Structural Geotechnical Report
IDOT PTB 198-003
FAI-80 (I-80) over Des Plaines River
Proposed Retaining Wall #6 along Center Street
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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September 29, 2022

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Structural Geotechnical Report
PTB 198-003, FAI-80 (I-80) over Des Plaines River
Proposed Retaining Wall #6 along Center Street
Will County, IL

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 #6 and embankment included advancing eight (8) soil borings to depths ranging from 4.5 to 20 feet and five (5) rock cores.

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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1.0 INTRODUCTION

GSG Consultants, Inc. (GSG) completed a geotechnical investigation for the proposed Retaining Wall #6 and associated embankment to be constructed as part of the I-80 improvements 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.

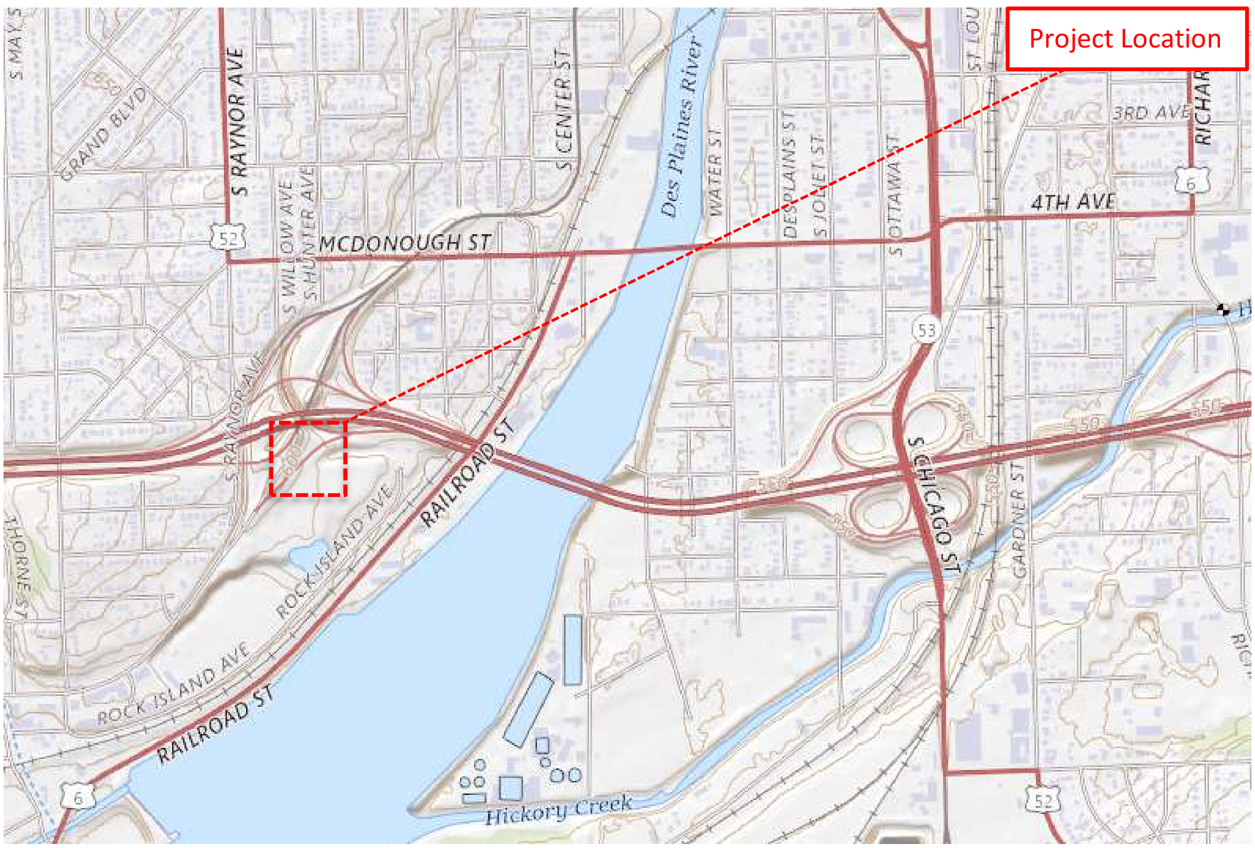


Exhibit 1 – Project Location Map
(Source: USGS Topographic Maps, usgs.gov)

1.1 Existing Conditions

The overall proposed improvements at this location will include the realignment of Center Street and the entrance/exit ramps for I-80 as part of the relocation of the Center Street bridge. The realignment of Center Street Ramp will require a retaining wall for the construction of a new embankment based on the existing IDOT right of way. According to the proposed Phase 1 plan drawings provided, the proposed retaining wall will be in a "fill" section. There is an existing storm sewer along the Center Street ramp, perpendicular to the proposed retaining wall. **Exhibit 2a and 2b** shows the existing conditions where the proposed retaining wall and embankment will be constructed.

Exhibit 2a – Existing Center Street Exit Ramp to EB I-80, Looking North

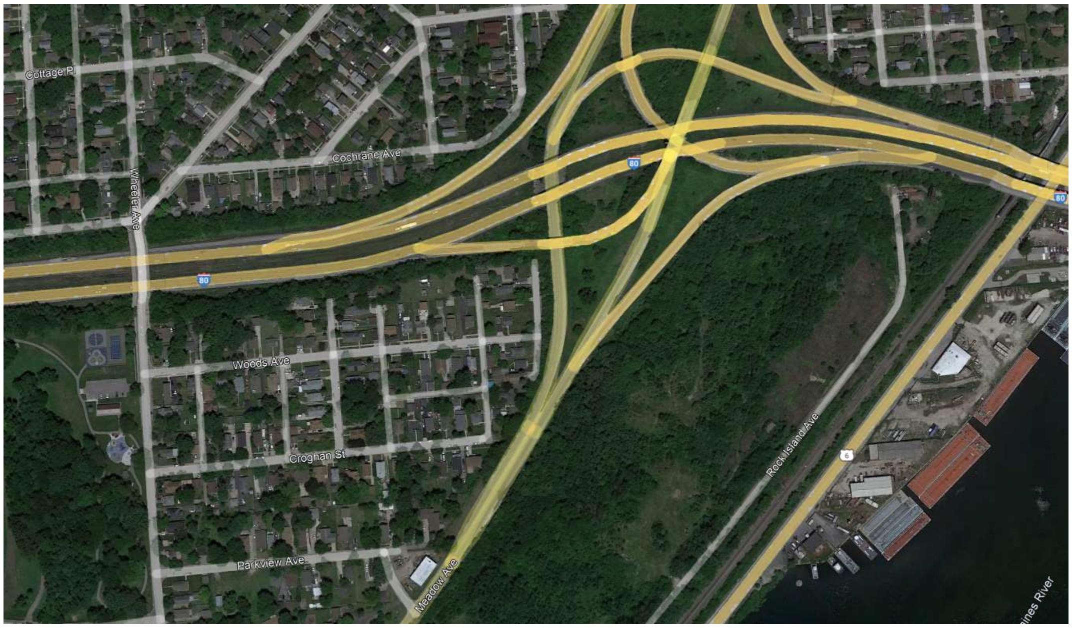


Exhibit 2b – Existing Retaining Wall Location, Looking from Top

1.2 Proposed Retaining Wall Information

Based on preliminary design information provided by WSP (**Appendix A**), a new bridge will be constructed to carry Center Street over I-80 and I-80 eastbound ramp. A new retaining wall and embankment will be constructed due to the realignment of Center Street and the entrance/exit ramps for I-80. Based on the preliminary design information and on the site topography, the proposed wall will be in a “fill” section. It is anticipated that the proposed wall will have a maximum exposed height of approximately 19 feet, for a maximum total height of 22.5 feet. The proposed embankment for the ramp leading to the new bridge will have a maximum height of up to approximately 40 feet. At the highest point of the embankment, near the bridge, in order to limit the overall height of the wall, the proposed wall will be constructed within the embankment and includes a 3H:1V slope below the wall for the lower embankment support. It is anticipated that a new detention pond may be constructed at the base of the slope. The southern portion of the embankment/retaining wall is anticipated to be supported near existing grade, with minimal grading in front of the wall. The proposed retaining wall and embankment will be approximately 460 feet in length along Center Street between Sta. 23+50 and Sta. 28+00. It is anticipated that the proposed structure will consist of a MSE wall.

Table 1 presents a summary of the proposed retaining wall and embankment.

Table 1 – Preliminary Improvement Summary

Structure Name	* Wall Stations	Approximate Length (ft)	Maximum Anticipated Exposed Wall Height (ft)	Maximum Anticipated Embankment Height (ft)
Retaining Wall #6	23+50 to Sta. 28+00	450	19	35
Embankment	23+50 to Sta. 28+00	450	n/a	40

* Based on proposed Ramp 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 locations and depths of the soil borings were selected in accordance with IDOT requirements. The borings were completed in the field based on field conditions and accessibility.

2.1 Subsurface Exploration and Laboratory Testing

The site subsurface exploration for the proposed retaining wall structure was conducted between July 7 and August 31, 2022. The investigation included advancing eight (8) borings along the proposed alignment to depths of 4.5 to 20 feet. The locations of these soil borings were adjusted in the field as necessary based on utilities and access. Elevations and as-drilled locations for the borings were gathered by GSG's field crew using GPS surveying equipment. The approximate as-drilled locations of the soil borings are shown on the Soil Boring Location Plan & Subsurface Profiles **(Appendix B)**. **Table 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-44	23+81.14	22.20 RT	1764728.162	1048519.596	13.0	592.90
RWB-45	24+48.30	28.78 RT	1764780.129	1048565.213	19.0*	590.12
RWB-46	25+8.31	35.69 RT	1764828.859	1048604.055	10.0	587.65
RWB-47	25+67.62	43.22 RT	1764879.285	1048640.32	14.0*	584.25
RWB-48	26+37.37	41.35 RT	1764945.898	1048669.427	20.0*	582.13
RWB-49	26+84.61	41.31 RT	1764991.629	1048687.6	4.5	582.13
RWB-99	27+35.73	57.43 RT	1765037.140	1048720.213	20.0*	575.54
RWB-100	28+6.39	48.17 RT	1765111.298	1048731.059	17.0*	591.27

* Depth includes Bedrock Core (10 to 15 feet)

** Based on proposed Ramp Stationing

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

The soil borings were drilled using truck mounted Diedrich D-50 TM (hammer efficiency 96.2%), Diedrich D-50 ATV (hammer efficiency 101.6%) and CME-75 (hammer efficiency 91.1%) drill rigs, each equipped with 3¼-inch I.D. hollow stem augers and an automatic hammer. Soil sampling was performed according to AASHTO T 206, "Penetration Test and Split Barrel Sampling of Soils." Soil samples were obtained at 2.5-foot intervals to the planned boring termination depths or auger refusal on bedrock. Water level measurements were made in each boring when evidence of free groundwater was detected on the drill rods or in the samples. The boreholes were also checked for free water immediately after auger removal, and before filling the open boreholes with soil cuttings.

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 & Subsurface Profiles. The soil boring logs provide specific conditions encountered at each boring location and include soil descriptions, stratifications, penetration resistance, elevations, location of the samples, and laboratory test data. Unless otherwise noted, soil descriptions indicated on boring logs are visual identifications. The stratifications shown on the boring logs represent the conditions only at the actual boring locations and represent the approximate boundary between subsurface materials; however, the actual transition may be gradual.

RWB-44 to RWB-49

The surface elevations of the borings ranged between 592.9 and 582.1 feet. The boring initially noted 2 to 3 inches of topsoil except for boring RWB-47, which was drilled in the shoulder of the existing ramp, noted 5 inches of asphalt. Below these materials, boring RWB-44 encountered silty clay loam fill to a depth of 3.5 feet followed by hard brown and gray silty clay to a depth of 6.0 feet; boring RWB-45 encountered silty clay fill to a depth of 6.0 feet; and boring RWB-49 encountered very stiff brown silty clay to a depth of 2.5 feet followed by very dense brown silty loam to a depth of 4.5 feet upon encountering auger refusal. Below these cohesive materials, and below the topsoil at the remaining locations, the borings encountered loose to very dense brown sand and gravel to the boring termination depth. All the borings encountered practical auger refusal or split-spoon refusal on bedrock at depths ranging from 4.0 to 13.0 feet below grade (elevations 570.5 to 584.3 feet).

The native and fill cohesive soils have unconfined compressive strength value ranging between 2.5 to 4.5 tsf. The native gravel and sand have an SPT blow count (N) values ranging from 5 to 100 blows per foot (bpf).

RWB-99 and RWB-100

The surface elevations of the borings were 575.5 and 591.3 feet, respectively, on the Center Street ramp and the I-80 eastbound shoulder. Boring RWB-99 initially noted 4 inches of topsoil; boring RWB-100 noted 12 inches of asphalt. Below these surface materials, boring RWB-99 encountered dark gray silty clay to a depth of 3.5 feet followed by very dense light gray gravel to a depth of 6.0 feet and then encountered auger refusal on bedrock. Boring RWB-100 encountered silty clay fill to a depth of 3.5 feet followed by stiff brown silty clay loam to a depth of 5.0 feet and very dense light brown sand until encountering auger refusal on bedrock at a depth of 7.0 feet.

