Preliminary Structural Geotechnical Report

Proposed Retaining Wall #7B along Ramp A
Structure No. 099-W127
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

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

Dear Mr. Skaleski:

Attached is a copy of the Preliminary 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 #7B and embankment included advancing two (2) soil borings to depths of 22 to 31 feet including one (1) 10 feet rock core. Additional borings at the proposed structure location will be performed once site access to the area is available.

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

Sincerely,

Ehab Shaheen

Ehab Shaheen, Ph.D., P.E.
Project Engineer

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Principal

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1.0 INTRODUCTION

GSG Consultants, Inc. (GSG) completed a preliminary geotechnical investigation for the proposed Retaining Wall #7B and associated embankment along Ramp A 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.

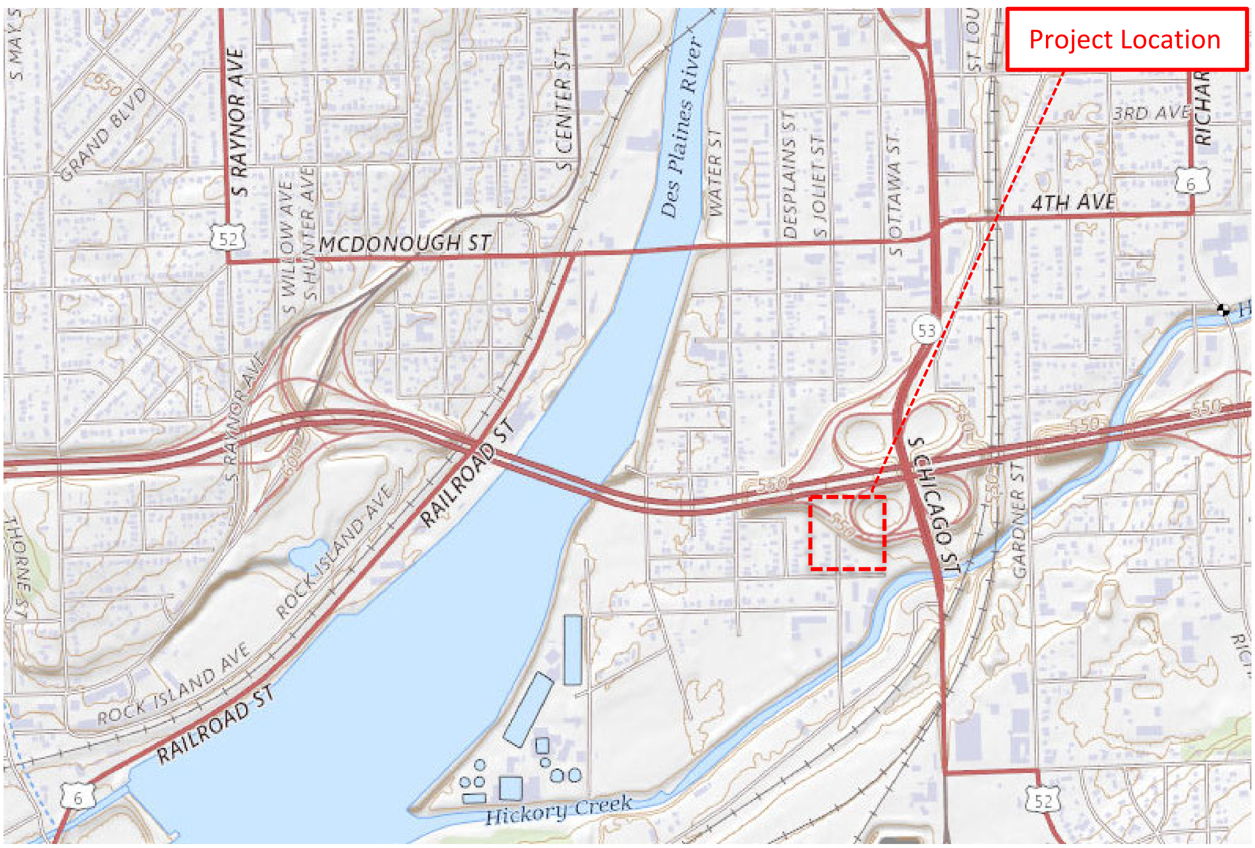


Exhibit 1 – Project Location Map
(Source: USGS Topographic Maps, usgs.gov)

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 streets to the existing Ramp A. The area where the proposed wall will be constructed is partially outside of the existing IDOT Right-of-Way, between the I-80/Chicago Street interchange and the neighboring residential area to the south. The area is currently overgrown with trees and vegetation at the end of S. Joliet Street.

Exhibit 2a and 2b 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

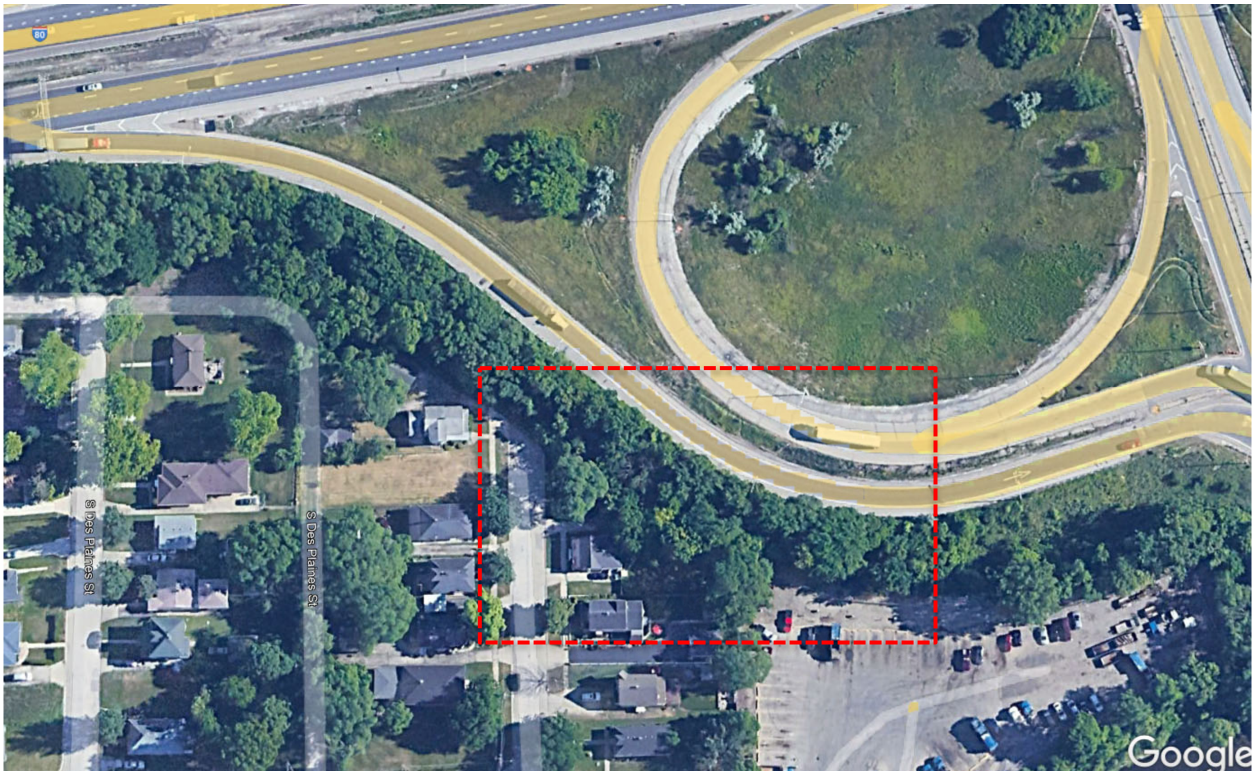


Exhibit 2b – Proposed Retaining Wall Location, Looking from Top

1.2 Proposed Structure Information

Based on preliminary design information 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 18.5 feet, for a maximum total height of 22.0 feet. The proposed retaining wall will be approximately 345 feet in length along Ramp A between Sta. 612+92.72 and Sta. 616+36.00. It is anticipated that the proposed structure will be a MSE wall. A new embankment will be constructed along Ramp A between Sta. 612+92.72 and Sta. 616+36.00. It is anticipated that the new embankment will have a maximum height of 18 feet. The new embankment will be sloped away from the wall to the new ramp roadway at a 1V:3H slope between Sta. 612+92.72 and Sta. 613+80.31. A new noise abatement wall will be constructed at the top of the new embankment. A Temporary Soil Retention System is anticipated to be necessary between the existing roadway ramp and the new retaining wall excavation. Recommendations for the proposed noise abatement wall will be provided 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 #7B</td><td rowspan="2">Sta. 612+92.72 to Sta. 616+36.00</td><td rowspan="2">345</td><td>18.5</td><td>n/a</td></tr>
<tr><td>Wall Embankment</td><td>n/a</td><td>28</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 were selected in accordance with IDOT requirements and reviewed with WSP based on the Phase 1 plans. Additional borings are proposed along the final wall alignment and will be completed once the area is accessible. 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 between October 20, 2022 and October 28, 2022. The investigation included advancing two (2) borings to depths of 22 to 31 feet including one (1) 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 analyses.

