Structural Geotechnical Report - Preliminary

Proposed Retaining Wall #9B – Ramp C
Structure No. 099-W129
IDOT PTB 198-003
FAI-80 over Des Plaines River Bridge
Will County, Illinois

Prepared for

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

Project Design Engineer Team
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Structural Geotechnical Report - Preliminary
Proposed Retaining Wall #9B – Ramp C
FAI-80 over Des Plaines River Bridge
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 included advancing two (2) soil borings to depths between 32 and 40 feet. Additional soil borings are proposed along the proposed wall alignment once access to those locations is available.

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

Sincerely,

Matthew Heron, P.E.
Project Engineer

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

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1.0 INTRODUCTION

GSG Consultants, Inc. (GSG) completed a preliminary geotechnical investigation for the proposed retaining wall #9B along Ramp C as part of the FAI-80 over Des Plaines 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 retaining wall. **Exhibit 1** shows the general project location.

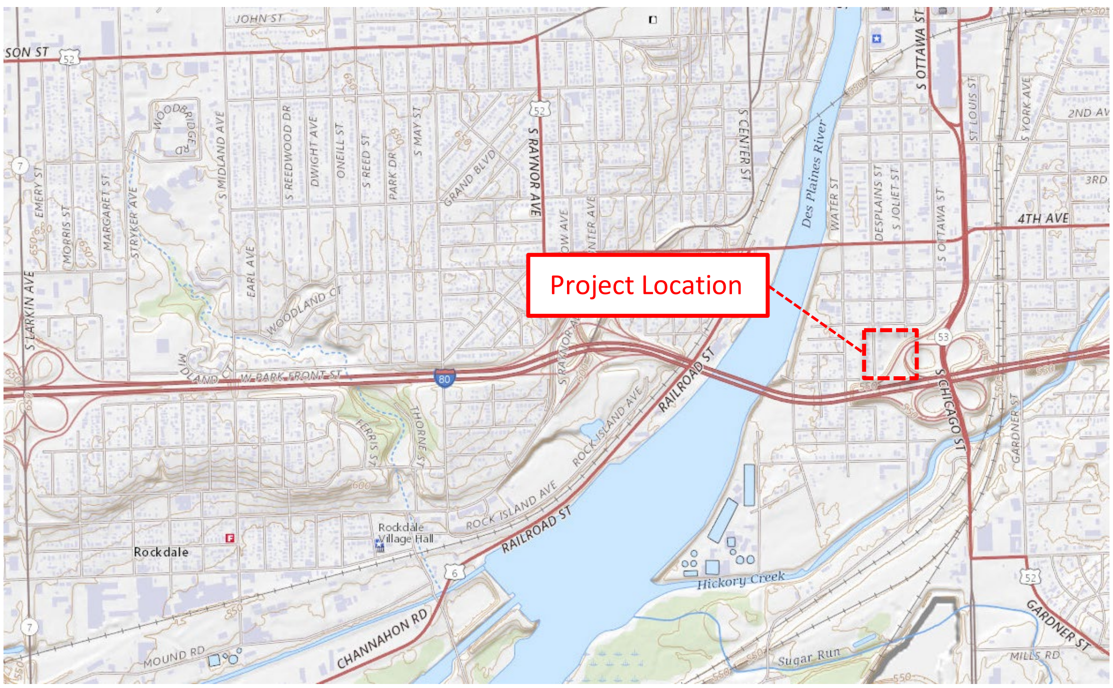


Exhibit 1 – Project Location Map
(Source: USGS Topographic Maps, usgs.gov)

1.1 Existing Site Conditions

The proposed improvements at this location will shift the existing Ramp C alignment to the west by approximately 40 to 50 feet requiring the proposed retaining wall to separate the roadway from the adjacent residential area. According to the proposed Phase 1 plan drawings provided, the proposed retaining wall will primarily be a "fill" section for construction of the new Ramp C. The area where the proposed wall will be constructed is outside of the existing IDOT Right-of-Way, between the I-80/Chicago Street interchange and the neighboring residential area to the north and west. **Exhibits 2a, 2b and 2c** show the existing conditions where the proposed retaining wall will be constructed.

Exhibit 2a – Existing Ramp C, Looking Northeast

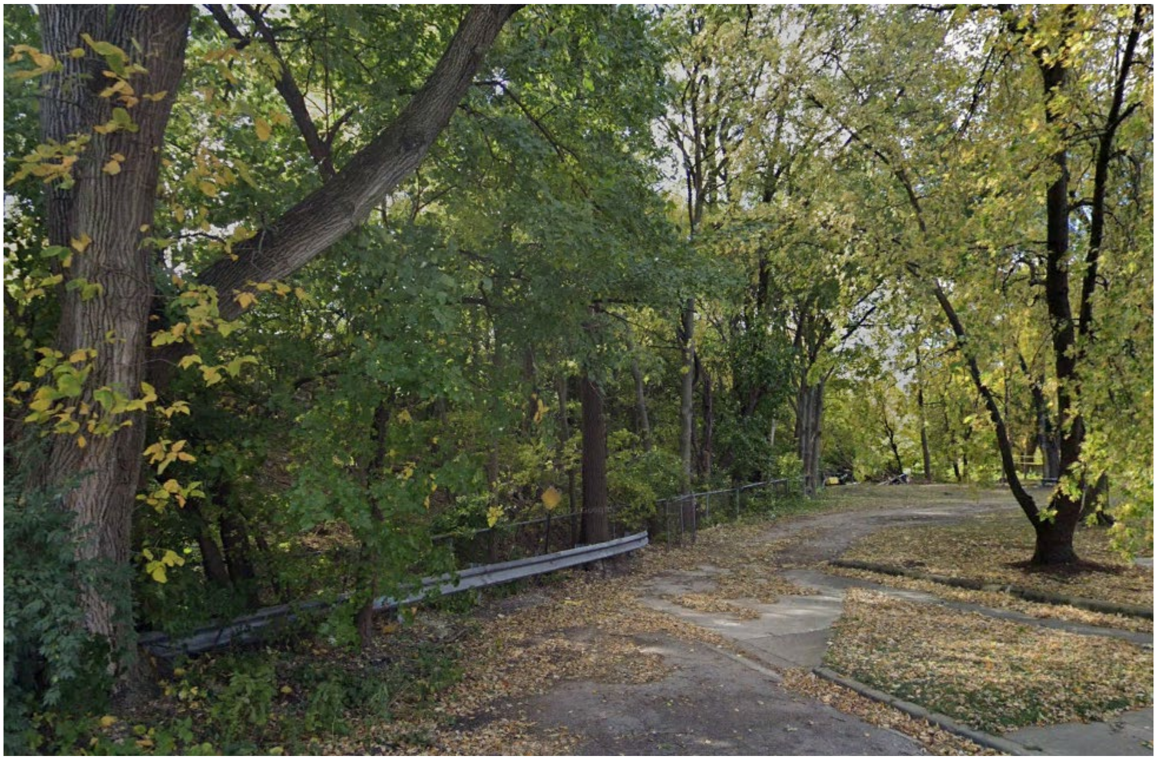


Exhibit 2b – Existing S. Joliet Street, Looking Southwest

Exhibit 2c – Proposed Project Area, Aerial

1.2 Proposed Retaining Wall Information

Based on preliminary Phase I design information provided and a review of site topography, the proposed wall will be in a predominantly "fill" section along the new Ramp C alignment, with a maximum exposed wall height of up to approximately 13.7 feet, and maximum retained height of 17.3 feet. The proposed retaining wall will be approximately 228 feet in length. The wall will be at least 68 feet from the Ramp C center line between Sta. 10+60.67 and Sta. 12+74.85. It is anticipated that the proposed structure will be a mechanically stabilized earth (MSE) wall. A new embankment will be constructed along Ramp C between Sta. 10+60.67 and Sta. 12+74.85. It is anticipated that the new embankment will have a maximum height of 14 feet. The new embankment will be sloped away from the wall to the new ramp roadway at a 1V:3H slope. A new noise abatement wall will be constructed at the top of the slope. Recommendations for the proposed noise abatement wall will be included in a separate report.