The native and fill cohesive soils have unconfined compressive strength value ranging between 1.5 to 4.0 tsf. The native gravel and sand have an SPT blow count (N) values of 50 blows per 2 inches or 50 blows per 6 inches.

2.4 Subsurface Bedrock Conditions

When bedrock was encountered, the extracted bedrock cores were 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 respective 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

Where sufficient sample was obtained, laboratory compressive strength tests were completed on sections of the rock cores. **Table 4** provides a summary of the RQD values and unconfined compressive strength values 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-45	1	9.0-15.0	Limestone	8.3	Very Poor	-
	2	15.0-19.0	Limestone	0	Very Poor	-
RWB-47	1	4.0-9.0	Limestone	13.0	Very Poor	-
	2	9.0-14.0	Limestone	29.0	Poor	-
RWB-48	1	10.0-20.0	Limestone	26.6	Poor	-
RWB-99	1	5.0 – 15.0	Limestone	76.0	Good	8.0 / 8,741
	2	15.0 – 20.0	Limestone	93.0	Excellent	13.0 / 17,164
RWB-100	1	7.0 – 17.0	Limestone	6.7	Very Poor	-

The soil boring logs provide bedrock conditions encountered at each location. The bedrock cores consisted of limestone that was slightly to moderately weathered and moderately to heavily fractured. RQD values ranged from 0 to 93.0 percent: Very Poor 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 any of the borings. None of the borings were 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 borings, it is anticipated that the long-term groundwater level may be near the bedrock interface due to the proximity of the Des Plaines River. Perched water may also be present within the fill 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 Embankment Settlement

It is anticipated that new engineered fill soils will be required to construct the proposed wall and embankment. Up to 40 feet of new fill may be required to construct the proposed embankment as it approaches the new Center Street Bridge.

The proposed new embankment was evaluated with respect to settlement. Based on the proposed embankment maximum height of 40 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** where 90% of the total settlement is estimated to be completed within 3 months**.** The settlement values provided in **Table 5** do not include any potential settlement of the new constructed embankment materials as it is assumed the new embankment will be compacted and constructed per the IDOT specifications.

Table 5 – Anticipated Embankment Settlement

Boring No.	Structure Stations *	Embankment Height (ft)	Anticipated Settlement (inches)
RWB-44	23+81.14	20	1.3
RWB-45	24+48.30	20	1.0
RWB-46	25+8.31	20	<0.5
RWB-47	25+67.62	25	<0.5
RWB-48	26+37.37	25	<0.5
RWB-49	26+84.61	25	1.2
RWB-99	27+35.73	40	1.2
RWB-100	28+6.39	10	0.7

* Based on proposed Ramp 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

Building Code Reference	PGA	S_{DS}	S_{D1}
2020 AASHTO Guide for LRFD Seismic Bridge Design	0.049g	0.126g	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. 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

Based on the proposed grading plan and location of the wall within a fill area, an MSE wall may be considered for this project. Design plans indicate that the wall location would require a new embankment to reach the proposed roadway subgrade.

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 the 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)	1.50		1.00	1.00	1.00
	Active		1.50			
	At-Rest		1.35			
	AEP for anchored walls		1.35			
	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** present soil design properties for the retaining wall for the anticipated soil types at the site and provide recommended lateral soil modulus and soil strain parameters that can be used for laterally loaded pile analysis via the p-y curve method based on the encountered subsurface conditions. Additional soil design properties are included in **Table D-1 (Appendix D).**

Table 8 – Lateral Soil Parameters

Depth Range (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
RWB-44 to RWB-49				
0 – 10.0 (590.0-580.0)	Loose to Very dense brown Sand and Gravel	0.20	5.04	0.33
10.0 – 20.0 (580.0-570.0)	Gray Limestone	0.17	5.83	0.29
0 - 6.0 (591.0-585.0) RWB-44 & 45	FILL: Brown and Gray Silty Clay	0.41	2.46	0.58
0 – 3.5 (582.0-578.5) RWB-49	Very Stiff Brown Silty Clay	0.41	2.46	0.58
RWB-99 only				
0 - 3.5 (575.5-572.0)	Very Stiff Brown and Gray Silty Clay	0.41	2.46	0.58
3.5 – 5.0 (572.0-570.5)	Very Dense Gray Sand and Gravel	0.20	5.04	0.33
3.5 – 5.0 (572.0-570.5)	Gray Limestone	0.17	5.83	0.29
RWB-100 only				
1 – 3.5 (590.0-587.5)	FILL: Brown Silty Clay	0.41	2.46	0.58
3.5 – 5.0 (587.5-586.0)	Stiff Brown Silty Clay Loam	0.36	2.77	0.53
5.0 – 7.0 (586.0-584.0)	Very Dense Brown Sand	0.20	5.04	0.33
7.0 – 17.0 (584.0-574.0)	Gray Limestone	0.17	5.83	0.29

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 soils 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. This could be accomplished by placing a Geocomposite Wall Drain over the entire length of the back face of the wall connected to 6-inch diameter perforated drain pipe and backfilling a minimum of 2 feet of free draining materials, Porous Granular Embankment, as measured laterally from the back of the wall. The backfill should be placed in accordance with the IDOT SSRBC.

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

Wall Type	Stations	Assumed 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)	Bearing Resistance for 3-inch Settlement Service Limit (ksf)	Anticipated Bearing Soil
MSE Wall	23+50 to 24+00 *	601.5 to 596.0	22.9	14.8	6.0	11.0	14.0	New Engineered Fill
	24+00 to 25+00	586.5 to 596.0	27.9	18.1	6.0	11.0	14.0	
	25+00 to 28+6.39 *	586.5 to 591.0	22.9	14.8	6.0	11.0	14.0	

Note: Assumed Friction Angle of New Engineered Granular Backfill Ø = 32°;

* Assuming footing on slope with reduction factor = RC_{BC} = 0.82

The minimum depth of the wall bearing should be 3.5 feet below the final exterior grade to alleviate the effects of frost. The subgrade soils encountered at the bearing elevations should be cleared of any unsuitable material, such as topsoil. 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, undercuts may be necessary in isolated areas due to the existing fill silty clay soils at some locations prior to the construction of the new embankment. Undercut locations and depths are provided in **Table 11**. Cohesive materials exhibiting moisture contents greater than 27% and unconfined compressive strengths less than 2.0 tsf, if encountered, should be removed during construction.

Table 11 - Recommended Subgrade Undercuts

Boring #	Ground Elevation (ft)	Anticipated Bearing Elevation	Recommended Undercut Depth	Description of Soils Encountered	Recommended Treatment
RWB-45	590.0	586.5	2.5	FILL: Brown and Gray Silty Clay	Remove and replace with structural fill
RWB-100	592.5	589.0	3.0		

Undercut areas 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.

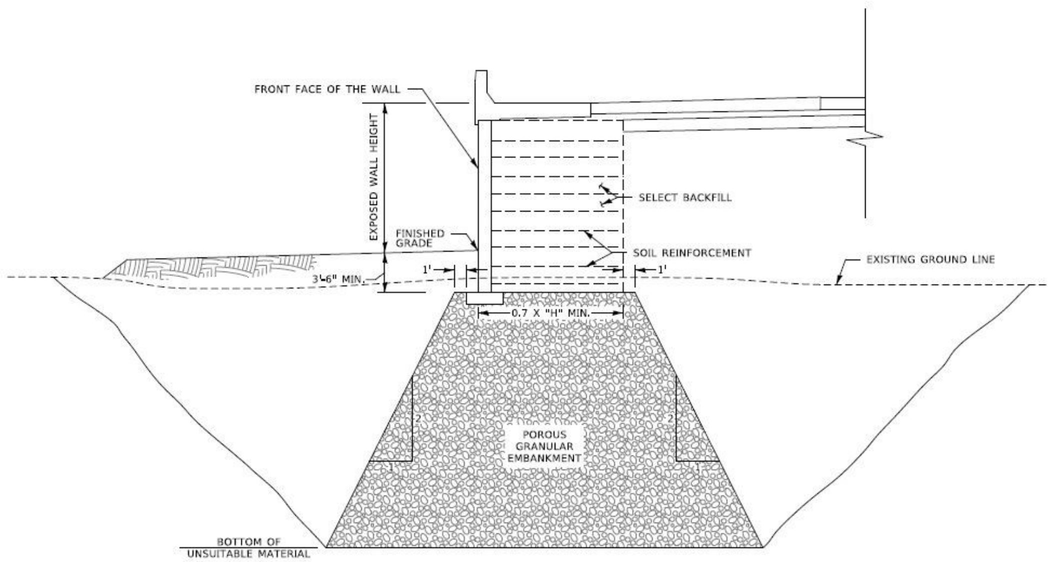


Exhibit 3 - Structural Fill Placement below MSE Wall

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 cohesive or 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 the less of the cohesion of the clay under the footing or 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 and Embankment 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 7** are used, the settlement of the retaining wall will be on the order of 1 inch. The differential settlement between two points 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 the external stability of the wall system. The parameters in **Table 12** were used to evaluate the proposed MSE wall types in order to reach a minimum Factor of Safety of 1.5.

Table 12 – MSE Wall Description

*Based on preliminary drawings provided

Description	Values	
	Station 24+50	Station 27+50
Maximum total retained height of retaining wall (H)*	21.5 feet	22.5 feet
Minimum length of reinforcement to meet FoS of 1.5	15.0 feet	19.0 feet
Unit weight of the retained soil (embankment)	125 pcf	
Unit weight of the reinforced soil mass	120 pcf	
Assumed Drained Cohesion of New Embankment (c’)	200 psf	
Assumed Drained Friction Angle of New Embankment (Ø’)	28°	
Assumed Friction Angle of Granular Backfill (Ø)	32°	
Assumed bearing elevation (feet)	586.5	590.5

The actual wall width, and total height of the wall 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 (**Tables 12**) for the proposed MSE retaining wall scenarios. The analyses were performed at Station 24+50 and 27+50 based on the overall wall heights and slope geometry. The results of the analyses are shown in **Table 13**.

Table 13 – Retaining Wall Global Slope Stability Analyses Results

Analysis Exhibit	Location	Wall Type	Analysis Type	Factor of Safety	Minimum Factor of Safety
Exhibit 1	Station 24+50	MSE Wall	Circular – Short Term	3.5	1.5
Exhibit 2			Circular – Long Term	2.2	1.5
Exhibit 3	Station 27+50		Circular – Short Term	2.3	1.5
Exhibit 4			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 drainage system to prevent the buildup of hydrostatic forces behind the wall. This could be accomplished with the installation of drainage blankets, geocomposite drainage panels, or gravel drains behind the facing of the wall with outlet pipes below the facing to collect and remove surface water away from the face of the MSE. If weep holes are to be used, it is recommended that a geocomposite wall drain to 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, including an existing storm sewer along the Center Street ramp that is perpendicular to the proposed retaining wall alignment. 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 a minimum of 2 feet of 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 to provide 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 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 fill. 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.

