Parameters

Depth Range (Elevation, feet) *	Soil Description	Long-term/Drained		
		Active Earth Pressure Coefficient (K_a)	Passive Earth Pressure Coefficient (K_p)	At-Rest Earth Pressure Coefficient (K_o)
	New Engineered Clay Fill	0.41	2.46	0.58
	New Engineered Granular Fill	0.33	3.00	0.50
1.0 – 9.5 (519.0 – 516.5)	Loose to Dense Gray Gravel with Silty Clay	0.20	5.04	0.33
1.0 – 3.5 (516.5 – 512.5) **Only Boring RWB-201**	Stiff Brown Silty Clay with Gravel	0.36	2.77	0.53
3.5 – 6.0 (519.0 – 516.5) **Only Boring RWB-56**	Stiff Brown Silty Clay with Gravel	0.36	2.77	0.53
6.0 – 8.5 (516.5 – 512.5) **Only Boring RWB-203**	Medium Dense Silty Loam	0.25	4.02	0.40

*Based on assumed ground elevation = 523.9 feet

Traffic and other surcharge loads should be included in the retaining wall design as applicable. 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 (Heq) of soil. **Table 9** provides the equivalent heights of soil for vehicular loadings on retaining walls.

Table 9 - Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic

Retaining Wall Height (ft)	H_{eq} 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 retaining wall design should include a drainage system to allow movement of any water behind the wall, and not allowing hydrostatic (seepage) pressures to develop in the active soil wedge behind the wall.

Heavy compaction equipment should not be allowed closer than five (5) feet to the retaining wall to prevent inducing high lateral earth pressures and causing wall yielding and/or other damage. The passive lateral earth pressure coefficient (Kp) 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.

4.2.2 Bearing Resistance – MSE Wall

It is anticipated that the retaining wall will bear on new engineered granular fill or native gravel with sand. Bearing resistance for the retaining wall shall be evaluated at the strength limit state using load factors (see **Table 7**), and factored bearing resistances. The bearing resistance factor, ϕ_b, for a MSE wall is 0.65 per AASHTO Table 11.5.7-1. The bearing resistance shall be checked for the extreme limit state with a resistance factor of 1.0.

Table 10 – Recommended Bearing Resistance for Retaining Wall

Stations	App. Elevation (feet)	Nominal Resistance (ksf)	Factored Bearing Resistance (ksf)	Bearing Resistance for 1-inch Settlement Service Limit (ksf)	Anticipated Bearing Soil
608+76.21 to 609+94.24	522.9 to 530.0	21.5	14.0	14.0	New Engineered Fill/ Medium Dense Gravel

The minimum depth of the wall should be 3.5 feet below the final exterior grade to alleviate the effects of frost. The subgrade soils encountered at the bearing elevation should be cleared of any unsuitable material. Based on the results of the subsurface exploration, we anticipate the wall would be supported upon the soil types noted in **Table 10.**

4.2.3 Subgrade Undercut Areas

Based on the soil conditions along the wall alignment, little to no undercuts are anticipated. Undercut areas (if needed) should be replaced with granular structural fill in accordance with IDOT standard construction requirements. The lateral limit of the structural fill should extend a minimum of 1 foot beyond the edge of the footing, then an additional 1 foot laterally for every 2 feet of structural fill depth as depicted in **Exhibit 3**. The granular structural fill should be placed and compacted to a minimum of 95% of the maximum dry density, as determined by AASHTO T-180: Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures (ASTM D1557) in accordance with IDOT standard construction requirements.

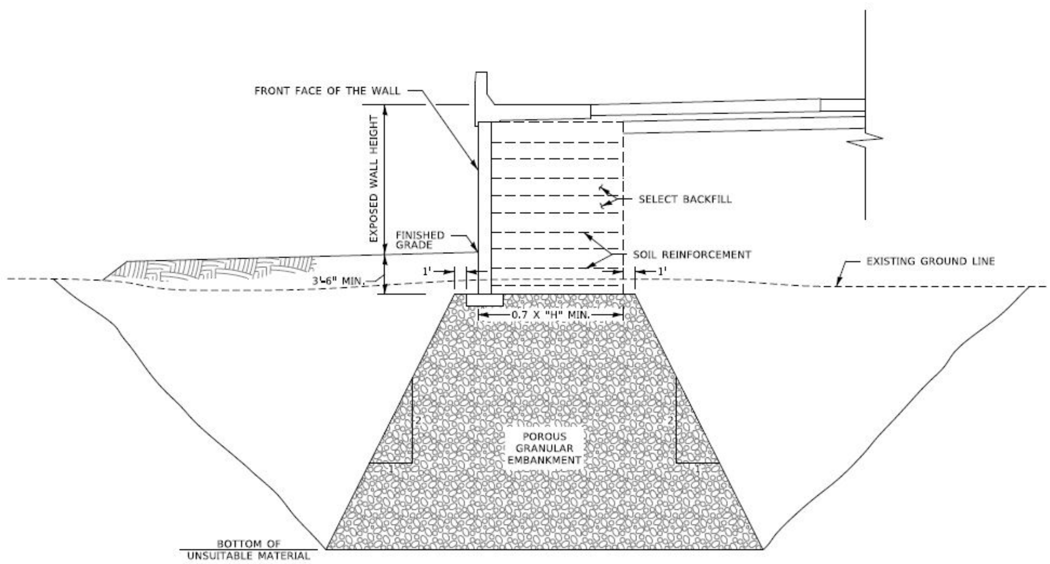


Exhibit 3 - Structural Fill Placement below MSE Wall

4.3 Sliding and Overturning Stability

The wall base width should be sufficient to resist sliding. The frictional resistance shall include the friction between granular backfill for the wall and supportive granular soils, and the friction between the wall foundation and bearing soils.

The factored resistance against sliding should be calculated using equation 10.6.3.4-1 in the AASHTO LRFD manual. A sliding resistance factor, ϕ, of 1.0 (Table 11.5.7-1) shall be applied to the nominal sliding resistance of soil beneath the wall footing. Assuming a layer of compacted granular material under the footing, the sliding resistance may be taken as one-half the normal stress on the interface between the footing and soil. The width of the footing must be wide enough to resist overturning forces. The location of the resultant of the forces shall be within the middle two-thirds of the base width.

4.4 Wall Settlement

Settlement of the proposed wall system depends on the foundation size and bearing resistance, as well as the strength and compressibility characteristics of the underlying bearing soil. Assuming the foundation subgrade has been prepared as recommended above and the service bearing resistance as noted in **Table 10** is used, the settlement of the retaining wall will be less than 1 inch. Differential settlement between two points of 100 feet apart along the length of the wall will be ½ inch or less.

4.5 Global Slope Stability

Based on the information provided by WSP, the retaining wall should be designed for external stability of the wall system. The parameters in **Table 11** were used to evaluate the proposed MSE wall to reach a minimum Factor of Safety of 1.5.

Table 11 – MSE Wall Description
*Based on drawings provided

Description	Value at Station
Maximum total height of retaining wall (H), feet	15.9
Minimum length of reinforcement 0.7XH or 8.0 feet*	11.0
Unit weight of the retained soil (embankment), pcf	125
Unit weight of the reinforced soil mass, pcf	120
Assumed bearing elevation, feet	523.0

*Actual minimum length may be greater than 0.7H depending on structural analyses.