A retaining wall is proposed for this location as shown on the General Plan & Elevation dated 12/13/22 (**Appendix A)**. **Table 1** presents a summary of the proposed structure.

Table 1 – Preliminary Retaining Wall Summary

Wall Name	Wall Stations*	Approximate Length (ft)	Maximum Anticipated Exposed Wall Height (ft)
Retaining Wall #9B	Sta. 10+60.67 RT to Sta. 12+74.85 RT	228.25	13.7

* Based on proposed Ramp C 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 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

The preliminary site subsurface exploration for the proposed retaining wall structure was conducted between June 20 and June 22, 2022. The investigation included advancing two (2) borings in the vicinity of the proposed retaining wall to depths between 32.0 and 40.0 feet. Boring RWB-76 was terminated upon encountering difficult drilling conditions and auger refusal. The locations of the 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 proposed retaining wall analysis.

Table 2 – Summary of Subsurface Exploration Borings[1]

Boring ID	Station †	Offset (ft) †	Northing	Easting	Depth (ft)	Surface Elevation (ft)
RWB-76	10+58.10	22.95 LT	1765151.53	1052489.578	32.0**	547.81
RWB-77	11+30.17	22.17 LT	1765091.54	1052449.87	40.0*	547.50

* Depth includes Bedrock Core (10 feet)

** Terminated upon encountering practical auger refusal

† Based on proposed Ramp C Stationing

[1] Additional borings are proposed along the proposed alignment of the retaining wall, but are currently unable to be drilled based on difficult site access; final locations are to be determined when accessibility is available

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

The soil borings were drilled using a truck mounted CME-75 (hammer efficiency 79.8%) drill rig, 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 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 and surface patching with asphalt where necessary to match the existing pavement.

GSG's field representative inspected, visually classified, and logged the soil samples during the subsurface exploration activities and performed unconfined compressive strength tests on cohesive soil samples using a calibrated Rimac compression tester and a calibrated hand penetrometer in accordance with IDOT procedures and requirements. Representative soil samples were collected from each sample interval and were placed in jars and returned to the laboratory for further testing and evaluation.

2.2 Laboratory Testing Program

All samples were inspected in the laboratory to verify the field classifications. A laboratory testing program was undertaken to characterize and determine engineering properties of the subsurface soils encountered in the area. 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 shown along with the field test results in the Soil Boring Logs **(Appendix C)** and in the Laboratory Results (**Appendix D)**.

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. 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.

The surface elevations of the borings ranged between 547.5 and 547.8 feet. The borings initially encountered 13 inches of asphalt pavement along the shoulder of the existing Ramp C. Beneath the surficial pavement, brown and gray silty clay fill materials were encountered to depths of 17.5 to 18.5 feet. Beneath the silty clay fill, light brown gravel with sand fill materials were encountered to a depth of 21 feet (elevation 526.5 feet). Cobbles were encountered in boring RWB-76 at depths of 13.5, 16 and 18.5 feet.

Beneath the fill materials, the borings encountered loose to dense sand with gravel to a depth of 26 feet (elevation 521.5 feet). Cobbles were encountered in boring RWB-77 at a depth of 21 feet. The borings then encountered medium dense to very dense, dark brown sandy loam and sand soils to depths of 30 to 32 feet (elevations 515.8 to 519.0 feet), where the borings encountered auger refusal.

The native light brown sand with gravel had SPT blow count (N) values ranging from 10 to 38 blows per foot (bpf), with an average value of 28 blows per foot. The sandy loam had a SPT blow count (N) values between 21 blows per foot to 59 blows per 9 inches, with an average value of 40 blows per foot. The brown sand with gravel in RWB-77 had a SPT blow count (N) value of 56 bpf.

2.4 Subsurface Bedrock Conditions

Rock core samples were collected from boring RWB-77 after reaching auger refusal. The bedrock cores consisted of light gray limestone, with moderate weathering and moderate fracturing. 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**. 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	Length (ft)	Core Depth (feet)	Type of Rock	RQD (%)	RQD Description	Depth (ft)/ Compressive Strength (psi)
RWB-77	10	30.0 – 40.0	Limestone	44.2	Poor	37.0/23,447

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 in the borings. None of the borings were left open after leaving the site due to safety concerns.

Based on lack of observed water, it is anticipated that the long-term groundwater level may be at an approximate elevation of 515.8 to 517.5 feet or deeper, within the bedrock. Perched water may also be present within the fill materials observed at the surface of 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 up to 14 feet of new fill behind the wall may be required to construct the new retaining wall embankment and Ramp C alignment.

The embankment behind the proposed wall was evaluated with respect to settlement. Based on the proposed new fill of up to 14 feet, an analysis was performed at the boring locations to evaluate the anticipated amount of total settlement. The maximum estimated settlement within the existing soils were calculated as shown in **Table 5.** This estimate does not include any settlement of the new embankment materials which is anticipated to occur during the construction process.

Table 5 – Anticipated Embankment Settlement

Max. Embankment Height (ft)	Max. Anticipated Settlement (in)	Differential Settlement (%)
14.0	0.07	<0.5

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 D. The Seismic Performance Zone (SPZ) was determined using Figure 2.3.10-2 in the IDOT Bridge Manual and was found to be Seismic Performance Zone 1.

The AASHTO Seismic Design Parameters program was used to determine the peak ground acceleration coefficient (PGA), and the short (S_{DS}) and long (S_{D1}) period design spectral acceleration coefficients for each of the proposed structures. For this section of the project, the S_{DS} and the S_{D1} were determined using 2020 AASHTO Guide Specifications as shown in **Table 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	0.112g	0.146g	0.110g

4.0 PRELIMINARY GEOTECHNICAL WALL DESIGN RECOMMENDATIONS

This section provides retaining wall design parameters including preliminary recommendations on foundation type, bearing capacity, settlement, and lateral earth pressures. The foundations for the proposed retaining walls 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 proposed retaining wall will be constructed predominantly within a fill section. There are various types of retaining walls that could be utilized for retaining earthen pressures in fill areas. This section discusses several earth retaining structures that could be used for the proposed project. Based on the proposed grading, the proposed wall will be in a fill area, adjacent to the proposed roadway. Several typical wall types are described in the section below.