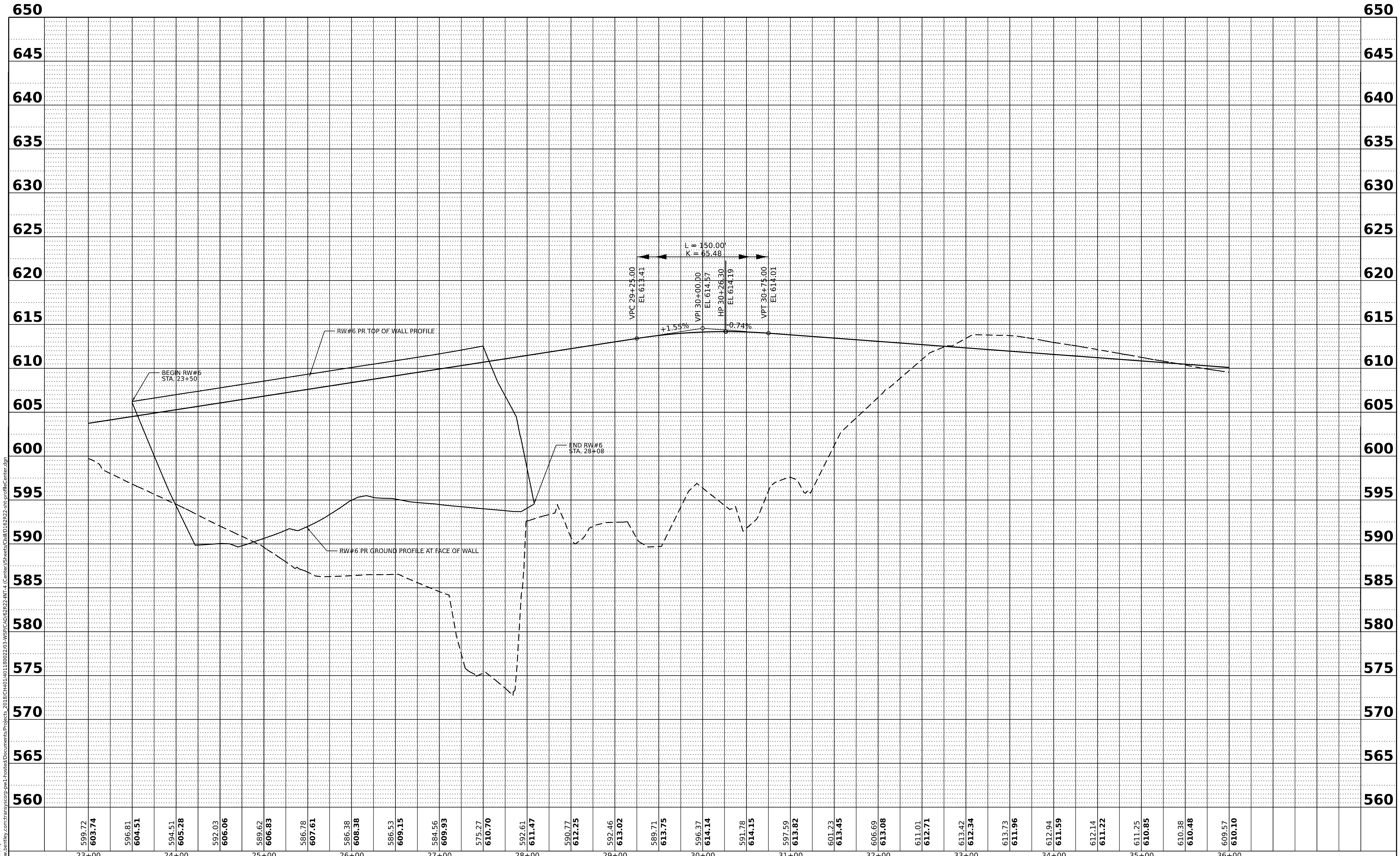
If water seepage occurs during excavations or where wet conditions are encountered such that the water cannot be removed with conventional sumping, we recommend placing open grade stone similar to IDOT CA-7 to stabilize the bottom of the excavation below the water table. The CA-7 stone should be placed 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.

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



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WSP USA Inc.
30 N. LASALLE STREET
SUITE 4200
CHICAGO, IL 60602
TEL: (312) 782-8150

USER NAME = donahuek1	DESIGNED -	REVISED -	STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION
	DRAWN -	REVISED -	
PLOT SCALE = 0.16666667 ' / in.	CHECKED -	REVISED -	
PLOT DATE = 9/12/2022	DATE -	REVISED -	

ROADWAY PROFILE
CENTER STREET

SCALE: HORIZ: 1"=50' VERT: 1"=5' | SHEET OF SHEETS | STA. TO STA.

F.A.I. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
80	FAI 80 21 INTERCHANGE	WILL	596	68
		CONTRACT NO. 62R22		
	ILLINOIS	FED. AID PROJECT		

Benchmark:	Chiseled "X" on top of SE bolt of Fire Hydrant at South ROW of Jasper St. (in front of 640 Jasper St. address), Elev. 585.86.
Existing Structure:	None.
Traffic Control:	Traffic will be detoured during construction.
Salvage:	None.

DESIGN STRESSES

PRECAST UNITS
f'c = 4,500 psi

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

DESIGN SPECIFICATIONS

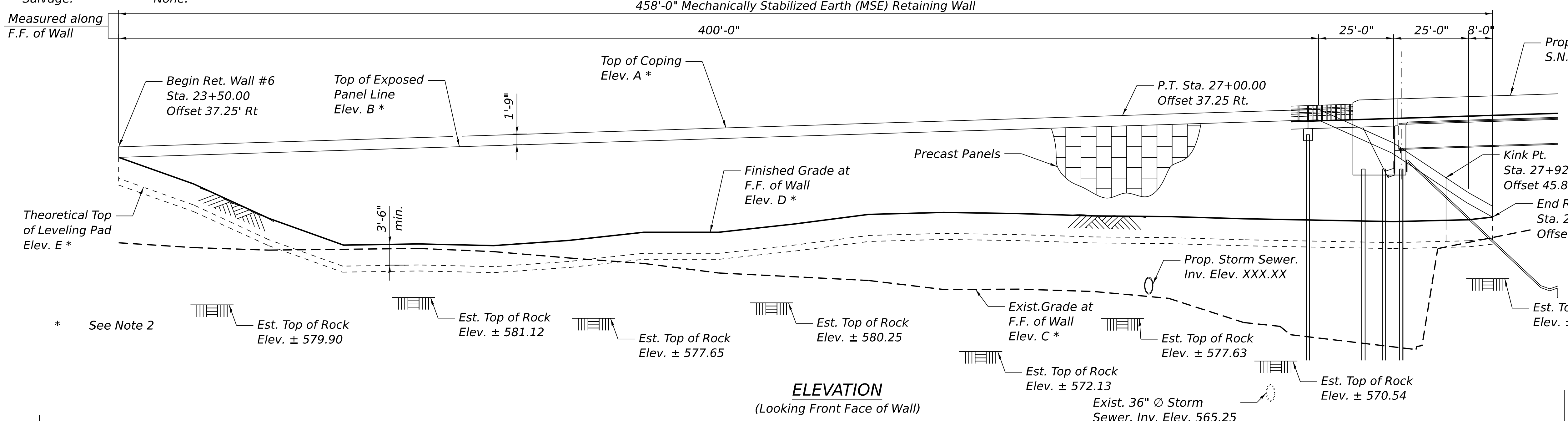
2020 AASHTO LRFD Bridge Design Specifications, 9th Edition

HIGHWAY SPECIFICATION

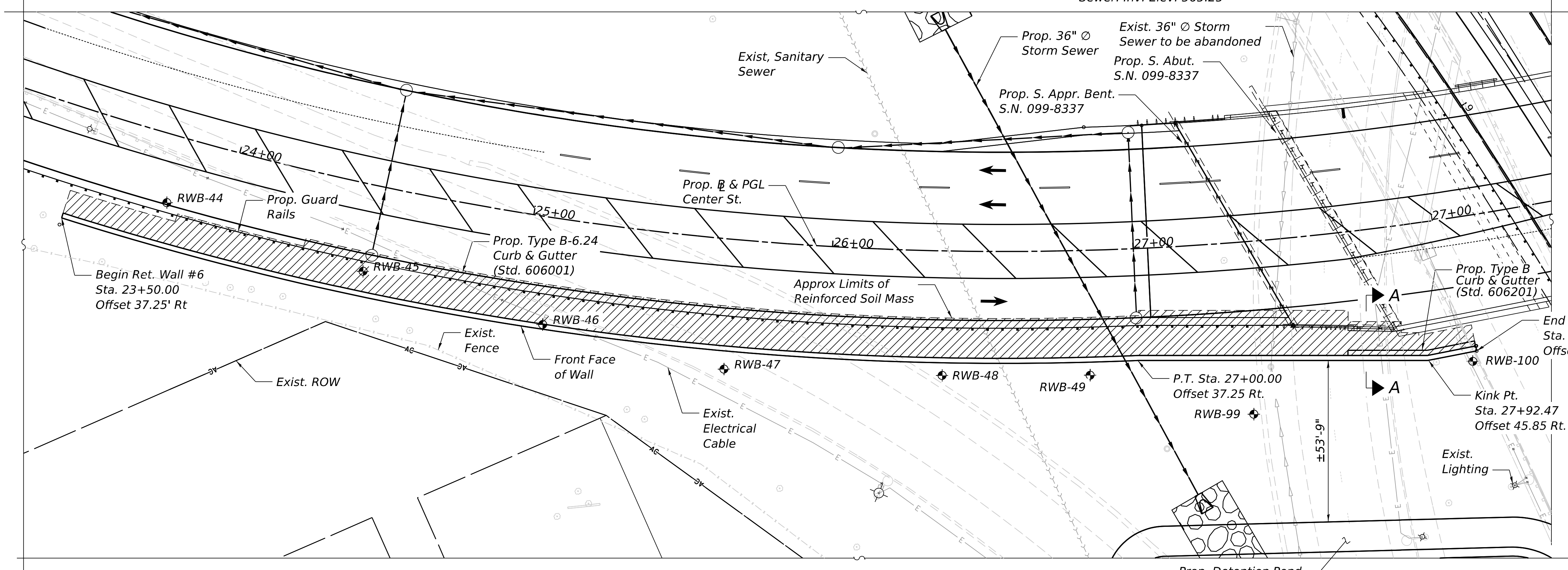
F.A.I. Rte. 80 - I-80
Functional Class: Interstate
ADT: 91,100 (2017); 133,500 (2040)
ADTT: 19,240 (2017); 28,195 (2040)
DHV: 14,685 (2032)
Design Speed: 70 m.p.h.
Posted Speed: 70 m.p.h.
2-Way Traffic
Direction Distribution: 50-50

Center Street
Functional Class: Minor Arterial
ADT: 18,900 (2019); 29,100 (2040)
ADTT: 1,266 (2019); 1,950 (2040)
DHV: 3,201 (2040)
Design Speed: 35 m.p.h.
Posted Speed: 35 m.p.h.
2-Way Traffic
Direction Distribution: 50-50

Center Street Ramp B
Functional Class: Interstate
ADT: 4,650(2017); 2,930 (2040)
ADTT: 400 (2017); 252 (2040)
DHV: 512 (2040)
Design Speed: 40 m.p.h.
Posted Speed: 40 m.p.h.
1-Way Traffic



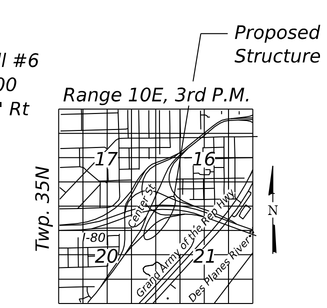
ELEVATION
(Looking Front Face of Wall)



PLAN

LEGEND

- Exist. Fence
- Exist. Electrical Cable
- Exist. Storm Sewer
- Exist. Light Pole
- Prop. Storm Sewer
- Tree
- Soil Boring
- Approx. Limits of Reinforced Soil Mass
- Exist. ROW Access Control
- Exist. Sanitary Sewer



LOCATION SKETCH

NOTES:

1. Stations and offsets are measured along the ℄ of Center Street to the front face of the precast panels.
2. For Table 1- Wall Elevations, Profile Grade Lines and Curve Data, See Sheet 2 of 3.
3. For Sections thru MSE Walls, See Sheets 2 thru 3 of 3.
4. For Section A-A, see Sheet 3 of 3.

IN PROGRESS

GENERAL PLAN & ELEVATION
RETAINING WALL 6
ALONG CENTER STREET
F.A.I. ROUTE I-80
SECTION C-91-109-22
WILL COUNTY
STA. 23+50.00 TO STA. 28+08.00
STRUCTURE NO. 099-WXXX

HBM ENGINEERING GROUP, LLC

USER NAME = prakarsh.gurmule	DESIGNED -	PG	REVISED -		
	DRAWN -	PG	REVISED -		
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STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

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

F.A.I. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
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CONTRACT NO. 62R22
ILLINOIS FED. AID PROJECT

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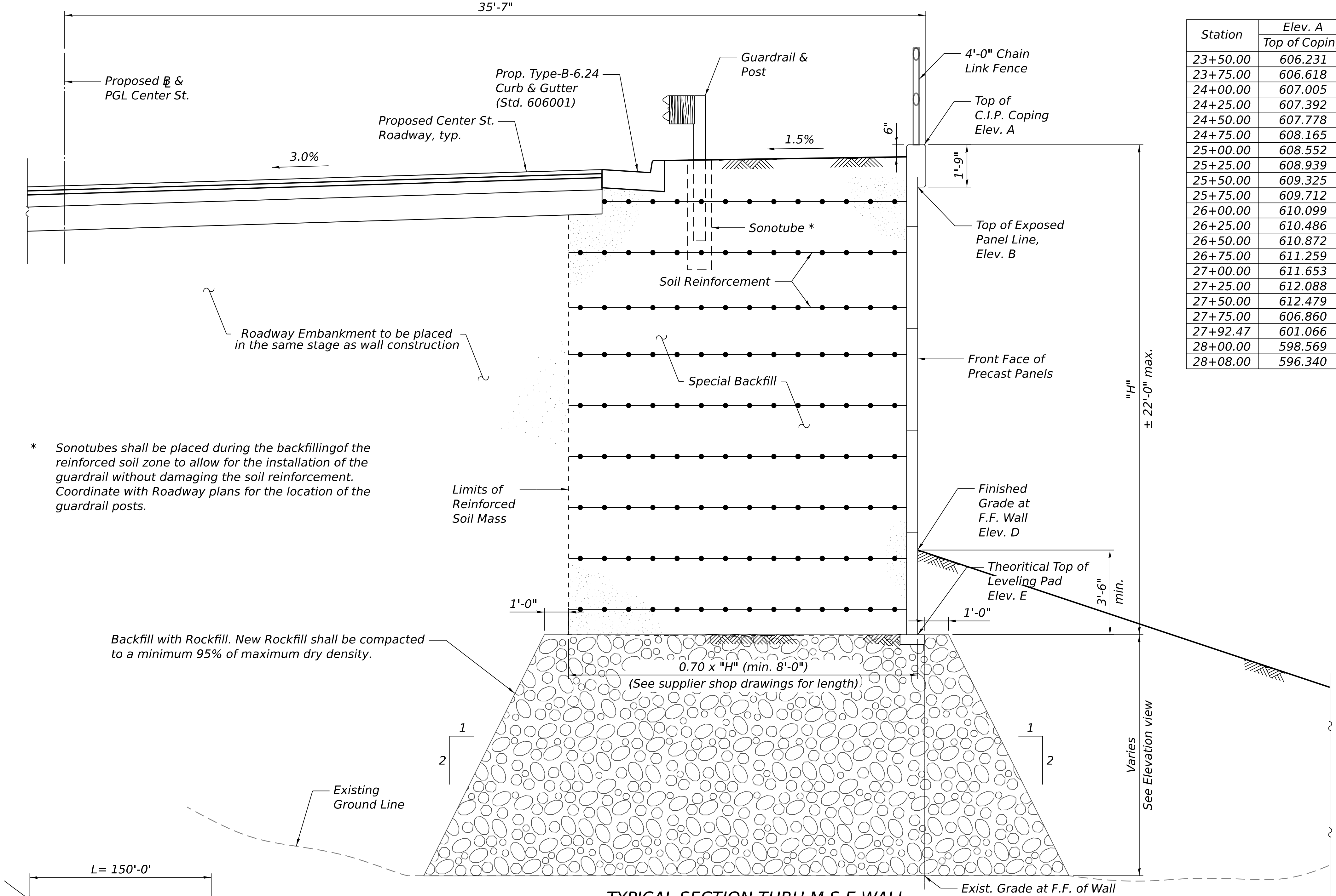