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

Slide2 is a comprehensive slope stability analysis software used to evaluate the proposed wall for the project based on the limit equilibrium method. The proposed wall was analyzed based on the grading and the soils encountered while drilling. Circular failure analyses were evaluated using the simplified Bishops analyses methods for the proposed wall geometries. Based on the proposed geometry and the soil borings, global stability analyses were performed.

4.5.1 Global Slope Stability Results

Circular failure analyses were evaluated for both a short term (undrained) and long term (drained) condition based on the proposed geometries (**Table 11**) for the proposed MSE retaining wall. The analyses were performed at the tallest section of the wall at Station 609+42.8 and one additional section with varying soil conditions. The results of the analyses are shown in **Table 12**.

Table 12 – Retaining Wall Global Slope Stability Analyses Results

Analysis Exhibit	Location	Wall Type	Analysis Type	Factor of Safety	Minimum Factor of Safety
Exhibit 1	Station 609+42.8	MSE Wall	Circular – Short Term	1.7	1.5
Exhibit 2			Circular – Long Term	1.5	1.5

Based on the analyses performed, the proposed retaining wall meets the minimum factor of safety of 1.5. Copies of the slope stability analyses are included in the Slope Stability Analyses Exhibits (**Appendix F**).

4.6 Drainage Recommendations

The wall design should include a drainage system to prevent the buildup of hydrostatic forces behind the 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.

5.0 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.

5.1 Site Preparation

All trees, vegetation, landscaping, and surface topsoil should be cleared and removed from the vicinity of the proposed construction. Where possible, the engineer may require proof-rolling of the subgrade with a 35-ton loaded truck or other pneumatic-tired vehicle of similar size and weight. The purpose of the proof-rolling is to locate soft, weak, or excessively wet soils present at the time of construction. Proof-rolling should be performed during a time of good weather and not while the site is wet, frozen, or severely desiccated. Any unsuitable materials observed during the evaluation and proof-rolling operations should be undercut and replaced with compacted structural fill and/or stabilized in-place. The possible need for, and extent of, undercutting and/or in-place stabilization required can best be determined by the geotechnical engineer at the time of construction. Once the site has been properly prepared, at grade construction may proceed.

Foundation aggregate fill should not be placed upon wet or frozen subgrade soils. If the subgrade or structural fill becomes frozen, desiccated, wet, disturbed, softened, or loose, the affected materials should be scarified, dried and moisture conditioned, and compacted to the full depth of the affected area or the soils should be removed. Rainfall and runoff can soften soils and affect the load bearing capacity of the soils. All water entering the foundation excavation should be removed prior to placement of backfill materials above the wall bottom.

5.2 Existing Utilities

Based on the existing site conditions, utilities exist along the project corridor. Based on the GPE plan, an existing gas line runs perpendicular to the proposed wall and embankment. The plan shows the gas line is an existing 4-inch diameter line at elevation 519.25 feet, which will be abandoned during construction. 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 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.

5.3 Site Excavation

Site excavations are expected to encounter various types of soils as described in the Subsurface Exploration section of this report. The contractor will be responsible for providing a safe excavation during the construction activities of the project. All excavations should be conducted in accordance with applicable federal, state, and local safety regulations, including, but not limited to the Occupational Safety and Health Administration (OSHA) excavation safety standards. Excavation stability and soil pressures on temporary shoring are dependent on soil conditions, depth of excavations, installation procedures, and the magnitude of any surcharge loads on the ground surface adjacent to the excavation. Excavation near existing structures and underground utilities should be performed with extreme care to avoid undermining existing structures. Excavations should not extend below the level of adjacent existing foundations or utilities unless underpinning or other support is installed. It is the responsibility of the contractor for field determinations of applicable conditions and providing adequate shoring (if needed) for all excavation activities.

5.4 Borrow Material and Compaction Requirements

If borrow material is to be used for onsite construction, it should conform to Section 204 “Borrow and Furnish Excavations” of the IDOT Construction Manual (2022). The fill material should be free of organic matter and debris and should be placed and compacted in accordance with Section 205, Embankment, of the IDOT Construction Manual. 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 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.

5.5 Groundwater Management

Based on the general lack of water levels and color change from brown to gray observed in the soil borings, it is anticipated that the long-term groundwater level may be near the bedrock interface. GSG does not anticipate that significant groundwater related issues will occur during construction activity, however perched water may be encountered within the existing granular soil. If rainwater run-off or groundwater is accumulated at the base of excavations, 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. Water should not be allowed to accumulate in the foundation area either during or after construction. 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.

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 beneath the footings should be backfilled using approved structural fill.

6.0 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 project area. The analysis has 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 during 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

(05/15/2024)

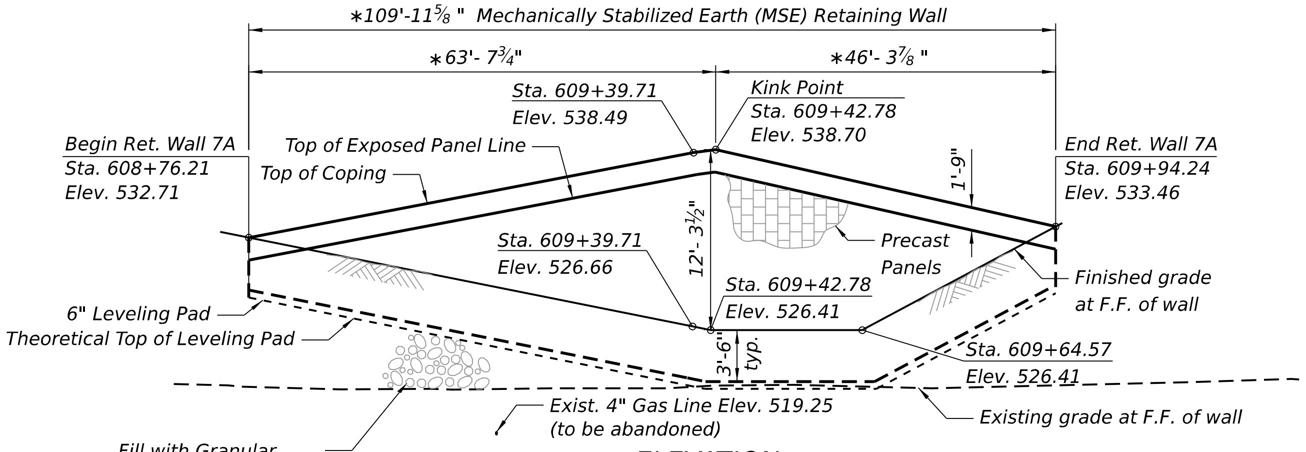
Benchmark: Iron Rod with Cap Sta. 67+63.19, 388.39' RT
N 1,764,330.037 and E 1,052,698.888
Elev.= 550.720

Existing Structures: None.

