# STRUCTURE GEOTECHNICAL REPORT
# INTERSTATE 80 BRIDGES
# OVER ROCK RUN CREEK
# EX SNs 099-0046 (EB) AND 099-0047 (WB)
# PR SNs 099-8318 (EB) AND 099-8319 (WB)
# WILL COUNTY, ILLINOIS

**For**
**Stantec**

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**Submitted by**

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**11. Abstract**

Two new, three-span bridges will replace the existing bridges carrying Interstate 80 over Rock Run Creek in Will County, Illinois. The proposed structures will have back-to-back of abutment lengths of 128.3 feet and out-to-out widths of 84.8 feet (EB) and 74.8 feet (WB). The proposed east and west abutment cap base elevations are 549.00 and 548.54 feet, respectively, whereas the pile cap base elevations at the new piers are both estimated the bottom of the encasement walls at elevation 537.8 feet. This report provides geotechnical recommendations for the design and construction of the proposed approach embankments, approach slabs, and bridge foundations.

The pavement structure along I-80 consists of 14 to 20 inches of asphalt over aggregate base. The bridge deck consists of 18 inches of concrete pavement. Beneath the I-80 pavement structure, the general lithologic profile includes up to 15.5 feet of embankment materials consisting of medium dense to very dense gravelly silty loam to loam fill overlying stiff to hard silty clay to silty clay loam and clay loam with intermittent layers of medium dense gravelly sandy loam. Up to 30 inches of very soft silty clay loam followed by very dense sandy gravel is present at the streambed elevation just above the bedrock. Dolostone bedrock was encountered at elevations of about 536 to 534 feet. The groundwater level was observed in boreholes at elevations ranging from 542 to 536 feet.

The approach embankments behind the east and west abutments will undergo up to 0.7 inches of total long-term settlement under the new embankment loads. Global stability analyses at the embankments show FOS meeting the IDOT minimum requirement of 1.5. The maximum factored soil bearing resistance for the bridge approach slab is 2,500 psf.

The bridge abutments could be supported on driven or drilled piles. To support the integral abutments, driven HP12x53, HP 12x74, HP 14x73, and HP14x89 steel piles will provide 230 to 388 kips of factored resistance at total lengths of 18 to 19 feet. HP12x53, HP 12x74, HP 14x73, and HP14x89 steel piles set in rock will provide about 542 to 914 kips of factored resistance. We do not anticipate the need for downdrag allowances on the piles. The piers could be supported on either drilled piles, rock-socketed drilled shafts, or spread footings. HP12x53, HP 12x74, HP 14x73, and HP14x89 steel piles set in rock will provide about 542 to 914 kips of factored resistance. Rock-socketed shafts have factored resistances of about 810 to 2650 kips for 3.0- to 5.0-foot diameter sockets. Spread footings at the piers can be designed based on a maximum factored bearing resistance of 15 ksf.


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# TABLE OF CONTENTS

**1.0 INTRODUCTION ..... 1**

1.1 EXISTING STRUCTURES AND GROUND CONDITIONS ..... 1

1.2 PROPOSED STRUCTURES ..... 2

**2.0 METHODS OF INVESTIGATION ..... 3**

2.1 FIELD INVESTIGATION ..... 3

2.2 LABORATORY TESTING ..... 4

**3.0 INVESTIGATION RESULTS ..... 4**

3.1 LITHOLOGICAL PROFILE ..... 4

3.2 GROUNDWATER CONDITIONS ..... 6

**4.0 FOUNDATION ANALYSIS AND RECOMMENDATIONS ..... 6**

4.1 SEISMIC DESIGN CONSIDERATIONS ..... 7

4.2 SCOUR CONSIDERATIONS ..... 7

4.3 APPROACH EMBANKMENTS AND SLABS ..... 9

*4.3.1 Settlement ..... 9*

*4.3.2 Global Stability ..... 9*

*4.3.3 Approach Slabs ..... 9*

4.4 STRUCTURE FOUNDATIONS ..... 10

*4.4.1 Driven Piles ..... 11*

*4.4.2 Piles Set in Rock ..... 12*

*4.4.3 Drilled Shafts ..... 14*

*4.4.4 Spread Footings ..... 16*

*4.4.3 Lateral Loading ..... 16*

4.5 STAGE CONSTRUCTION ..... 18

**5.0 CONSTRUCTION CONSIDERATIONS ..... 19**

5.1 SITE PREPARATION ..... 19

5.2 EXCAVATION, DEWATERING, AND UTILITIES ..... 19

5.3 FILLING AND BACKFILLING ..... 20

5.4 EARTHWORK OPERATIONS ..... 20

5.5 PILE INSTALLATION ..... 20

5.6 DRILLED SHAFTS ..... 20

**6.0 QUALIFICATIONS** ..... **21**

REFERENCES

EXHIBITS

*1. Site Location Map*

*2. Site and Regional Geology*

*3. Boring Location Plan*

*4. Soil Profile*

APPENDIX A

*Boring Logs*

APPENDIX B

*Laboratory Test Results*

APPENDIX C

*Bedrock Core Photographs*

APPENDIX D

*Global Stability Analysis*

APPENDIX E

*General Plan and Elevation Drawing*

APPENDIX F

*Cross Sections*

## List of Tables

Table 1: Recommended Seismic Design Parameters ..... 7

Table 2: Project Scour Data ..... 8

Table 3: Project Design Scour Elevations ..... 8

Table 4: Preliminary Factored Loads and Proposed Pile Cap Elevations ..... 10

Table 5: Estimated Pile Lengths and Tip Elevations for Steel H-Piles Driven to $R_{NMAX}$ ..... 12

Table 6: Estimated Resistances for Piles Set in Rock ..... 13

Table 7: Estimated Drilled Shaft Resistances and Base Elevations (Rock-Socketed Shafts) ..... 15

Table 8: Recommended Soil Parameters for Lateral Load Analysis at West Abutments ..... 17

Table 9: Recommended Soil Parameters for Lateral Load Analysis at Pier 1 ..... 17

Table 10: Recommended Soil Parameters for Lateral Load Analysis at Pier 2 ..... 17

Table 11: Recommended Soil Parameters for Lateral Load Analysis at East Abutments ..... 18

Table 12: Bedrock Parameters for Lateral Load Analysis ..... 18

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## 1.0 INTRODUCTION

This report presents the results of our subsurface investigation, laboratory testing, geotechnical evaluations, and recommendations in support of the design and reconstruction of the bridges carrying eastbound (EX SN 099-0046) and westbound (EX SN 099-0047) Interstate 80 (I-80) over Rock Run Creek in Will County, Illinois. On the USGS *Channahon Quadrangle 7.5 Minute Series* map, the project is located at Section 27, Tier 35 N, Range 9 E of the Third Principal Meridian. A *Site Location Map* is presented as Exhibit 1. The bridge replacements are part of the proposed widening and reconstruction of I-80 from east of Ridge Road to west of Houbolt Road in Will County, Illinois. These bridges will be reconstructed as part of Contract ML-4.

The purpose of this investigation was to characterize the site soil and groundwater conditions, perform geotechnical analyses, and provide recommendations for the design and construction of the proposed bridge foundations, approach embankments, and approach slabs.

### 1.1 Existing Structures and Ground Conditions

Based on the *Bridge Condition Report (BCR),* dated March 11, 2020 and provided by Stantec, Wang Engineering, Inc. (Wang) understands the existing bridges were originally built in 1963 as three-span structures supported by cast-in-place reinforced concrete stub abutments and solid pier stems. The piers are supported on spread footings on rock, whereas the abutments are supported on 37-ton capacity steel piles. The approach slabs and wingwalls are supported on creosoted timber piles. The bridges have lengths of 104.8 feet from back-to-back of abutments and out-to-out widths of 44.0 feet. There are also reinforced concrete wingwalls and concrete slope walls at the ends of the structures. The structures were repaired in 1992.

In general, the relief within the project area is flat to gently undulatory. At the bridge crossing, the Rock Run Creek runs through a 50-foot wide channel. The surface elevation at the I-80 roadway level is about 555 feet and elevation below deck at the Rock Run Creek level is about 543 feet.

In the project area (see Exhibit 2), an about 10-foot thick overburden covers the shallow bedrock. The surficial cover within the project area consists of about 2-foot thick clay silt and sandy loam of the Cahokia Alluvium, over sand and gravel outwash deposits of the Henry Formation resting unconformably over bedrock (Bauer et al. 1991, Hansel and Johnson 1996, Leighton et al. 1948 Willman et al. 1971). The top of the bedrock is highly to slightly weathered. The bedrock is made up of shaly and vuggy Silurian-age dolostone resting over Ordovician-age bedrock made of interbedded shale, mudstone, and dolostone. Within the project area, the top of bedrock is mapped at about 535 feet elevation. The site is located on the eastern flank of the Wisconsin Arch (Willman 1971), on the northern, downthrown block of the inactive Sandwich Fault Zone. The shallow bedrock is highly to moderately weathered and may show the presence of cavities more likely filled with fine sediment. The fault zone runs northwest to southeast and can be traced about 100 feet south of the bridge area (Kolata 2005). No underground mines are known in the area. The nearest coal mines are located about 15.0 miles southwest, in the vicinity of Morris, Grundy County (ISGS 2021). No active faults or underground mines are known in the area.

### 1.2 Proposed Structures

Based on the *General Plan and Elevation Drawings* (Appendix E), prepared by Lin Engineering, Ltd, (Lin) and dated December 19, 2022, Wang understands the existing three-span bridges will be removed and replaced with two new, three-span bridges with integral abutments and two solid wall encased bent piers. The new bridges will have back-to-back of abutments length of 128.3 feet and out-to-out widths of 84.8 feet (EB) and 74.8 feet (WB) to accommodate two 12-foot wide travel lanes, a 12-foot wide inside shoulder, a 12-foot wide future lane/shoulder, a 12-foot wide auxiliary lane, a 12-foot wide outside shoulder (WB only), a 10-foot wide outside shoulder (EB only), a 12-foot wide ramp (EB only), and 1.4-foot wide parapets.

Based on the provided *Cross Sections* (Appendix F), the existing grade elevation along I-80 is approximately 555 to 556 feet and the grade will be raised by up to 12 inches along each centerline at the west and east bridge approaches. From the design drawings, we estimate the new east and west abutments will be constructed about 13.0 feet behind the existing ones, whereas the new piers will be constructed at almost the same location as the existing piers. About 2.0 to 3.0 feet of new fill will be placed along the existing median at the west and east approaches, respectively, to facilitate the inward widening of the bridges by about 25.0 feet at the north and south sides of the eastbound and westbound bridges.

As per the *Cross Sections* (Appendix F), the side slopes at the west approach will be graded at slopes of 1:3 and 1:3.5 (V: H) and up to 4.0 feet of new fill will be placed along the existing slopes. The east approach will be graded at a slope of 1:4 (V: H) and up to 5.0 feet of new fill will be placed along the existing eastbound embankment slope. The *GPE* (Appendix E) shows end slopes armored with riprap and graded at 1:2 (V: H) with an 18-foot wide future bike path proposed on the east and west ends.

## 2.0 METHODS OF INVESTIGATION

The following sections outline the subsurface and laboratory investigations performed by Wang.

### 2.1 Field Investigation

The subsurface investigation consisted of six bridge borings, designated as RRC-BSB-01 to RRC-BSB-06, drilled by Wang in August of 2021. A Shelby tube sample was collected at a depth of 6.0 feet in Boring RRC-BSB-01-ST. The borings were drilled from elevations of 555.7 to 554.9 feet and were advanced to depths of 27.0 to 46.0 feet below the ground surface (bgs). The as-drilled northings and eastings were acquired with a mapping-grade GPS unit. Stations, offsets, and elevations were provided by Stantec. Boring location data are presented in the *Boring Logs* (Appendix A) and the as-drilled boring locations are shown in the *Boring Location Plan* (Exhibit 3).

To supplement our subsurface investigation, we considered the historical borings drilled as part of the original bridge design and designated as Borings 1 to 8. These borings are also included in the *Boring Logs* (Appendix A) and shown in the *Boring Location Plan* (Exhibit 3).

A truck-mounted drilling rig, equipped with hollow stem augers, was used to advance and maintain open boreholes. Mud rotary drilling techniques were used from depths of 10.0 to 26.0 feet bgs. Soil sampling was performed according to AASHTO T206, *"Penetration Test and Split Barrel Sampling of Soils."* The soil was sampled at 2.5-foot intervals to the top of bedrock. Bedrock cores were collected from all the borings in 2- to 8-foot runs with an NWD4-sized core barrel. Soil samples collected from each sampling interval were placed in sealed jars and rock cores were placed into marked core boxes and transported to the laboratory for further examination and testing.

Field boring logs, prepared and maintained by Wang field engineers, included lithological descriptions, visual-manual soil (IDH Textural) classifications, results of Rimac and pocket penetrometer unconfined compressive strength tests, and results of Standard Penetration Tests (SPT) recorded as blows per 6 inches of penetration.

Groundwater levels were measured while drilling and at completion of drilling. Since mud rotary drilling techniques were used to advance and maintain open boreholes from the I-80 embankment level, groundwater level recordings were not available at completion of the borings. Prior to being backfilled, Boring RRC-BSB-01, drilled from I-80, was left open to record 24-hour water level readings. Each borehole location was backfilled upon completion with lean grout, soil cuttings, and/or bentonite chips and, where necessary, the pavement surface was restored as much as possible to its original condition.