* Sonotubes shall be placed during the backfillingof the reinforced soil zone to allow for the installation of the guardrail without damaging the soil reinforcement. Coordinate with Roadway plans for the location of the guardrail posts.

TYPICAL SECTION THRU M.S.E WALL

(Sta. 23+50.00 to 24+04.00 &
Sta. 25+17 to 28+08)
(Looking North)

TABLE 1- WALL ELEVATIONS

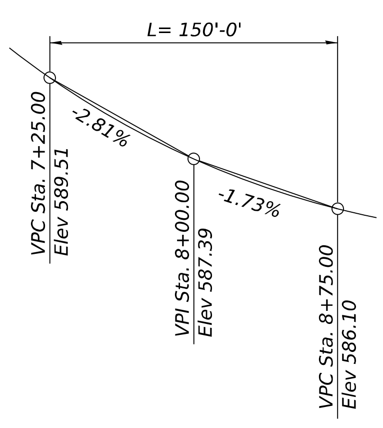
Station	Elev. A	Elev. B	Elev. C	Elev. D	Elev. E
	Top of Coping	Bottom of Coping	Exist. Grade at FF Wall	Top of Prop. Grade	Top of Levelling Pad
23+50.00	606.231	604.481	590.246	604.481	600.98
23+75.00	606.618	604.868	589.430	600.044	596.544
24+00.00	607.005	605.255	589.025	594.337	590.837
24+25.00	607.392	605.642	589.116	589.863	586.363
24+50.00	607.778	606.028	589.301	590.047	586.547
24+75.00	608.165	606.415	588.777	589.779	586.279
25+00.00	608.552	606.802	587.712	590.624	587.124
25+25.00	608.939	607.189	586.787	591.554	588.054
25+50.00	609.325	607.575	585.216	591.992	588.492
25+75.00	609.712	607.962	584.597	593.343	589.843
26+00.00	610.099	608.349	583.965	594.977	591.477
26+25.00	610.486	608.736	582.462	595.290	591.790
26+50.00	610.872	609.122	582.531	595.115	591.615
26+75.00	611.259	609.509	582.124	594.713	591.213
27+00.00	611.653	609.903	580.984	594.603	591.103
27+25.00	612.088	610.338	576.957	594.237	590.737
27+50.00	612.479	610.729	575.660	594.009	590.509
27+75.00	606.860	605.110	573.461	593.779	590.279
27+92.47	601.066	599.223	589.561	593.980	590.479
28+00.00	598.569	596.819	588.217	594.066	590.566
28+08.00	596.340	594.531	590.979	594.531	591.025

PR CURVE CENTER-3

PI Sta. = 25+64.87
Δ = 33° 44' 53" (LT)
D = 5° 43' 46"
R = 1.000.00'
T = 303.33'
L = 589.01'
E = 44.99'
e =
T.R. =
S.E. RUN =
P.C. STA. = 22+61.54
P.T. STA. = 28+50.55

PR CURVE CENTER-B-3

PI Sta. = 7+27.63
Δ = 24° 44' 45" (RT)
D = 06° 52' 42"
R = 833.00'
T = 182.73'
L = 359.77'
E = 19.81'
e = 0.045
T.R. =
S.E. RUN = 144
P.C. STA. = 5+44.90
P.T. STA. = 9+04.67



RAMP B PROFILE GRADE

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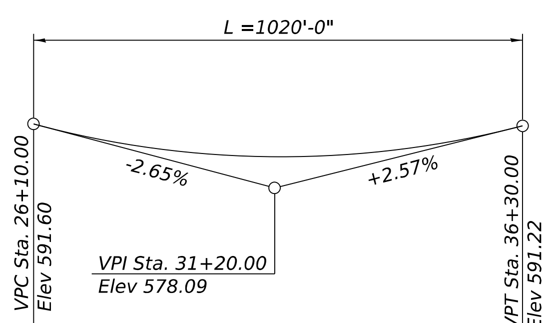

I-80 PROFILE GRADE

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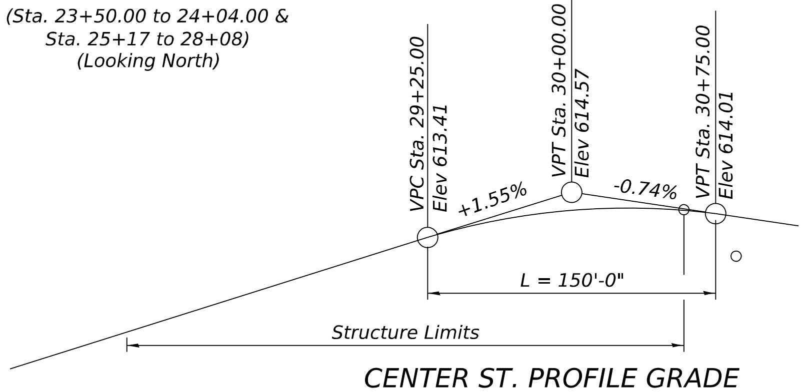
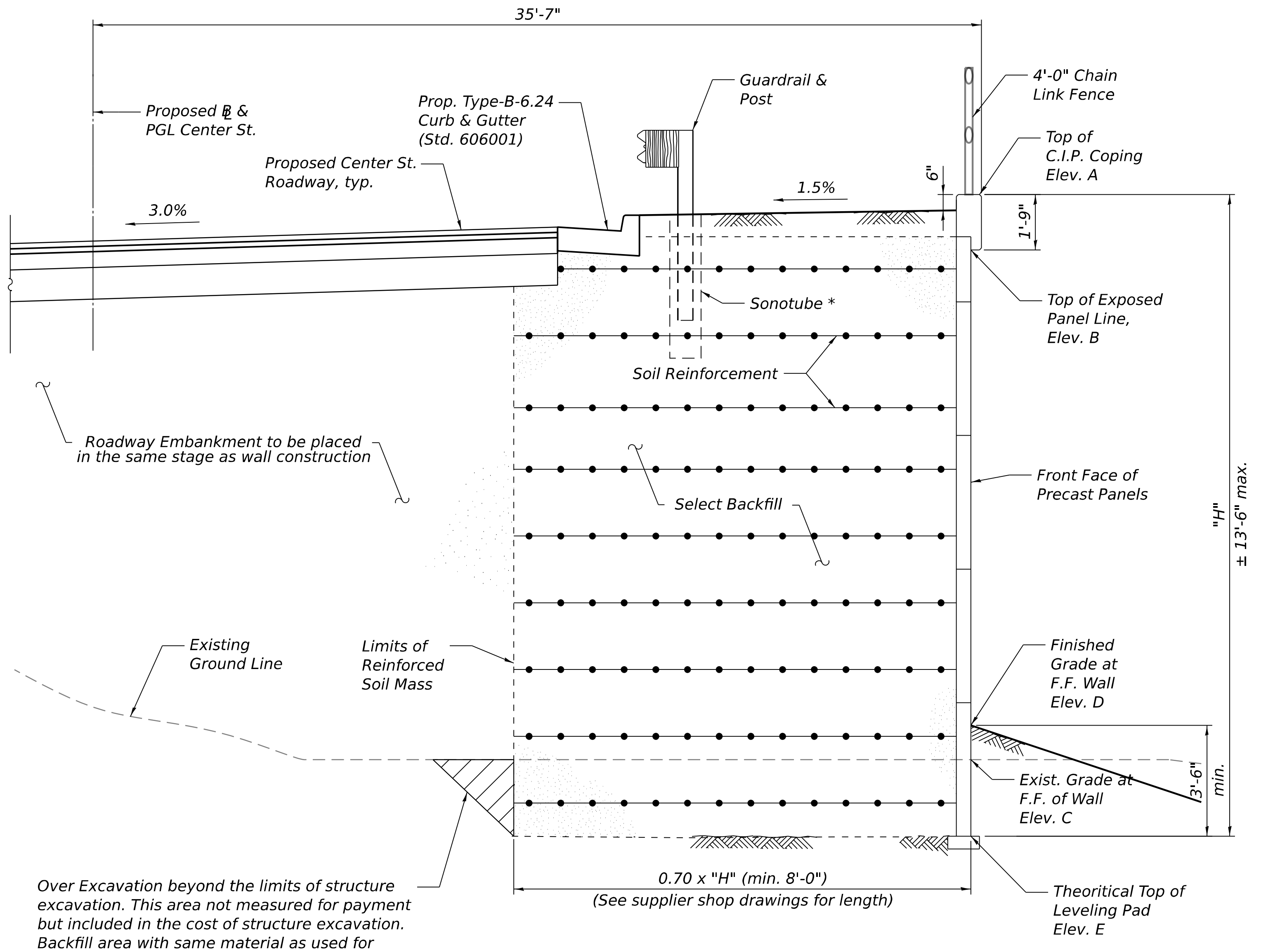


CENTER ST. PROFILE GRADE

(Along ℄ Roadway)

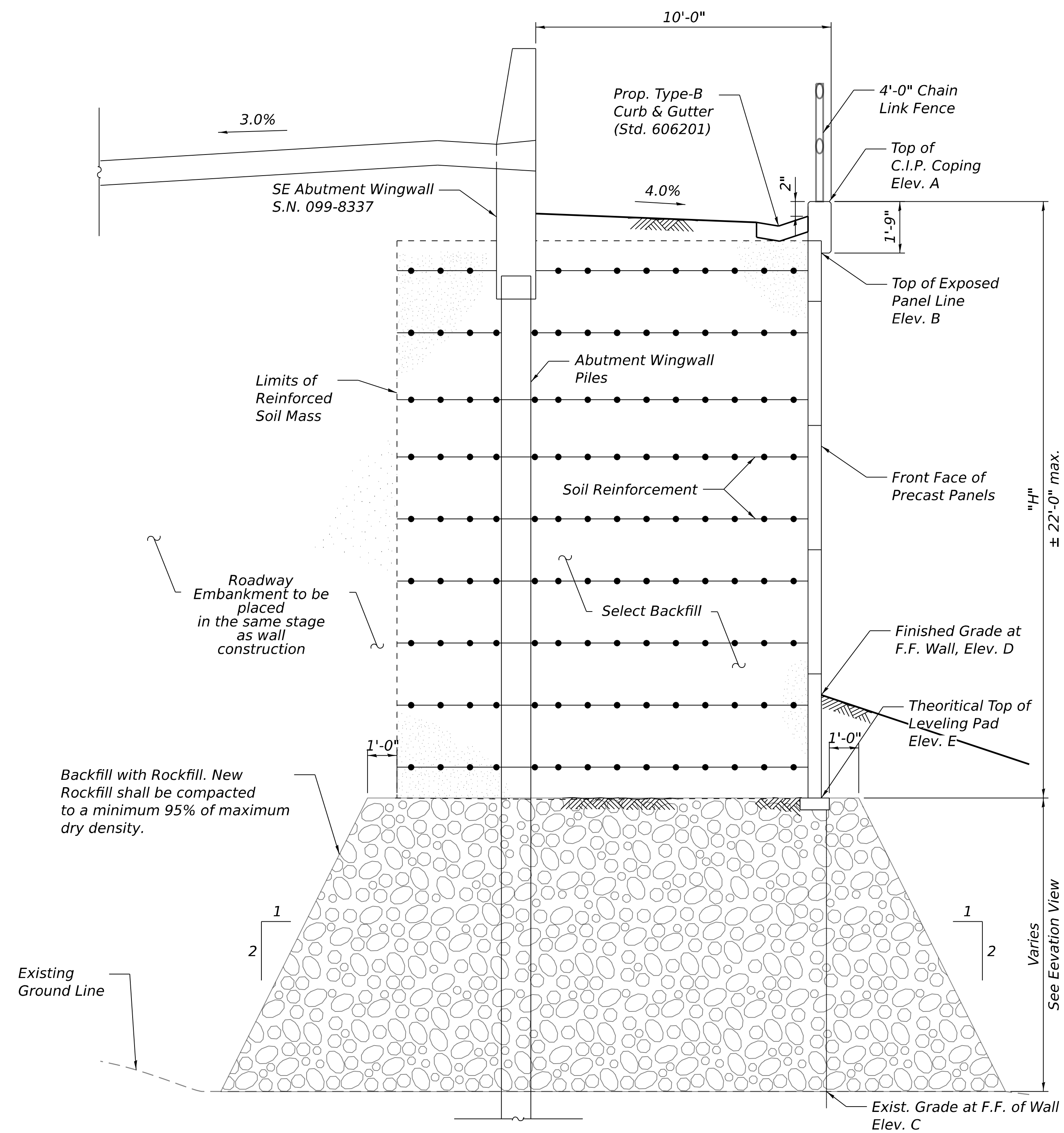
DETAILS
RETAINING WALL 6
ALONG CENTER STREET
F.A.I. ROUTE I-80
SECTION C-91-109-22
WILL COUNTY
STA. 23+50.00 TO STA. 28+08.00
STRUCTURE NO. 099-WXXX