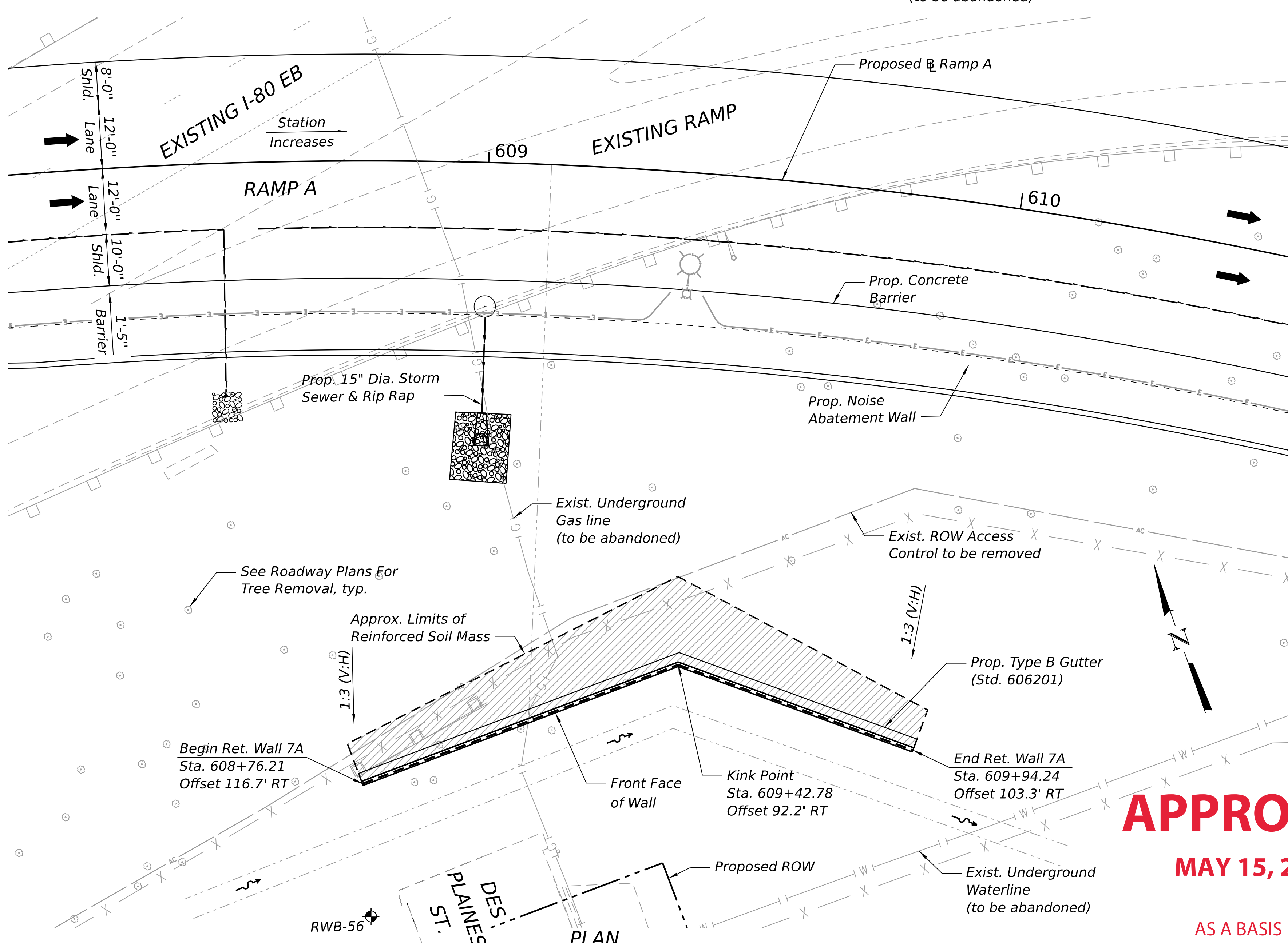
Traffic Control: EB I-80 traffic to Chicago St. will be detoured to the east at Richards St.

Salvage: None.

∗ Measured along F.F. of Wall



ELEVATION
(Looking at F.F. of Wall)



PLAN

LEGEND

⯁	Soil Borings
—G—	Exist. Underground Gasline
—W—	Exist. Underground Waterline
—AC—	Exist. ROW Access Control
— —	Exist. ROW
— - —	Prop. ROW
—E—	Prop. Underground Electric
▸▸	Prop. Storm Sewer
- -	Prop. Pipe Underdrain
F.F.	Front Face
B.F.	Back Face
~▸	Proposed Ditch
▨	Approx. Limits of Reinforced Soil Mass
⊙	Existing Tree

NOTE:

1. Stations and offsets are measured from the ℄ of Ramp A to the front face of precast panels.

HIGHWAY CLASSIFICATION

I-80 EB
Functional Class: Interstate
ADT: 91,100 (2017); 133,500 (2040)
ADTT: 19,241 (2017); 28,169 (2040)
DHV: 14,685 (2040)
Design Speed: 70 m.p.h.
Posted Speed: 65 m.p.h.
One-Way Traffic
Directional Distribution: 100% EB

Ramp A
Functional Class: Interstate
ADT: 4,860 (2017); 11,400 (2040)
ADTT: 810 (2017); 1,900 (2040)
DHV: 1,254 (2040)
Design Speed: 35 m.p.h.
Posted Speed: 35 m.p.h.
One-Way Traffic
Directional Distribution: 100%

DESIGN STRESSES

PRECAST UNITS

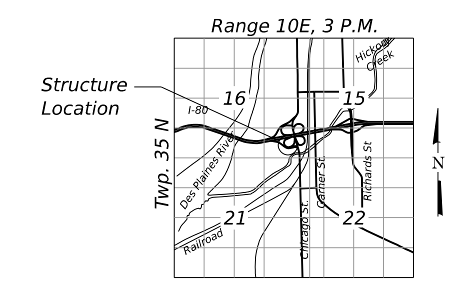
f'c = 4,500 psi

FIELD UNITS

f'c = 3,500 psi
fy = 60,000 psi (Reinforcement)

DESIGN SPECIFICATIONS

2020 AASHTO LRFD Bridge Design Specifications, 9th Edition



LOCATION SKETCH

APPROVED
MAY 15, 2024
AS A BASIS FOR PREPARATION OF DETAILED PLANS

GENERAL PLAN AND ELEVATION
RETAINING WALL ALONG RAMP A
F.A.I. RTE. 80 - SEC 2017-057F
WILL COUNTY
STA. 608+76.21 TO STA. 609+94.24
STRUCTURE NO. 099-W126

MODEL: Default
FILE NAME: 099XXXX-62F94-sht-TSL_W7A_001.dgn

SEE
1111 BURLINGTON AVE., SUITE 111 - LISLE, IL 60532
(630) 641-9900

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	CHECKED - RRD	REVISED -	
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STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