### 2.2 Laboratory Testing

The soil samples were tested in the laboratory for moisture content (AASHTO T265). Atterberg limits (AASHTO T89 and T90) and particle size analysis (AASHTO T88) tests were performed on selected samples. Unconfined compressive strength tests were performed on selected bedrock cores. Field visual descriptions of the soil samples were verified in the laboratory and index tested soils were classified according to the IDH soil Classification System. The laboratory test results are shown in the *Boring Logs* (Appendix A) and in the *Laboratory Test Results* (Appendix B).

## 3.0 INVESTIGATION RESULTS

Detailed descriptions of the soil conditions encountered during the subsurface investigation are presented in the attached *Boring Logs* (Appendix A) and in the *Soil Profile* (Exhibit 4). Please note that strata contact lines represent approximate boundaries between soil types. The actual transition between soil types in the field may be gradual in horizontal and vertical directions.

Our subsurface investigation results fit into the local geologic context. The borings drilled in the project area encountered man-made ground consisting of gravelly silty loam and native sediments consisting of silty clay alluvium, gravel, and sand outwash of the Henry Formation unconformably resting over highly to moderately weathered dolostone. The bedrock was encountered at about 535 feet elevation.

### 3.1 Lithological Profile

Borings RRC-BSB-01, RRC-BSB-02, RRC-BSB-05, and RRC-BSB-06 were drilled along the I-80 shoulders and revealed the pavement structure consists of 14 to 20 inches of asphalt overlying sandy gravel aggregate base. Borings RRC-BSB-03 and RRC-BSB-04 were drilled through 18 inches of bridge deck. In descending order, the general lithologic succession encountered beneath the pavement includes: 1) man-made ground (fill); 2) stiff to hard silty clay to silty clay loam and clay loam; 3) very dense weathered bedrock; and 4) medium strong to strong, very poor to poor dolostone bedrock.

*1) Man-made ground (fill)*

Beneath the pavement structure, borings RRC-BSB-01, RRC-BSB-01-ST, RRC-BSB-02, RRC-BSB-05, and RRC-BSB-06, drilled along I-80, encountered up to 15.5 feet of fill. The fill mainly consists of medium dense to very dense, brown and gray, gravelly silty loam to loam with SPT N values of 14 blows/foot to more than 50 blows per 2 inches and moisture contents of 5 to 15%. Laboratory index testing on samples from this layer showed liquid limit ($L_L$) values of 22 to 25% and plastic limit ($P_L$) values of 15 to 16%.

An 18-inch thick layer of buried, black silty clay topsoil with a moisture content value of 30% was sampled beneath the fill in Boring RRC-BSB-06. The presence of this layer most likely indicates the boundary between fill and natural soils.

*2) Stiff to hard silty clay to silty clay loam and clay loam*

Beneath the fill, at elevations of 545 to 544 feet, Borings RRC-BSB-01, RRC-BSB-02, and RRC-BSB-06, advanced through up to 7.5 feet of stiff to hard, brown and gray clay to silty clay, silty clay loam, and clay loam with unconfined compressive strength ($Q_u$) values of 3.1 to 4.5 tsf and moisture content values of 8 to 18%. Laboratory index testing on a sample from the silty loam layer showed $L_L$ and $P_L$ values of 33 and 16%, respectively.

A 2.5-foot thick granular layer, consisting of brown, damp, medium dense gravelly sandy loam was found within the cohesive soil in Boring RRC-BSB-02. This layer has an SPT N-value of 20 blows per foot and a moisture content of 8%.

At an elevation of 540 feet Borings RRC-BSB-03 and RRC-BSB-04 sampled a 4- to 30-inch thick layer of very soft, black and gray silty clay loam with $Q_u$ values of 0.1 to less than 0.2 tsf and a moisture content value of 94%. Laboratory index testing on a sample from this layer showed $L_L$ and $P_L$ values of 69 and 32%, respectively. This layer was followed in both borings by a 1.5- to 2.5-foot thick layer of very dense, gray, saturated sandy gravel.

*3) Very dense weathered bedrock*

At elevations of 545 to 537 feet, the borings advanced through 2.0 to 9.5 feet of very dense, weathered bedrock. This soil unit has N values of greater than 50 blows per 6 inches. The weathered bedrock shows spoon refusal and moisture content values of 1 to 9%.

*4) Medium strong to very strong, very poor to poor dolostone bedrock*

At elevations of 535.7 to 534.5 feet (20 to 21 feet bgs), the borings encountered medium strong to very strong, light gray and light bluish gray, thinly bedded, shaly and vuggy dolostone bedrock with greenish silty and clayey infill. The dolostone bedrock RQD ranges from 0 to 41% with rock $Q_u$ values of 7,494 to 11,678 psi. The bedrock core data is shown in the *Bedrock Core Photographs* (Appendix C).

The original bridge design borings (1 to 8) encountered and cored bedrock described as limestone at elevations ranging from 538 to 536 feet and generally consistent with the current borings.

### 3.2 Groundwater Conditions

Borings RRC-BSB-02, RRC-BSB-03, RRC-BSB-04, and RRC-BSB-06 were drilled with hollow stem augers to depths of 17 to 26 feet bgs. While drilling, the groundwater was measured at elevations of 542 to 537 feet (13.5 to 18.5 feet bgs) and at completion of drilling at elevations of 542 to 538 feet (13.5 to 18.0 feet bgs). Boring RRC-BSB-01 was left open to measure the 24-hour groundwater level. The groundwater level was recorded after 24 hours and was measured at an elevation of 536 feet (19 feet bgs). The Rock Run Creek water level was observed at an elevation 542 feet (13.5 to 14.0 feet below the bridge level) during and upon completion of drilling. The Estimated Water Surface Elevation (EWSE) as shown in the GPE is 541.29 feet. It should be noted that groundwater levels might change with seasonal rainfall patterns and long-term climate fluctuations.

## 4.0 FOUNDATION ANALYSIS AND RECOMMENDATIONS

The *Cross Sections* (Appendix F) indicate the grade along I-80 will be raised by up to 12 inches along west and east approach centerlines. We understand the east and west integral abutments will be constructed about 13.0 feet behind the existing ones, whereas the piers will be constructed in almost the same location as the existing piers. About 2.0 to 3.0 feet of new fill will be placed along the existing median at the west and east bridge approaches, respectively, to facilitate the inward widening of the bridges by about 25.0 feet.

As per the *Cross Sections* (Appendix F), the side slope of the west approach will be graded at a slope of 1:3 and 1:3.5 (V: H) with up to 4.0 feet of new fill placed along the existing slopes. The east approach will be graded at a slope of 1:4 (V: H) and up to 5.0 feet of new fill will be placed along the existing eastbound embankment slope. The *GPE* (Appendix E) shows rip rap end slopes graded at 1:2 (V: H) with an 18-foot wide bike path proposed on the east and west ends.

The integral abutments could be supported on driven or drilled H-pile foundations whereas the piers could be supported on either shallow foundations on bedrock, drilled H-pile foundations, or rock-socketed drilled shafts. Geotechnical evaluations and recommendations for the approach embankments, approach slabs, and substructure foundations are included in the following sections.

## 4.1 Seismic Design Considerations

The seismic site class was determined in accordance with the IDOT *Geotechnical Manual* (IDOT 2020a). The soils within the top 100 feet have a weighted average $S_u$ value of 4.01 ksf and the results classify the site in the Seismic Site Class C. The project location belongs to the Seismic Performance Zone 1 (IDOT 2020a). The seismic spectral acceleration parameters recommended for design in accordance with the AASHTO *LRFD Bridge Design Specifications* (AASHTO 2020) are summarized in Table 1. According to the IDOT *Bridge Manual* (IDOT 2012), liquefaction analysis is not required for sites located in Seismic Performance Zone 1.

Table 1: Recommended Seismic Design Parameters

| Spectral Acceleration Period (sec) | Spectral Acceleration Coefficient[1] (% g) | Site Factors | Design Spectrum for Site Class C[2] (% g) |
|---|---|---|---|
| 0.0 | PGA= 4.9 | $F_{pga}$= 1.2 | **$A_s$= 5.9** |
| 0.2 | $S_s$= 10.6 | $F_a$= 1.2 | **$S_{DS}$= 12.7** |
| 1.0 | $S_1$= 4.0 | $F_v$=1.7 | **$S_{D1}$= 6.8** |

1) Spectral acceleration coefficients based on Site Class C

2) Site Class C Spectrum to be included on plans; $A_s = PGA*F_{pga}$; $S_{DS}= S_s*F_a$; $S_{D1}= S_1*F_v$

## 4.2 Scour Considerations

A hydraulic report was prepared by HBP Partners Illinois and revised by 2IM Group, LLC in October 2020. The hydraulic report provides 100-year and 500-year scour depths at the proposed main span piers where the maximum scour occurs near the substructures. The estimated 100 and 500 year scour depths are 9.29 and 10.43 feet, respectively; these results place the Q100 and Q500 scour elevations at about 531.01 and 529.87 feet. The scour data provided in the hydraulic report are summarized in Table 2, while the Q100, Q200, Design and Check scour elevations, as required in accordance with IDOT *All Bridge Designers Memo 14.2*, are provided in Table 3. Since the Q200 scour data was not available in the hydraulic report, we have considered the Q500 data. Dolostone bedrock is expected to be encountered within the scour depth at the piers. Non-weathered limestone or dolomite is generally not considered susceptible to scour and, in most cases, should be assumed to arrest scour from extending below the nonweathered elevation (IDOT 2012). The top of bedrock is at elevations of 535.3 and 534.5 feet at Pier 1 and Pier 2, respectively. The abutments end slopes will be armored with riprap along with the proposed 18-foot wide bike path; therefore, the scour elevations are placed at the proposed abutment base elevations (IDOT 2012). As indicated on the GPE, the design high water elevation (DHWE) is 545.47 feet and the Estimated Water Surface Elevation within the channel is 541.05 feet. The streambed elevation is 540.30 feet.

Table 2: Project Scour Data

| | West Abutment | Pier 1 | Pier 2 | East Abutment |
|---|---|---|---|---|
| Streambed Elevation (feet) | NA | 540.30 | 540.30 | NA |
| Foundation Base Elevation (feet) | 548.54 | 537.80(1) | 537.80(1) | 549.00 |
| 100-year Combined Scour Estimate (feet) | NA | 9.29 | 9.29 | NA |
| 500-year Combined Scour Estimate (feet) | NA | 10.43 | 10.43 | NA |

(1) Pile cap base elevations are assumed to be at the bottom of the encasement walls.

Table 3: Project Design Scour Elevations

| | West Abutment | Pier 1 | Pier 2 | East Abutment | Item 113 |
|---|---|---|---|---|---|
| Q100 Elevation (feet) | NA | 531.01 | 531.01 | NA | 8 |
| Q500 Elevation (feet) | NA | 529.87 | 529.87 | NA | |
| Design Elevation (feet) | 548.54 | 535.3 | 534.50 | 549.00 | |
| Check Elevation (feet) | 548.54 | 535.3 | 534.50 | 549.00 | |

### 4.3 Approach Embankment and Slabs

Wang has performed evaluations of the settlement and global stability of the approach embankments. The drawings indicate the grade along the I-80 approach embankments near the bridge will be raised by up to 12 inches along each centerline. About 2.0 to 3.0 feet of new fill will be placed along the existing median at the west and east approaches, respectively, to facilitate the inward widening of the bridges by about 25 feet. We understand the side slopes along the west approach will be graded at a slope of 1:3 and 1:3.5 (V: H) and up to 4.0 feet of new fill will be placed along the existing slopes. The east approach will be graded at a slope of 1:4 (V: H) and up to 5.0 feet of new fill will be placed along the existing eastbound embankment slope.

#### *4.3.1 Settlement*

To facilitate the bridge widenings, up to 3.0 feet of new fill will be placed along the existing medians and up to 5.0 feet of new fill will be placed along the existing west and east approach embankment slopes, respectively. Settlement estimates have been made based on correlations to measured index properties obtained from the laboratory tests (Appendix B). Based on the soil conditions, we estimate the foundation soils at the approaches will undergo up to 0.7 inches of long-term consolidation settlement under the applied load of the new approach embankment fill material with less than 0.4 inches remaining after placement of the new fill. These settlements are appropriate for the construction of the approach slabs and we do not anticipate downdrag allowances for the proposed abutment piles.

#### *4.3.2 Global Stability*

The global stability of the approach embankment side slopes was analyzed at the critical sections based on the soil profile described in Section 3.1 and the information provided in the *GPE* and *Cross Sections* (Appendixes E and F). The minimum required FOS for both short (undrained) and long-term (drained) conditions is 1.5 (IDOT 2012). *Slide v6.0* evaluation exhibits employing the Bishop Simplified method of analysis are shown in Appendix D. The FOS values meet the minimum requirement.

#### *4.3.3 Approach Slabs*

We estimate the approach slabs will be supported on spread footing foundations (IDOT 2012). Based on the design drawings and soil conditions revealed in Borings RRC-BSB-01, RRC-BSB-02, RRC-BSB-05, and RRC-BSB-06, the approach footings will be supported on the new fill to be placed along the approaches and the existing silty loam and clay loam fill. We estimate the newly placed and compacted fill, as required by the IDOT *Standard Specifications* (IDOT 2022), could be designed based on a maximum factored soil bearing resistance of 2,500 psf calculated for a geotechnical resistance factor ($\Phi_b$) of 0.45 (AASHTO 2020). The settlement of approach footings is estimated to be less than 1.0 inch.