TYPICAL SECTION THRU M.S.E WALL

(Sta. 24+04 to 25+17)
(Looking North)

* Sonotubes shall be placed during the backfillingof the reinforced soil zone to allow for the installation of the guardrail without damaging the soil reinforcement. Coordinate with Roadway plans for the location of the guardrail posts.



SECTION A-A

(Sta. 27+67.17 to 28+08.00)
(Looking North)

DETAILS
RETAINING WALL 6
ALONG CENTER STREET
F.A.I. ROUTE I-80
SECTION C-91-109-22
WILL COUNTY
STA. 23+50.00 TO STA. 28+08.00
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HBM ENGINEERING GROUP, LLC

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

F.A.I. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
80	C-91-109-22	WILL	3	3
		CONTRACT NO. 62R22		
		ILLINOIS	FED. AID PROJECT	

Appendix B

Soil Boring Location Plan and Subsurface Profile

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GSG CONSULTANTS, INC. 735 E. REMINGTON RD, SCHAUMBURG, IL 60173 TEL: +1630.994.2600 | WWW.GSG-CONSULTANTS.COM	USER NAME = $USER$	DESIGNED - ES	STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION	RETAINING WALL NO. 6
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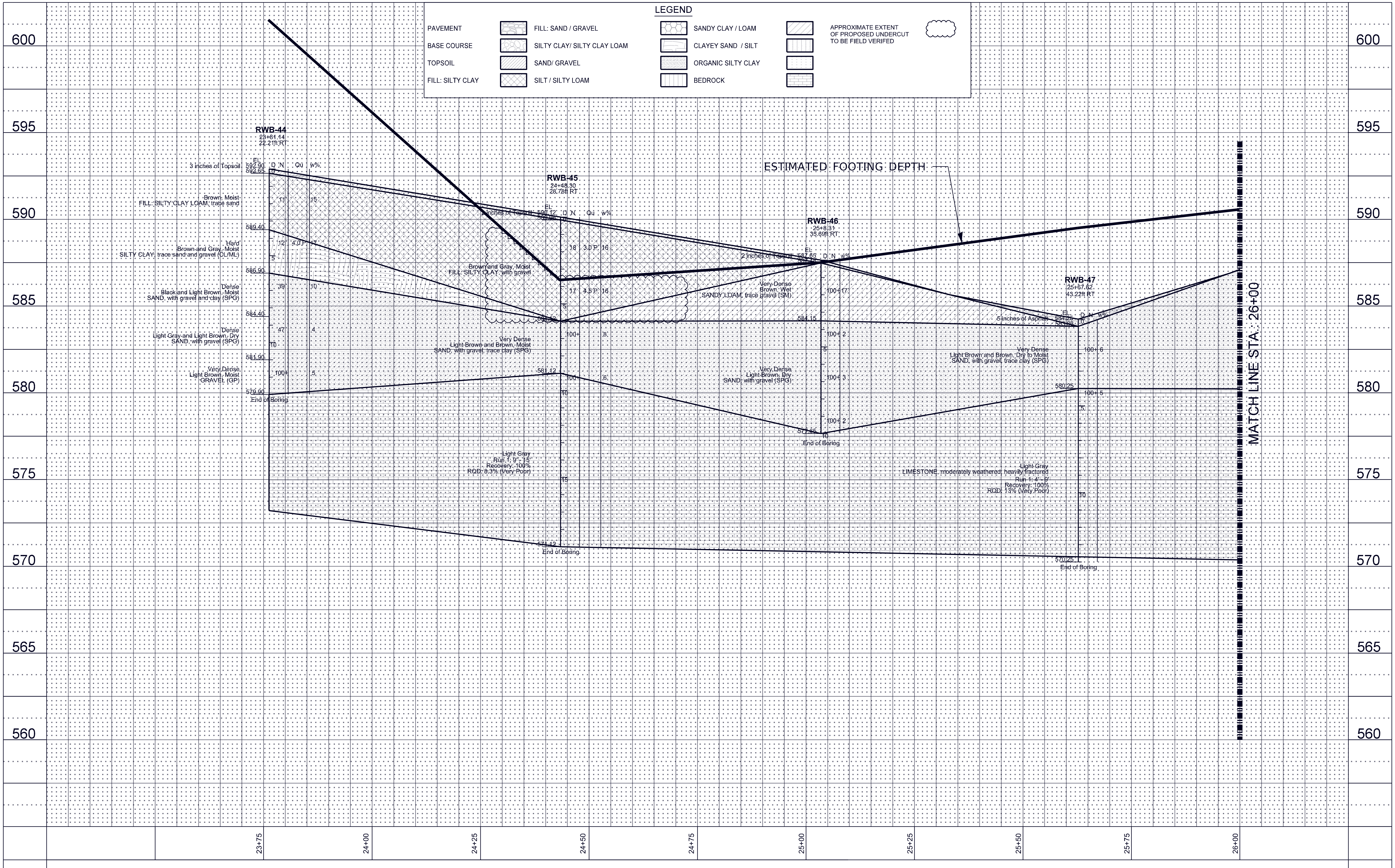
		CONTRACT NO. PTB 198-003		
		ILLINOIS FED. AID PROJECT		



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

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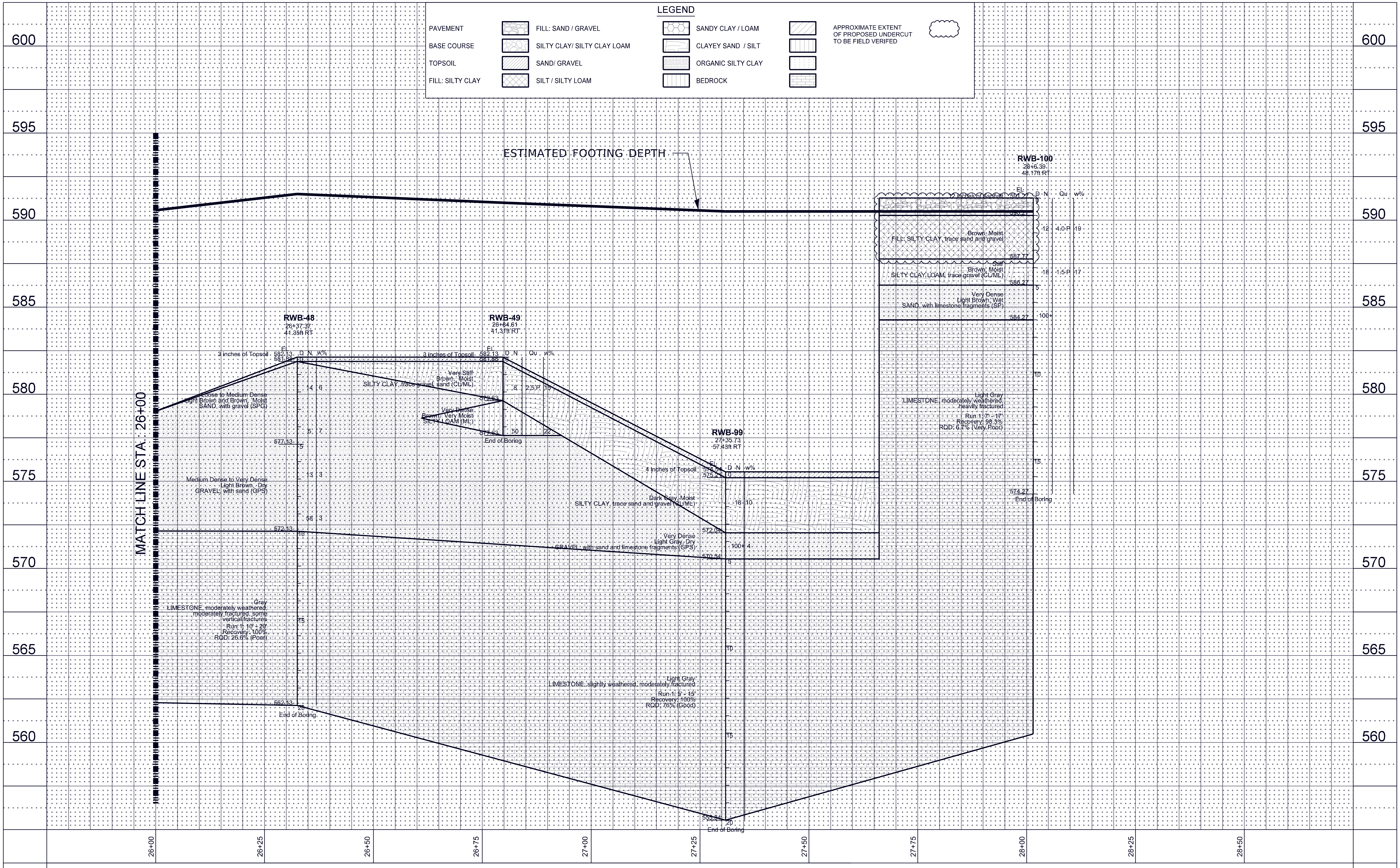
STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

RETAINING WALL NO. 6
SOIL BORING PROFILE
SCALE: AS NOTED | SHEET 1 OF 2 SHEETS | STA. 23+25 TO STA. 19+25

F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
		WILL	3	2

CONTRACT NO. PTB- 198-003
ILLINOIS | FED. AID PROJECT



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

USER NAME = nnano		DESIGNED -	ES
SHEET SIZE = 1.41667x0.916667 (ft.)		DRAWN -	NN
PLOT SCALE = 1200:0.0000 ':" / ft.		CHECKED -	DE
PLOT DATE = 9/20/2022		DATE -	09/12/2022

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

RETAINING WALL NO. 6

SOIL BORING PROFILE

SCALE: AS NOTED | SHEET 2 OF 2 SHEETS | STA. 26+00 TO STA. 28+75

F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
		WILL	3	3

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 7/7/22

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 6 - Center Street Sta 23+50 **LOGGED BY** KA

SECTION C-91-109-22 **LOCATION** , **SEC.** 16, **TWP.** 35 N, **RNG.** 10 E, **Latitude** 41.5116499, **Longitude** -88.098671

COUNTY Will **DRILLING RIG** Diedrich D-50 ATV **HAMMER TYPE** Auto
DRILLING METHOD HSA **HAMMER EFF (%)** 102

STRUCT. NO.
Station

BORING NO. RWB-44
Station 23+81.1439
Offset 22.21ft RT
Ground Surface Elev. 592.90 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 (%)
3 inches of Topsoil	592.65				
Brown, Moist FILL: SILTY CLAY LOAM, trace sand			16 7 4		15
Cobbles at 2.5 feet	589.40				
Hard Brown and Gray, Moist SILTY CLAY, trace sand and gravel (CL/ML)	586.90	-5	4 3 9	4.0 P	17
Dense Black and Light Brown, Moist SAND, with gravel and clay (SPG) Cobbles at 6 feet	584.40		20 19 20		10
Dense Light Gray and Light Brown, Dry SAND, with gravel (SPG) Cobbles at 8.5 feet	581.90	-10	12 22 25		4
Very Dense Light Brown, Moist GRAVEL (GP)	579.90		50/3"		5
Auger refusal at 13.0 feet End of Boring		-15 -20			

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

BBS, form 137 (Rev. 8-99)

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

SOIL BORING LOG

Date 7/8/22

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 6 - Center Street Sta 23+50 **LOGGED BY** KA