GENERAL PLAN AND ELEVATION
STRUCTURE NO. 099-W126
SHEET 1 OF 2 SHEETS

F.A.I. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
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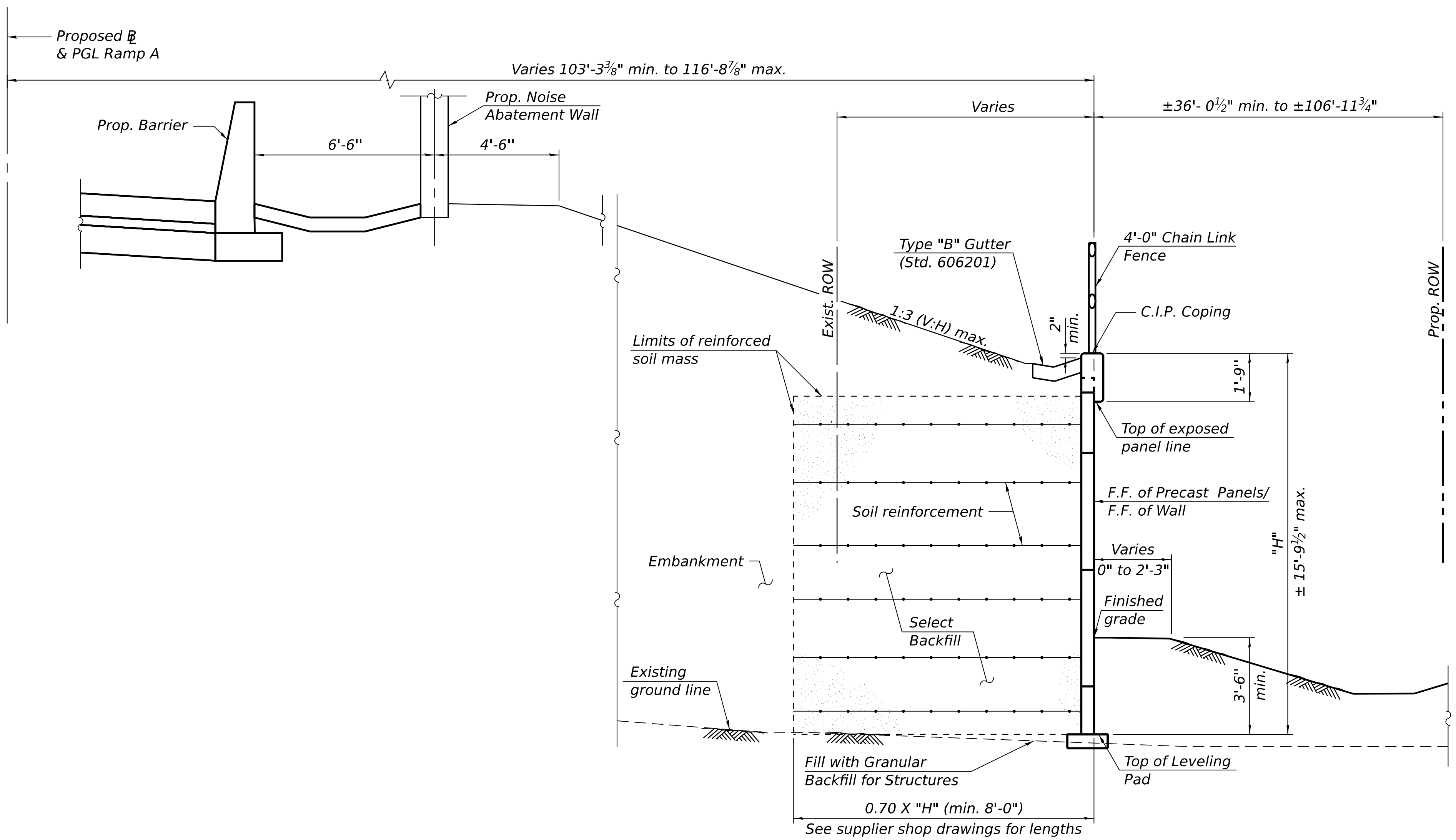
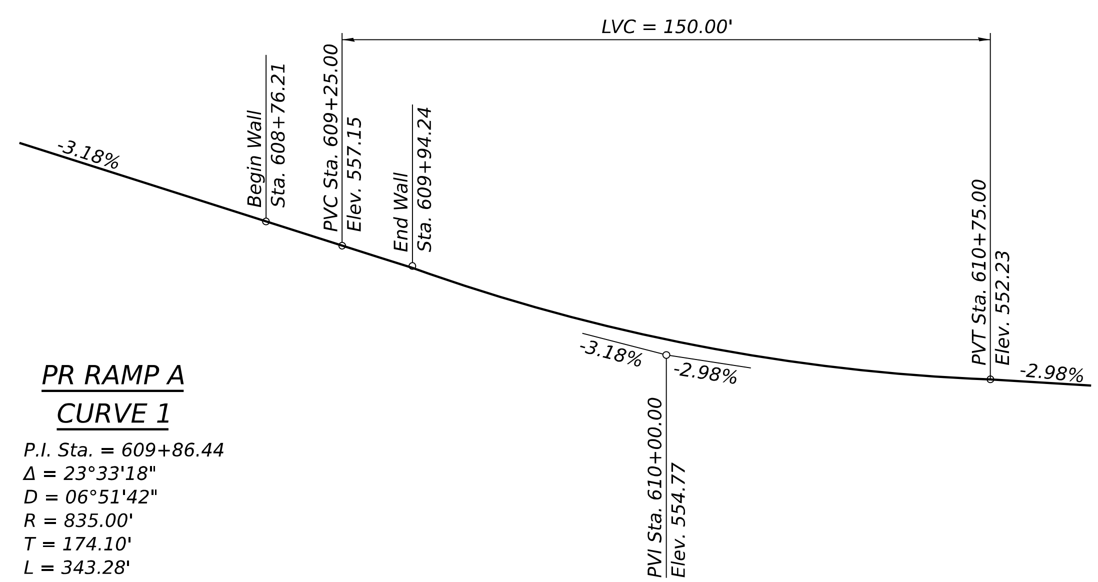
		CONTRACT NO. 62F94		
	ILLINOIS	FED. AID PROJECT		

2/28/2024 4:41:43 PM



TYPICAL SECTION THRU M.S.E WALL

(Looking East)



RAMP A PROFILE GRADE

(Along Prop. ℄ Ramp A)

PR RAMP A
CURVE 1

P.I. Sta. = 609+86.44
Δ = 23°33'18"
D = 06°51'42"
R = 835.00'
T = 174.10'
L = 343.28'
E = 17.96'
e = 6.0%
P.C. Sta. = 608+12.34
P.T. Sta. = 611+55.62