4.1.1 CIP Concrete Cantilever Walls

Cast-In-Place (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, formwork, 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

A Mechanically Stabilized Earth (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 30 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 either retaining wall for this project. The wall section should be analyzed to determine adequate factors of safety relative to overturning failure. The contractor is responsible for providing a detailed internal stability design for the wall. The final wall design should be submitted to the structural design team for review prior to commencing construction of the wall.

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 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. Soil parameters for the site are included in **Appendix F.**

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 (H_{eq}) of soil. **Table 8** provides the equivalent heights of soils for vehicular loadings on retaining walls.

Table 8 - 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 free draining system through panel joints 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 three (3) 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 suitable native sandy loam and sand with gravel soils. Bearing resistance for the retaining wall shall be evaluated at the strength limit state using load factors (See **Table 7**), and factored bearing resistance. The bearing resistance factor, ϕ_b, is 0.65 for an MSE wall 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 9** presents the proposed bearing elevation and recommended bearing resistances of suitable materials to support the wall system.

Table 9 – Recommended Bearing Resistance (MSE Wall)

Elevation* (feet)	Nominal Resistance (ksf)	Factored Bearing Resistance (ksf)	Bearing Resistance for 1-inch Settlement Service Limit (ksf)	Anticipated Bearing Soil
520.4 - 522.1	91.5	59.5	27.5	Medium Dense to Very Dense Sandy Loam

*Elevations estimated from GPE dated 12/13/2022

The minimum depth of the wall foundation 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, such as topsoil or unsuitable debris laden fill materials. Based on the results of the subsurface exploration, we anticipate the wall would be supported upon the soil types noted in **Table 9.**

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-on-soil beneath the retaining wall. A maximum frictional coefficient of 0.53 (tan 28 degrees) could be used for determining the sliding resistance for the soil-to-soil interfaces. The retaining wall must be wide enough to resist overturning forces, and the location of the resultant forces shall be within the middle two-thirds of the base width.

4.4 Wall Settlement

Settlement of the proposed wall system depends on the foundation size and bearing resistance, as well as the strength and compressibility characteristics of the underlying bearing soil. Assuming the foundation subgrade has been prepared as recommended above and the service bearing resistances as noted in **Table 9** are used, the settlement of the retaining wall is anticipated to be less than 1 inch. Differential settlement between two points of 100 feet apart along the length of the wall will be ½ inch or less.

4.5 Global Slope Stability

Based on the preliminary information provided by WSP, the retaining wall should be designed for external stability of the wall system. The geometry in **Table 10** was used to evaluate the proposed MSE wall.

Table 10 – MSE Wall Description at Station 11+95

**Based on preliminary drawings provided

Description	Value
Maximum total height of the retaining wall (H)*	17 feet
Minimum length of reinforcement (MSE Wall) 0.7XH or 8.0 feet**	12 feet
Unit weight of the retained soil (embankment)	134 pcf
Unit weight of the reinforced soil mass	120 pcf

* Wall height is calculated from top of wall coping to top of the leveling pad
**Actual minimum length may be greater than 0.7H depending on structural analysis

The actual wall width and the total height of the wall should be based on the 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 10**) for the proposed MSE retaining wall. The analyses were performed at Station 11+95. The results of the analyses are shown in **Table 11**.

Table 11 – Retaining Wall Global Slope Stability Analyses Results

Analysis Exhibit	Location	Wall Type	Analysis Type	Factor of Safety	Minimum Factor of Safety
Exhibit 1	Station 11+95	MSE	Circular – Short Term	4.0	1.5
Exhibit 2			Circular – Long Term	2.5	1.5

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

4.6 Drainage Recommendations

The wall design should include a free draining system through panel joints to prevent the buildup of hydrostatic forces behind the wall. A drainage system could also 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 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 (SSRBC) (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 foundations. 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 foundation excavation should be removed prior to placement backfill materials above the wall bottom.

5.2 Existing Utilities and Structures

Before proceeding with construction, all existing underground utility lines or structures that will interfere with construction should be completely relocated from the proposed construction areas. Where possible, existing utility lines that are to be abandoned in place should be removed and/or plugged with a minimum of 2 feet of cement grout. All excavations resulting from underground utilities or structure removal activities should be cleaned of loose and disturbed materials, including all previously placed backfill, and backfilled with suitable fill materials in accordance with the requirements of this section. During the clearing and stripping operations, positive surface drainage should be maintained to prevent the accumulation of water.

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 the Construction Manual. Earth-moving operations should be avoided during excessively cold or wet weather to avoid freezing of softening subgrade soils.

Suitable structural fill materials shall be of a nature that will compact and develop stability satisfactory to the geotechnical engineer. Structural fill shall consist of crushed limestone or recycled concrete consistent with IDOT CA-6 gradation or medium plasticity silty clays. Suitable structural fill should meet the IDOT SSRBC requirements.

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

5.5 Groundwater Management

The long-term groundwater level may be at an approximate elevation of 515.8 to 517.5 feet or deeper, within the bedrock. GSG does not anticipate that groundwater related issues occur during construction activity, however, perched water may be encountered within the existing fill materials. 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 retaining wall 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 I-80 Sta. 67+29.78, 390.65' LT
N 1,765,102.773 and E 1,052,594.602
Elev.= 546.815

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

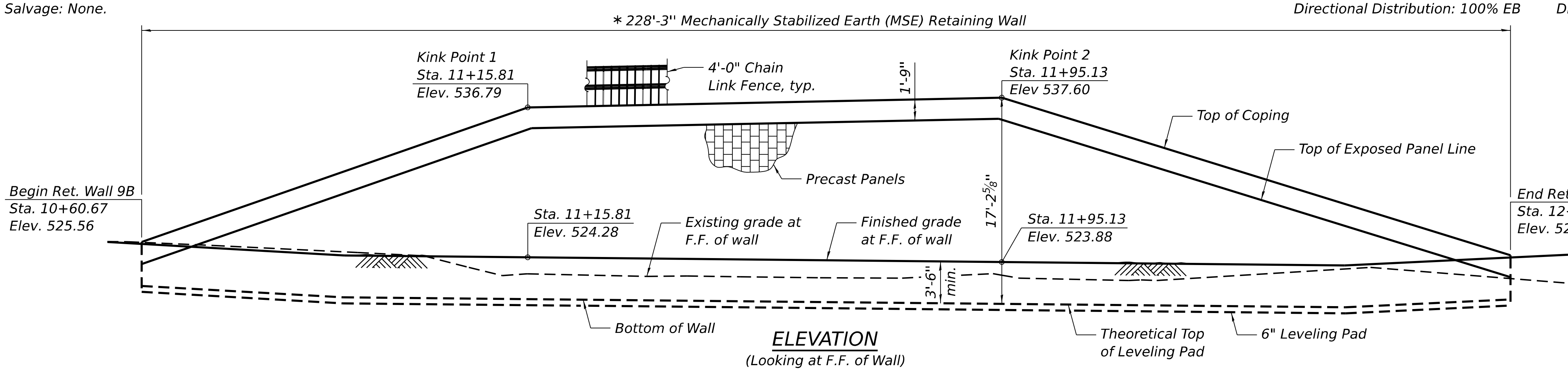
f'c = 3,500 psi
fy = 60,000 psi (Reinforcement)