Table 2 – Summary of Subsurface Exploration Borings

Boring ID	Station **	Offset (ft)	Northing	Easting	Depth (ft)	Surface Elevation (ft)
RWB-57	613+21.30	36.96 RT	1764294.919	1052547.708	22.0	525.80
RWB-58	613+41.20	60.34 RT	1764264.244	1052548.894	31.0*	525.45
RWB-64***	618+41.25	34.94 RT			20.0	532.0

* Depth includes Bedrock Core (10 feet)

** Based on proposed Ramp A Stationing

***completed for adjacent wall 7D

Copies of the Soil Boring Logs are provided in **Appendix C**.

The soil borings were drilled using truck mounted Diedrich D-50 and B-57 Mobile (hammer efficiency 99.5 and 89.0%) drill rigs 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 boreholes were 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 borings 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.

The surface elevation of the borings was between 525.4 and 525.8 feet. The borings initially noted 4 inches of asphalt underlain by 8 inches of aggregate subbase. Below the pavement materials, the borings encountered loose to medium dense brown and gray sand/gravel to elevations between 517.0 and 512.3 feet. Boring RWB-57 then noted a layer of soft brown and gray silty clay to elevations between 509.5 and 507.3 feet. Below this cohesive material, the borings then encountered loose to very dense gray gravel and weathered limestone to the boring termination depth (auger refusal) at elevations between 504.4 and 503.8 feet. Cobbles were noted throughout the borings.

The upper native brown and gray sand/gravel had SPT blow count (N) values ranging from 4 to 24 blows per foot (bpf). The native cohesive silty clay soils had an unconfined compressive strength value of 0.4 tsf. The lower native gravel had SPT blow count (N) values of 5 bpf to 50 blows per 2 inches.

2.4 Subsurface Bedrock Conditions

A 10-foot bedrock core was collected upon encountering bedrock in boring RWB-58. 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**. A photograph of the rock core is included with the soil boring 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 core 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-58	1	21.0-26.0	Limestone	45.0	Poor	30.0 / 10,223
	2	26.0-31.0	Limestone	83.0	Good	

The soil boring logs provides bedrock conditions encountered at the boring locations. The bedrock core consisted of limestone that was slightly weathered and slightly fractured. RQD values ranged from 45.0 to 83.0 percent: Poor to Good 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 encountered at boring RWB-58 during drilling at a depth of 16 feet below grade (Elev. of 509.4 feet) within the gravel deposit. Groundwater was not encountered in boring RWB-57 either while drilling or after drilling. The borings were not left open after leaving the site due to safety concerns.

Based on the color change of the soils from brown to gray, GSG estimates the long-term groundwater level to be at a depth of 16.0 feet below existing grade (elev. of 509.6 feet). Perched water may be present within the upper granular soil 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 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 Preliminary Embankment Settlement

It is anticipated that new fill soils will be required to construct the proposed wall and embankment. Up to 28 feet of new fill may be required to construct the new embankment.

The proposed new embankment behind the proposed wall was evaluated with respect to settlement. Based on the proposed embankment heights of 28 feet, analyses were performed at the borings 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

Boring Number	Structure Stations *	Embankment Height (ft)	Anticipated Settlement (inches)
RWB-57	Sta. 612+92.72 to Sta. 613+50	28	1.0
RWB-58			

* 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 PRELIMINARY GEOTECHNICAL WALL DESIGN RECOMMENDATIONS

This section provides preliminary 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. This section discusses several earth retaining structures that could be used for the proposed project. Several typical wall types are described in the section below.

4.1.1 CIP Concrete Cantilever Walls

CIP concrete cantilever retaining walls are typically used in fill areas. They are constructed with a footing that extends laterally both in front of and behind the wall. They can be designed to resist horizontal loading with or without tie-backs by changing the geometry of the foundation. This type of wall typically requires that the area behind the wall be excavated to facilitate construction or are constructed where new fill embankments are necessary.

The advantages of a CIP wall include that it is a conventional system with well-established design procedures and performance characteristics; it is durable; and it has the ability to easily be formed, textured, or colored to meet aesthetic requirements. Disadvantages include a relatively long construction period due to undercutting, excavation, form work, steel placement, and curing of the concrete. This wall system is also sensitive to total and differential settlements.

4.1.2 Mechanically Stabilized Earth Walls

An MSE wall is typically associated with fill wall construction and consists of facing such as segmental precast units, dry block concrete or CIP concrete facing units connected to horizontal steel strips, bars or geosynthetic to create a reinforced soil mass. The reinforcement is typically placed in horizontal layers between successive layers of granular backfill. A free draining backfill is required to provide adequate performance of the wall. MSE walls can be used in cut situations as well. The additional cost of the excavations for an MSE wall is usually offset by the savings in construction costs and schedule as compared to a CIP wall on spread footings.

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. Facing panels can be designed for various architectural finishes.

The design of MSE walls for internal stability is the Contractor's responsibility and will need to be designed by a licensed Structural Engineer in the State of Illinois. The length of the reinforced soil mass from the outside face should be a minimum of 8 feet, but not less than 70% of the wall height. The length should be determined to satisfy eccentricity and sliding criteria and provide adequate length to prevent structural failure with respect to pullout and rupture of reinforcement. The MSE wall could be designed using a unit weight of 120 pcf and a friction angle of 34 degrees for the reinforced backfill soil.

4.1.3 Prefabricated Modular Gravity Walls

This type of wall typically consists of interlocking soil or rock-filled concrete, steel, or wire modules or bins (such as gabions). The combined weight of the wall materials resists the lateral loads from the soil embankment being retained. This type of wall may be used where conventional reinforced concrete walls are also being considered but are typically selected when the overall wall height will be less than 25 feet.

The advantage of this type of wall is that less select fill is required for the backfill behind the wall and the construction is relatively more economical compared to other wall types; however, this type of wall may require additional soil excavation for placement of the modules. The additional cost of the excavations could be offset by the savings in construction costs and schedule as compared to other walls.

4.1.4 Recommended Wall Type

GSG understands that a MSE Wall will be selected for the design of the retaining wall the project. The retaining wall is considered a "fill" wall. 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 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 - 10.0 (525.0 - 516.0)	Loose to Medium Dense Brown and Gray Sand/Gravel	0.22	4.60	0.36
10.0 - 16.0 (516.0 - 510.0)	Loose to Very Dense Brown Clayey Gravel	0.26	3.85	0.41
16.0 - 18.5 (510.0 – 507.5)	Soft Brown and Gray Silty Clay	0.36	2.77	0.53
18.5 - 21.0 (507.5 - 505.0)	Loose to Very Dense Gray Gravel	0.20	5.04	0.33
21.0 - 31.0 (505.0 - 495.0)	Gray Limestone	0.17	5.83	0.29

*Based on assumed ground elevation = 526.0 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 sand/gravel. 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** presents the proposed bearing elevation and recommended bearing resistances of suitable materials to support the wall system.

Table 10 – Recommended Bearing Resistance for Retaining Wall

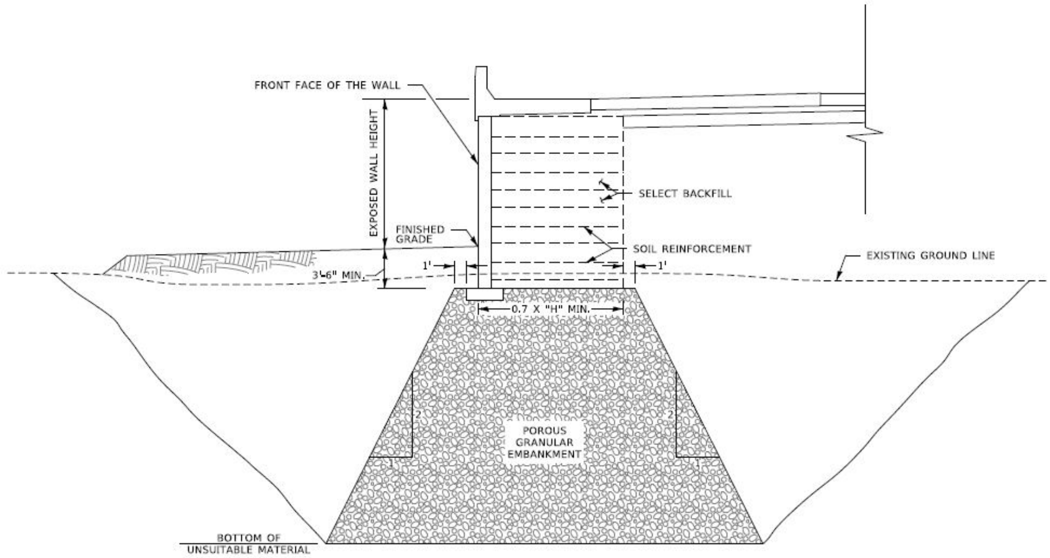
Stations	Approx. Bearing Elevation (feet)	Nominal Resistance (ksf)	Factored Bearing Resistance (ksf)	Bearing Resistance for 1-inch Settlement Service Limit (ksf)	Bearing Resistance for 2-inch Settlement Service Limit (ksf)	Anticipated Bearing Soil
612+92.72 to 615+64.250	519.5 to 521.5	29.7	19.3	3.5	6.5	Native Sand/Gravel
615+64.25 to 616+36.00	519.5 to 535.5	18.0	11.7	3.5	6.5	New Engineered Fill or Native Sand/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. Once additional borings are completed along the remaining section of the alignment, these recommendations should be reviewed and reevaluated.