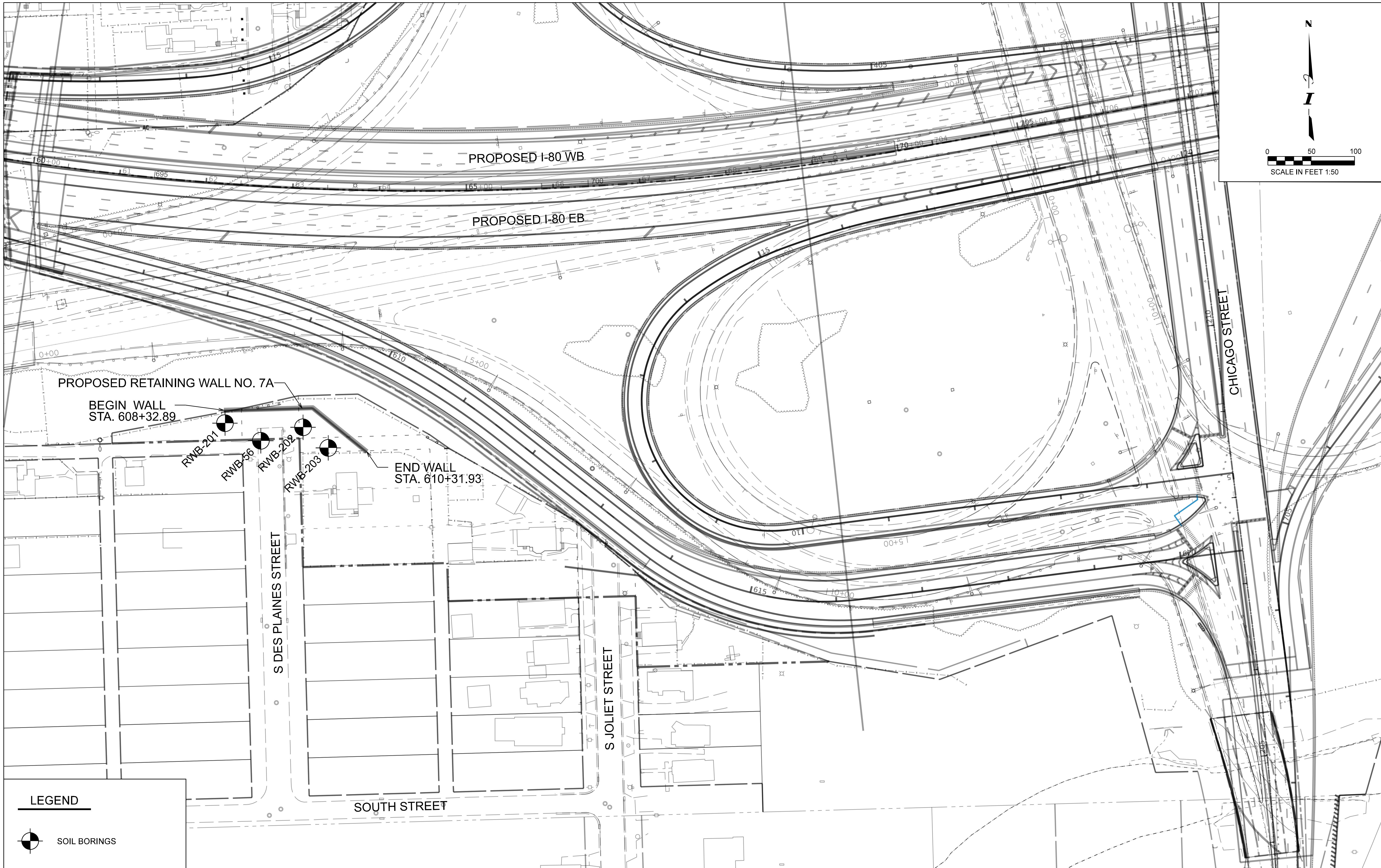
APPROVED

MAY 15, 2024

AS A BASIS FOR
PREPARATION OF DETAILED PLANS

WALL DETAIL AND TYPICAL SECTIONS
RETAINING WALL ALONG RAMP A
F.A.I. RTE. 80 - SEC 2017-057F
WILL COUNTY
STA. 608+76.21 TO STA. 609+94.24
STRUCTURE NO. 099-W126

Appendix B
Soil Boring Location Plan



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

USER NAME = nnano		DESIGNED -	BG
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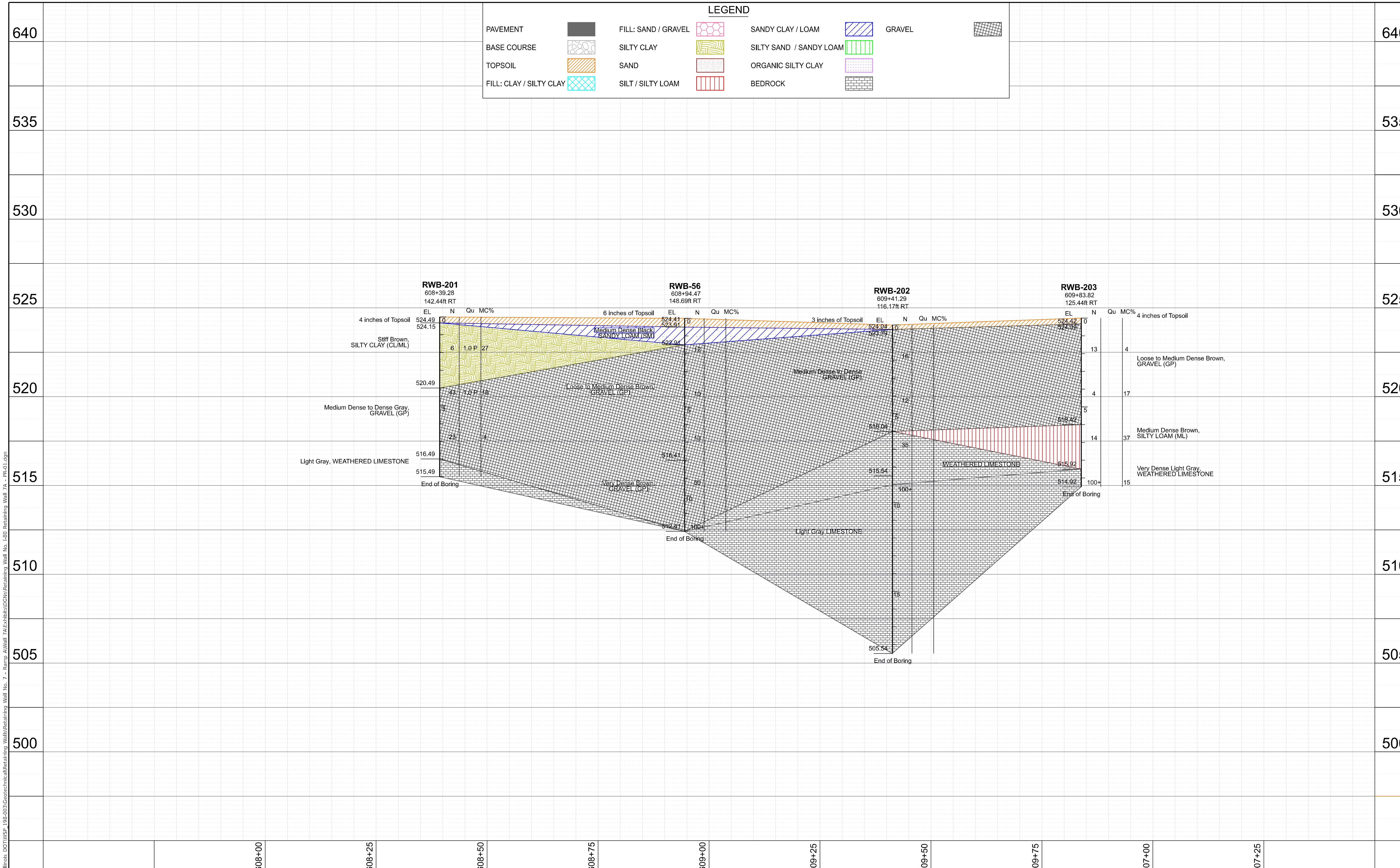
PLOT SCALE = $SCALE$		CHECKED -	DE
PLOT DATE = 6/25/2025		DATE -	06/18/2025

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

RETAINING WALL NO. 7A
SOIL BORING LOCATION PLAN
JOLIET, ILLINOIS

SCALE: 1:50 | SHEET 1 OF 1 SHEETS | STA. | TO STA.