HIGHWAY CLASSIFICATION

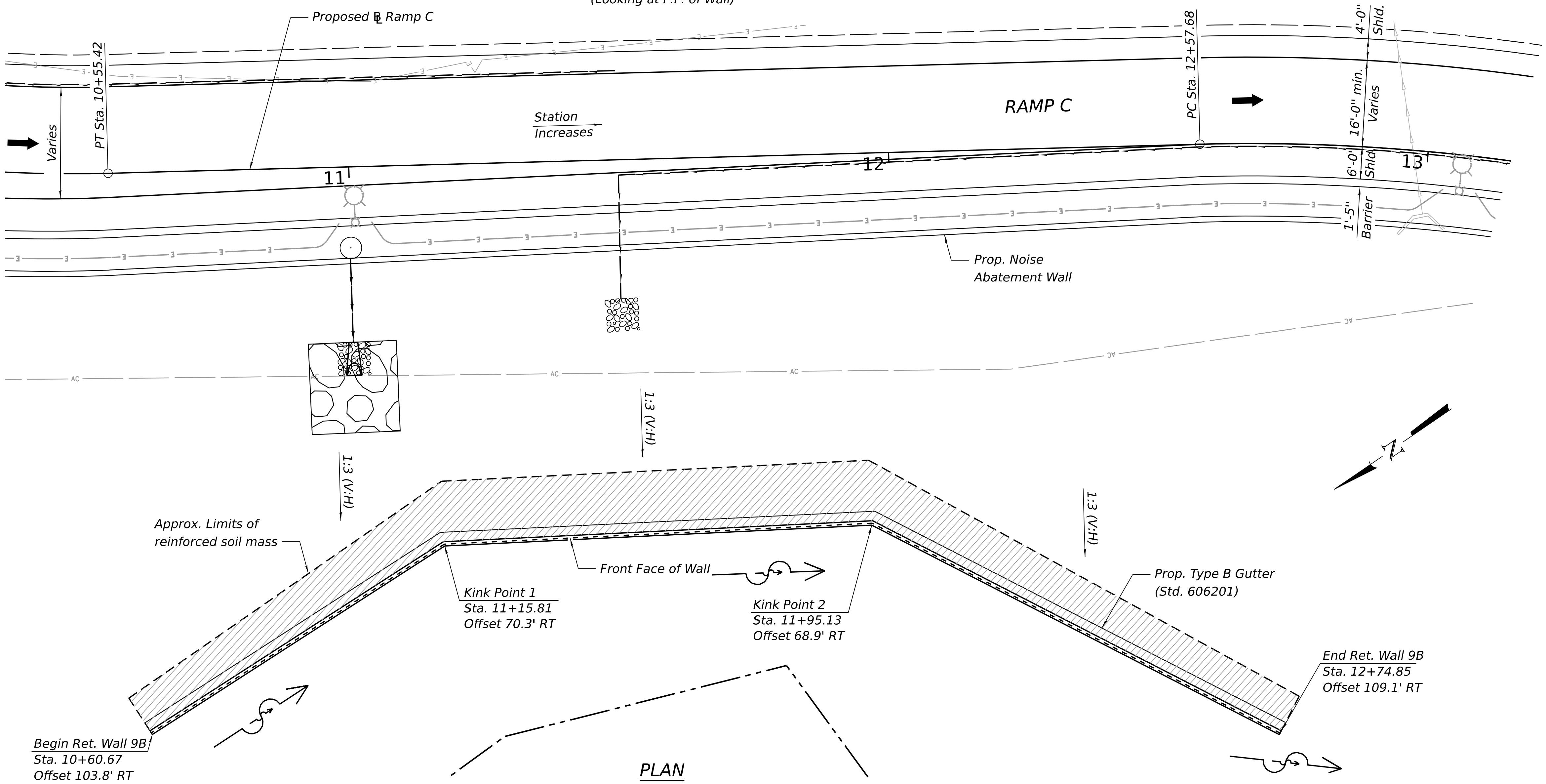
I-80	Ramp C
Functional Class: Interstate	Functional Class: Interstate
ADT: 91,100 (2017); 133,500 (2040)	ADT: 4,590 (2017); 12,500 (2040)
ADTT: 19,241 (2017); 28,169 (2040)	ADTT: 395 (2017); 1,075 (2040)
Design Speed: 70 m.p.h.	Design Speed: 35 m.p.h.
Posted Speed: 65 m.p.h.	Posted Speed: 35 m.p.h.
One-Way Traffic	One-Way Traffic
Directional Distribution: 100% EB	Directional Distribution: 100% WB

NOTES:

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



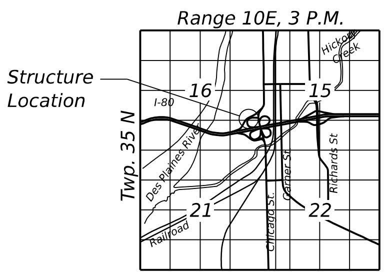
ELEVATION
(Looking at F.F. of Wall)



PLAN

LEGEND

- Soil Borings
- Exist. Underground Electric
- Exist. Underground Storm Sewer
- Exist. ROW Access Control (To be removed)
- Prop. Underground Electric
- Prop. Storm Sewer
- Prop. Pipe Underdrain
- Prop. ROW
- F.F. Front Face
- B.F. Back Face



LOCATION SKETCH

GENERAL PLAN AND ELEVATION
RETAINING WALL ALONG RAMP C
F.A.I. RTE. 80 - SEC 2017-057F
WILL COUNTY
STA. 10+60.67 TO STA. 12+74.85
STRUCTURE NO. 099-W129

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	CHECKED - RRD	REVISED -	
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PLOT DATE = 12/13/2022	CHECKED - 10/7/2022	REVISED -	