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.



NOT TO SCALE

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 wall must be wide enough to resist overturning forces, and 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 and embankment 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 resistances as noted in **Table 10** are used, settlement of the retaining wall will be on the order of 1 to 2 inches. Settlement of the embankment behind the wall could be on the order of 1 to 2 inches. 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 preliminary 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 preliminary wall to reach a minimum Factor of Safety of 1.5.

Table 11 – MSE Wall Description
*Based on preliminary drawings provided

Description	Value at Station	
	613+80	614+00
Maximum total retained height of retaining wall (H), feet	22.0	22.0
Minimum length of reinforcement 0.7XH or 8.0 feet*	16.0	16.0
Unit weight of the retained soil (embankment), pcf	120	120
Unit weight of the reinforced soil mass, pcf	120	120
Assumed bearing elevation, feet	521.5	521.5

*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 preliminary 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 Station 613+80 and station 614+00. 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 613+80	MSE Wall (1V:3H slope above the wall)	Circular – Short Term	2.0	1.5
Exhibit 2			Circular – Long Term	2.0	1.5
Exhibit 3	Station 614+00	MSE Wall	Circular – Short Term	2.4	1.5
Exhibit 4			Circular – Long Term	2.4	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, pavements, 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 TSL plan, several water lines, a gas line and 48-inch diameter sewer run perpendicular to the proposed wall and embankment. The depth, size and location of these line are not known at this time. 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 to 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 color change of the soils from brown to gray, GSG estimates the long-term groundwater level to be at a depth of 16.0 feet below existing grade (elev. of 509.6). 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 to 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.

5.6 Temporary Earth Structure Lateral Earth Pressures

It is anticipated that staged construction will be utilized for construction of the new ramp and wall; therefore, a temporary soil retention system (TSRS) is anticipated to be required. The Temporary Soil Retention System shall be designed by an Illinois licensed structural engineer in accordance with the IDOT Bridge Design Manual, Section 3.13.1, Temporary Sheet Piling Design, Temporary Soil Retention Systems. The design of the Temporary Soil Retention System is the responsibility of the contractor.

The IDOT Temporary Sheet Piling Design procedures include limitations if the required embedment depths fall below soil layers with a Qu value larger than 4.5 tsf or N-values larger than 45 blows or rock, because the sheet piling may not penetrate these layers. Refer to the soil boring logs for the elevations to the hard stratum. If adequate retained heights cannot be obtained using the IDOT Temporary Sheet Piling Design Guide, then a Temporary Soil Retention System shall be designed by the Contractor. The Temporary Soil Retention Systems should include surcharge loads from the excavated materials, construction equipment and truck traffic as necessary. The retention system should extend to a sufficient depth below excavation bottom to provide the required lateral passive resistance if the active case is used for the design. Embedment depths should be determined based on the principles of force and moment equilibrium. The retention system should be designed for at-rest condition if the adjacent railroad embankment cannot withstand the anticipated horizontal and vertical movements of the construction excavation.

The retention system shall be designed by an Illinois licensed structural engineer in accordance with the IDOT Bridge Design Manual. The design of the temporary soil retention system (TSRS) is the responsibility of the contractor. The contractor should submit the TSRS plans to the structural design team for review prior to commencing construction of the TSRS.

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 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 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
Preliminary GPE

Benchmark: Iron Rod with Cap Sta. 67+63.19, 388.39' RT
N 1,764,330.037 and E 1,052.698.88
Elev.= 550.720

Existing Structures: None.

Traffic Control: Traffic will be detoured during construction.

Salvage: None.

∗ Measured along F.F. of Wall

DESIGN SPECIFICATIONS

2020 AASHTO LRFD Bridge Design Specifications, 9th Edition

DESIGN STRESSES

PRECAST UNITS

f'c = 4,500 psi

FIELD UNITS

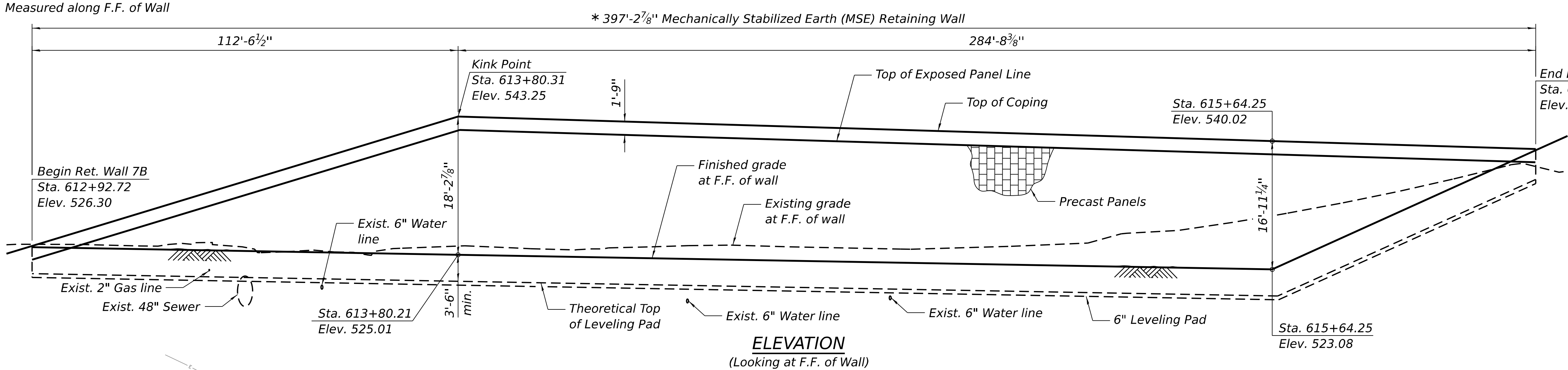
f'c = 3,500 psi
fy = 60,000 psi (Reinforcement)

HIGHWAY CLASSIFICATION

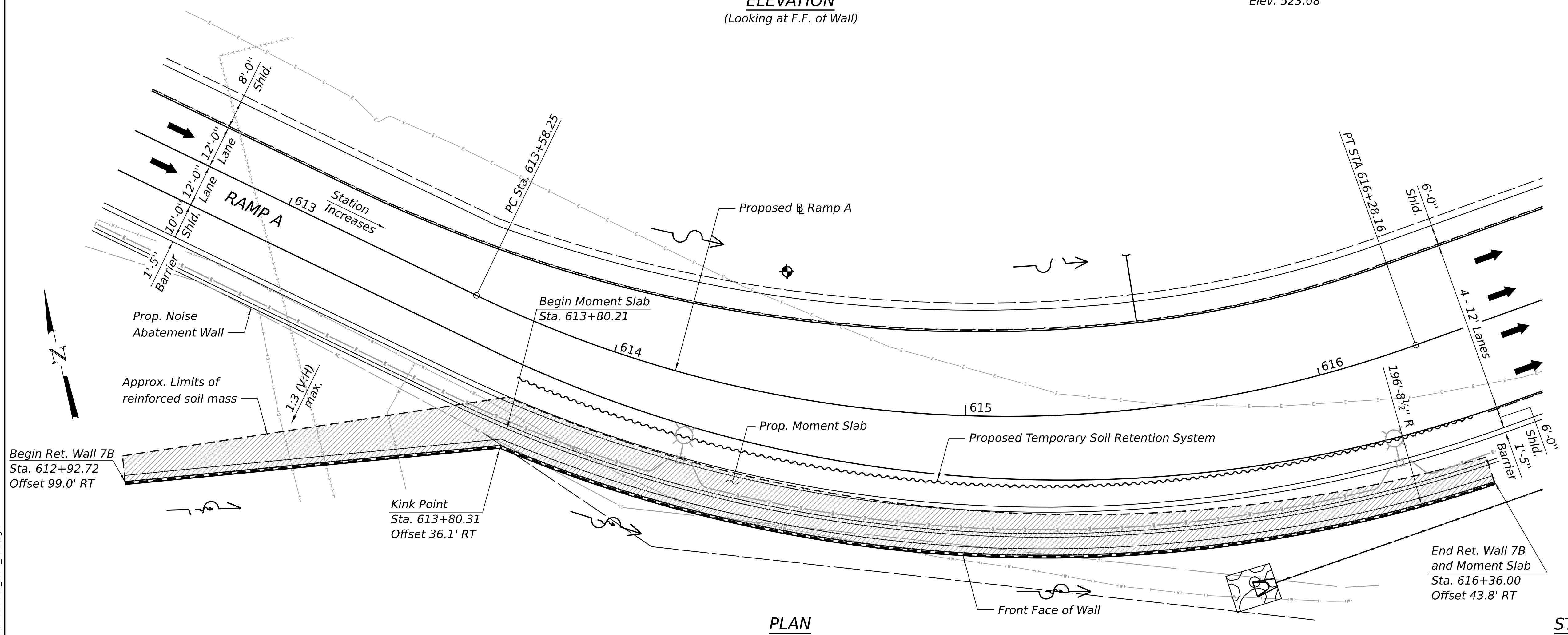
I-80 EB	Ramp A
Functional Class: Interstate	Functional Class: Interstate
ADT: 91,100 (2017); 133,500 (2040)	ADT: 4,860 (2017); 11,400 (2040)
ADTT: 19,241 (2017); 28,169 (2040)	ADTT: 810 (2017); 1,900 (2040)
Design Speed: 70 m.p.h.	Design Speed: 35 m.p.h.
Posted Speed: 65 m.p.h.	Posted Speed: 35 m.p.h.
One-Way Traffic	One-Way Traffic
Directional Distribution: 100% EB	Directional Distribution: 100%