F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
		WILL	2	1
		CONTRACT NO. PTB-198-003		
	ILLINOIS	FED. AID PROJECT		



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

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	DRAWN - NN	REVISED -			
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PLOT DATE = 7/21/2025	DATE - 06/18/2025	REVISED -			

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

RETAINING WALL NO. 7A

BORING PROFILE

SCALE: AS NOTED | SHEET 1 OF 1 SHEETS | STA. 608+00 TO STA. 607+25

F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
		WILL	2	2
		CONTRACT NO. PTB 198-003		
		ILLINOIS FED. AID PROJECT		

Appendix C
Soil Boring Logs

Illinois Department of Transportation
Division of Highways
GSG Consultants

SOIL BORING LOG

Date 3/19/25

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 7 - Ramp A **LOGGED BY** SB

SECTION C-91-109-22 **LOCATION** , **SEC.** 16, **TWP.** 35 N, **RNG.** 10 E, **Latitude** 41.5108154, **Longitude** -88.0855262

COUNTY Will **DRILLING RIG** Mobile B-57 **HAMMER TYPE** Auto
DRILLING METHOD HSA **HAMMER EFF (%)** 89

STRUCT. NO. 099-W126
Station 608+76.21 to 609+94.24

BORING NO. RWB-201
Station 608+39.28
Offset 142.44ft RT
Ground Surface Elev. 524.49 ft

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

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
4 inches of Topsoil	524.15				
Stiff Brown, Moist SILTY CLAY, little gravel, trace roots (CL/ML)			3 4 2	1.0 P	27
	520.49		3		
Medium Dense to Dense Gray, Moist GRAVEL (GP)		-5	12 31	1.0 P	18
			7 15 8		4
	516.49				
Light Gray, Moist WEATHERED LIMESTONE	515.49	-9			
Auger refusal at 9 feet 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

SOIL BORING LOG

Date 3/20/25

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 7 - Ramp A **LOGGED BY** AGK

SECTION C-91-109-22 **LOCATION** , **SEC.** 16, **TWP.** 35 N, **RNG.** 10 E, **Latitude** 41.5108016, **Longitude** -88.0851979

COUNTY Will **DRILLING RIG** Mobile B-57 **DRILLING METHOD** HSA **HAMMER TYPE** Auto **HAMMER EFF (%)** 89

STRUCT. NO. 099-W126
Station 608+76.21 to 609+94.24

BORING NO. RWB-202
Station 609+41.29
Offset 116.17ft RT
Ground Surface Elev. 524.04 ft

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

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

Description	Elev. (ft)	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
3 inches of Topsoil	523.79				
Medium Dense to Dense Gray, Moist GRAVEL, with silty clay (GP)			5 7 9		20
		-5	7 6 6		32
WEATHERED LIMESTONE	518.04		16 17 18		9
Auger refusal at 8.5 feet	515.54		8 50/1"		
Light Gray LIMESTONE, slightly weathered, moderately fractured Run 1: 8.5' - 18.5' Recovery: 100% RQD: 92.5% (Excellent)		-10			
		-15			
End of Boring	505.54	-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)

Retaining Wall #7A
Boring Number: RWB-202

Depth =8.5 ft
Elev. = 515.5 ft

Top

Depth = 18.5 ft
Elev. = 505.5 ft

Bottom

Boring No.	Run	Depth (ft)	Recovery (%)	RQD (%)	RQD Classification	Description
RWB-202	1	8.5' – 18.5'	100	92.5	Excellent	Light Gray, Slightly Weathered Moderately Fractured, Trace Chert, LIMESTONE

Illinois Department of Transportation
Division of Highways
GSG Consultants

SOIL BORING LOG

Date 3/19/25

ROUTE I-80 DESCRIPTION Retaining Wall No. 7 - Ramp A LOGGED BY SB

SECTION C-91-109-22 LOCATION , **SEC.** 16, **TWP.** 35 N, **RNG.** 10 E, **Latitude** 41.5107362, **Longitude** -88.0850919

COUNTY Will DRILLING RIG Mobile B-57 HAMMER TYPE Auto
DRILLING METHOD HSA HAMMER EFF (%) 89

STRUCT. NO. 099-W126
Station 608+76.21 to 609+94.24

BORING NO. RWB-203
Station 609+83.82
Offset 125.44ft RT
Ground Surface Elev. 524.42 ft

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

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
4 inches of Topsoil	524.09				
Loose to Medium Dense Brown, Moist GRAVEL, little clay, trace roots (GP)			6 6 7		4
		-5	7 2 2		17
	518.42				
Medium Dense Brown, Moist SILTY LOAM, little clay, trace sand, trace gravel (ML)			2 4 10		37
	515.92				
Very Dense Light Gray, Moist WEATHERED LIMESTONE	514.92		50/2"		15
Auger refusal at 9.5 feet End of Boring		-10			

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

SOIL BORING LOG

Date 10/28/22

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 7 - Ramp A **LOGGED BY** AA

SECTION C-91-109-22 **LOCATION** , **SEC.** 16, **TWP.** 35 N, **RNG.** 10 E, **Latitude** 41.510754, **Longitude** -88.084677

COUNTY Will **DRILLING RIG** Mobile B-57 **HAMMER TYPE** Auto
DRILLING METHOD HSA **HAMMER EFF (%)** 89

STRUCT. NO. 099-W126
Station 608+76.21 to 609+94.24

BORING NO. RWB-56
Station 608+94.47
Offset 148.70ft RT
Ground Surface Elev. 522.58 ft

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

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
4 inches of Asphalt	522.25				
8 inches of Aggregate Subbase	521.58				
Medium Dense Dark Brown and Gray, Moist GRAVEL, with sand, trace clay seams (GPS)			5 15 8		12
	519.08				
Stiff Dark Brown, Moist SILTY CLAY, with gravel (CL/ML)		-5	2 2 2	1.0 P	20
	516.58				
Medium Dense to Very Dense Brown, Dry to Moist GRAVEL, trace sand and clay seams (GP)			2 6 6		15
			16 50/4"		3
	513.08				
WEATHERED LIMESTONE	512.58	-10			
Gray LIMESTONE, slightly weathered, moderately fractured Run 1: 10' - 15' Recovery: 92% RQD: 52% (Fair)					
	507.58	-15			
Run 2: 15' - 20' Recovery: 100% RQD: 75% (Good)					
	502.58	-20			

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)

Retaining Wall #7A
Boring Number: RWB-56

Depth =10.0 ft
Elev. = 512.6 ft

Depth =15.0 ft
Elev. = 507.6 ft

Boring No.	Run	Depth (ft)	Recovery (%)	RQD (%)	RQD Classification	Description	Depth (ft)/ Unconfined Compression Strength (psi)
RWB-56	1	10' – 15'	92.0	52.0	Fair	Gray, Slightly Weathered, Moderately Fractured, LIMESTONE	17.0 / 17,041
	2	15' – 20'	100.0	75.0	Good		