### 4.4 Structure Foundations

The soil conditions along the structures show medium dense to very dense silty loam to loam soils as embankment fill followed by stiff to hard silty clay to silty clay loam and gravelly sandy loam overlying dolostone bedrock. The integral abutments could be supported on driven steel H-piles. We do not recommend metal shell piles due to the presence of shallow bedrock. Additionally, based on Borings RRC-BSB-01, RRC-BSB-02, RRC-BSB-05, and RRC-BSB-06, drilled near the proposed abutment locations, the bedrock surface is interpreted to be at elevations of 535 to 536 feet or about 13.0 feet below the proposed pile cap elevations. In these instances, the integral abutment piles may be required to be drilled and set into the bedrock to provide sufficient lateral resistance. Alternatively, the use of semi-integral abutments could be considered in the design (IDOT 2012).

Considering the presence of shallow bedrock, as shown by Borings RRC-BSB-03 and RRC-BSB-04, the piers could be supported on shallow foundations on bedrock, piles drilled and set into bedrock, or rock-socketed drilled shafts. The preliminary factored loading information provided by Lin on July 12, 2022 and proposed abutment cap base elevations as shown in the *GPE* are summarized in Table 4.

Table 4: Preliminary Factored Loads and Proposed Pile Cap Elevations

| Direction | Substructure | Pile Cap Elevations (feet) | Total Factored Load (kips) |
|---|---|---|---|
| Eastbound | West Abutment | 548.54 | 1653 |
| | Pier 1 | 537.80(1) | 2775 |
| | Pier 2 | 537.80(1) | 2775 |
| | East Abutment | 549.00 | 1653 |
| Westbound | West Abutment | 548.54 | 1512 |
| | Pier 1 | 537.80(1) | 2535 |
| | Pier 2 | 537.80(1) | 2535 |
| | East Abutment | 549.00 | 1512 |

(1) Pile cap base elevations are assumed to be at the bottom of the encasement wall elevation.

*4.4.1 Driven Piles*

IDOT specifies the maximum nominal required bearing ($R_{NMAX}$) for each pile and states the factored resistance available ($R_F$) for steel H-piles should be based on a geotechnical resistance factor ($\Phi_G$) of 0.55 (IDOT 2012). Nominal tip and side resistance were estimated using the methods and empirical equations presented in the latest *IDOT Geotechnical Pile Design Guide* (IDOT 2020a). Based on the loads provided by Lin and the proposed width of the substructures, the load per pile at the abutments and piers will range between about 54 and 177 kips and 90 to 297 kips for one row of piles spaced at 3- to 8-feet, respectively.

Based on IDOT standards, piles with greater than 0.4-inch of relative settlement along the sides require allowances for downdrag loads. We estimate that downdrag allowances will not be required for the abutment piles.

The foundation soils within 10.0 feet below the abutment pile cap elevations consist of dense to very dense silty loam soils and very stiff clayey soils with average $Q_u$ values of greater than 3.0 tsf. In accordance with the *All Bridge Designers Memo 19.8* (IDOT 2019), when the average soil strengths at an integral abutment exceed 3.0 tsf, the piles at the abutments should be precored for a depth of 10.0 feet below the abutment cap elevation and backfilled with bentonite having a $Q_u$ value of 1.0 tsf to increase pile flexibility (IDOT 2019). The pile capacity evaluations have been performed assuming pile driving begins about 10.0 feet below the proposed abutment pile cap elevations.

High blow counts and sampler refusal were noted within the borings below an approximate elevation of 540 feet indicating the possible presence of cobbles. As such, pile shoes should be used for piles driven to or below an elevation of 540 feet to avoid damage to the piles. Additionally, to achieve the maximum nominal required bearing at the abutments, the analysis shows the H-piles would need to be driven about 1.0 to 3.0 feet into the weathered bedrock and/or shaly dolostone. In these instances, the piles should be considered end bearing and designed for the maximum capacity of the pile. IDOT generally recommends that H-piles be driven to their maximum nominal required bearing.

As per Section 6.13.2.4.2.2 of the IDOT *Geotechnical Manual,* when bedrock is within 10.0 to 15.0 feet below the bottom of a substructure, such as at the proposed abutment locations, soil-structure interaction analysis would need to be performed to determine if piles need to be set into rock. If the analysis indicates excessive pile head deflection, the piles will likely need to be set into rock to satisfy the deflection requirements (IDOT 2020). We recommend contacting IDOT Bureau of Bridges and Structures (BBS) for integral abutment special design approval or considering the use of semi-integral abutments in the design (IDOT 2012).

Table 5 provides estimated pile tip elevations and pile lengths for HP 12x53, HP12x74, HP 14x73, and HP14x89 steel H-piles driven to maximum nominal bearing. The pile lengths shown in Table 5 include a 2-foot pile embedment into the abutment pile cap elevations as shown on the *GPE* (Appendix E) and the precored length of the pile.

Table 5: Estimated Pile Lengths and Tip Elevations for Steel H-Piles Driven to $R_{NMAX}$

| Structure Unit (Reference Boring) | Pile Cap Base Elevations (feet) | Pile Size | Maximum Nominal Bearing, $R_N$ (kips) | Factored Geotechnical Loss (kips) | Factored Geotechnical Load Loss (kips) | Factored Resistance Available, $R_F$ (kips) | Total Estimated Pile Length (feet) | Estimated Pile Tip Elevation (feet) |
|---|---|---|---|---|---|---|---|---|
| Westbound and Eastbound West Abutments RRC-BSB-01 and RRC-BSB-02 | 548.54 | HP 12x53 | 418 | 0 | 0 | 230 | 18 | 532.5 |
| | | HP 12x74 | 589 | 0 | 0 | 324 | 19 | 531.5 |
| | | HP 14x73 | 578 | 0 | 0 | 318 | 19 | 531.5 |
| | | HP 14x89 | 705 | 0 | 0 | 388 | 19 | 531.5 |
| Westbound and Eastbound East Abutment RRC-BSB-05 and RRC-BSB-06 | 549.00 | HP 12x53 | 418 | 0 | 0 | 230 | 18 | 533.0 |
| | | HP 12x74 | 589 | 0 | 0 | 324 | 19 | 532.0 |
| | | HP 14x73 | 578 | 0 | 0 | 318 | 19 | 532.0 |
| | | HP 14x89 | 705 | 0 | 0 | 388 | 19 | 532.0 |

### *4.4.2 Piles Set in Rock*

Piles drilled and set into bedrock could be considered to increase axial capacity and provide sufficient resistance to lateral loads at the integral abutments and piers The top of the bedrock elevation at the abutments as noted in Borings RRC-BSB-01 to RRC-BSB-02, RRC-BSB-05, and RRC-BSB-06, ranges from elevations of 535.7 to 534.7 feet or about 13.0 to 14.0 feet below the proposed west and east bottom of pile cap elevations of 548.54 and 549.00 feet, respectively, shown on the GPE (Appendix E). At the piers, the top of bedrock elevation was recorded in Borings RRC-BSB-03 and RRC-BSB-04 at elevations of 535.3 to 534.5 feet or about 2.5 to 3.5 feet below the estimated bottom of pile cap elevations measured at the bottom of the encasement walls shown on the GPE (Appendix E). The socket diameter should be specified in 6-inch increments and be just large enough to allow a pile to be placed into the socket with sufficient room to permit placement of concrete such that it completely encases the pile. The socket length should be checked to determine if it adequately carries the lateral load and, if necessary, the socket length can be increased to carry the lateral load (IDOT 2020).

As per the IDOT *Geotechnical Manual* (IDOT 2020), the design axial capacity of a pile set in rock can be larger than the maximums allowed in the IDOT *Bridge Manual* (IDOT 2012) for driven piles. This is because piles set in rock are not subjected to high driving stresses, which limit the maximum nominal capacity of driven piles. The maximum nominal capacity of driven H-piles is limited to 54% of its yield strength, while the nominal capacity of piles set in rock is 100% of its yield strength. Additionally, piles set in rock use a more favorable resistance factor of 0.7 for non-driven undamaged piles according to Article 6.5.4.2 of the "*AASHTO LRFD Bridge Design Specifications*" (AASHTO 2020) compared to 0.55 used for driven piles (IDOT 2020). The $R_F$, $R_N$ for HP12x53, HP 12x74, HP14x73, and HP 14x89 along with approximate top of bedrock elevations are summarized in Table 6.

Table 6: Estimated Resistances for Piles Set in Rock

| Structure Unit (Reference Boring) | Proposed Pile Cap Base Elevations (feet) | Approximate Top of Bedrock Elevation (feet) | Pile Size | Cross-sectional Area (square inches) | Nominal Resistance, $R_N$ (kips) | Factored Resistance Available, $R_F$ (kips) |
|---|---|---|---|---|---|---|
| Westbound and Eastbound West Abutment RRC-BSB-01 and RRC-BSB-02 | 548.54(1) | 534.9 to 536.9 | HP 12x53 | 15.49 | 775 | 542 |
| | | | HP12x74 | 21.80 | 1090 | 763 |
| | | | HP14x73 | 21.40 | 1070 | 749 |
| | | | HP 14x89 | 26.11 | 1305 | 914 |
| Westbound and Eastbound Pier 1 RRC-BSB-03 | 537.80(2) | 535.3 | HP 12x53 | 15.49 | 775 | 542 |
| | | | HP12x74 | 21.80 | 1090 | 763 |
| | | | HP14x73 | 21.40 | 1070 | 749 |
| | | | HP 14x89 | 26.11 | 1305 | 914 |
| Westbound and Eastbound Pier 2 RRC-BSB-04 | 537.80(2) | 534.5 | HP 12x53 | 15.49 | 775 | 542 |
| | | | HP12x74 | 21.80 | 1090 | 763 |
| | | | HP14x73 | 21.40 | 1070 | 749 |
| | | | HP 14x89 | 26.11 | 1305 | 914 |

| Structure Unit (Reference Boring) | Proposed Pile Cap Base Elevations (feet) | Approximate Top of Bedrock Elevation (feet) | Pile Size | Cross-sectional Area (square inches) | Nominal Resistance, $R_N$ (kips) | Factored Resistance Available, $R_F$ (kips) |
|---|---|---|---|---|---|---|
| Westbound and Eastbound East Abutment RRC-BSB-05 and RRC-BSB-06 | 549.00(1) | 534.7 to 535.7 | HP 12x53 | 15.49 | 775 | 542 |
| | | | HP12x74 | 21.80 | 1090 | 763 |
| | | | HP14x73 | 21.40 | 1070 | 749 |
| | | | HP 14x89 | 26.11 | 1305 | 914 |

(1) Pile cap base elevations as shown on the GPE at the proposed abutment locations.

(2) Pile cap base elevations estimated at the bottom of the proposed encasement walls at the pier locations.

Due to the presence of groundwater and granular soils above the bedrock, the recommended construction method for piles socketed into bedrock, similar to that of shaft construction, is to install casing to the top of the rock to maintain clean open holes during excavation. Loss of water circulation was also not noted while coring, however, cavities are known to be present in this area, as discussed in Section 1.1. The quality of bedrock at the abutments should be verified during construction. A value engineering analysis is recommended to select the most suitable type of foundation system at the abutments. The pile installation should be as per IDOT Section 512, *Piling* (IDOT 2022).

### *4.4.3 Drilled Shafts*

The piers could be supported on drilled shafts socketed 5.0- to 10-feet into the bedrock. As per the 2012 IDOT *Bridge Manual*, drilled shafts extending into rock, in most cases, should be designed utilizing only end bearing or side resistance in rock, whichever is larger. For shafts socketed into the bedrock less than 10-foot long, we estimate the end bearing will give more capacity than the side resistance. Therefore, we recommend considering only the end bearing resistance. The shafts should be designed for end bearing with a tip resistance factor ($\phi_{stat}$) of 0.50 (AASHTO 2020). Above the bedrock, the shafts should have diameters 6 inches larger than the sockets.

The bedrock resistance was evaluated in accordance with the Geologic Strength Index (GSI) method provided by AASHTO (2020). The $R_F$, $R_N$, and estimated base elevations for rock-socketed shafts are summarized in Table 7. The shaft lengths were estimated assuming the shafts start from the proposed bike path grade elevation shown on the GPE (Appendix E). For the anticipated loads (Table 4), we estimate shaft settlements of less than 0.5 inch.

Due to the presence of groundwater and granular soils above the bedrock, the recommended construction method for shafts socketed into bedrock is to install casing to the top of the rock to maintain clean, open shafts during excavation. The quality of bedrock at the piers should be verified during construction. A value engineering analysis is recommended to select the most suitable type of foundation system at the pier.