NOTE:

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



ELEVATION
(Looking at F.F. of Wall)



PLAN

LEGEND

Soil Borings

Exist. Underground Electric

Exist. Underground Gasline

Exist. Underground Waterline

Exist. Underground Sanitary Sewer

Exist. ROW Access Control (To be removed)

Exist. ROW

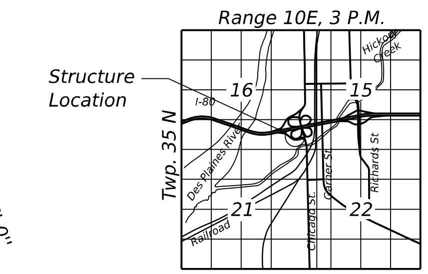
Prop. Storm Sewer

Prop. Pipe Underdrain

Prop. Temporary Soil Retention System

F.F. Front Face

B.F. Back Face



LOCATION SKETCH

GENERAL PLAN AND ELEVATION
RETAINING WALL ALONG RAMP A
F.A.I. RTE. 80 - SEC 2017-057F
WILL COUNTY
STA. 612+92.72 TO STA. 616+36.00
STRUCTURE NO. 099-W127

MODEL: Default
FILE NAME: 099XXXX-62F94-sht-TSL_W7B_001.dgn

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

USER NAME = jsandoval	DESIGNED - JS	REVISED -
	CHECKED - RRD	REVISED -
PLOT SCALE = 0:1.9999 ':" / in.	DRAWN - SVJ	REVISED -
PLOT DATE = 12/13/2022	CHECKED - 10/7/2022	REVISED -

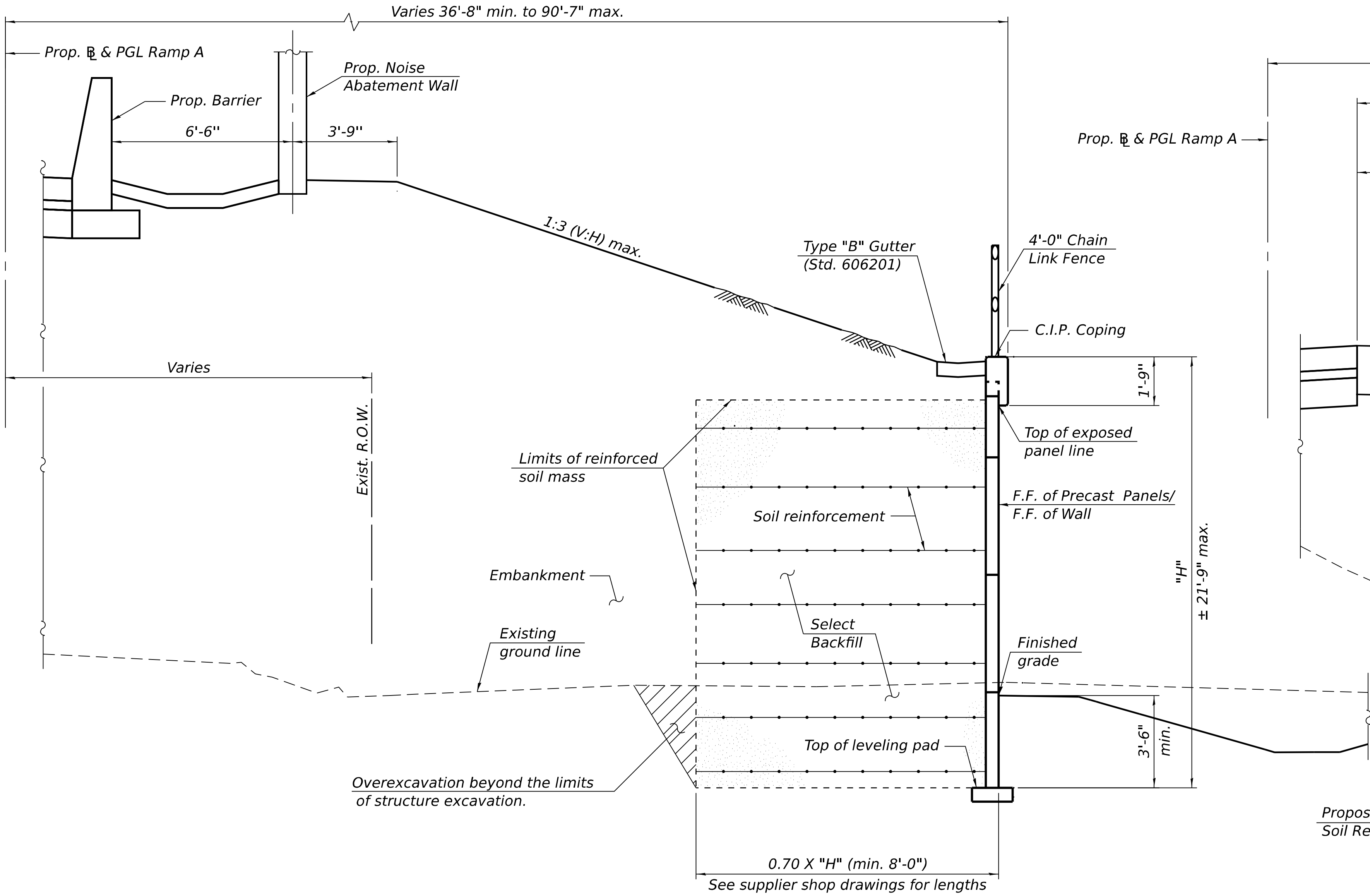
STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

GENERAL PLAN AND ELEVATION
STRUCTURE NO. 099-W127

SHEET 1 OF 2 SHEETS

F.A.I. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
80	2017-057F	WILL	2	1
		CONTRACT NO. 62F94		
	ILLINOIS	FED. AID PROJECT		