Appendix D
Soil Parameter Table

Table D-1 – Summary of Soil Parameters

Depth Range (Elevation, feet) *	Soil Description	In situ Unit Weight γ (pcf)	Undrained		Drained	
			Cohesion C (psf)	Friction Angle ϕ (°)	Cohesion C (psf)	Friction Angle ϕ (°)
	New Engineered Clay Fill	125	1,000	0	100	25
	New Engineered Granular Fill	125	0	30	0	30
1.0 – 9.5 (519.0 – 516.5)	Loose to Dense Gray Gravel with Silty Clay	126	0	42	0	42
1.0 – 3.5 (516.5 – 512.5) **Only Boring RWB-201**	Stiff Brown Silty Clay with Gravel	129	1,000	0	100	28
3.5 – 6.0 (519.0 – 516.5) **Only Boring RWB-56**	Stiff Brown Silty Clay with Gravel	129	1,000	0	100	28
6.0 – 8.5 (516.5 – 512.5) **Only Boring RWB-203**	Medium Dense Gray Silty Loam	122	0	37	0	37
6.0 – 10.0 (517.9 – 513.9)	Weathered Limestone	150	0	45	0	45

Appendix E
Laboratory Test Results

Order # 955

Compressive Strength of Rock by ASTM D7012 - Method C

GSG CONSULTANTS, INC.
735 Remington Road, Schaumburg, IL 60173
Tel: 630.994.2600, www.gsg-consultants.com

Project Name:	WSP_198-003 I-80	Project No:	21-2007
Boring ID:	RWB-56		
Sample Depth (ft):	17-18		
Lithological Description:	Limestone		
Formation Name:			
Load Direction:	Vertical	Angle Drilled:	Vertical
Appearance (e.g. cracks, shearing, spalling):			

	Bulk/Prep	MC/CS
Tester:	AJ	AJ
Date:	11/02/22	11/02/22

Bulk Density Determination

	1	2	3	Average
Height, *in.*	4.8840	4.8830	4.8800	4.8823
Diameter, *in.*	1.9855	1.9825	1.9850	1.9843

Specimen Mass, *g*	676.3	Ratio (2.0-2.5)	
Bulk Density, *pcf*	170.7		2.46

Moisture Condition - D2216

Container ID	Φ5
container, *g*	518.6
container + wet rock, *g*	854.6
container + dry soil, *g*	849.6
moisture content, *w%*	1.5

Preparation Check

Preparation Check	Yes	No	Reason/Readings If No:
Ends Flat within 0.02 mm prior to capping?	X		
Ends perpendicular to side within 0.25 degrees?	X		
Ends parallel to each other within 0.25 degrees?	X		

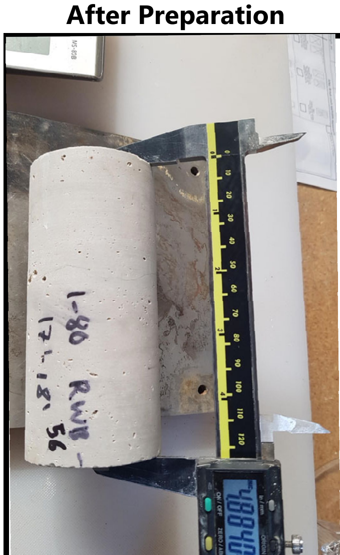
Axial Loading

Seating Load (≤1000 psi)	1000
Rate of Loading (73-145 psi/s)	75
Time to Failure (2-15 min)	3 min 43 sec
Load @ Failure, *lbf*	52,700
Uniaxial Compressive Strength, *psi*	17,041

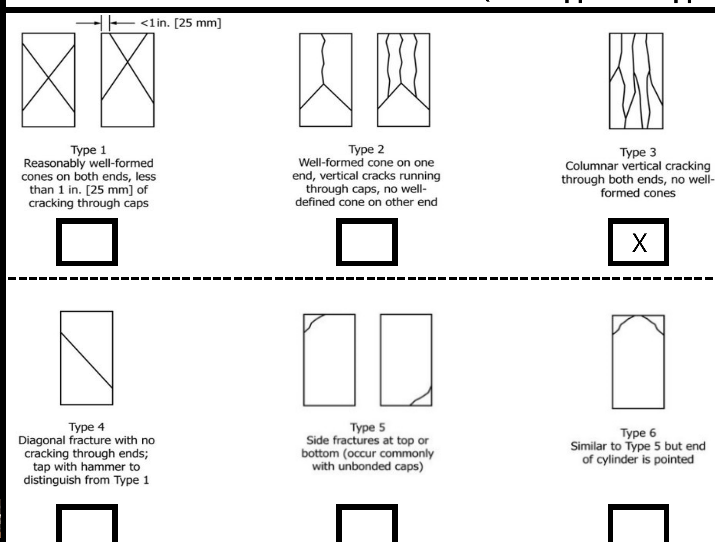
Remarks

Best efforts have been made for the specimen to meet the required tolerances of D4543. See IH3 Procedure for efforts made.

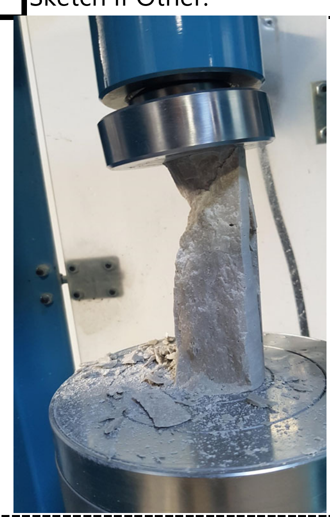
After Preparation



After Break (check applicable appearance)



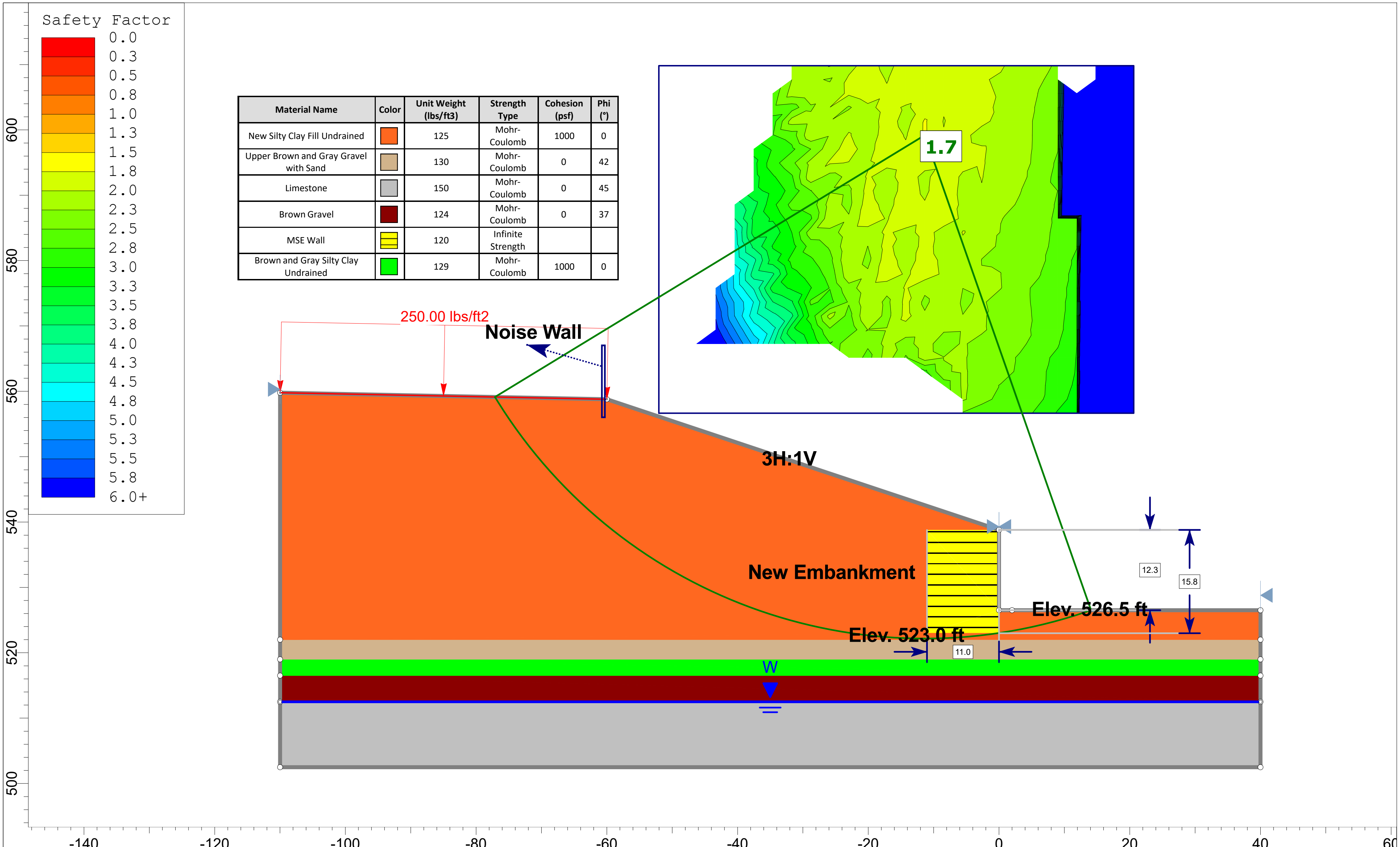
Sketch if Other:



Form ID	TF-RCS	Reviewed By	
Revision Date	10/21/2021	Review Date	

Appendix F

Slope Stability Analysis Exhibits

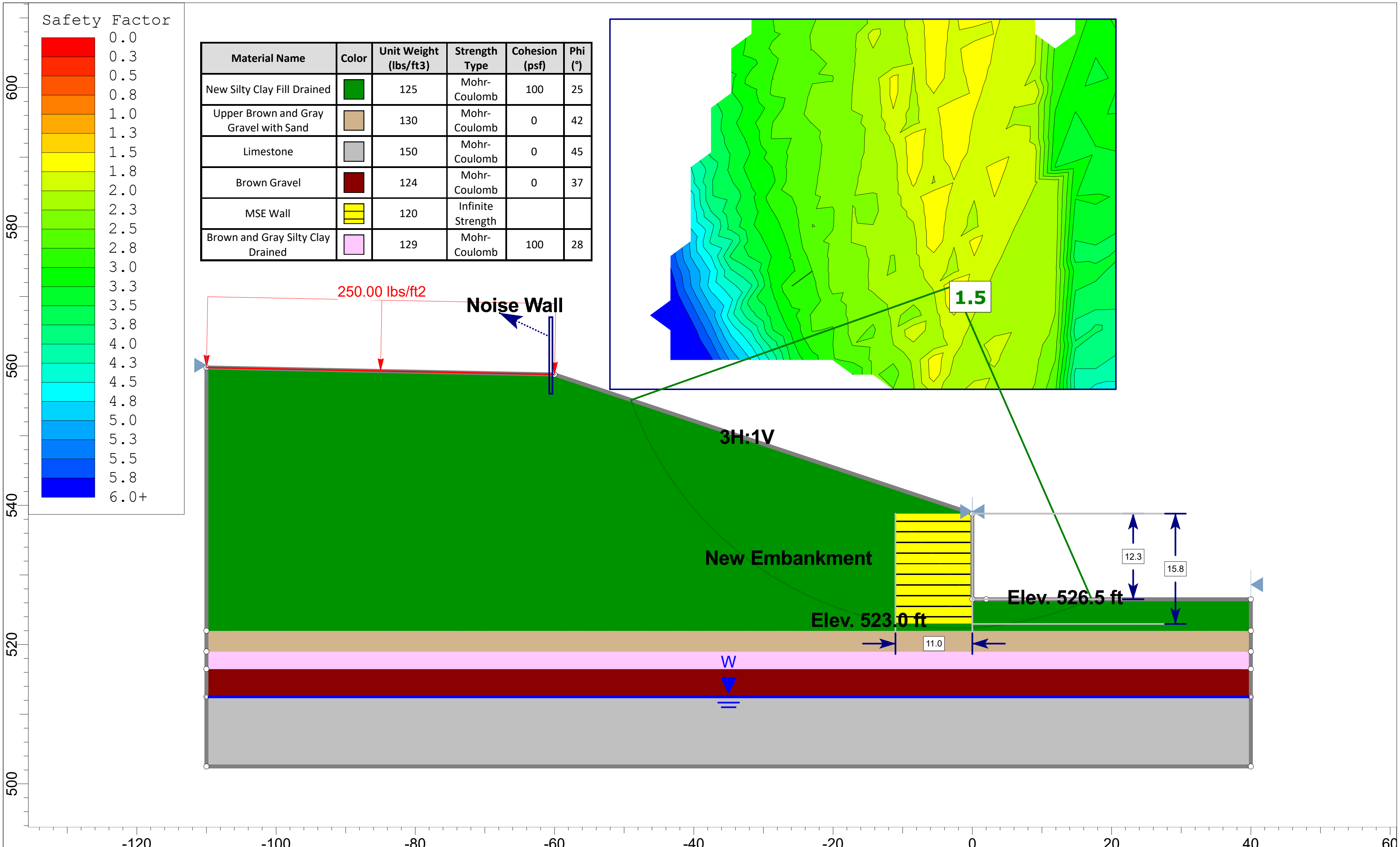


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Silty Clay Fill Undrained		125	Mohr-Coulomb	1000	0
Upper Brown and Gray Gravel with Sand		130	Mohr-Coulomb	0	42
Limestone		150	Mohr-Coulomb	0	45
Brown Gravel		124	Mohr-Coulomb	0	37
MSE Wall		120	Infinite Strength		
Brown and Gray Silty Clay Undrained		129	Mohr-Coulomb	1000	0

GSG CONSULTANTS, INC.
735 Remington Road, Schaumburg, IL 60173
Tel: 630.994.2600, www.gsg-consultants.com

SLIDEINTERPRET 9.031

Project	I-80: IDOT PTB 198-003 - Retaining Wall #7A - MSE Wall - Station 609+43		
Group	Group 1	*Scenario*	Short Term Stability
Drawn By	RM	*Company*	GSG Consultants, Inc.
Date	9/27/2019, 2:18:40 PM	*File Name*	MSE Wall 609+43_v1.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
New Silty Clay Fill Drained		125	Mohr-Coulomb	100	25
Upper Brown and Gray Gravel with Sand		130	Mohr-Coulomb	0	42
Limestone		150	Mohr-Coulomb	0	45
Brown Gravel		124	Mohr-Coulomb	0	37
MSE Wall		120	Infinite Strength		
Brown and Gray Silty Clay Drained		129	Mohr-Coulomb	100	28

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SLIDEINTERPRET 9.031

Project	I-80: IDOT PTB 198-003 - Retaining Wall #7A - MSE Wall - Station 609+43
Group	Group 1
Scenario	Long Term Stability
Drawn By	RM
Company	GSG Consultants, Inc.
Date	9/27/2019, 2:18:40 PM
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