SECTION C-91-109-22 **LOCATION** , **SEC.** 16, **TWP.** 35 N, **RNG.** 10 E,
Latitude 41.5117922, **Longitude** -88.0985039

COUNTY Will **DRILLING RIG** Diedrich D-50 ATV **HAMMER TYPE** Auto
DRILLING METHOD HSA **HAMMER EFF (%)** 102

STRUCT. NO.
Station

BORING NO. RWB-45
Station 24+48.2989
Offset 28.78ft RT
Ground Surface Elev. 590.12 ft

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

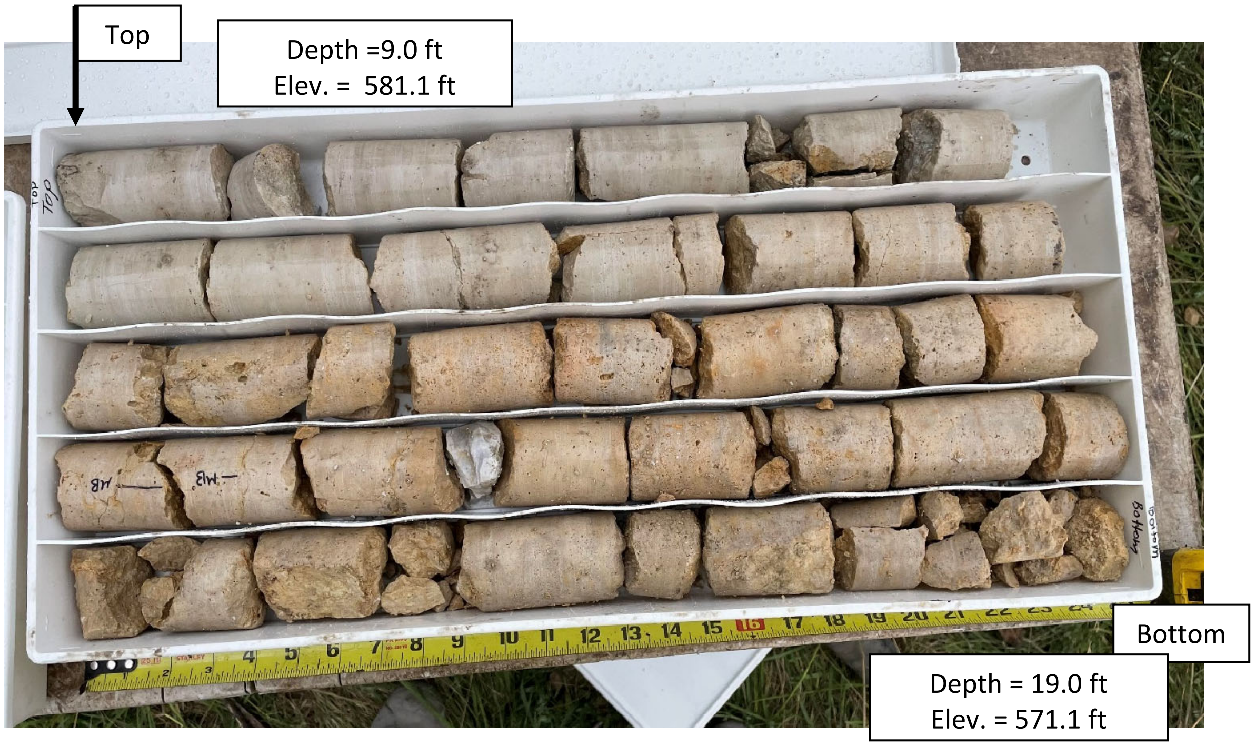
Groundwater Elev.:
First Encounter Dry ft
Upon Completion N/A ft
After ___ **Hrs.** N/A ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
2 inches of Topsoil	589.96				
Brown and Gray, Moist FILL: SILTY CLAY, with gravel			13 10 8	3.0 P	16
Cobbles at 2.5 feet			9 9 8	4.5 P	16
		-5			
	584.12				
Very Dense Light Brown and Brown, Moist SAND, with gravel, trace clay (SPG) Cobbles at 6.0 feet			50/5"		8
Cobbles at 8.5 feet	581.12		50/3"		
Auger refusal at 9.0 feet					6
Light Gray LIMESTONE, moderately weathered, heavily fractured		-10			
Run 1: 9' - 15' Recovery: 100% RQD: 8.3% (Very Poor)					
Run 2: 15' - 19' Recovery: 100% RQD: 0% (Very Poor)		-15			
	571.12				
End of Boring		-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)

BBS, form 137 (Rev. 8-99)

Retaining Wall #6
Boring Number: RWB-45



Boring No.	Run	Depth (ft)	Recovery (%)	RQD (%)	RQD Classification	Description
RWB-45	1	9.0' – 15.0'	100.0	8.3	Very Poor	Light Gray Limestone Heavily Fractured, Moderately Weathered
RWB-45	2	15.0' – 19.0'	100.0	0	Very Poor	

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

SOIL BORING LOG

Date 7/7/22

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 6 - Center Street Sta 23+50 **LOGGED BY** KA

SECTION C-91-109-22 **LOCATION** , **SEC.** 16, **TWP.** 35 N, **RNG.** 10 E,
Latitude 41.5119256, **Longitude** -88.0983616

COUNTY Will **DRILLING RIG** Diedrich D-50 ATV **HAMMER TYPE** Auto
DRILLING METHOD HSA **HAMMER EFF (%)** 102

STRUCT. NO. ____
Station ____

BORING NO. RWB-46
Station 25+8.3101
Offset 35.69ft RT
Ground Surface Elev. 587.65 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 (%)
2 inches of Topsoil	587.48				
Very Dense Brown, Wet SANDY LOAM, trace gravel (SM)			9 20 50/3"		17
	584.15				
Very Dense Light Brown, Dry SAND, with gravel (SPG) Cobbles at 3.5 feet		-5	2 16 50/5"		2
			50/3"		3
Cobbles at 8.5 feet			20 50/3"		2
	577.65	-10			
Auger refusal at 10.0 feet End of Boring					
		-15			
		-20			

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

Illinois Department of Transportation

Division of Highways
GSG Consultants, Inc.

SOIL BORING LOG

Date 7/11/22

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 6 - Center Street Sta 23+50 **LOGGED BY** KA

SECTION C-91-109-22 **LOCATION** , **SEC.** 16, **TWP.** 35 N, **RNG.** 10 E,
Latitude 41.5120638, **Longitude** -88.0982287

COUNTY Will **DRILLING RIG** Diedrich D-50 ATV **HAMMER TYPE** Auto
DRILLING METHOD HSA **HAMMER EFF (%)** 102

STRUCT. NO.
Station

BORING NO. RWB-47
Station 25+67.6210
Offset 43.22ft RT
Ground Surface Elev. 584.25 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 Asphalt	583.83				
Very Dense Light Brown and Brown, Dry to Moist SAND, with gravel, trace clay (SPG) Cobbles at 2.5 feet			7 50/3"		6
	580.25		50/1"		
Auger refusal at 4.0 feet					5
Light Gray LIMESTONE, moderately weathered, heavily fractured Run 1: 4' - 9' Recovery: 100% RQD: 13% (Very Poor) Run 2: 9' - 14' Recovery: 100% RQD: 29% (Poor)		-5 -10			
	570.25				
End of Boring		-15 -20			

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

Retaining Wall #6
Boring Number: RWB-47

Boring No.	Run	Depth (ft)	Recovery (%)	RQD (%)	RQD Classification	Description
RWB-47	1	4.0' – 9.0'	100.0	13	Very Poor	Light Gray Limestone Heavily Fractured, Moderately Weathered
RWB-47	2	9.0' – 14.0'	100.0	29	Poor	

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

SOIL BORING LOG

Date 8/16/22

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 6 - Center Street Sta 23+50 **LOGGED BY** KA

SECTION C-91-109-22 **LOCATION** , **SEC.** 16, **TWP.** 35 N, **RNG.** 10 E,
Latitude 41.5122464, **Longitude** -88.0981218

COUNTY Will **DRILLING RIG** Diedrich D-50 ATV **HAMMER TYPE** Auto
DRILLING METHOD HSA **HAMMER EFF (%)** 102

STRUCT. NO.
Station

BORING NO. RWB-48
Station 26+37.3684
Offset 41.35ft RT
Ground Surface Elev. 582.13 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	581.88				
Loose to Medium Dense Light Brown and Brown, Moist SAND, with gravel (SPG) Cobbles at 1 foot			6 7 7		6
			7 3 2		7
	577.13	-5			
Medium Dense to Very Dense Light Brown, Dry GRAVEL, with sand (GPS)			7 7 6		3
			7 8 50/3"		3
	572.13	-10			
Auger refusal at 10.0 feet					
Gray LIMESTONE, moderately weathered, moderately fractured, some vertical fractures Run 1: 10' - 20' Recovery: 100% RQD: 26.6% (Poor)					
		-15			
	562.13	-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)

BBS, form 137 (Rev. 8-99)

Retaining Wall #6
Boring Number: RWB-48

Depth =10.0 ft
Elev. = 572.1 ft

Top

Bottom

Depth = 20.0 ft
Elev. = 562.1 ft

Boring No.	Run	Depth (ft)	Recovery (%)	RQD (%)	RQD Classification	Description
RWB-48	1	10' – 20'	100.0	26.6	Poor	Gray Limestone Moderately Weathered, Moderately Fractured, Some Vertical Fractures

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

SOIL BORING LOG

Date 8/16/22

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 6 - Center Street Sta 23+50 **LOGGED BY** KA

SECTION C-91-109-22 **LOCATION** , **SEC.** 16, **TWP.** 35 N, **RNG.** 10 E,
Latitude 41.5123717, **Longitude** -88.098055

COUNTY Will **DRILLING RIG** Diedrich D-50 ATV **HAMMER TYPE** Auto
DRILLING METHOD HSA **HAMMER EFF (%)** 102

STRUCT. NO.
Station

BORING NO. RWB-49
Station 26+84.6061
Offset 41.31ft RT
Ground Surface Elev. 582.13 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 (%)
3 inches of Topsoil	581.88				
Very Stiff Brown, Moist SILTY CLAY, trace gravel, sand (CL/ML)	579.63		5 5 3	2.5 P	15
Very Dense Brown, Very Moist SILTY LOAM (ML)	577.63		50/4"		22
Auger refusal at 4.5 feet End of Boring		-5			
		-10			
		-15			
		-20			

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

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

SOIL BORING LOG

Date 8/31/22

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 6 - Center Street Sta 23+50 **LOGGED BY** EH

SECTION C-91-109-22 **LOCATION** , **SEC.** , **TWP.** , **RNG.** ,
Latitude 41.5124964, **Longitude** -88.0979354

COUNTY Will **DRILLING RIG** CME-75 **HAMMER TYPE** Auto
DRILLING METHOD HSA **HAMMER EFF (%)** 91

STRUCT. NO.
Station

BORING NO. RWB-99
Station 27+35.7341
Offset 57.43ft RT
Ground Surface Elev. 575.54 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	575.21				
Very Stiff Dark Gray, Moist SILTY CLAY, trace sand and gravel (CL/ML)			9 11 7		10
	572.04				
Very Dense Light Gray, Dry GRAVEL, with sand and limestone fragments (GPS)			50/6"		4
	570.54	-5			
Light Gray LIMESTONE, slightly weathered, moderately fractured Run 1: 5' - 15' Recovery: 100% RQD: 76% (Good)					
		-10			
		-15			
Run 2: 15' - 20' Recovery: 100% RQD: 93% (Excellent)					
	555.54	-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 #6
Boring Number: RWB-99

Boring No.	Run	Depth (ft)	Recovery (%)	RQD (%)	RQD Classification	Description	Depth (ft)/ Unconfined Compression Strength (psi)
RWB-99	1	5.0' – 15.0'	100.0	76.0	Good	Light Gray Limestone Slightly Weathered, Moderately Fractured	8.0 / 8,741
RWB-99	2	15.0' – 20.0'	100.0	93.0	Excellent		13.0 / 17,164

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

SOIL BORING LOG

Date 8/31/22

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 6 - Center Street Sta 23+50 **LOGGED BY** DD

SECTION C-91-109-22 **LOCATION** , **SEC.** , **TWP.** , **RNG.** ,
Latitude 41.5126998, **Longitude** -88.0978951

COUNTY Will **DRILLING RIG** Diedrich D-50 **HAMMER TYPE** Auto
DRILLING METHOD HSA **HAMMER EFF (%)** 98

STRUCT. NO.
Station

BORING NO. RWB-100
Station 28+6.3942
Offset 48.17ft RT
Ground Surface Elev. 591.27 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 (%)
12 inches of Asphalt	590.27				
Brown, Moist FILL: SILTY CLAY, trace sand and gravel			5 4 8	4.0 P	19
	587.77				
Stiff Brown, Moist SILTY CLAY LOAM, trace gravel (CL/ML)	586.27	-5	2 3 15	1.5 P	17
Very Dense Light Brown, Wet SAND, with limestone fragments (SP) Auger refusal at 7.0 feet	584.27		50/2"		
Light Gray LIMESTONE, moderately weathered, heavily fractured Run 1: 7' - 17' Recovery: 98.3% RQD: 6.7% (Very Poor)		-10			
		-15			
	574.27				
End of Boring		-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)