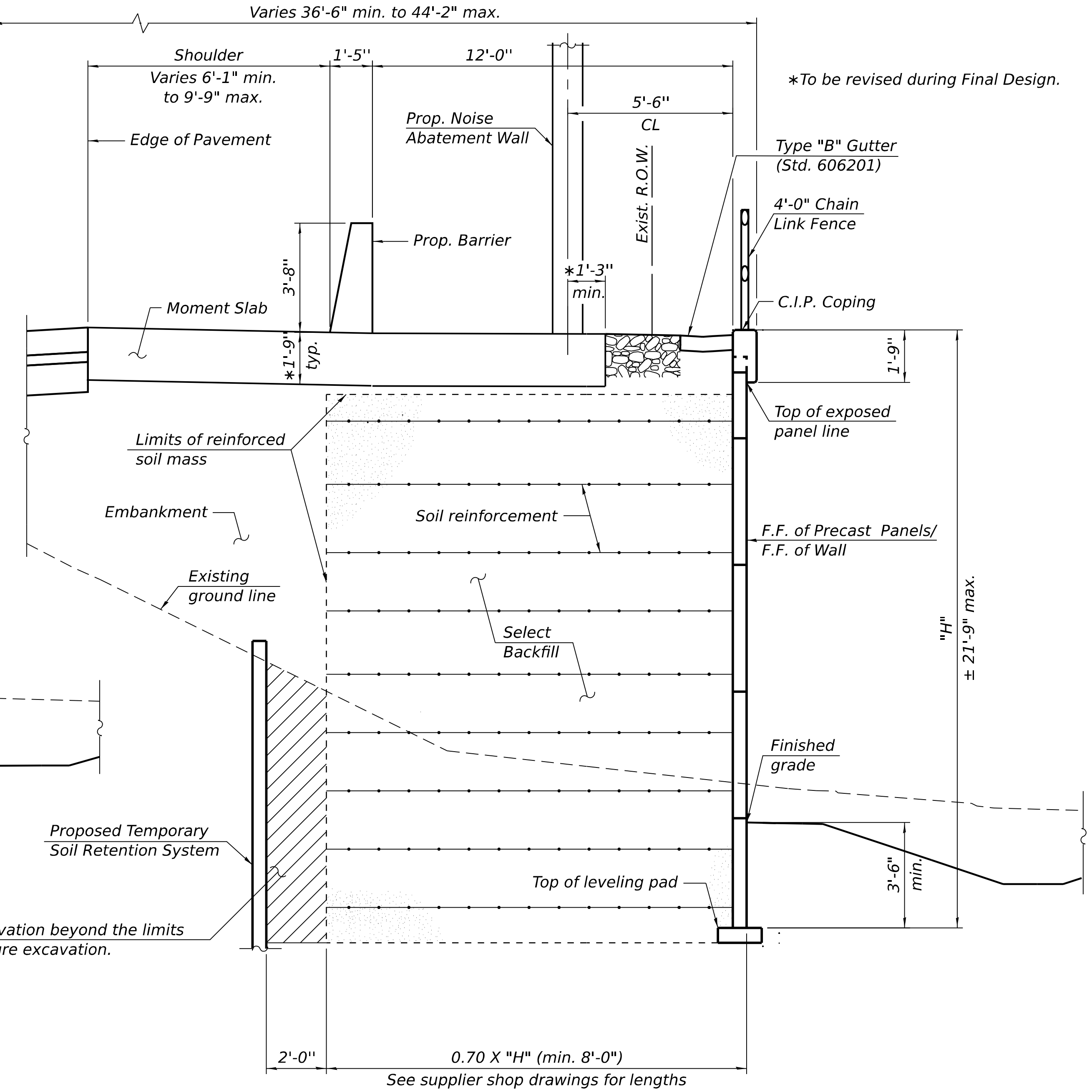
12/13/2022 3:12:53 PM



TYPICAL WALL SECTION

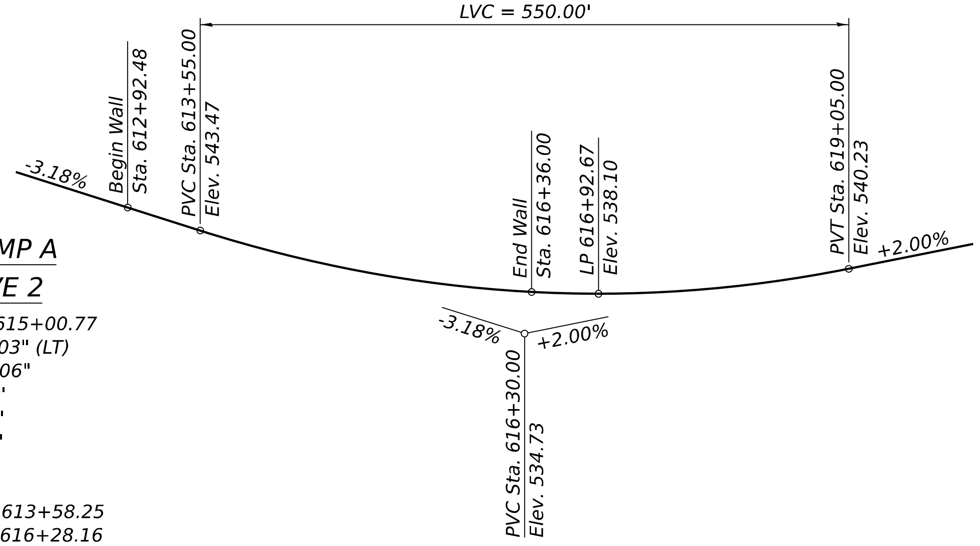
(Sta. 612+92.72 thru Sta. 613+80.21)
(Looking East)

*To be revised during Final Design.



TYPICAL WALL SECTION

(Sta. 613+80.21 thru Sta. 616+36.00)
(Looking East)



RAMP A PROFILE GRADE

(Along Prop. ℄ Ramp A)

PR RAMP A

CURVE 2

P.I. Sta. = 615+00.77
Δ = 45°29'03" (LT)
D = 16°51'06"
R = 340.00'
T = 142.42'
L = 269.91'
E = 28.66'
e = 6.0%
P.C. Sta. = 613+58.25
P.T. Sta. = 616+28.16

WALL DETAIL AND TYPICAL SECTIONS
RETAINING WALL ALONG RAMP A
F.A.I. RTE. 80 - SEC 2017-057F
WILL COUNTY
STA. 612+92.72 TO STA. 616+36.00
STRUCTURE NO. 099-W127

MODEL: Default
FILE NAME: 099XXXX-62F94-sht-TSL_W78_002.dgn

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

USER NAME = Jsandoval	DESIGNED - JS	REVISED -
	CHECKED - RRD	REVISED -
PLOT SCALE = 0:1.9999 ':" / in.	DRAWN - SVJ	REVISED -
PLOT DATE = 12/13/2022	CHECKED - 10/7/2022	REVISED -

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

WALL DETAIL AND TYPICAL SECTION
STRUCTURE NO. 099-W127

SHEET 2 OF 2 SHEETS

F.A.I. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
80	2017-057F	WILL	2	2

CONTRACT NO. 62F94

ILLINOIS | FED. AID PROJECT

12/13/2022 3:13:01 PM

Appendix B

Soil Boring Location Plan

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

USER NAME	= $USER$	DESIGNED	- ES
SHEET SIZE	= $SHEETSIZE$	DRAWN	- NN
PLOT SCALE	= $SCALE$	CHECKED	- DE
PLOT DATE	= $DATE$	DATE	- 11/21/2022

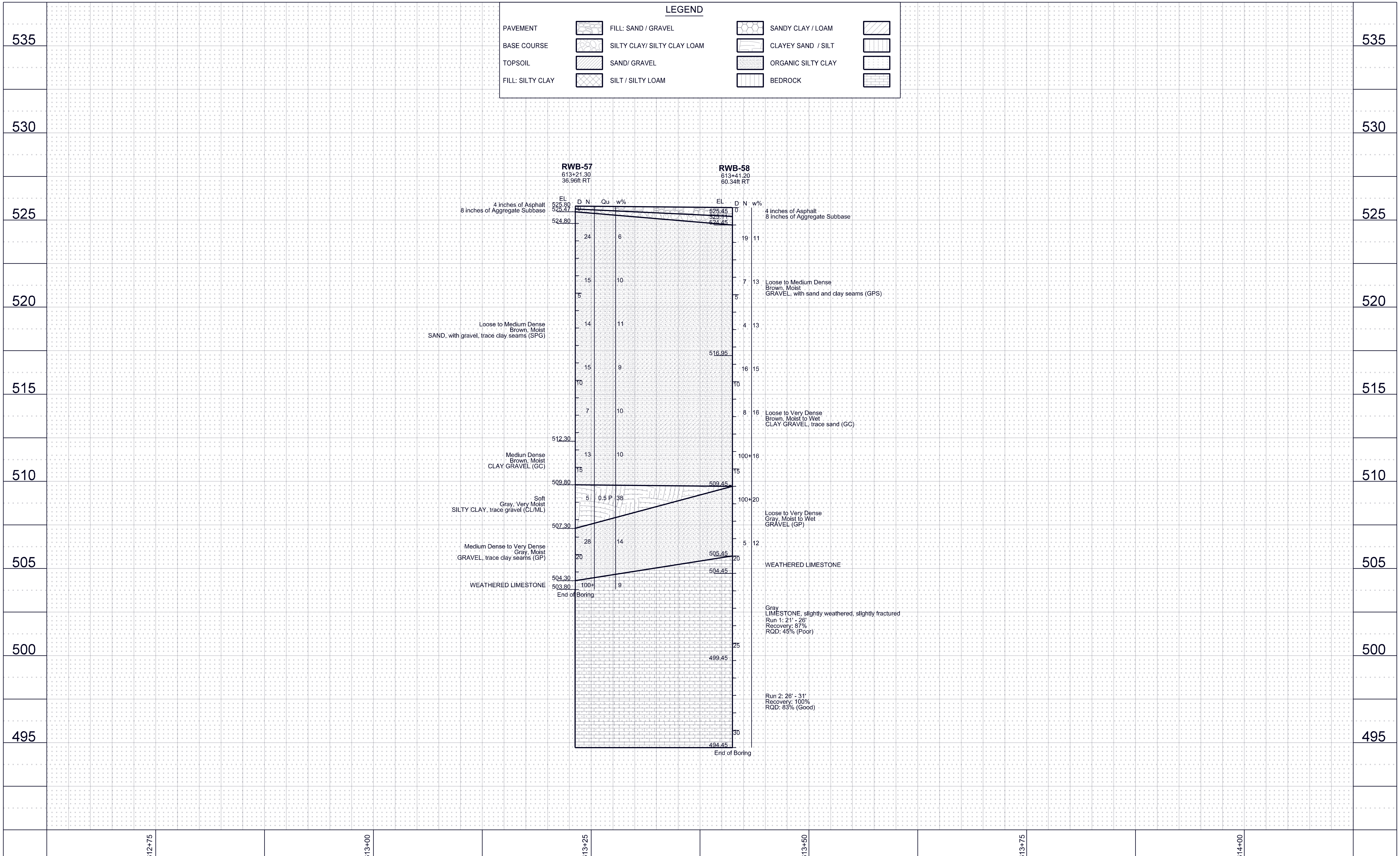
STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

RETAINING WALL NO. 7B
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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PLOT SCALE	= 1200:0.0000 ':" / ft.	CHECKED	- DE
PLOT DATE	= 1/4/2023	DATE	- 11/21/2022

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

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

RETAINING WALL NO. 7B

SOIL BORING PROFILE

SCALE: AS NOTED | SHEET 1 OF 1 SHEETS | STA. 612+75 TO STA. 614+00

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, Inc.