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

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

GENERAL PLAN AND ELEVATION
STRUCTURE NO. 099-W129

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

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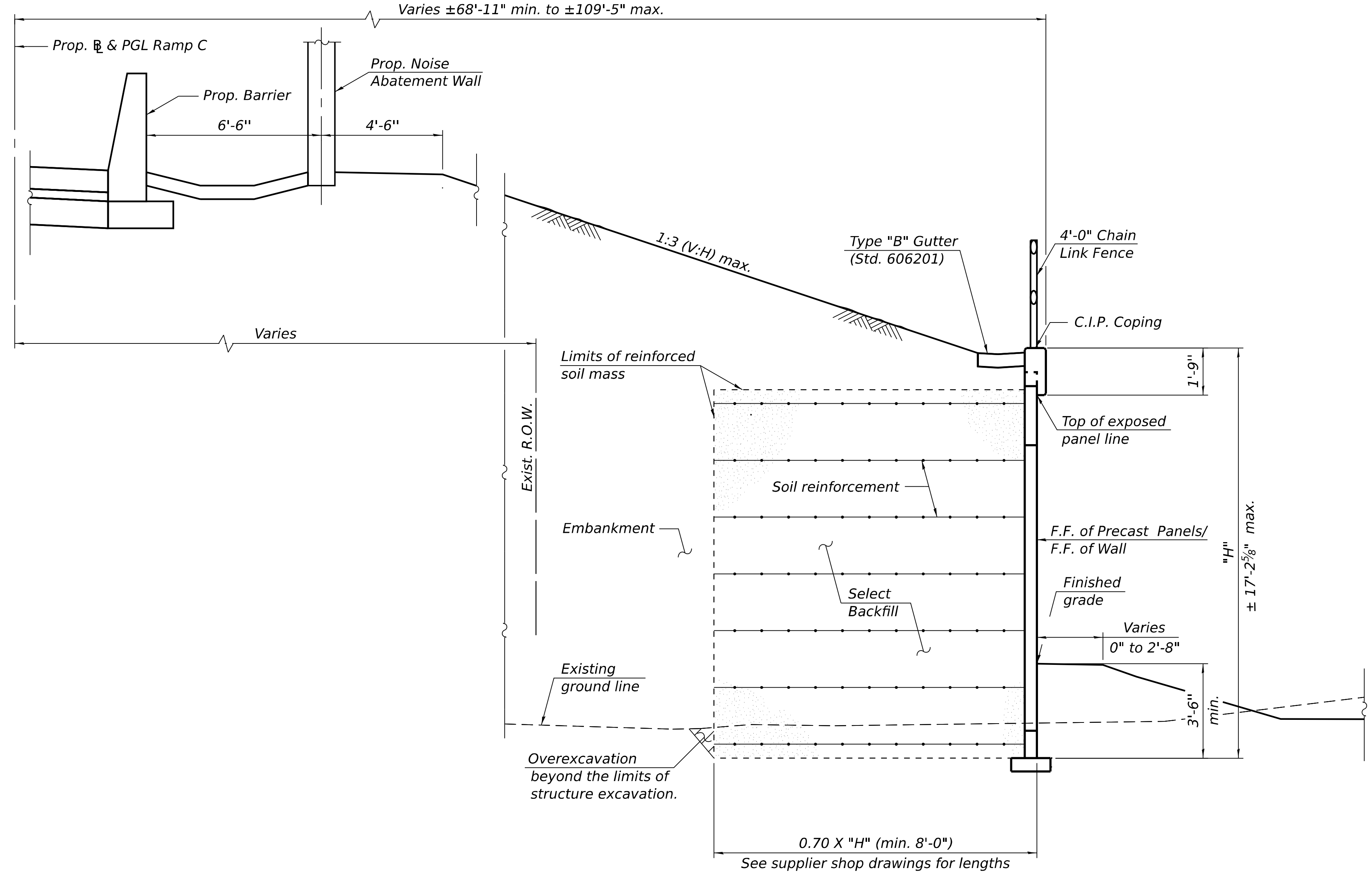
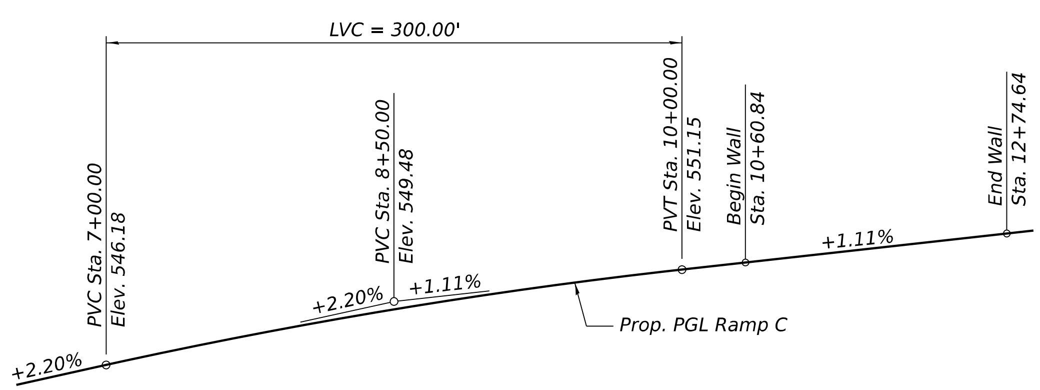
12/13/2022 3:13:45 PM



TYPICAL SECTION THRU MSE WALL
(Looking West)



RAMP C PROFILE GRADE
(Along Prop. ℄ Ramp C)

PR RAMP C CURVE 1

P.I. Sta. = 9+40.44
Δ = 58°19'14" (LT)
D = 22°55'06"
R = 250.00'
T = 139.49'
L = 254.47'
E = 36.28'
e = 6.0%
P.C. Sta. = 8+00.95
P.T. Sta. = 10+55.42

PR RAMP C CURVE 2

P.I. Sta. = 14+34.93
Δ = 52°25'33" (RT)
D = 15°54'56"
R = 360.00'
T = 177.24'
L = 329.40'
E = 41.27'
e = 6.0%
P.C. Sta. = 12+57.68
P.T. Sta. = 15+87.08

WALL DETAIL AND TYPICAL SECTIONS
RETAINING WALL ALONG RAMP C
F.A.I. RTE. 80 - SEC 2017-057F
WILL COUNTY
STA. 10+60.67 TO STA. 12+74.85
STRUCTURE NO. 099-W129

MODEL: Default
FILE NAME: 099XXXX-62F94-sht-TSL_W98_002.dgn

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

USER NAME = jsandoval	DESIGNED - JS	REVISED -	STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION
	CHECKED - RRD	REVISED -	
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PLOT DATE = 12/13/2022	CHECKED - 10/7/2022	REVISED -	

WALL DETAIL AND TYPICAL SECTION
STRUCTURE NO. 099-W129

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 3:13:53 PM

Appendix B

Soil Boring Location Plan and Subsurface Profile

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PLOT SCALE = $SCALE$
USER NAME = $USER$

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

USER NAME	= $USER$	DESIGNED	- MH
SHEET SIZE	= $SHEETSIZE$	DRAWN	- NN
PLOT SCALE	= $SCALE$	CHECKED	- DE
PLOT DATE	= $DATE$	DATE	- 12/13/2022