BBS, form 137 (Rev. 8-99)

Retaining Wall #6
Boring Number: RWB-100

Boring No.	Run	Depth (ft)	Recovery (%)	RQD (%)	RQD Classification	Description
RWB-100	1	7.0' – 17.0'	98.3	6.7	Very Poor	Light Gray Limestone Moderately Weathered, Heavily Fractured

Appendix D
Soil Parameter Table

Table D-1 – Summary of Soil Parameters

Retaining Wall #6

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	2,000	0	200	28
	New Engineered Granular Fill	125	0	30	0	30
RWB-44 to RWB-49						
0 – 10.0 (590.0-580.0)	Loose to Very dense brown Sand and Gravel	129	0	42	0	42
10.0 – 20.0 (580.0-570.0)	Gray Limestone	150	0	45	0	45
0 - 6.0 (591.0-585.0) RWB-44 & 45	FILL: Brown and Gray Silty Clay	138	3,000	0	300	25
0 – 3.5 (582.0-578.5) RWB-49	Very Stiff Brown Silty Clay	138	2,500	0	250	25
RWB-99 only						
0 - 3.5 (575.5-572.0)	Very Stiff Brown and Gray Silty Clay	129	1,000	0	100	25
3.5 – 5.0 (572.0-570.5)	Very Dense Gray Sand and Gravel	138	0	42	0	42
3.5 – 5.0 (572.0-570.5)	Gray Limestone	150	0	45	0	45
RWB-100 only						
1 – 3.5 (590.0-587.5)	FILL: Brown Silty Clay	138	4,000	0	400	25
3.5 – 5.0 (587.5-586.0)	Stiff Brown Silty Clay Loam	134	1,500	0	150	28
5.0 – 7.0 (586.0-584.0)	Very Dense Brown Sand	138	0	42	0	42
7.0 – 17.0 (584.0-574.0)	Gray Limestone	150	0	45	0	45

Appendix E

Laboratory Test Results

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 99		
Sample Depth (ft):	8		
Lithological Description:	Lime Stone		
Formation Name:		Load Direction:	Vertical
Appearance (e.g. cracks, shearing, spalling):		Angle Drilled:	Vertical

	Bulk/Prep	MC/CS
Tester:	AJ	AJ
Date:	09/07/22	09/07/22

Bulk Density Determination

	1	2	3	Average
Height, *in.*	4.8690	4.8940	4.8810	4.8813
Diameter, *in.*	1.9855	1.9830	1.9855	1.9847

Specimen Mass, *g*	662.6	Ratio (2.0-2.5)	
Bulk Density, *pcf*	167.2		2.46

Moisture Condition - D2216

Container ID	Audrey
container, *g*	226.2
container + wet rock, *g*	821.0
container + dry soil, *g*	812.9
moisture content, *w%*	1.4

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		

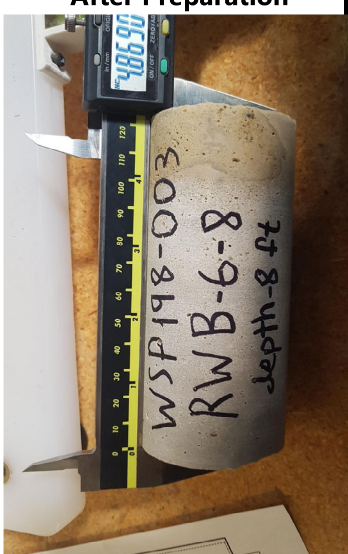
Axial Loading

Seating Load (≤1000 psi)	1000
Rate of Loading (73-145 psi/s)	75
Time to Failure (2-15 min)	4 min 16 sec
Load @ Failure, *lbf*	27,042
Uniaxial Compressive Strength, *psi*	8,741

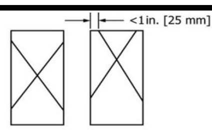
Remarks

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

After Preparation

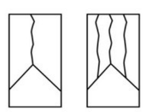


After Break (check applicable appearance)



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

☐

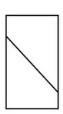


Type 2
Well-formed cone on one end, vertical cracks running through caps, no well-defined cone on other end

☐

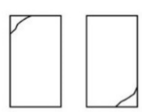
Type 3
Columnar vertical cracking through both ends, no well-formed cones

X



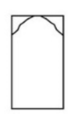
Type 4
Diagonal fracture with no cracking through ends; tap with hammer to distinguish from Type 1

☐



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

☐

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

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-99	Bulk/Prep	MC/CS
Sample Depth (ft):	13	Tester: AJ	Tester: AJ
Lithological Description:	Lime Stone	Date: 09/07/22	Date: 09/07/22
Formation Name:		Load Direction: Vertical	Angle Drilled: Vertical
Appearance (e.g. cracks, shearing, spalling):	Cracks		

Bulk Density Determination

	1	2	3	Average
Height, *in.*	4.9205	4.9220	4.9195	4.9207
Diameter, *in.*	1.9920	1.9910	1.9895	1.9908
Specimen Mass, *g*	675.0		Ratio (2.0-2.5)	
Bulk Density, *pcf*	167.9		2.47	

Moisture Condition - D2216

Container ID	Oreo
container, *g*	226.4
container + wet rock, *g*	833.5
container + dry soil, *g*	821.3
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		

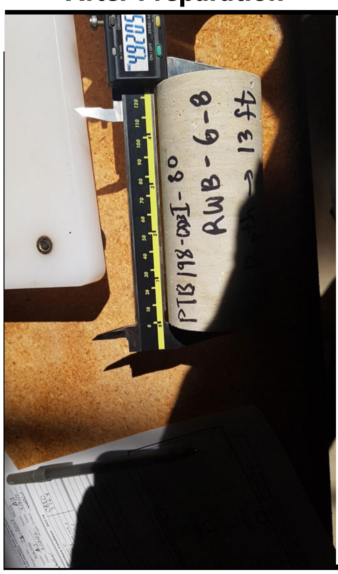
Axial Loading

Seating Load (≤1000 psi)	1000
Rate of Loading (73-145 psi/s)	75
Time to Failure (2-15 min)	3 min 44 sec
Load @ Failure, *lbf*	53,430
Uniaxial Compressive Strength, *psi*	17,164

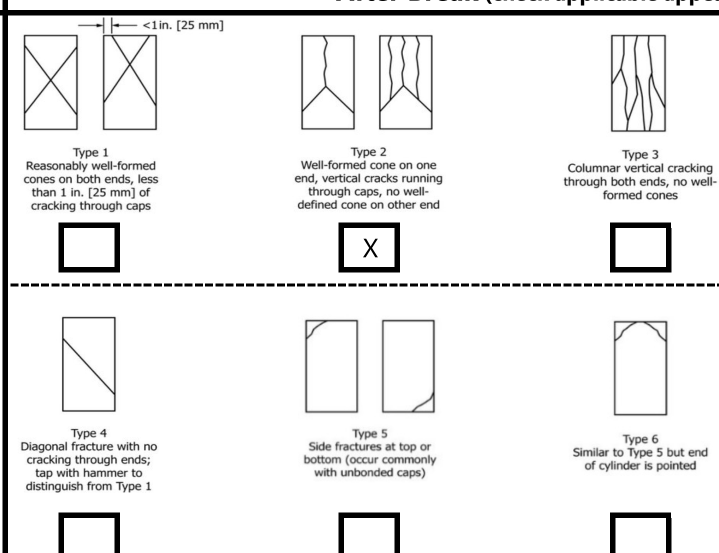
Remarks

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

After Preparation

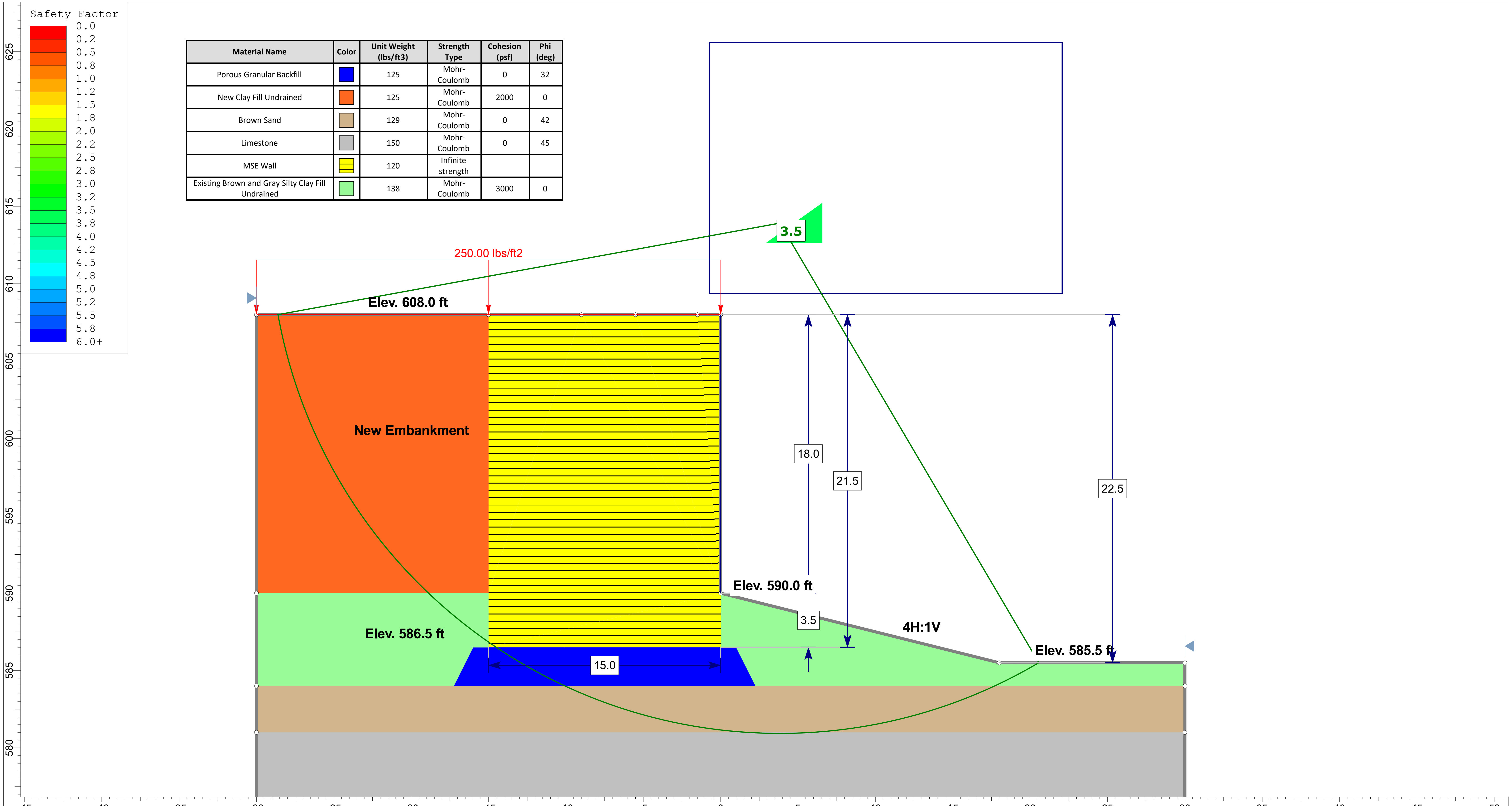


After Break (check applicable appearance)



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)
Porous Granular Backfill		125	Mohr-Coulomb	0	32
New Clay Fill Undrained		125	Mohr-Coulomb	2000	0
Brown Sand		129	Mohr-Coulomb	0	42
Limestone		150	Mohr-Coulomb	0	45
MSE Wall		120	Infinite strength		
Existing Brown and Gray Silty Clay Fill Undrained		138	Mohr-Coulomb	3000	0