SOIL BORING LOG

Date 10/20/22

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** , **Longitude**

COUNTY Will **DRILLING RIG** Diedrich D-50 **HAMMER TYPE** Auto
DRILLING METHOD HSA **HAMMER EFF (%)** 95.5

STRUCT. NO. 099-W127
Station

BORING NO. RWB-57
Station 613+21.30
Offset 36.96ft RT
Ground Surface Elev. 525.80 ft

Surface Water Elev. 17.00 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	525.47				
8 inches of Aggregate Subbase	524.80				
Loose to Medium Dense Brown, Moist SAND, with gravel, trace clay seams (SPG)			28 / 14 / 10		6
			4 / 6 / 9		10
		-5			
			5 / 6 / 8		11
Cobbles at 9 feet			5 / 7 / 8		9
		-10			
			4 / 3 / 4		10
	512.30				
Medium Dense Brown, Moist CLAYEY GRAVEL (GC)			4 / 6 / 7		10
		-15			
	509.80				
Soft Gray, Very Moist SILTY CLAY, trace gravel (CL/ML)			2 / 2 / 3	0.4 P	38
	507.30				
Medium Dense to Very Dense Gray, Moist GRAVEL, trace clay seams (GP)			14 / 23 / 5		14
		-20			
Medium Dense to Very Dense Gray, Moist GRAVEL, trace clay seams (GP) *(continued)*	504.30		50/2"		
WEATHERED LIMESTONE	503.80				9
Auger refusal at 22.0 feet End of Boring					
		-25			
		-30			
		-35			
		-40			

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

Illinois Department of Transportation

Division of Highways
GSG Consultants, Inc.

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** , **Longitude**

COUNTY Will **DRILLING RIG** Mobile B-57 **HAMMER TYPE** Auto

DRILLING METHOD HSA **HAMMER EFF (%)** 89

STRUCT. NO. 099-W127
Station

BORING NO. RWB-58
Station 613+41.20
Offset 60.34ft RT
Ground Surface Elev. 525.45 ft

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

Groundwater Elev.:
First Encounter 509.4 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	525.11				
8 inches of Aggregate Subbase	524.45				
Loose to Medium Dense Brown, Moist GRAVEL, with sand and clay seams (GPS)			4 11 8		11
			2 3 4		13
		-5			
			2 2 2		13
	516.95				
Loose to Very Dense Brown, Moist to Wet CLAYEY GRAVEL, trace sand (GC)			1 5 11		15
		-10			
			3 2 6		16
			8 50/3"		16
		-15			
	509.45 ▼				
Loose to Very Dense Gray, Moist to Wet GRAVEL (GP)			50/5"		20
			2 2 3		12
	505.45	-20			
WEATHERED LIMESTONE					
	504.45				
Gray LIMESTONE, slightly weathered, slightly fractured Run 1: 21' - 26' Recovery: 87% RQD: 45% (Poor)					
		-25			
	499.45				
Run 2: 26' - 31' Recovery: 100% RQD: 83% (Good)					
		-30			
	494.45				
End of Boring					
		-35			
		-40			

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

BBS, form 137 (Rev. 8-99)

Retaining Wall #7B
Boring Number: RWB-58

Depth =21.0 ft
Elev. = 504.5 ft

Depth =26.0 ft
Elev. = 499.5 ft

Boring No.	Run	Depth (ft)	Recovery (%)	RQD (%)	RQD Classification	Description	Depth (ft)/ Unconfined Compression Strength (psi)
RWB-58	1	21’ – 26’	87.0	45.0	Poor	Gray Limestone Slightly Weathered, Slightly Fractured	30.0 / 10,223
	2	26’ – 31’	100.0	83.0	Good		

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

SOIL BORING LOG

Date 9/16/22

ROUTE I-80 DESCRIPTION Retaining Wall No. 7D LOGGED BY AA

SECTION C-91-109-22 LOCATION , SEC. 16, TWP. 35 N, RNG. 10 E, Latitude 41.51023, Longitude -88.0820664

COUNTY Will DRILLING RIG Geoprobe 7822DT HAMMER TYPE Auto
DRILLING METHOD HSA HAMMER EFF (%) 92

STRUCT. NO.
Station

BORING NO. RWB-64
Station 618+41.2529
Offset 34.94ft RT
Ground Surface Elev. 532.00 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 (%)
5 inches of Topsoil	531.58				
Brown, Moist FILL: SILTY CLAY, trace gravel			3, 2, 3	1.3 B	19
			4, 10, 4	1.8 P	15
Cobbles at 4.5 feet		-5			
			1, 1, 3	1.5 P	16
	523.50				
Very Stiff Dark Brown, Very Moist SILTY CLAY, trace gravel (CL/ML)			2, 2, 4	2.0 P	29
		-10			
	521.00				
Medium Dense to Dense Light Brown, Moist SAND, with gravel (SPG)			12, 11, 22		8
			19, 16, 9		7
		-15			
	516.00				
Dense to Extremely Dense Brown, Moist SAND, with gravel (SPG)			11, 23, 11		10
			10, 36, 50/3"		5
	512.00	-20			

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Auger refusal at 20 feet End of Boring				
	-25			
	-30			
	-35			
	-40			

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

Appendix D
Soil Parameter Table

Table D-1 – Summary of Soil Parameters

Retaining Wall #7B

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 - 10.0 (525.0 - 516.0)	Loose to Medium Dense Brown and Gray Sand/Gravel	125	0	40	0	40
10.0 - 16.0 (516.0 - 510.0)	Loose to Very Dense Brown Clayey Gravel	124	0	36	0	36
16.0 - 18.5 (511.0 – 507.5)	Soft Brown and Gray Silty Clay	118	400	0	40	28
18.5 - 21.0 (508.5 - 505.0)	Loose to Very Dense Gray Gravel	130	0	42	0	42
21.0 - 31.0 (505.0 - 495.0)	Gray 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-58		
Sample Depth (ft):	30-31		
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.*	4.4660	4.4650	4.4670	4.4660
Diameter, *in.*	1.9845	1.9865	1.9870	1.9860
Specimen Mass, *g*	608.4		Ratio (2.0-2.5)	
Bulk Density, *pcf*	167.6		2.25	

Moisture Condition - D2216

Container ID	Φ7
container, *g*	513.0
container + wet rock, *g*	980.8
container + dry soil, *g*	971.2
moisture content, *w%*	2.1

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

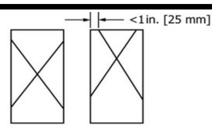
Seating Load (≤1000 psi)	1000
Rate of Loading (73-145 psi/s)	75
Time to Failure (2-15 min)	2 min 3 sec
Load @ Failure, *lbf*	31,667
Uniaxial Compressive Strength, *psi*	10,223

Remarks

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

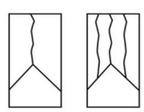
After Preparation

After Break (check applicable appearance)



Type 1
Reasonably well-formed cones on both ends, less than 1 in. [25 mm] of cracking through caps

☐

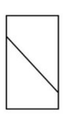


Type 2
Well-formed cone on one end, vertical cracks running through caps, no well-defined cone on other end

☐

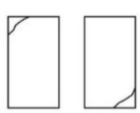
Type 3
Columnar vertical cracking through both ends, no well-formed cones

☒ X



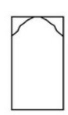
Type 4
Diagonal fracture with no cracking through ends; tap with hammer to distinguish from Type 1

☐



Type 5
Side fractures at top or bottom (occur commonly with unbonded caps)