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

RETAINING WALL NO. 9B

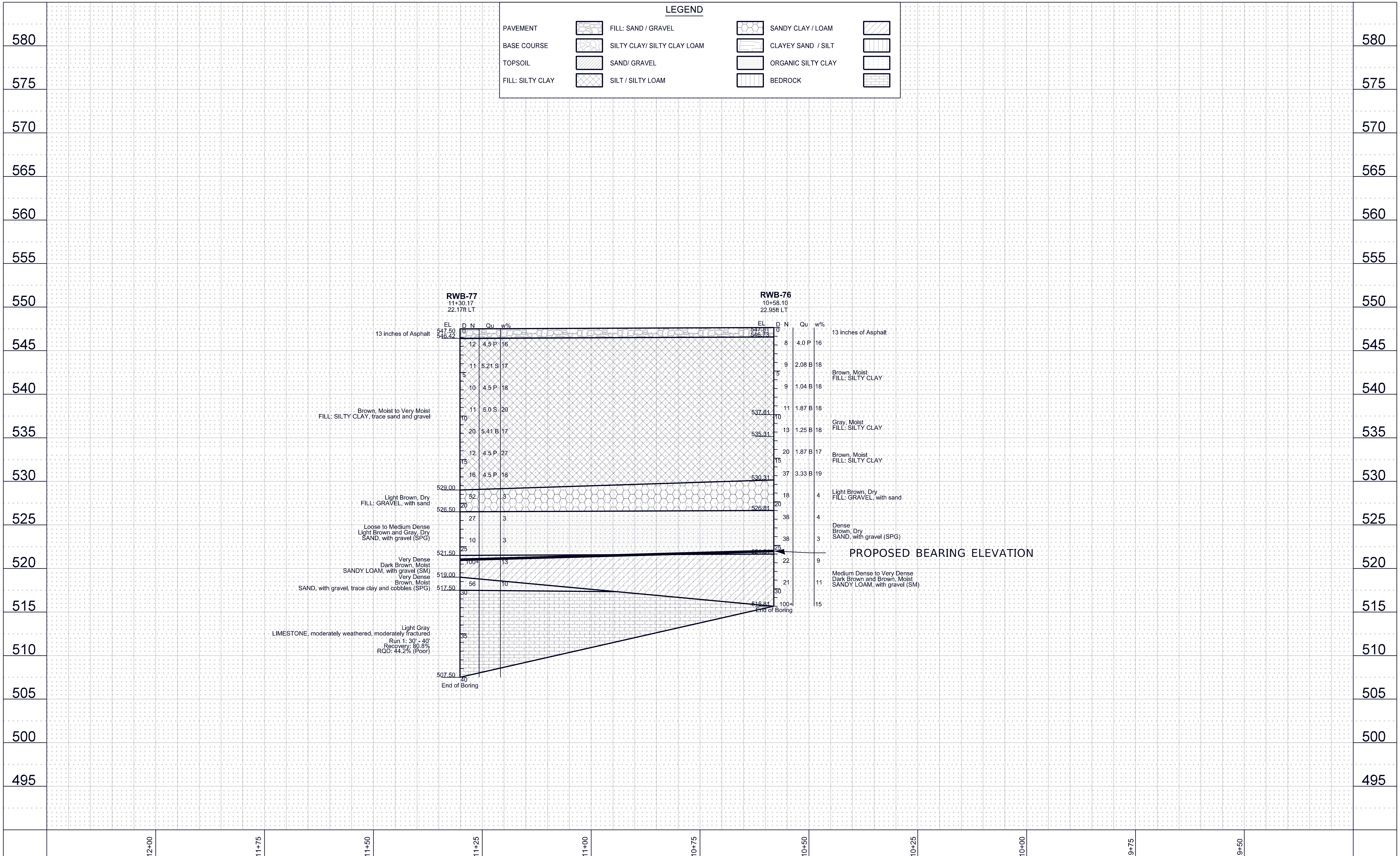
SOIL BORING LOCATION PLAN
JOLIET, ILLINOIS

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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TIME = 2:02:43 PM
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USER NAME = nnano

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

USER NAME = nnano	DESIGNED - MH		
SHEET SIZE = 1.41667x0.916667 (ft.)	DRAWN - NN		
PLOT SCALE = 1200:0.0000 ':" / ft.	CHECKED - DE		
PLOT DATE = 12/13/2022	DATE - 12/13/2022		

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

RETAINING WALL NO. 9B
SOIL BORING PROFILE

SCALE: AS NOTED | SHEET 1 OF 1 SHEETS | STA. 12+00 TO STA. 9+50

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 6/22/22

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 10 - Ramp C Sta 7+89 **LOGGED BY** DD

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

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

STRUCT. NO.
Station

BORING NO. RWB-76
Station 10+58.1017
Offset 22.95ft LT
Ground Surface Elev. 547.81 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 (%)
13 inches of Asphalt	546.73				
Brown, Moist FILL: SILTY CLAY			3 3 5	4.0 P	16
		-5	1 3 6	2.1 B	18
			1 4 5	1.0 B	18
	537.81	-10	1 4 7	1.9 B	18
Gray, Moist FILL: SILTY CLAY	535.31		2 4 9	1.3 B	18
Brown, Moist FILL: SILTY CLAY Cobbles at 13.5 feet		-15	6 9 11	1.9 B	17
Cobbles at 16 feet	530.31		2 6 31	3.3 B	19
Light Brown, Dry FILL: GRAVEL, with sand Cobbles at 18.5 feet		-20	7 10 8		4
Light Brown, Dry FILL: GRAVEL, with sand *(continued)* Cobbles at 20 feet	526.81		6 27 11		4
Dense Brown, Dry SAND, with gravel (SPG)		-25	40 25 13		3
	521.81				
Medium Dense to Very Dense Dark Brown and Brown, Moist SANDY LOAM, with gravel (SM)			10 13 9		9
		-30	12 9 12		11
	515.81		5 10 50/2"		15
Auger refusal at 32 feet 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)

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

SOIL BORING LOG

Date 6/20/22

ROUTE I-80 **DESCRIPTION** Retaining Wall No. 10 - Ramp C Sta 7+89 **LOGGED BY** KA

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

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

STRUCT. NO. ______
Station ______

BORING NO. RWB-77
Station 11+30.1683
Offset 22.17ft LT
Ground Surface Elev. 547.50 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	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
13 inches of Asphalt				
546.42				
Brown, Moist to Very Moist FILL: SILTY CLAY, trace sand and gravel		16 5 7	4.5 P	16
	-5	6 5 6	5.2 S	17
		4 4 6	4.5 P	18
	-10	4 6 5	5.0 S	20
		7 9 11	5.4 B	17
	-15	4 4 8	4.5 P	27
		5 5 11	4.5 P	18
529.00				
Light Brown, Dry FILL: GRAVEL, with sand	-20	17 37 15		3

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Light Brown, Dry FILL: GRAVEL, with sand *(continued)*				
526.50				
Loose to Medium Dense Light Brown and Gray, Dry SAND, with gravel (SPG) Cobbles at 21 feet		13 18 9		3
	-25	17 7 3		3
521.50				
Very Dense Dark Brown, Moist SANDY LOAM, with gravel (SM)		6 9 50/3"		13
519.00				
Very Dense Brown, Moist SAND, with gravel, trace clay and cobbles (SPG) Auger refusal at 30 feet		4 11 45		10
517.50	-30			
Light Gray LIMESTONE, moderately weathered, moderately fractured Run 1: 30' - 40' Recovery: 80.8% RQD: 44.2% (Poor)				
	-35			
507.50	-40			
End of Boring				

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

I-80 Retaining Wall #9B
RWB-77
Will County, IL

Top

Depth = 30 ft
Elev. = 517.5ft

Depth = 40 ft
Elev. = 507.5 ft

Bottom

Boring No.	Run	Depth (ft)	Recovery (%)	RQD (%)	RQD Classification	Compressive Strength (psi)	Description
RWB-77	1	30' – 40'	80.8	44.2	Poor	23,447	Gray Limestone Moderately Weathered, Moderately Fractured

Appendix D
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-77	Bulk/Prep		MC/CS	
Sample Depth (ft):	37-37.5	Tester:	AJ	Tester:	AJ
Lithological Description:	Limestone	Date:	06/28/22	Date:	06/28/22
Formation Name:		Load Direction:	Vertical	Angle Drilled:	Vertical
Appearance (e.g. cracks, shearing, spalling):					

Bulk Density Determination

	1	2	3	Average
Height, *in.*	4.2460	4.2425	4.2445	4.2443
Diameter, *in.*	1.9910	1.9855	1.9855	1.9873