GSG CONSULTANTS, INC.
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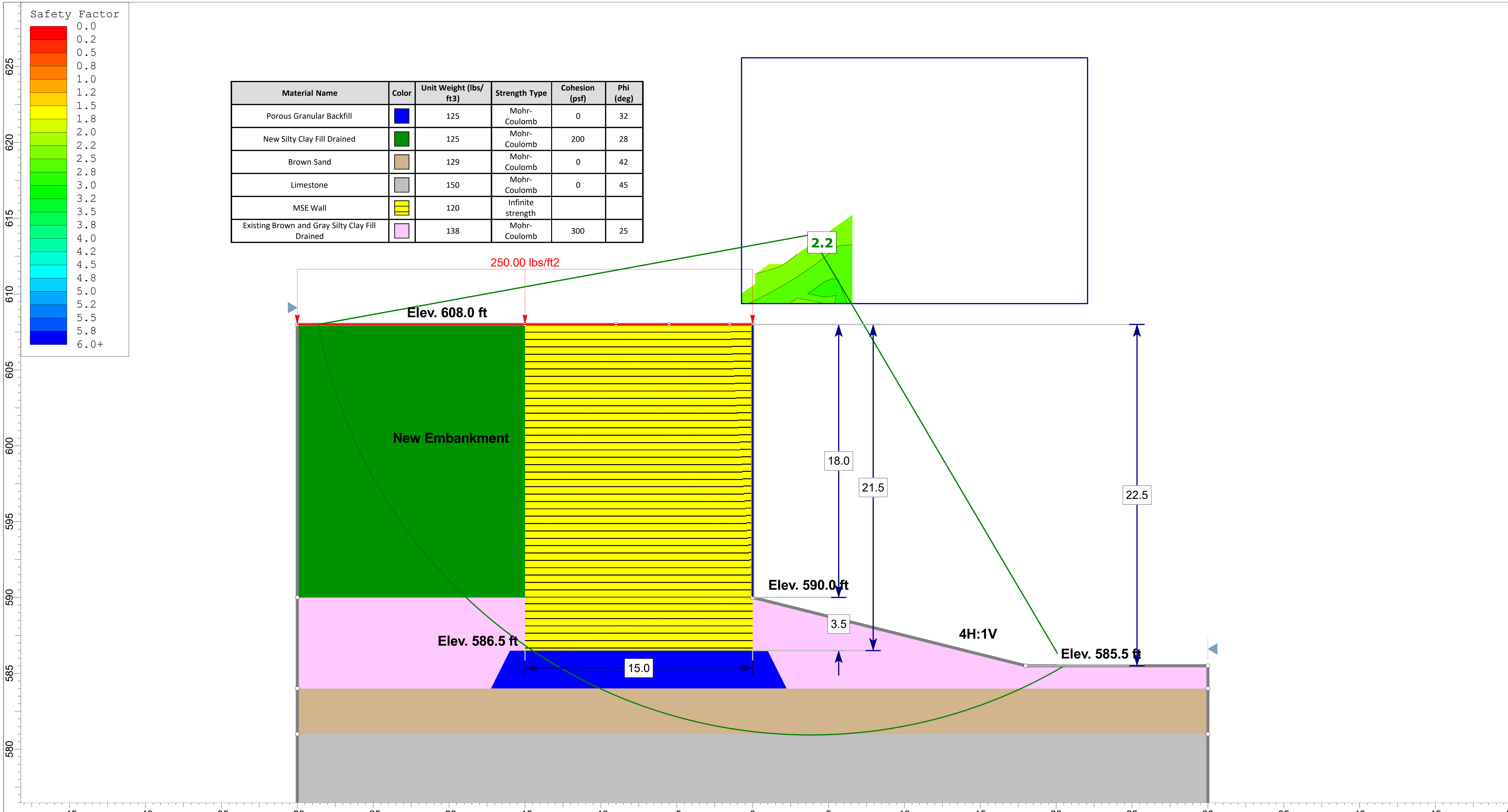
SLIDEINTERPRET 9.018

Project	I-80: IDOT PTB 198-003 - Retaining Wall #6 - MSE Wall - Station 24+50		
Analysis Description	Exhibit 1 - Circular Failure Short Term		
Drawn By	ES	Company	GSG Consultants, Inc.
Date	9/22/2022	File Name	MSE Wall 24+50.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Porous Granular Backfill		125	Mohr-Coulomb	0	32
New Silty Clay Fill Drained		125	Mohr-Coulomb	200	28
Brown Sand		129	Mohr-Coulomb	0	42
Limestone		150	Mohr-Coulomb	0	45
MSE Wall		120	Infinite strength		
Existing Brown and Gray Silty Clay Fill Drained		138	Mohr-Coulomb	300	25

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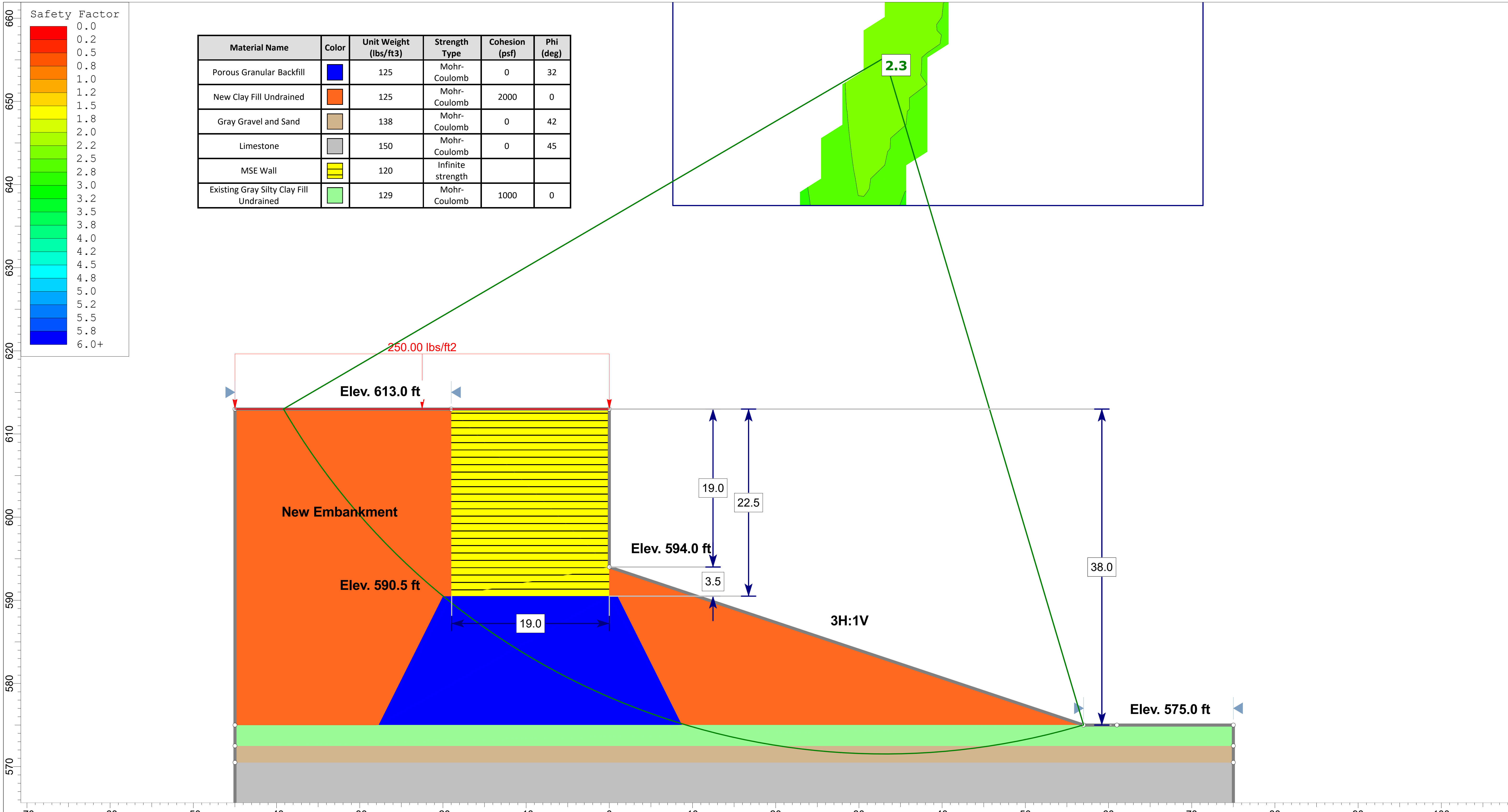
Project	I-80: IDOT PTB 198-003 - Retaining Wall #6 - MSE Wall - Station 24+50		
Analysis Description	Exhibit 2 - Circular Failure Long Term		
Drawn By	ES	*Company*	GSG Consultants, Inc.
Date	9/22/2022	*File Name*	MSE Wall 24+50.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Porous Granular Backfill		125	Mohr-Coulomb	0	32
New Clay Fill Undrained		125	Mohr-Coulomb	2000	0
Gray Gravel and Sand		138	Mohr-Coulomb	0	42
Limestone		150	Mohr-Coulomb	0	45
MSE Wall		120	Infinite strength		
Existing Gray Silty Clay Fill Undrained		129	Mohr-Coulomb	1000	0

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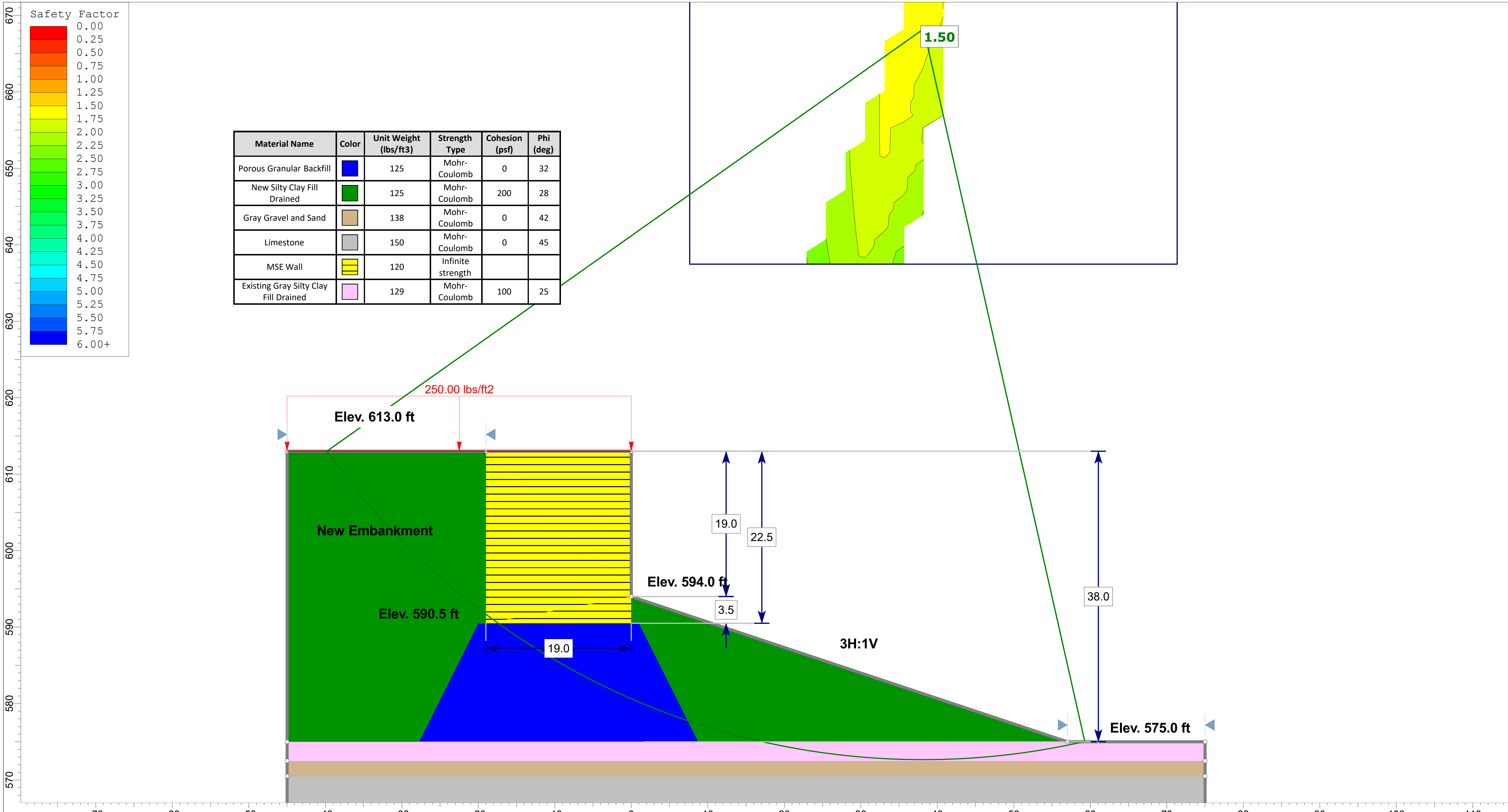
Project	I-80: IDOT PTB 198-003 - Retaining Wall #6 - MSE Wall - Station 27+50
Analysis Description	Exhibit 3 - Circular Failure Short Term
Drawn By	ES
Company	GSG Consultants, Inc.
Date	9/22/2022
File Name	MSE Wall 27+50.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Porous Granular Backfill		125	Mohr-Coulomb	0	32
New Silty Clay Fill Drained		125	Mohr-Coulomb	200	28
Gray Gravel and Sand		138	Mohr-Coulomb	0	42
Limestone		150	Mohr-Coulomb	0	45
MSE Wall		120	Infinite strength		
Existing Gray Silty Clay Fill Drained		129	Mohr-Coulomb	100	25

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SLIDEINTERPRET 9.018

Project	I-80: IDOT PTB 198-003 - Retaining Wall #6 - MSE Wall - Station 27+50
Analysis Description	Exhibit 4 - Circular Failure Long Term
Drawn By	ES
Company	GSG Consultants, Inc.
Date	9/22/2022
File Name	MSE Wall 27+50.slmd