☐



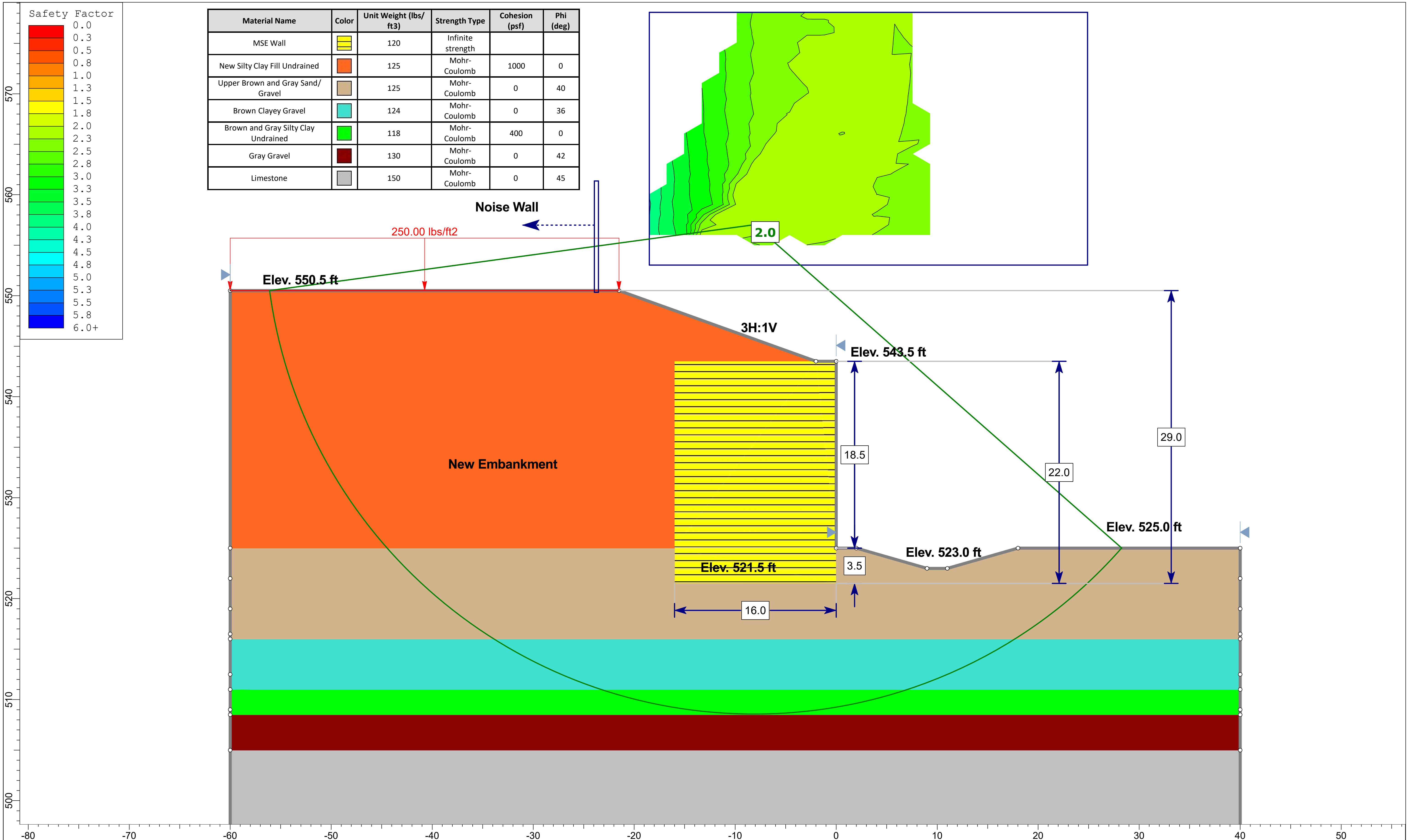
Type 6
Similar to Type 5 but end of cylinder is pointed

☐

☐ 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 (deg)
MSE Wall		120	Infinite strength		
New Silty Clay Fill Undrained		125	Mohr-Coulomb	1000	0
Upper Brown and Gray Sand/Gravel		125	Mohr-Coulomb	0	40
Brown Clayey Gravel		124	Mohr-Coulomb	0	36
Brown and Gray Silty Clay Undrained		118	Mohr-Coulomb	400	0
Gray Gravel		130	Mohr-Coulomb	0	42
Limestone		150	Mohr-Coulomb	0	45

GSG CONSULTANTS, INC.
735 Remington Road, Schaumburg, IL 60173
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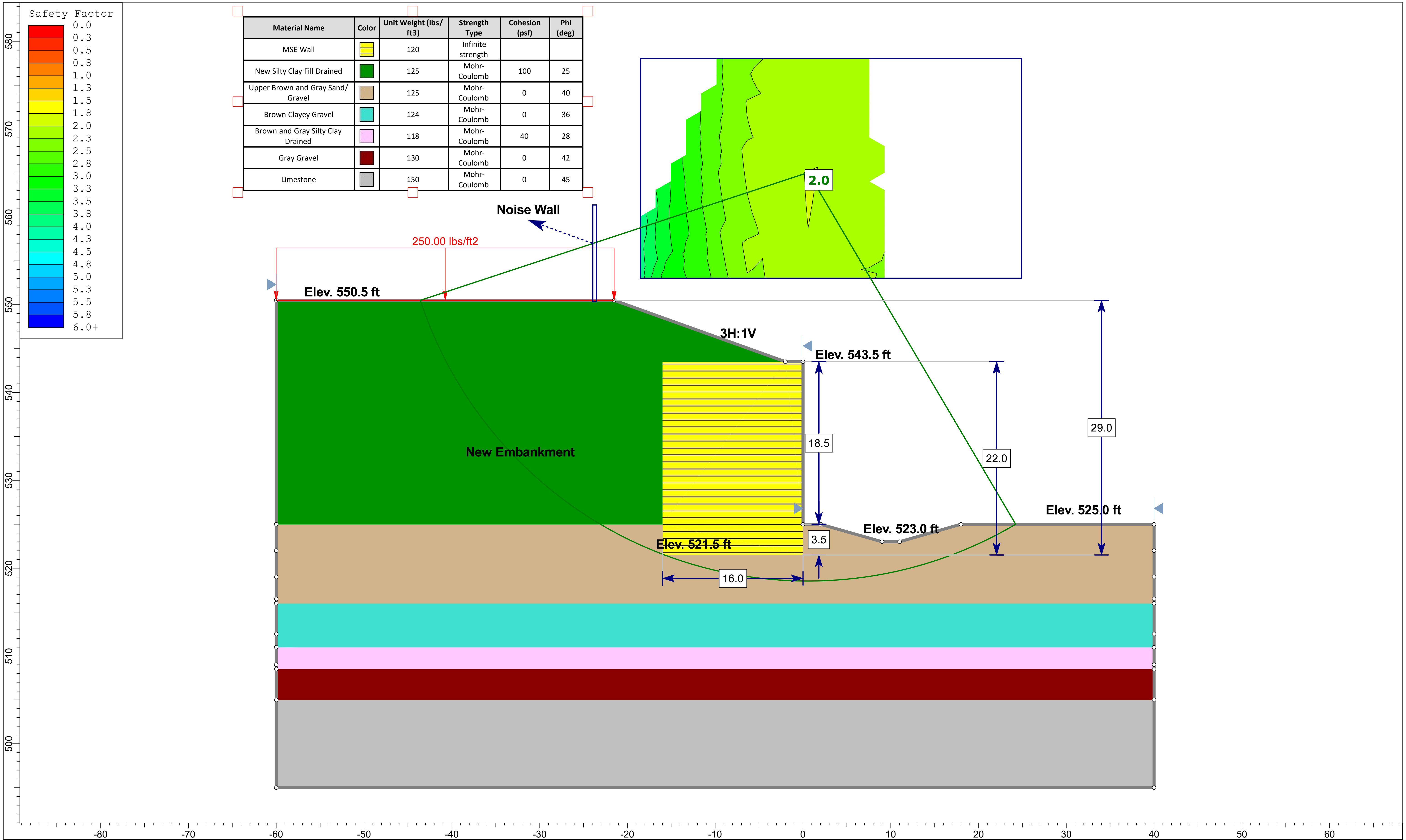
SLIDEINTERPRET 9.018

Project	I-80: IDOT PTB 198-003 - Retaining Wall #7B - MSE Wall - Station 613+80
Analysis Description	Exhibit 1 - Circular Failure Short Term
Drawn By	ES
Company	GSG Consultants, Inc.
Date	1/5/2023
File Name	MSE Wall 613+80_v1.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
MSE Wall		120	Infinite strength		
New Silty Clay Fill Drained		125	Mohr-Coulomb	100	25
Upper Brown and Gray Sand/Gravel		125	Mohr-Coulomb	0	40
Brown Clayey Gravel		124	Mohr-Coulomb	0	36
Brown and Gray Silty Clay Drained		118	Mohr-Coulomb	40	28
Gray Gravel		130	Mohr-Coulomb	0	42
Limestone		150	Mohr-Coulomb	0	45