Table 7: Estimated Drilled Shaft Resistances and Base Elevations (Rock-Socketed Shafts)

| Structure Unit (Reference Boring) | Shaft Cap Base Elevations(1) (feet) | Approximate Top of Bedrock Elevation (feet) | Socket Diameter (feet) | Nominal Unit Resistance (ksf) | Nominal Resistance, $R_N$ (kips) | Factored Resistance Available, $R_F$ (kips) | Total Socket Length (feet) | Estimated Total Shaft Length(2) (feet) |
|---|---|---|---|---|---|---|---|---|
| Westbound and Eastbound Pier 1 RRC-BSB-03 | 543.05 | 535.30 | 3.0 | 230 | 1620 | 810 | 5.0 | 13.0 |
| | | | 4.0 | | 2890 | 1445 | | |
| | | | 5.0 | | 4510 | 2255 | | |
| | | | 3.0 | 250 | 1760 | 880 | 10.0 | 18.0 |
| | | | 4.0 | | 3140 | 1570 | | |
| | | | 5.0 | | 4910 | 2455 | | |
| Westbound and Eastbound Pier 2 RRC-BSB-04 | 543.05 | 534.50 | 3.0 | 250 | 1760 | 880 | 5.0 | 14.0 |
| | | | 4.0 | | 3140 | 1570 | | |
| | | | 5.0 | | 4910 | 2455 | | |
| | | | 3.0 | 270 | 1900 | 950 | 10.0 | 19.0 |
| | | | 4.0 | | 3390 | 1695 | | |
| | | | 5.0 | | 5300 | 2650 | | |

(1) Drilled shaft cap base elevations are assumed to be the bike path grade elevation

(2) Total shaft lengths were measured from the assumed cap base elevation at the pier locations

Based on the geometry shown in the *GPE* (Appendix E), the existing pier footings may conflict with installation of some of the shafts. The proposed pier shaft locations should be selected to miss the existing footings.

### *4.4.4 Spread Footings*

A spread footing supported on dolostone bedrock could be considered at the pier locations. The top of the bedrock elevation ranges from 535.3 to 534.5 feet at Pier 1 and Pier 2, respectively. The bottom of the pier spread footing should be placed a minimum of 6 inches below the top of the bedrock (IDOT 2012). The quality of bedrock at the pier locations should be verified during construction.

Construction of spread footings supported on bedrock at the pier locations will require excavations of up to 6.0 feet under the river surface. Therefore, cofferdams will be needed for the excavation and construction of the piers.

According to Section 10.6.3.2.2 of the AASTHO LRFD *Bridge Design Specifications* (AASHTO 2020), the recommended bearing resistance for spread footings should be determined using empirical correlations with the Geomechanics RMR system. Based on our analysis, we estimate a factored bearing resistance of 15 ksf, considering a resistance factor of 0.45 (AASHTO 2020). The factored bearing resistance shall not be taken to be greater than either the unconfined compressive strength of rock or the factored compressive resistance of the footing concrete (AASHTO 2020). The laboratory rock unconfined compressive strength results ranged from 7,494 to 11,678 psi (1079 to 1682 ksf) and the factored nominal resistance of the concrete is 151 ksf based on the nominal concrete resistance as provided on the GPE of 3.5 ksi. Thus, the recommended bearing resistance of 15 ksf should be used for the design.

As per Section 10.6.2.4.4, for footings bearing on fair to very good rock, elastic settlements may be assumed to be less than 0.5 inches (AASHTO 2020). However, since the RQD value at the pier location is about 0 to 41% and the rock within the top 5.0 feet does not meet the fair to very good criteria, a settlement analysis was conducted. For our settlement evaluations, we considered a footing width of 8.0 feet and length of 76.0 feet, as estimated from the GPE, and no eccentric loads. Based on the proposed factored load and assumed dimensions, we estimate a settlement of less than 0.5 inches. The recommended friction coefficient between concrete and bedrock materials is 0.7 (AASHTO 2020).

### *4.4.3 Lateral Loading*

Lateral loads on the foundations should be analyzed for maximum moments and lateral deflections. Recommended lateral soil modulus and strain parameters required for analysis via the p-y curve method are included in Tables 8 to 12.

Table 8: Recommended Soil Parameters for Lateral Load Analysis at West Abutments
Reference Borings RRC-BSB-01 and RRC-BSB-02

| Elevation Range (feet) Soil Type (Layer) | Unit Weight, $\gamma$ (pcf) | Undrained Shear Strength, $c_u$ (psf) | Estimated Friction Angle, $\Phi$ (°) | Estimated Lateral Soil Modulus Parameter, k (pci) | Estimated Soil Strain Parameter, $\varepsilon_{50}$ |
|---|---|---|---|---|---|
| 548.54(1) to 538.5 New FILL (Bentonite) | 120 | 1000 | 0 | 500 | 0.007 |
| 539.0 to 537.0 Soft SILTY CLAY LOAM | 48(2) | 250 | 0 | 100 | 0.020 |
| 537.0 to 535.0(3) WEATHERED BEDROCK | 63(2) | 0 | 36 | 125 | -- |

(1) Pile cap base elevation

(2) Submerged unit weight

(3) Approximate top of bedrock

Table 9: Recommended Soil Parameters for Lateral Load Analysis at Pier 1
Reference Boring RRC-BSB-03

| Elevation Range (feet) Soil Type (Layer) | Unit Weight, $\gamma$ (pcf) | Undrained Shear Strength, $c_u$ (psf) | Estimated Friction Angle, $\Phi$ (°) | Estimated Lateral Soil Modulus Parameter, k (pci) | Estimated Soil Strain Parameter, $\varepsilon_{50}$ |
|---|---|---|---|---|---|
| 537.8(1) to 535.0(3) Very Dense SANDY GRAVEL | 63(2) | 0 | 34 | 125 | -- |

(1) Estimated bottom of encasement wall elevation at the proposed Pier 1 location.

(2) Submerged unit weight

(3) Approximate top of bedrock

Table 10: Recommended Soil Parameters for Lateral Load Analysis at Pier 2
Reference Boring RRC-BSB-04

| Elevation Range (feet) Soil Type (Layer) | Unit Weight, $\gamma$ (pcf) | Undrained Shear Strength, $c_u$ (psf) | Estimated Friction Angle, $\Phi$ (°) | Estimated Lateral Soil Modulus Parameter, k (pci) | Estimated Soil Strain Parameter, $\varepsilon_{50}$ |
|---|---|---|---|---|---|
| 537.8(1) to 537.0 Very Dense SANDY GRAVEL | 58(2) | 0 | 34 | 125 | -- |

| Elevation Range (feet) Soil Type (Layer) | Unit Weight, $\gamma$ (pcf) | Undrained Shear Strength, $c_u$ (psf) | Estimated Friction Angle, $\Phi$ (°) | Estimated Lateral Soil Modulus Parameter, k (pci) | Estimated Soil Strain Parameter, $\varepsilon_{50}$ |
|---|---|---|---|---|---|
| 537.0 to 534.0(3) Very Dense WEATHERED BEDROCK | 63(2) | 0 | 36 | 125 | -- |

(1) Estimated bottom of encasement wall at the proposed Pier 2 location.

(2) Submerged unit weight

(3) Approximate top of bedrock

Table 11: Recommended Soil Parameters for Lateral Load Analysis at East Abutments
Reference Borings RRC-BSB-05 and RRC-BSB-06

| Elevation Range (feet) Soil Type (Layer) | Unit Weight, $\gamma$ (pcf) | Undrained Shear Strength, $c_u$ (psf) | Estimated Friction Angle, $\Phi$ (°) | Estimated Lateral Soil Modulus Parameter, k (pci) | Estimated Soil Strain Parameter, $\varepsilon_{50}$ |
|---|---|---|---|---|---|
| 549.0(1) to 539.0 New FILL (Bentonite) | 120 | 1000 | 0 | 500 | 0.007 |
| 539.0 to 535.0(3) WEATHERED BEDROCK | 63(2) | 0 | 36 | 125 | -- |

(1) Pile cap base elevation

(2) Submerged unit weight

(3) Approximate top of bedrock

Table 12: Bedrock Parameters for Lateral Load Analysis
Reference Borings RRC-BSB-01 to RRC-BSB-06

| Bedrock | Total Unit Weight, $\gamma$ (pcf) | Modulus of Rock Mass (ksi) | Uniaxial Compressive Strength (psi) | RQD (%) | Strain Factor |
|---|---|---|---|---|---|
| Dolostone | 140 | 170 | 7,494 to 11,678 | 0 to 17 | 0.0005 |

## 4.5 Stage Construction

Stage construction is identified in the *GPE* (Appendix E). Wang understands that the bridge replacements will be performed utilizing two main stages of construction to maintain traffic on each bridge. During Stage I, two lanes of traffic would be moved to the outside lanes and shoulders of the existing bridges so that the widening can advance within the existing median area. During Stage II, the two lanes of traffic would utilize the roadway constructed during Stage I so that the existing bridges can be removed and the outside portion of the bridges can be replaced.

The construction activities will likely involve excavations of up to 7.0 feet along the sides of the existing east and west abutments, respectively. Temporary support systems will be required if the ground cannot be sloped at 1:2 (V: H). Due to the presence of very dense soils and shallow bedrock, we estimate temporary steel sheet piling, designed using the charts included in the *IDOT Design Guide-Simplified Temporary Sheet Piling Design Charts* will not be feasible and we recommend including the pay item, *Temporary Soil Retention System* for shoring. For the pier support, a Type II cofferdam will be required for construction of spread footings.

## 5.0 CONSTRUCTION CONSIDERATIONS

### 5.1 Site Preparation

Vegetation, surface topsoil, pavements, and debris should be cleared and stripped where the structures will be placed. If unstable or unsuitable materials are exposed during excavation, they should be removed and replaced with compacted structural fill as described in Section 5.3.

### 5.2 Excavation, Dewatering, and Utilities

Excavations should be performed in accordance with local, state, and federal regulations. The potential effect of ground movements upon nearby utilities should be considered during construction. Any slope that cannot be graded at 1:2 (V: H) for cohesive soils and at 1:2.5 (V:H) for saturated granular soils should be properly shored.

During the subsurface investigation, the groundwater was encountered at elevations ranging from 542 to 536 feet, as discussed in Section 3.2. At the abutments, the groundwater will be about 10.0 to 12.0 feet below the proposed pile cap base elevations. As such, we do not anticipate the need for dewatering at the abutments. Perched or temporary water may be encountered during times of heavy precipitation while excavating within the upper fill soils and will require dewatering efforts. Water that does accumulate in open excavations by seepage or runoff should be immediately removed by sump pump. For the proposed shallow footing for the piers, a Type II cofferdam with seal coat will be required.

## 5.3 Filling and Backfilling

Fill material used to attain final design elevations should be pre-approved, cohesive or granular soil conforming to Section 204, *Borrow and Furnished Excavation* (IDOT 2022). The fill material should be free of organic matter and debris and should be placed in lifts and compacted according to Section 205, *Embankment* (IDOT 2022). In accordance with IDOT Section 205, *Embankment*, any embankments proposed for widening should be properly benched or deeply plowed prior to placement of new fill along the slopes (IDOT 2022).

Backfill materials for the abutments must be pre-approved by the Resident Engineer. To backfill the abutments, we recommend porous granular material conforming to the requirements specified in the IDOT Supplemental Special and Recurring Special Provisions, *Granular Backfill for Structures* (IDOT 2020b).

## 5.4 Earthwork Operations

The required earthwork can be accomplished with conventional construction equipment. Moisture and traffic will cause deterioration of exposed subgrade soils. Precautions should be taken by the Contractor to prevent water erosion of the exposed subgrade. A compacted subgrade will minimize water runoff erosion.

Earth moving operations should be scheduled to not coincide with excessive cold or wet weather (early spring, late fall or winter). Any soil allowed to freeze or soften due to the standing water should be removed. Wet weather can cause problems with subgrade compaction.

It is recommended that an experienced geotechnical engineer be retained to inspect the exposed subgrade, monitor earthwork operations, and provide material inspection services during the construction phase of this project.

## 5.5 Pile Installation

The driven piles shall be furnished and installed according to the requirements of IDOT Section 512, *Piling* (IDOT 2022). Wang recommends performing one test pile at each substructure location. Since the piles will be driven to bedrock, pile shoes are required as indicated in Section 4.4.1.

## 5.6 Drilled Shafts

Drilled shafts should be installed as per Section 516, *Drilled Shafts* (IDOT 2022). Due to the presence of groundwater and granular soils above the bedrock, the recommended construction method for shafts socketed into bedrock is to install casing to the top of the rock to maintain clean, open shafts during excavation. Loss of water circulation was not noted while coring, however, cavities are known to be present in this area, as discussed in Section 1.1. The quality of bedrock at the proposed pier locations should be verified during construction.

## 6.0 QUALIFICATIONS

The analysis and recommendations submitted in this report are based upon the data obtained from the borings drilled at the locations shown on the boring logs and in Exhibit 3. This report does not reflect any variations that may occur between the borings or elsewhere on the site, variations whose nature and extent may not become evident until the course of construction. In the event that any changes in the design and/or location of the structure are planned, we should be timely informed so that our recommendations can be adjusted accordingly.

It has been a pleasure to assist Stantec, Lin Engineering, Ltd., and the Illinois Department of Transportation on this project. Please call if there are any questions, or if we can be of further service.

Respectfully Submitted,

WANG ENGINEERING, INC.