Specimen Mass, *g*	589.9	Ratio (2.0-2.5)
Bulk Density, *pcf*	170.7	2.14

Moisture Condition - D2216

Container ID	KITKAT
container, *g*	226.6
container + wet rock, *g*	478.2
container + dry soil, *g*	475.9
moisture content, *w%*	0.9

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

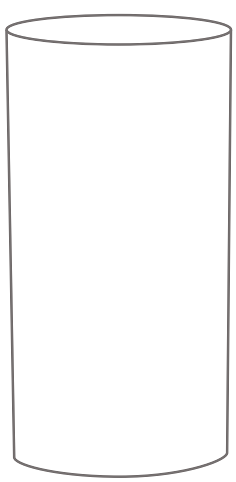
Seating Load (≤1000 psi)	1000
Rate of Loading (73-145 psi/s)	75
Time to Failure (2-15 min)	5 min 6 sec
Load @ Failure, *lbf*	72,732
Uniaxial Compressive Strength, *psi*	23,447

Remarks

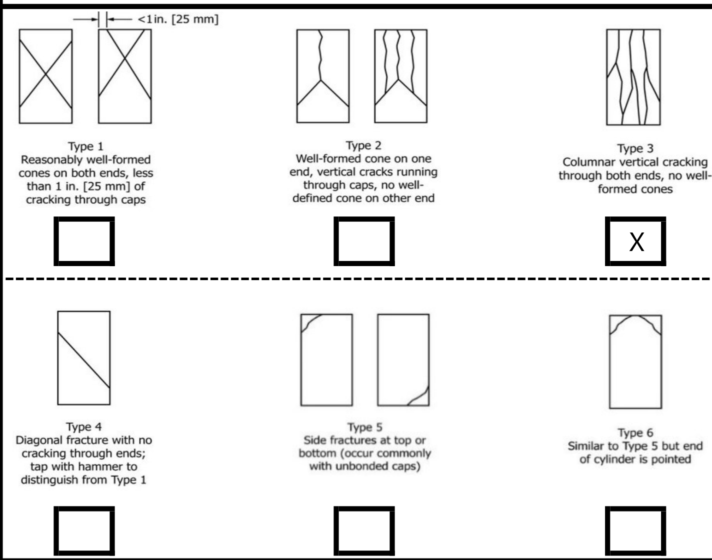
Best efforts have been made for the specimen to meet the required tolerances of D4543. See IH3 Procedure for efforts made.

After Preparation

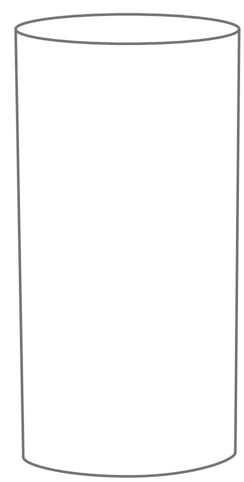
Sketch



After Break (check applicable appearance)

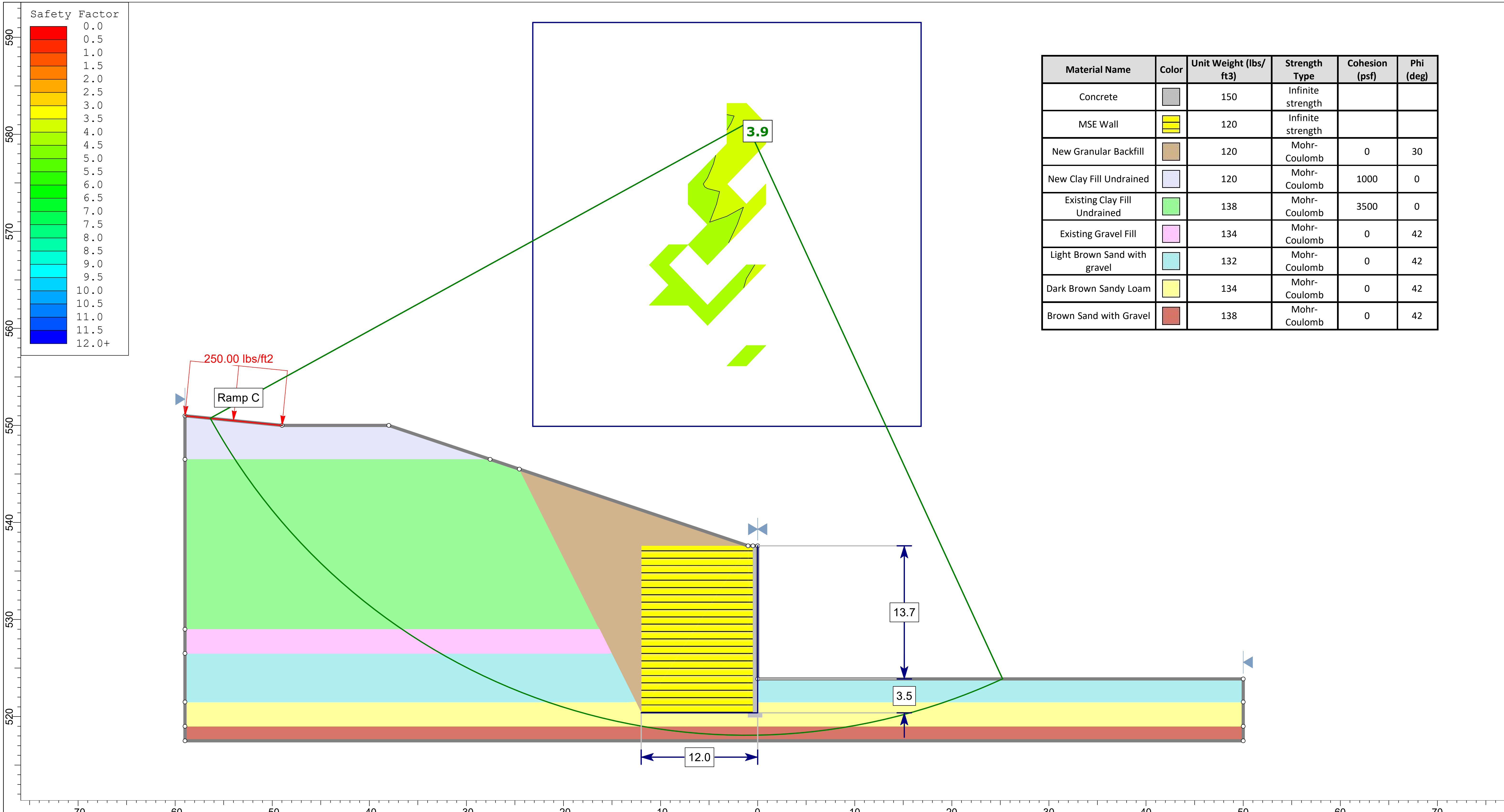


Sketch if Other:



Form ID	TF-RCS	Reviewed By	SL
Revision Date	10/21/2021	Review Date	06/30/22

Appendix E
Slope Stability Analysis Exhibits

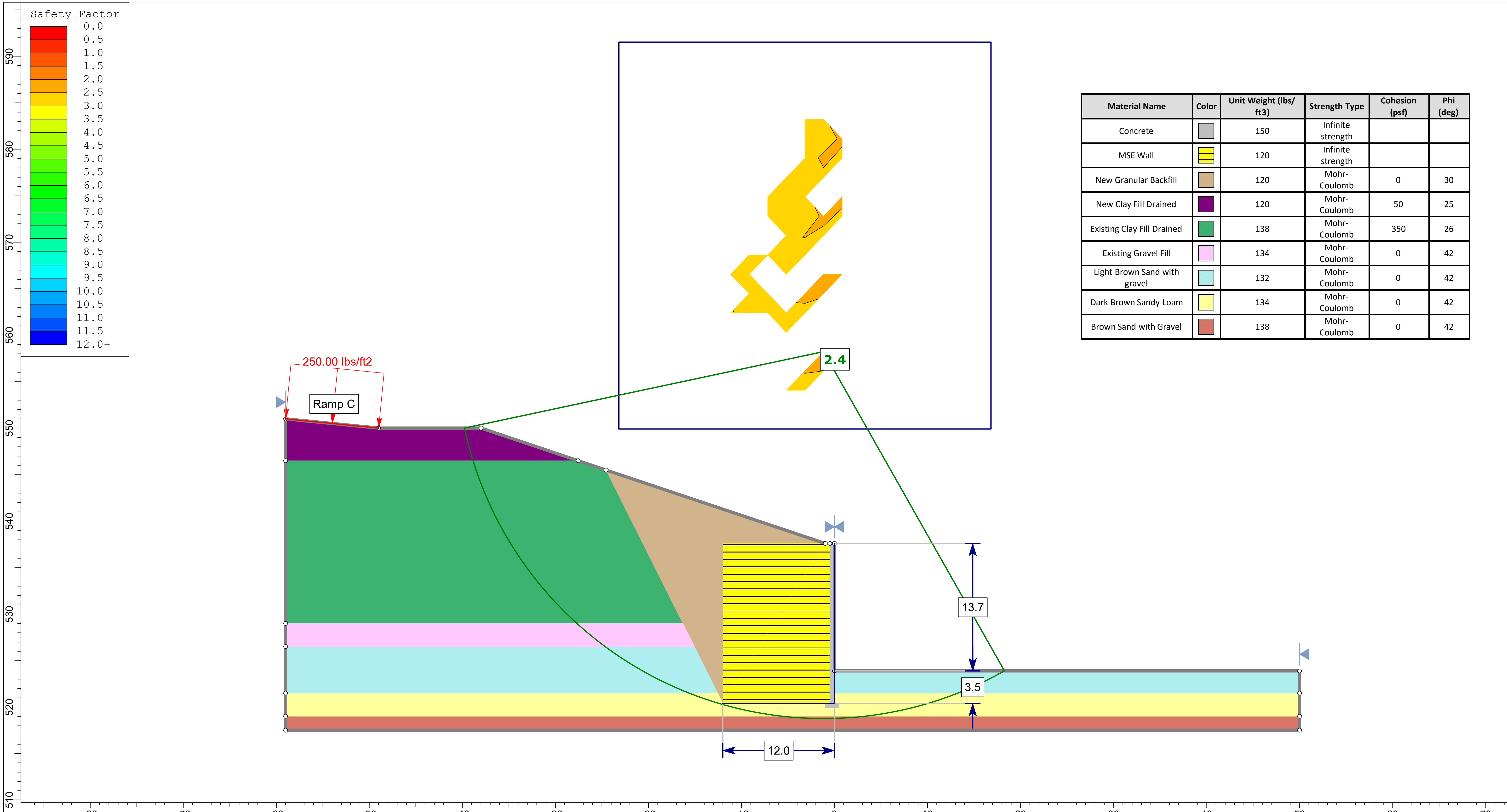


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Concrete		150	Infinite strength		
MSE Wall		120	Infinite strength		
New Granular Backfill		120	Mohr-Coulomb	0	30
New Clay Fill Undrained		120	Mohr-Coulomb	1000	0
Existing Clay Fill Undrained		138	Mohr-Coulomb	3500	0
Existing Gravel Fill		134	Mohr-Coulomb	0	42
Light Brown Sand with gravel		132	Mohr-Coulomb	0	42
Dark Brown Sandy Loam		134	Mohr-Coulomb	0	42
Brown Sand with Gravel		138	Mohr-Coulomb	0	42

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

SLIDEINTERPRET 9.020

Project	PTB 198-003 - Retaining Wall 9B - Station 11+95
Analysis Description	Exhibit 1 - Circular Failure Short Term
Drawn By	MH
Company	GSG Consultants, Inc.
Date	12/13/2022, 12:04:02 PM
File Name	rw-9b.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Concrete		150	Infinite strength		
MSE Wall		120	Infinite strength		
New Granular Backfill		120	Mohr-Coulomb	0	30
New Clay Fill Drained		120	Mohr-Coulomb	50	25
Existing Clay Fill Drained		138	Mohr-Coulomb	350	26
Existing Gravel Fill		134	Mohr-Coulomb	0	42
Light Brown Sand with gravel		132	Mohr-Coulomb	0	42
Dark Brown Sandy Loam		134	Mohr-Coulomb	0	42
Brown Sand with Gravel		138	Mohr-Coulomb	0	42

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

SLIDEINTERPRET 9.020

Project	PTB 198-003 - Retaining Wall 9B - Station 11+95
Analysis Description	Exhibit 2 - Circular Failure Long Term
Drawn By	MH
Company	GSG Consultants, Inc.
Date	12/13/2022, 12:04:02 PM
File Name	rw-9b.slmd

APPENDIX F

SOIL DESIGN PARAMETERS

Table F1 – Retaining Wall #9B Soil Parameters

Approximate Depth Range* (Elevation, feet)	Soil Description	In situ Unit Weight γ (pcf)	Undrained		Drained				
			Cohesion c (psf)	Friction Angle ϕ (Degrees)	Cohesion c (psf)	Friction Angle ϕ (Degrees)	Active Earth Pressure Coefficient (K_a)	Passive Earth Pressure Coefficient (K_p)	At-Rest Earth Pressure Coefficient (K_o)
	New Engineered Clay Fill	120	1,000	0	50	25	0.41	2.46	0.58
	New Engineered Granular Fill	120	0	30	0	30	0.33	3.00	0.50
1.0 to 18.0 (546.5 - 529.5)	Brown and Gray Silty Clay Fill	138	3,500	0	350	26	0.39	2.56	0.56
18.0 to 21.0 (529.5 - 526.5)	Light Brown Gravel with Sand Fill	134	0	42	0	42	0.20	5.04	0.33
21.0 to 26.0 (526.5 - 521.5)	Loose to Dense Light Brown and Gray Sand with Gravel	132	0	42	0	42	0.20	5.04	0.33
26.0 to 32.0 (521.5 - 515.5) RWB-76 1.0 to 18.0 (546.5 - 529.5) RWB-77	Medium Dense to Very Dense Dark Brown Sandy Loam with Gravel	134	0	42	0	42	0.20	5.04	0.33
28.5 to 30.0 (519.0 - 517.5) RWB-77 only	Very Dense Brown Sand with Gravel	138	0	42	0	42	0.20	5.04	0.33
Below Auger Refusal	Moderately Weathered Moderately Fractured Limestone	150	0	45	0	45	0.17	5.83	0.29

* Depth below approximate boring surface elevations of 547.5 feet