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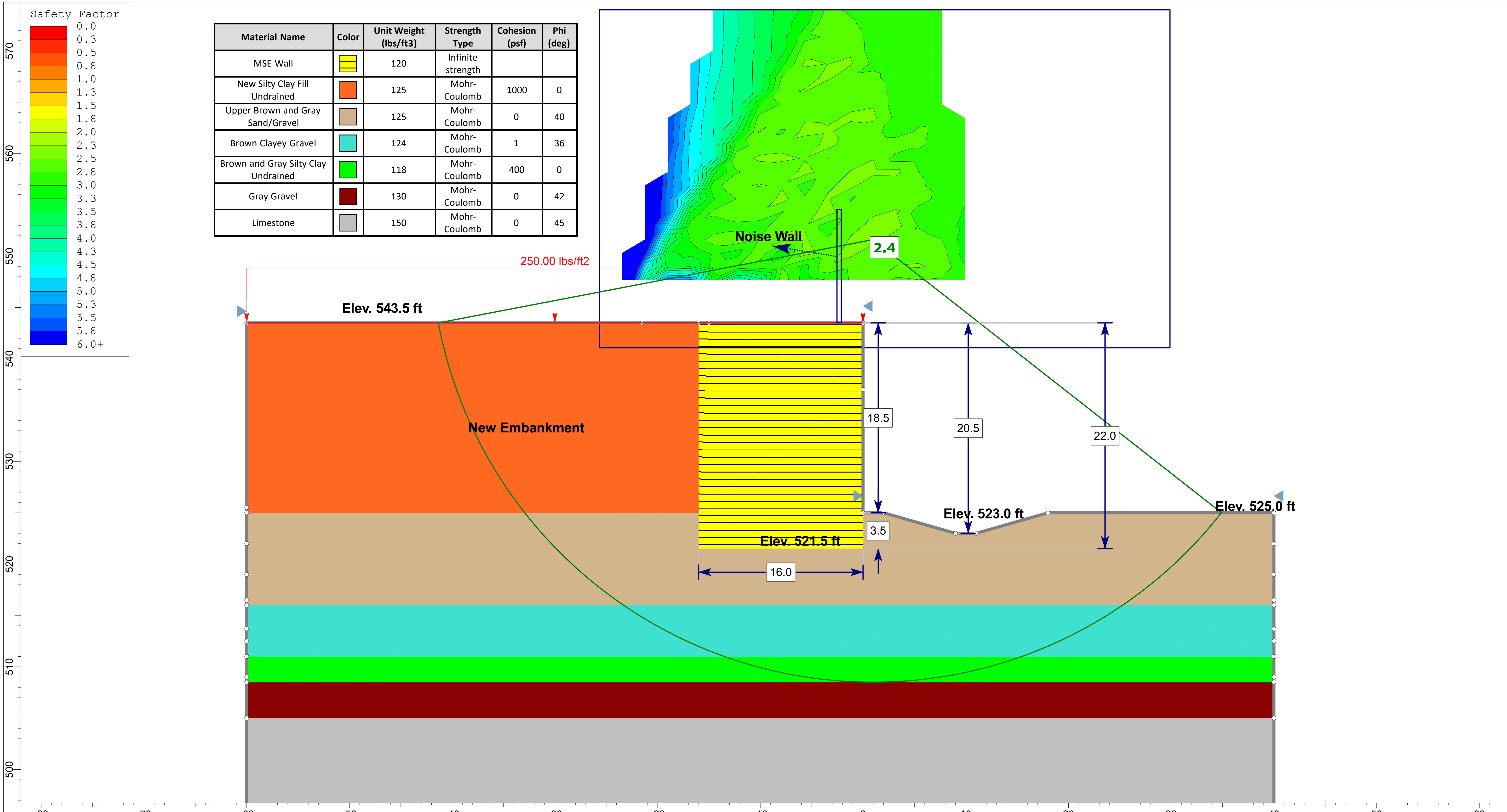
Project	I-80: IDOT PTB 198-003 - Retaining Wall #7B - MSE Wall - Station 613+80
Analysis Description	Exhibit 4 - Circular Failure Long Term

Drawn By	ES	*Company*	GSG Consultants, Inc.
Date	1/5/2023	*File Name*	MSE Wall 613+80_v1.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
MSE Wall		120	Infinite strength		
New Silty Clay Fill Undrained		125	Mohr-Coulomb	1000	0
Upper Brown and Gray Sand/Gravel		125	Mohr-Coulomb	0	40
Brown Clayey Gravel		124	Mohr-Coulomb	1	36
Brown and Gray Silty Clay Undrained		118	Mohr-Coulomb	400	0
Gray Gravel		130	Mohr-Coulomb	0	42
Limestone		150	Mohr-Coulomb	0	45

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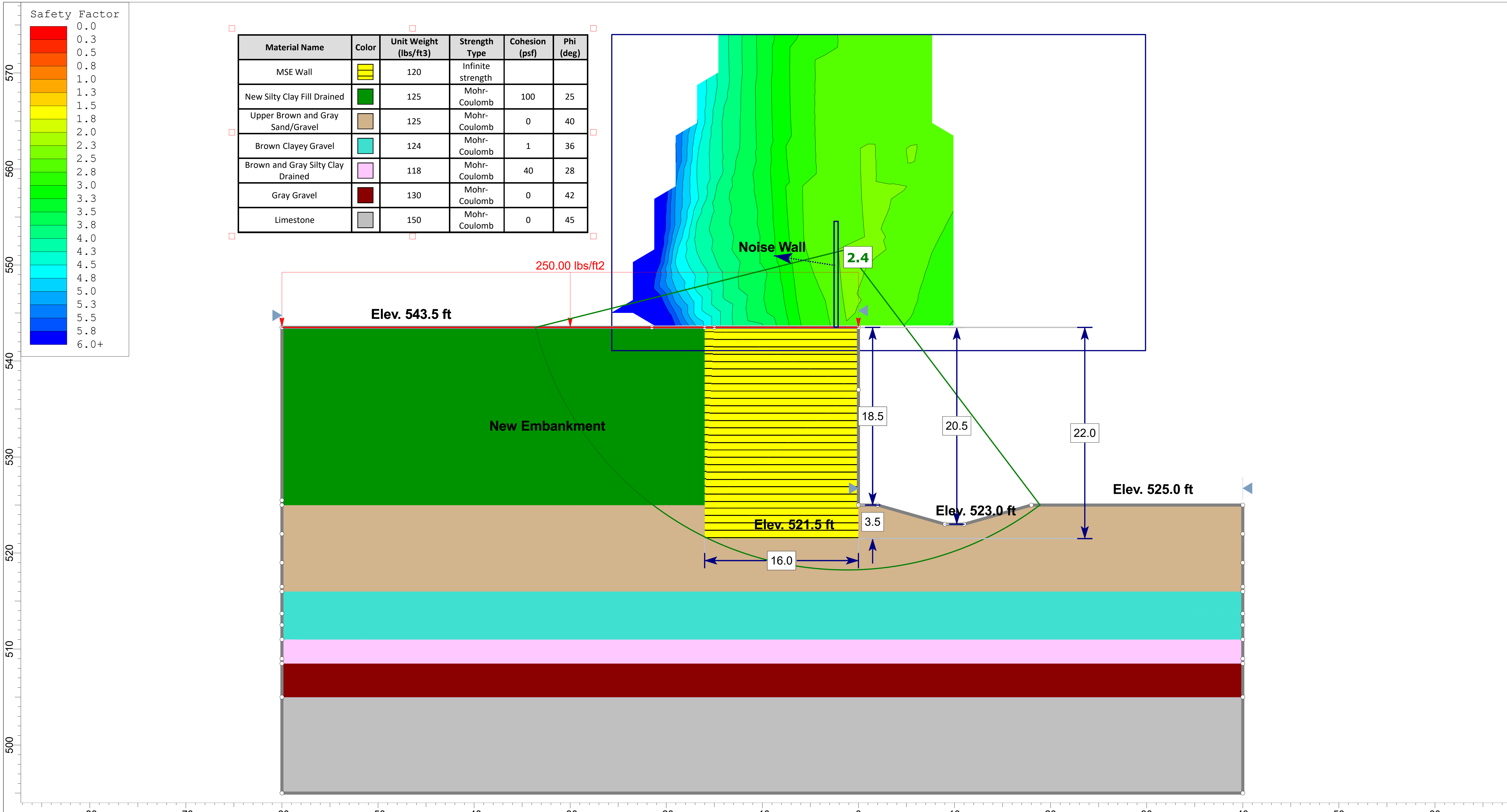
SLIDEINTERPRET 9.012

Project	I-80: IDOT PTB 198-003 - Retaining Wall #7B - MSE Wall - Station 614+00
Analysis Description	Exhibit 3 - Circular Failure Short Term
Drawn By	ES
Company	GSG Consultants, Inc.
Date	1/4/2023
File Name	MSE Wall 614+00_v1.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
MSE Wall		120	Infinite strength		
New Silty Clay Fill Drained		125	Mohr-Coulomb	100	25
Upper Brown and Gray Sand/Gravel		125	Mohr-Coulomb	0	40
Brown Clayey Gravel		124	Mohr-Coulomb	1	36
Brown and Gray Silty Clay Drained		118	Mohr-Coulomb	40	28
Gray Gravel		130	Mohr-Coulomb	0	42
Limestone		150	Mohr-Coulomb	0	45

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SLIDEINTERPRET 9.012

Project	I-80: IDOT PTB 198-003 - Retaining Wall #7B - MSE Wall - Station 614+00		
Analysis Description	Exhibit 4 - Circular Failure Long Term		
Drawn By	ES	*Company*	GSG Consultants, Inc.
Date	1/4/2023	*File Name*	MSE Wall 614+00_v1.slmd