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Corina Farez, P.E., P.G.
QC/QA Reviewer

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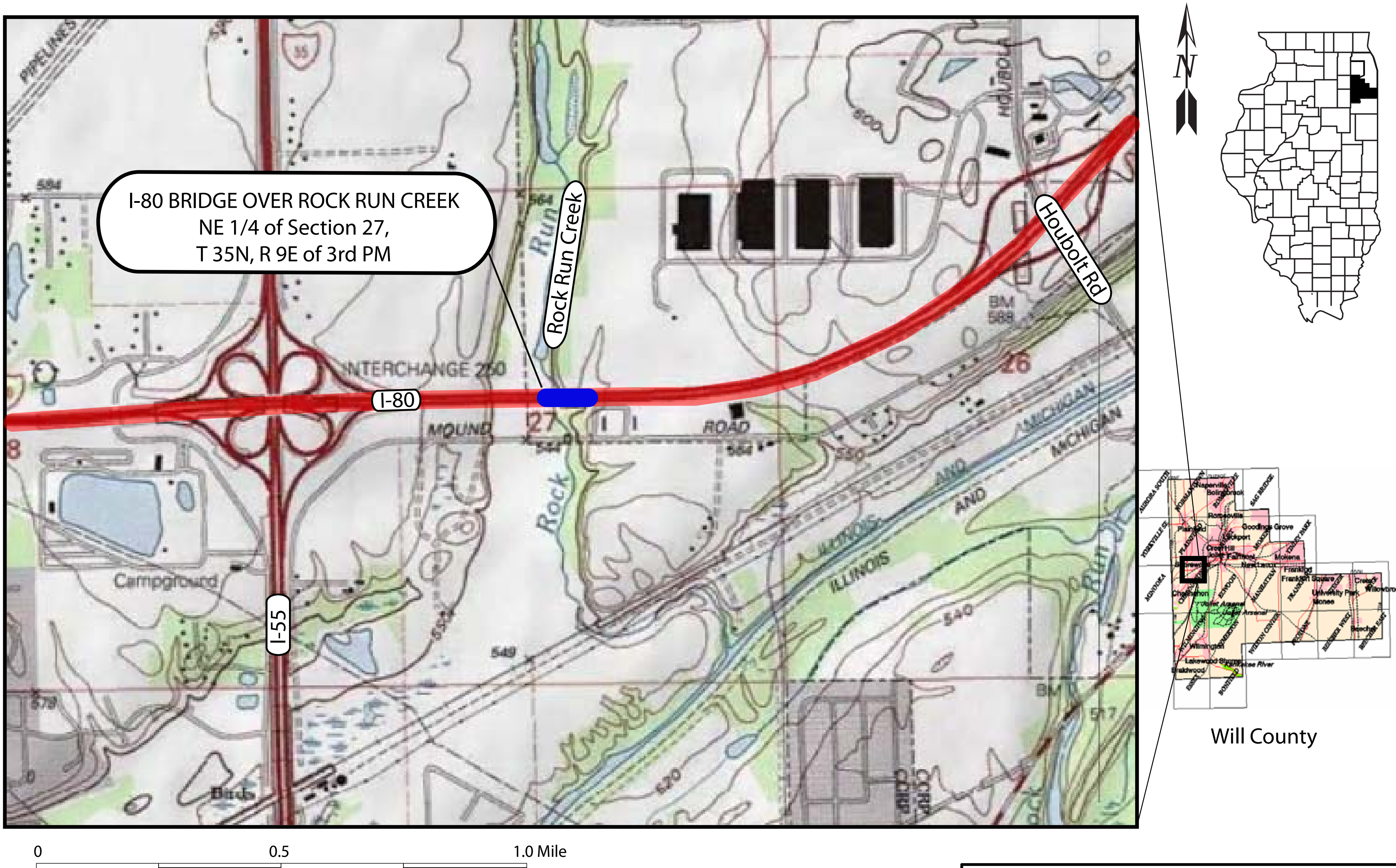
# EXHIBITS




SITE LOCATION MAP: I-80 BRIDGE OVER ROCK RUN CREEK; I-80 RECONSTRUCTION FROM EAST OF RIDGE RD TO HOUBOLT RD, WILL COUNTY, ILLINOIS

| | | |
|---|---|---|
| SCALE: GRAPHICAL | EXHIBIT 1 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |

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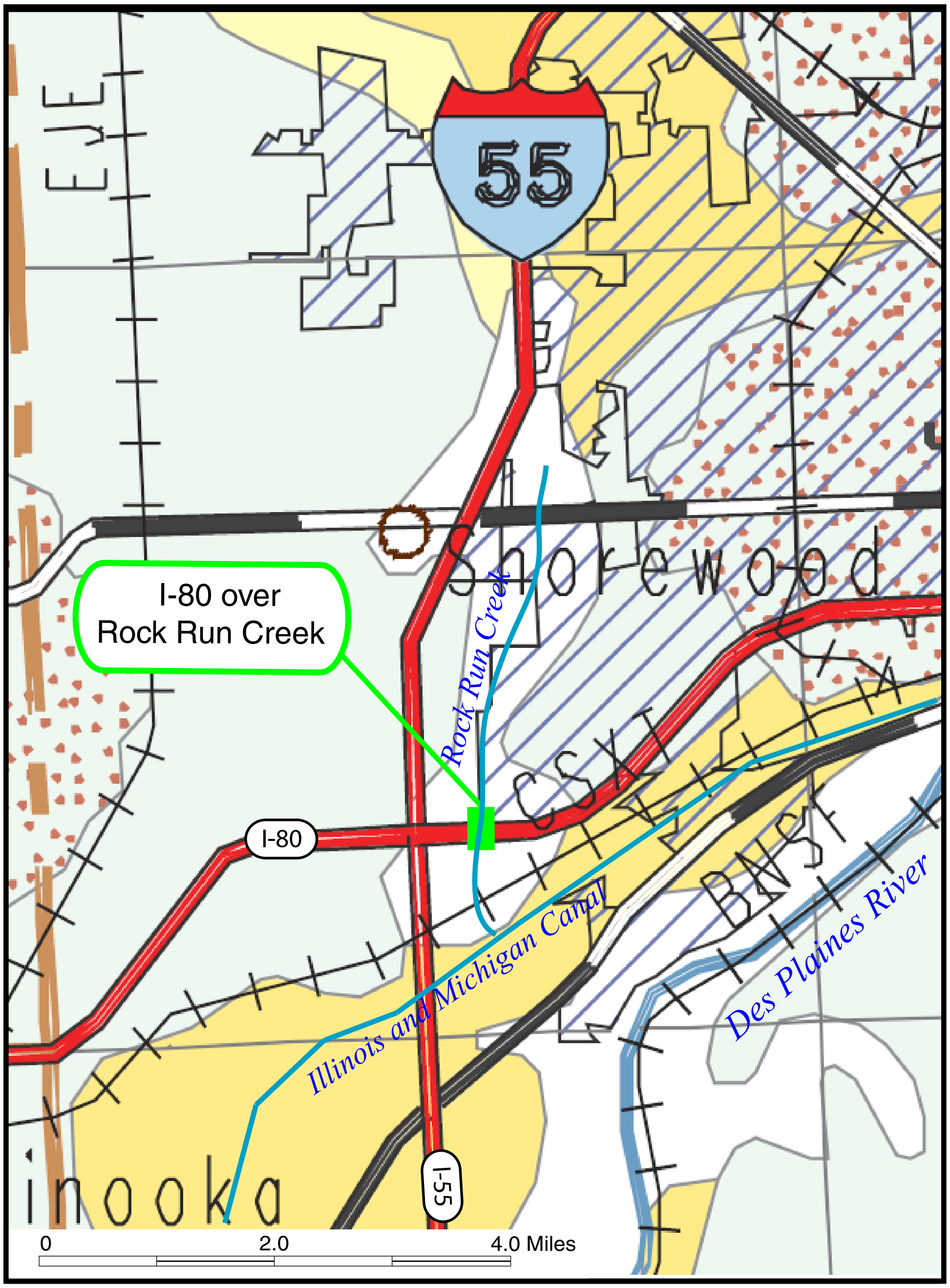
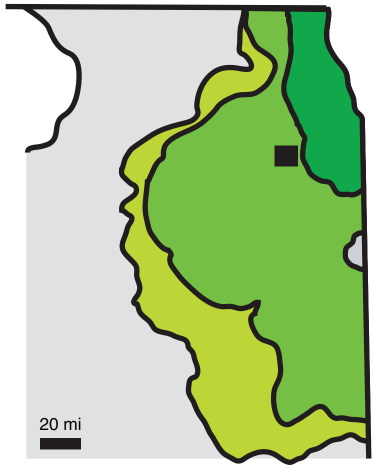
| | |
|---|---|
| FOR STANTEC | 255-39-01 |




Modified after William W. Shilts (2000)

## REGIONAL GEOLOGY




**Wedron Group**

- Wadsworth Formation
- Lemont Formation
- Tiskilwa Formation

Modified after Hansel and Johnson (1996)

## Legend

**Cahokia Formation**
Bedded silts, clays and sand and gravel deposited in floodplains and channels of modern rivers and streams

**Equality Formation**
Laminated silt, clay, or fine sand deposited in slackwater, glacial, and postglacial lakes; massive to bedded, calcareous

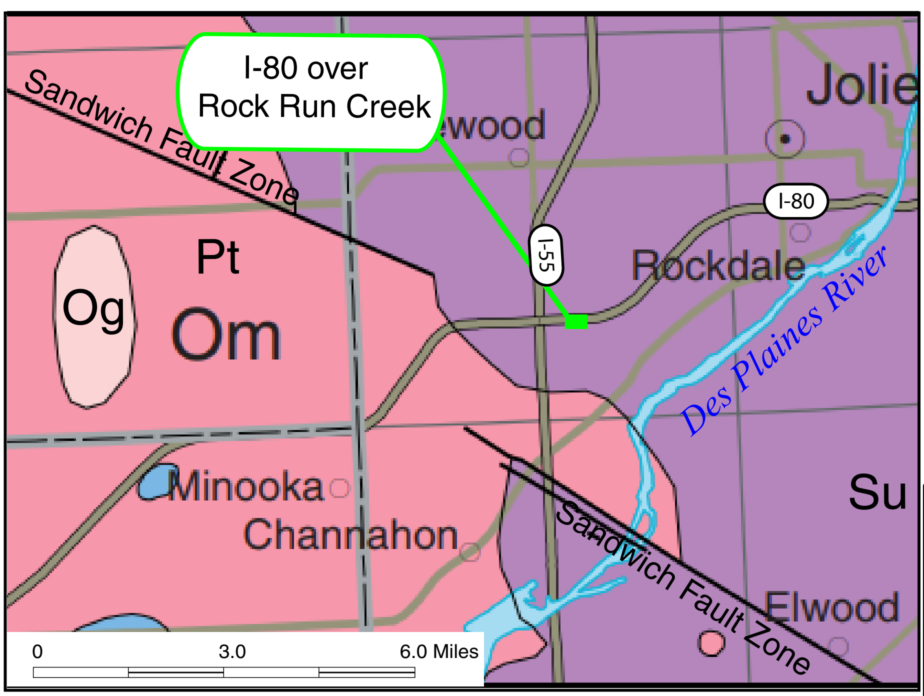
**Henry Formation**
Predominantly stratified sand and gravel; locally occurring silt, clay, organic debris, wood, shell lenses; outwash on floodplains

**Lemont Formation**
Diamicton; silty clay loam to silty clay; fine grained matrix; unsorted, unstratified mixture of gravel, sand, silt, and clay; containing lenses of sand, gravel and/or silt.

Bedrock near or at the surface

Major metropolitan area

Modified after William W. Shilts (2000)

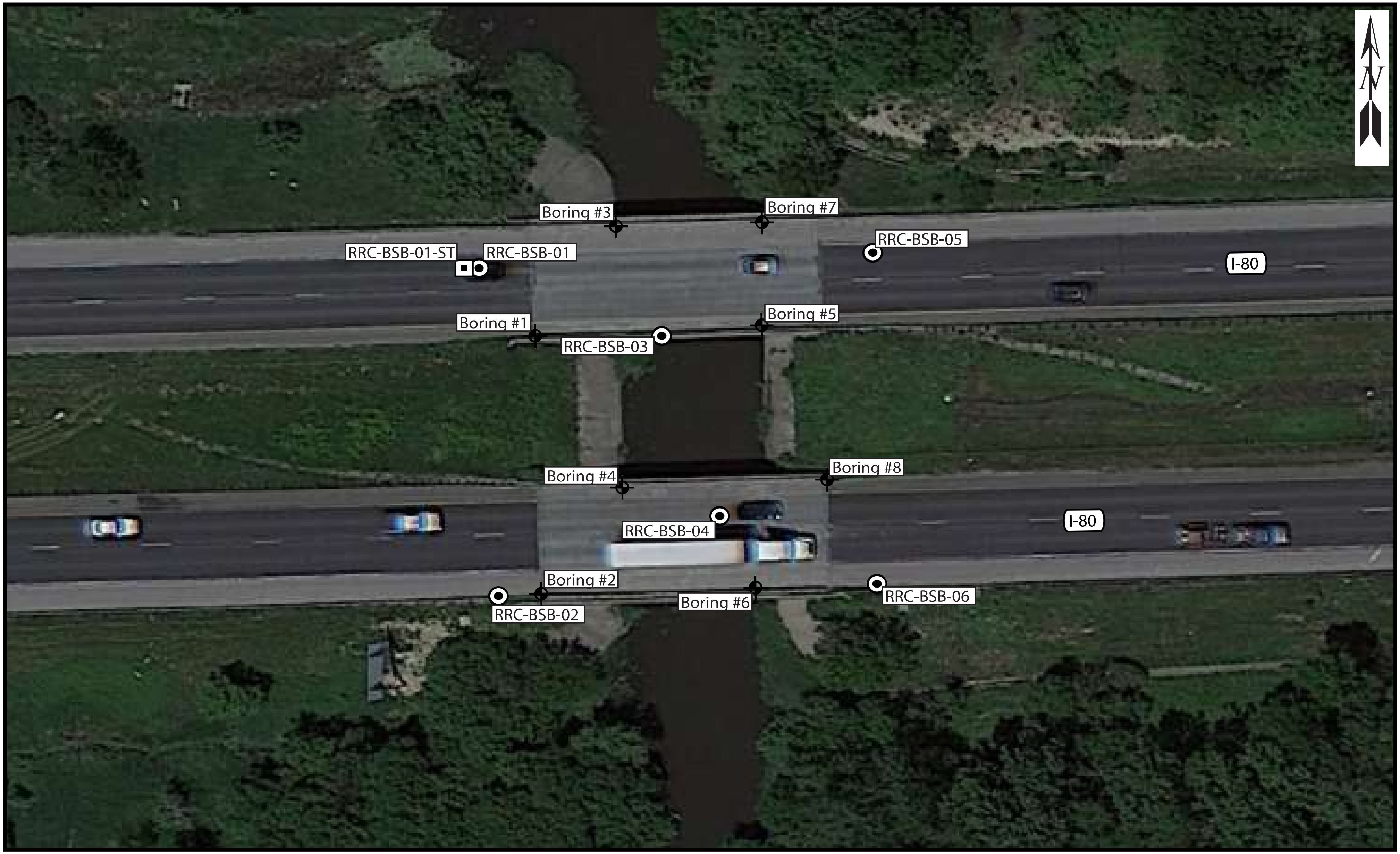



Modified after Dennis R. Kolata (2005)

## Legend

| | | |
|---|---|---|
| Pt | **Pennsylvanian Shale** | Tadewater Formation |
| Su | **Silurian Dolomitic Limestone** | |
| Om | **Ordovician Formation** | Maquoketa Group |
| Og | **Ordovician Formation** | Galena Group |

| | | |
|---|---|---|
| SITE AND REGIONAL GEOLOGY: I-80 RECONSTRUCTION, I-80 over ROCK RUN CREEK<br>WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | EXHIBIT 2 | DRAWN BY: C. Marin<br>CHECKED BY: L. Iordache |
| Wang Engineering | | 1145 N. Main Street<br>Lombard, IL 60148<br>www.wangeng.com |
| FOR STANTEC | | 255-39-01 |




0 100 200 Feet

## Legend

- Boring Location
- Historical Boring Location
  *Approximate location
- Shelby Tube Boring Location

BORING LOCATION PLAN: I-80 BRIDGE OVER ROCK RUN CREEK; I-80 RECONSTRUCTION FROM EAST OF RIDGE RD TO HOUBOLT RD, WILL COUNTY, ILLINOIS

| | | |
|---|---|---|
| SCALE: GRAPHICAL | EXHIBIT 3 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |

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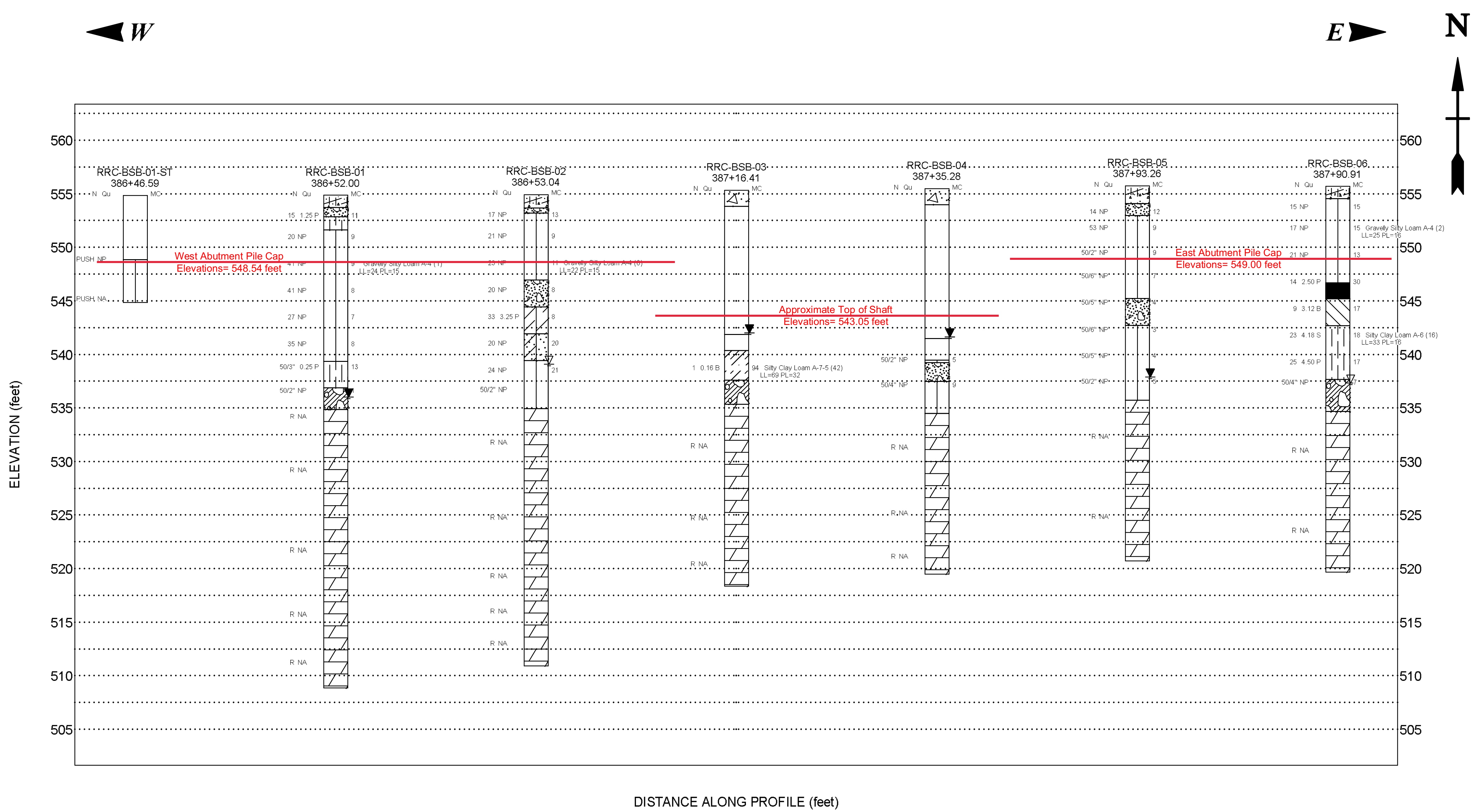
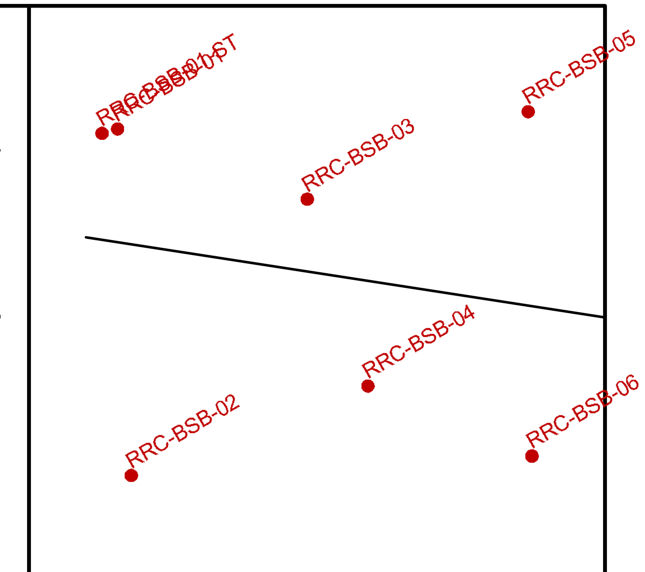
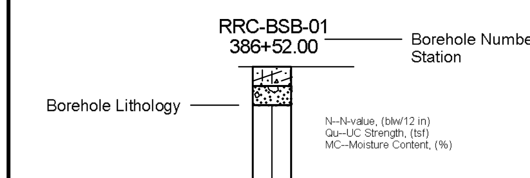
FOR STANTEC | 255-39-01




Site Map Scale 1 inch equals 65 feet

## Explanation:



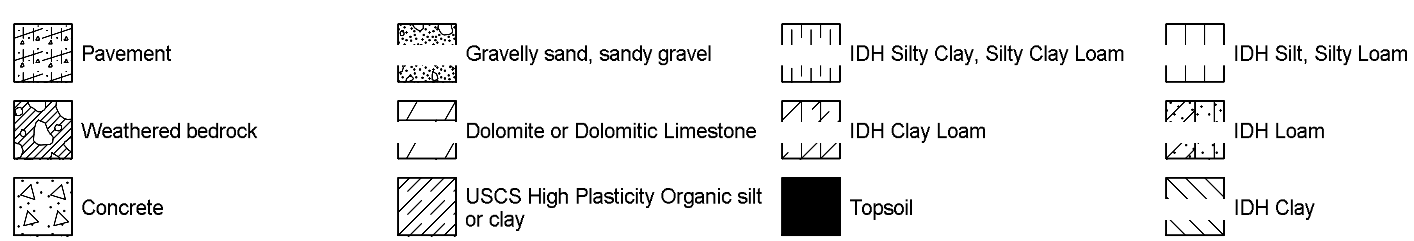

Water Level Reading at time of drilling.

Water Level Reading 24-hr after drilling or at end of drilling


Horizontal Scale (feet)

Vertical Exaggeration: 1.5x

## Lithology Graphics




**Wang Engineering, Inc.**
1145 N Main Street
Lombard, IL 60148

**I-80 Over Rock Run Creek**
**Soil Profile**

WEI
SINCE 1982

I-80 Reconstruction, Ridge Road to Houbolt Road
Will County, Illinois

| JOB NUMBER | PLATE NUMBER |
|---|---|
| 255-39-01 | EXHIBIT 4 |

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# APPENDIX A

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Fax: 630 953-9938

# BORING LOG RRC-BSB-01

**WEI Job No.: 255-39-01**

Client: **Stantec**
Project: **I-80 Reconstruction, Ridge Road to Houbolt Road**
Location: **Will County, Illinois**

Datum: NAVD 88
Elevation: 554.86 ft
North: 1755592.99 ft
East: 1024452.16 ft
Station: 386+52.00
Offset: 55.2 LT

| Elevation (ft) | SOIL AND ROCK DESCRIPTION | Depth (ft) | Sample No. | SPT Values (blw/6 in) | Qu (tsf) | Moisture Content (%) |
|---|---|---|---|---|---|---|
| 553.7 | 14-inch thick ASPHALT --PAVEMENT-- | | | | | |
| 552.9 | Brown SANDY GRAVEL; damp --AGGREGATE BASE-- | | | | | |
| 551.6 | Stiff, brown and gray SILTY CLAY LOAM, some gravel; damp --FILL-- --RDR 2-3-- | | 1 | 6 6 9 | 1.25 P | 11 |
| 539.4 | Medium dense to dense, brown and gray Gravelly SILTY LOAM to LOAM; damp to moist --FILL-- --RDR 2-3-- --$L_L$(%)=24, $P_L$(%)=15-- --%Gravel=30.0-- --%Sand=25.7-- --%Silt=35.1-- --%Clay=9.1-- --A-4 (1)-- | 5 | 2 | 7 9 11 | NP | 9 |
| | | | 3 | 6 16 25 | NP | 9 |
| | | 10 | 4 | 7 17 24 | NP | 8 |
| | | | 5 | 5 14 13 | NP | 7 |
| | | 15 | 6 | 6 17 18 | NP | 8 |
| 536.9 | Soft, gray and brown SILTY CLAY LOAM, some gravel; damp --RDR 3-- | | 7 | 27 50/3" | 0.25 P | 13 |
| 534.9 | Very dense, gray Weathered BEDROCK --RDR 3-4-- | | 8 | 50/2" | NP | 1 |
| | Strong, light bluish gray, very poor quality, shaly and vuggy DOLOSTONE; closely spaced, highly to moderately weathered, horizontal, oblique, and vertical joints, with <0.05 inch opening, slightly rough walls, and <0.2 inch thick clay and silty infill. --RUN 1: 20.0 to 22.0 feet-- | 20 | 9 | CORE | | |
| | | 25 | | CORE | | |
| | --Recovery: 100%-- --RQD: 0%-- | | 10 | | | |
| | --RUN 2: 22.0 to 30.0 feet-- --Recovery: 58%-- --RQD: 0%-- | | | | | |
| | --RUN 3: 30.0 to 37.0 feet-- --Recovery: 99%-- --RQD: 10%-- | 30 | 11 | CORE | | |
| | | 35 | | | | |
| | --RUN 4: 37.0 to 42.0 feet-- --Recovery: 55%-- --RQD: 15%-- | 40 | 12 | CORE | | |
| | --RUN 5: 42.0 to 46.0 feet-- --Recovery: 100%-- --RQD: 17%-- | 45 | 13 | CORE | | |
| 508.9 | Boring terminated at 46.00 ft | | | | | |
| | | 50 | | | | |

## GENERAL NOTES

Begin Drilling: **08-15-2021** Complete Drilling: **08-16-2021**
Drilling Contractor: **Wang Testing Services** Drill Rig: **20D50T [80%]**
Driller: **R&A** Logger: **M. Sadowski** Checked by: **C. Marin**
Drilling Method: **2.25" ID HSA; boring backfilled upon completion**

## WATER LEVEL DATA

While Drilling: **DRY**
At Completion of Drilling: **DRY**
Time After Drilling: **24 hours**
Depth to Water: **19.00 ft**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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# BORING LOG RRC-BSB-02

**WEI Job No.: 255-39-01**

Client **Stantec**
Project **I-80 Reconstruction, Ridge Road to Houbolt Road**
Location **Will County, Illinois**

Datum: NAVD 88
Elevation: 554.93 ft
North: 1755473.76 ft
East: 1024456.87 ft
Station: 386+53.04
Offset: 64.2 RT

| Elevation (ft) | SOIL AND ROCK DESCRIPTION | Depth (ft) | Sample No. | SPT Values (blw/6 in) | Qu (tsf) | Moisture Content (%) |
|---|---|---|---|---|---|---|
| | 15-inch thick ASPHALT --PAVEMENT-- | | | | | |
| 553.7 | | | | | | |
| 553.2 | Brown and gray SANDY GRAVEL; damp --AGGREGATE BASE-- | | 1 | 3 6 11 | NP | 13 |
| | Medium dense, brown and gray Gravelly SILTY LOAM to LOAM; damp --FILL-- --RDR 2-3-- | 5 | 2 | 5 7 14 | NP | 9 |
| | --L$_L$(%)=22, P$_L$(%)=15-- --%Gravel=29.1-- --%Sand=26.0-- --%Silt=37.4-- --%Clay=7.4-- --A-4 (0)-- | | 3 | 4 6 17 | NP | 11 |
| 546.9 | Medium dense, brown Gravelly SANDY LOAM; damp --RDR 3-- | 10 | 4 | 8 10 10 | NP | 8 |
| 544.4 | Very stiff, brown to gray Gravelly CLAY LOAM --RDR 2-3-- | | 5 | 7 15 18 | 3.25 P | 8 |
| 541.9 | Medium dense, brown and gray Gravelly LOAM; damp --Weathered BEDROCK-- --RDR 2-- | 15 | 6 | 6 7 13 | NP | 20 |
| 539.4 | Medium dense, gray, SILTY LOAM, some gravel; damp --Weathered BEDROCK-- --RDR 2-3-- | | 7 | 7 9 15 | NP | 21 |
| | | | 8 | 50/2" | NP | |
| 534.9 | Very strong, light bluish gray, very poor quality, vuggy DOLOSTONE, occasional shale partings; moderately to highly weathered, horizontal, oblique, and vertical joints, with 0.05 - 0.2 inch opening, slightly rough walls, and <0.2 inch thick clay and sity greenish infill. | 20 | 9 | CORE | | |
| | | 25 | | | | |

| Elevation (ft) | SOIL AND ROCK DESCRIPTION | Depth (ft) | Sample No. | SPT Values (blw/6 in) | Qu (tsf) | Moisture Content (%) |
|---|---|---|---|---|---|---|
| | --RUN 1: 20.0 to 27.0 feet-- --Recovery: 18%-- --RQD: 0%-- --poor water return top 4 feet; bottom 3 feet no water return-- --RUN 2: 27.0 to 34.0 feet-- --Recovery: 31%-- --RQD: 0%-- | 30 | 1 | CORE | | |
| | --no water return after 31 feet-- | | | | | |
| | --RUN 3: 34.0 to 38.0 feet-- --Recovery: 79%-- --RQD: 0%-- | 35 | 2 | CORE | | |
| | Medium strong, light bluish gray, poor quality, shaly and vuggy DOLOSTONE --RUN 4: 38.0 to 40.5 feet-- --Recovery: 97%-- --RQD: 0%-- | 40 | 3 | CORE | | |
| 510.9 | Strong, light gray to dark brownish and grayish gray, very poor quality, shaly and vuggy DOLOSTONE; closely spaced, moderately weathered, horizontal and oblique joints, with 0.05 - > 0.2 inch opening, slightly rough walls, and <0.2 inch thick silt infill. --RUN 5: 40.5 to 44.0 feet-- --Recovery: 50%-- --RQD: 0%-- | 45 | 4 | CORE | | |
| | Boring terminated at 44.00 ft | 50 | | | | |

## GENERAL NOTES

Begin Drilling **08-09-2021** Complete Drilling **08-09-2021**
Drilling Contractor **Wang Testing Services** Drill Rig **20D50T [80%]**
Driller **R&A** Logger **M. Sadowski** Checked by **C. Marin**
Drilling Method **2.25" ID HSA; boring backfilled upon completion**

## WATER LEVEL DATA

While Drilling **16.00 ft**
At Completion of Drilling **DRY**
Time After Drilling **NA**
Depth to Water **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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# BORING LOG RRC-BSB-03

**WEI Job No.: 255-39-01**

Client: **Stantec**
Project: **I-80 Reconstruction, Ridge Road to Houbolt Road**
Location: **Will County, Illinois**

Datum: NAVD 88
Elevation: 555.34 ft
North: 1755568.83 ft
East: 1024517.34 ft
Station: 387+16.41
Offset: 29 LT

| Elevation (ft) | SOIL AND ROCK DESCRIPTION | Depth (ft) | Sample No. | SPT Values (blw/6 in) | Qu (tsf) | Moisture Content (%) |
|---|---|---|---|---|---|---|
| | 18-inch thick CONCRETE --bridge deck pavement-- | | | | | |
| 553.8 | --Drilled from bridge deck-- | | | | | |
| 541.8 | Creek water level | | | | | |
| 540.3 | Very soft, black and gray SILTY CLAY LOAM, some organic matter; saturated --organic content= 11.2%-- --$L_L$(%)=69, $P_L$(%)=32-- --%Gravel=0.8-- --%Sand=4.6-- --%Silt=68.5-- --%Clay=26.1-- --A-7-5 (42)-- | 15 | 1 | 0 0 1 50/5" | 0.16 B | 94 |
| 537.6 | Gray SANDY GRAVEL; saturated | | | | | |
| 535.3 | Strong, light bluish gray, very poor quality, vuggy DOLOSTONE; very closely to closely spaced, moderately weathered, horizontal, oblique, and vertical joints, with <0.05 inch opening, slightly rough walls, and <0.2 inch thick, greenish clay and silty infill. --RUN 1: 20.0 to 28.5 feet-- --Recovery: 57%-- --RQD: 10%-- --no water return after 21 feet-- | 20 | 2 | CORE | | |
| | --Qu=11,678 psi-- --RUN 2: 28.5 to 33.5 feet-- --Recovery: 55%-- --RQD: 0%-- | 30 | 3 | CORE | | |
| | --RUN 3: 33.5 to 37.0 feet-- --Recovery: 90%-- --RQD: 0%-- | 35 | 4 | CORE | | |
| 518.3 | Boring terminated at 37.00 ft | | | | | |

## GENERAL NOTES

Begin Drilling **08-18-2021** Complete Drilling **08-18-2021**
Drilling Contractor **Wang Testing Services** Drill Rig **20D50T [80%]**
Driller **R&N** Logger **M. Sadowski** Checked by **C. Marin**
Drilling Method **2.25" ID HSA; boring backfilled upon completion**

## WATER LEVEL DATA

While Drilling **13.50 ft**
At Completion of Drilling **13.50 ft**
Time After Drilling **NA**
Depth to Water **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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# BORING LOG RRC-BSB-04

**WEI Job No.: 255-39-01**

Client: **Stantec**
Project: **I-80 Reconstruction, Ridge Road to Houbolt Road**
Location: **Will County, Illinois**

Datum: NAVD 88
Elevation: 555.47 ft
North: 1755504.55 ft
East: 1024538.19 ft
Station: 387+35.28
Offset: 35.9 RT

| Elevation (ft) | SOIL AND ROCK DESCRIPTION | Depth (ft) | Sample No. | SPT Values (blw/6 in) | Qu (tsf) | Moisture Content (%) |
|---|---|---|---|---|---|---|
| 554.0 | 18-inch thick CONCRETE --bridge deck pavement-- | | | | | |
| 541.5 | | | | | | |
| 539.5 | Creek water level | | | | | |
| 539.2 | | | | | | |
| 537.5 | Very soft (<0.25P), black and gray SILTY CLAY LOAM; saturated | | 1 | 22 50/2" | NP | 5 |
| | Very dense, gray SANDY GRAVEL; saturated --RDR 3-4-- | | 2 | 50/4" | NP | 9 |
| 534.5 | Very dense, gray SILTY LOAM, few gravel; wet --Highly weathered BEDROCK-- --RDR 3-4-- | 20 | | | | |
| 519.5 | Very strong, light bluish gray, very poor to poor quality, vuggy DOLOSTONE, few shale partings; moderately to closely spaced, highly fragmented, highly to moderately weathered, horizontal, oblique, and vertical joints, with 0-0.2 inch opening, slightly rough walls, and <0.2 inch thick, greenish clay and silty infill. --RUN 1: 21.0 to 28.0 feet-- --Recovery: 100%-- --RQD: 10%-- --Qu=7,494 psi-- --RUN 2: 28.0 to 33.3 feet-- --Recovery: 100%-- --RQD: 8%-- --RUN 3: 33.3 to 36.0 feet-- --Recovery: 100%-- --RQD: 41%-- | 25–35 | 3, 4, 5 | CORE | | |
| | Boring terminated at 36.00 ft | | | | | |

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

## GENERAL NOTES

Begin Drilling **08-08-2021** Complete Drilling **08-08-2021**
Drilling Contractor **Wang Testing Services** Drill Rig **20D50T [80%]**
Driller **R&A** Logger **M. Sadowski** Checked by **C. Marin**
Drilling Method **2.25" ID HSA; boring backfilled upon completion**

## WATER LEVEL DATA

While Drilling **14.00 ft**
At Completion of Drilling **14.00 ft**
Time After Drilling **NA**
Depth to Water **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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# BORING LOG RRC-BSB-05

**WEI Job No.: 255-39-01**

Client: **Stantec**
Project: **I-80 Reconstruction, Ridge Road to Houbolt Road**
Location: **Will County, Illinois**

Datum: NAVD 88
Elevation: 555.73 ft
North: 1755598.94 ft
East: 1024593.29 ft
Station: 387+93.26
Offset: 56.8 LT

| Elevation (ft) | SOIL AND ROCK DESCRIPTION | Depth (ft) | Sample No. | SPT Values (blw/6 in) | Qu (tsf) | Moisture Content (%) |
|---|---|---|---|---|---|---|
| | 20-inch thick ASPHALT --PAVEMENT-- | | | | | |
| 554.1 | Medium dense, brown and gray SANDY GRAVEL; damp --AGGREGATE BASE-- | | 1 | 7 8 6 | NP | 12 |
| 553.0 | Medium dense to very dense, brown and gray SILTY LOAM, little to some gravel; damp --FILL-- --RDR 2-- | 5 | 2 | 19 32 21 | NP | 9 |
| | | | 3 | 25 44 50/2" | NP | 9 |
| | | 10 | 4 | 50/6" | NP | 7 |
| 545.2 | Very dense, gray SANDY GRAVEL; dry --RDR 2-3-- | | 5 | 50/5" | NP | 4 |
| 542.7 | Very dense, gray SILTY LOAM, some gravel; dry --Highly weathered BEDROCK-- --RDR 2-3-- | 15 | 6 | 50/6" | NP | 3 |
| | | | 7 | 50/5" | NP | 4 |
| | | | 8 | 50/2" | NP | 5 |
| 535.7 | Strong, light bluish gray, very poor quality, vuggy DOLOSTONE, few shale partings; closely spaced, highly to moderately weathered, horizontal, oblique, and vertical joints, with <0.05 inch opening, slightly rough walls, and <0.2 inch thick greenish clay and silty infill. --RUN 1: 20.0 to 27.5 feet-- --Recovery: 98%-- --RQD: 17%-- --RUN 2: 27.5 to 35.0 feet-- --Recovery: 98%-- --RQD: 14%-- | 20–25 | 9 | CORE | | |
| | | 30 | 10 | CORE | | |
| 520.7 | Boring terminated at 35.00 ft | 35 | | | | |

## GENERAL NOTES

Begin Drilling: **08-15-2021** Complete Drilling: **08-15-2021**
Drilling Contractor: **Wang Testing Services** Drill Rig: **20D50T [80%]**
Driller: **R&A** Logger: **M. Sadowski** Checked by: **C. Marin**
Drilling Method: **2.25" ID HSA; boring backfilled upon completion**

## WATER LEVEL DATA

While Drilling: **DRY**
At Completion of Drilling: **18.00 ft**
Time After Drilling: **NA**
Depth to Water: **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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# BORING LOG RRC-BSB-06

**WEI Job No.: 255-39-01**

Client: **Stantec**
Project: **I-80 Reconstruction, Ridge Road to Houbolt Road**
Location: **Will County, Illinois**

Datum: NAVD 88
Elevation: 555.68 ft
North: 1755480.43 ft
East: 1024594.60 ft
Station: 387+90.91
Offset: 61.7 RT

| Elevation (ft) | SOIL AND ROCK DESCRIPTION | Depth (ft) | Sample Type / recovery | Sample No. | SPT Values (blw/6 in) | Qu (tsf) | Moisture Content (%) |
|---|---|---|---|---|---|---|---|
| 554.5 | 14-inch thick ASPHALT --PAVEMENT-- | | | | | | |
| 546.7 | Medium dense, brown and gray Gravelly SILTY LOAM to CLAY LOAM; damp --FILL-- --RDR 2-3-- --$L_L$(%)=25, $P_L$(%)=16-- --%Gravel=23.7-- --%Sand=22.3-- --%Silt=42.8-- --%Clay=11.2-- --A4 (2)-- | 5 | SS | 1 | 8 7 8 | NP | 15 |
| | | | SS | 2 | 7 6 11 | NP | 15 |
| | | | SS | 3 | 3 4 17 | NP | 13 |
| 545.2 | Very stiff, black SILTY CLAY; damp --Buried TOPSOIL-- | 10 | SS | 4 | 7 7 7 | 2.50 P | 30 |
| 542.7 | Very stiff, brown and gray CLAY to SILTY CLAY, trace wood fragments; damp --RDR 2-- | | SS | 5 | 3 3 6 | 3.12 B | 17 |
| 537.7 | Hard, brown and light gray SILTY CLAY LOAM; damp --RDR 2-- --$L_L$(%)=33, $P_L$(%)=16-- --%Gravel=0.1-- --%Sand=5.0-- --%Silt=69.6-- --%Clay=25.2-- --A-6 (16)-- | 15 | SS | 6 | 6 11 12 | 4.18 S | 18 |
| | | | SS | 7 | 3 9 16 | 4.50 P | 17 |
| 534.7 | Very dense, gray GRAVEL; damp to wet --Weathered BEDROCK-- --RDR 2-3-- | 20 | SS | 8 | 50/4" | NP | 7 |
| | --poor recovery of rock core possibly due to gummy shale in core sample-- Strong, light bluish gray, very poor quality, vuggy DOLOSTONE, SHALE and MUDSTONE interbedded; | 25 | CORE | 9 | CORE | | |
| 519.7 | closely spaced, moderately weathered, horizontal, oblique, and vertical joints, with 0.05 - 0.2 inch opening, slightly rough walls, and >0.2 inch thick, greenish clay and silty infill. --RUN 1: 21.0 to 29.0 feet-- --Recovery: 63%-- --RQD: 4%-- --RUN 2: 29.0 to 36.0 feet-- --Recovery: 95%-- --RQD: 4%-- | 30, 35 | CORE | 10 | CORE | | |
| | Boring terminated at 36.00 ft | 40, 45, 50 | | | | | |

## GENERAL NOTES

Begin Drilling: **08-05-2021** Complete Drilling: **08-05-2021**
Drilling Contractor: **Wang Testing Services** Drill Rig: **20D50T [80%]**
Driller: **R&R** Logger: **M. Sadowski** Checked by: **C. Marin**
Drilling Method: **2.25" ID HSA to 10 ft; mud rotary thereafter; boring backfilled upon completion**

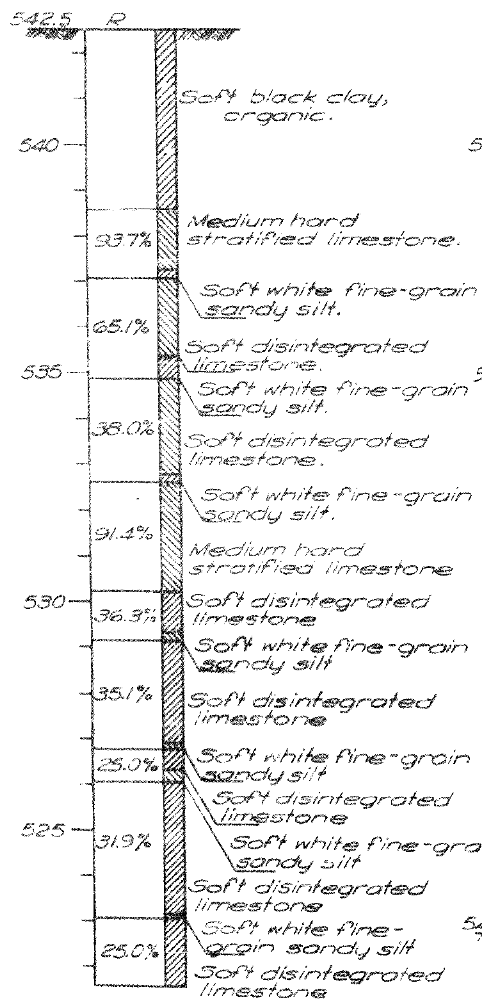
## WATER LEVEL DATA

While Drilling: **18.50 ft**
At Completion of Drilling: **mud in the borehole**
Time After Drilling: **NA**
Depth to Water: **NA**

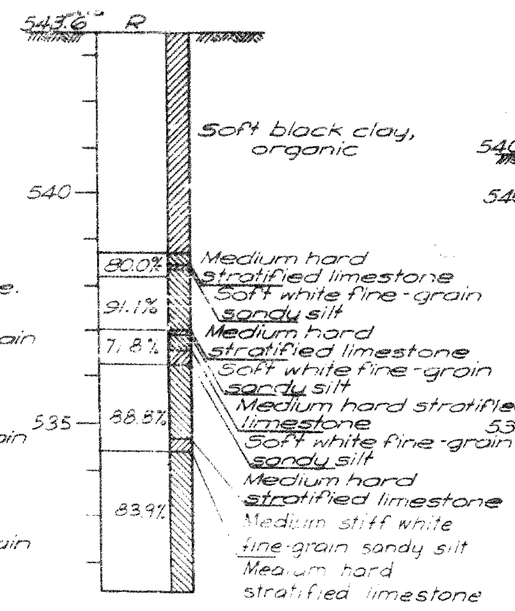
The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

STATE OF ILLINOIS
DEPARTMENT OF PUBLIC WORKS & BUILDINGS
DIVISION OF HIGHWAYS

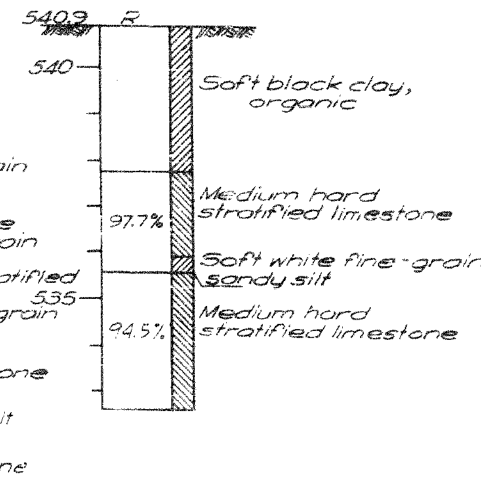
| ROUTE NO. | SECTION | COUNTY | TOTAL SHEETS | SHEET NO. |
|---|---|---|---|---|
| F.A.I.-80 | 99-28 | WILL | 14 | 7 |
| FED. ROAD DIST. NO. 7 | ILLINOIS | PROJECT | | |



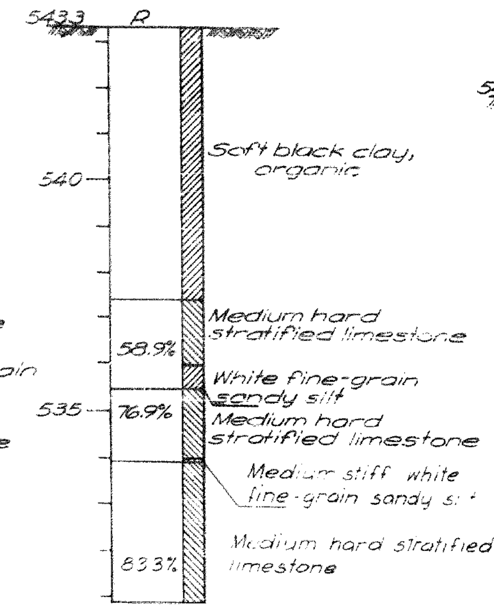

BORING NO. 1




BORING NO. 2




BORING NO. 3

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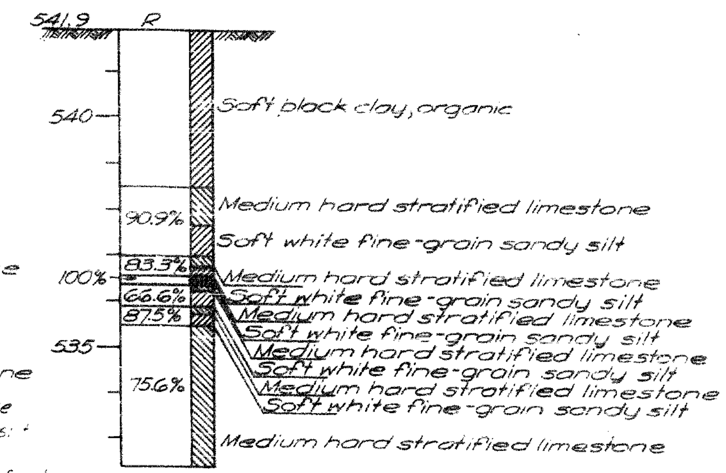


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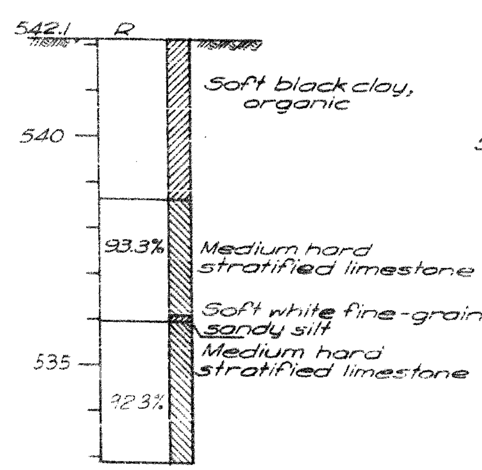
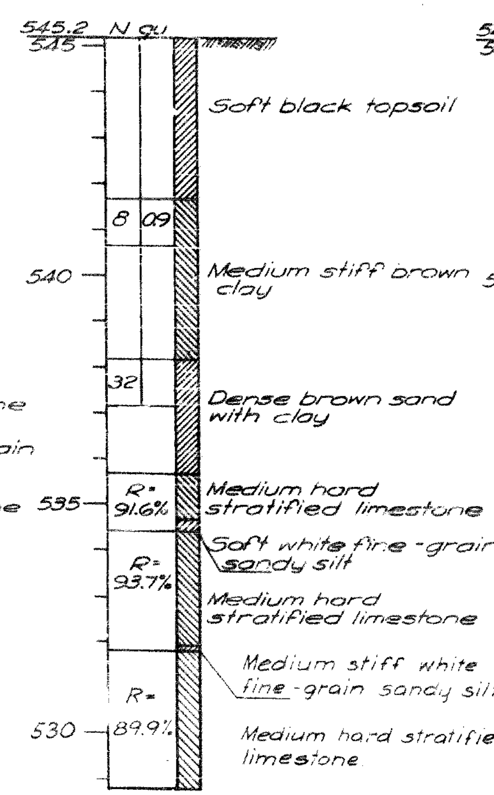
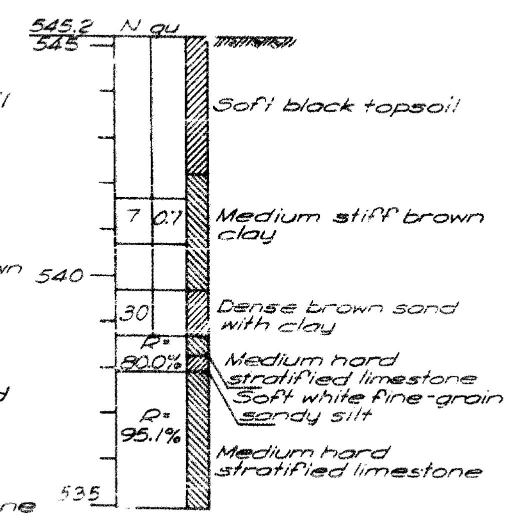



BORING NO. 6




BORING NO. 7




BORING NO. 8

BORING LOCATION

| No. | Station | Offset from ℄ |
|---|---|---|
| 1 | 93+89 | 28' Left |
| 2 | 93+89 | 65' Right |
| 3 | 94+20 | 65' Left |
| 4 | 94+20 | 28' Right |
| 5 | 94+60 | 28' Left |
| 6 | 94+60 | 65' Right |
| 7 | 94+90 | 65' Left |
| 8 | 94+90 | 28' Right |

R = Rock Core % Recovery
N = Blows per foot of penetration of sampling spoon. Hammer Weight = 140 lbs., Drop = 30"
qu = Unconfined compressive strength in tons per square foot.

| | |
|---|---|
| DESIGNED | R.H. & J.D. |
| CHECKED | N.B. & H.C.M. |
| DRAWN | C.L.N. |
| CHECKED | H.M. |

BORING LOGS
FAI-80 OVER
ROCK RUN

STA. 94+40.00

FAI ROUTE 80 PROJECT
SECTION 99-28 WILL COUNTY
Scale: 1" = 2' Date: March 31, 1960

CASLER & STAPLETON AND BLAUVELT ENGINEERING CO.
CONSULTING ENGINEERS
JACKSONVILLE, ILL. NEW YORK, N.Y. CRYSTAL LAKE, ILL.

Dwg. No. 174 B-3002

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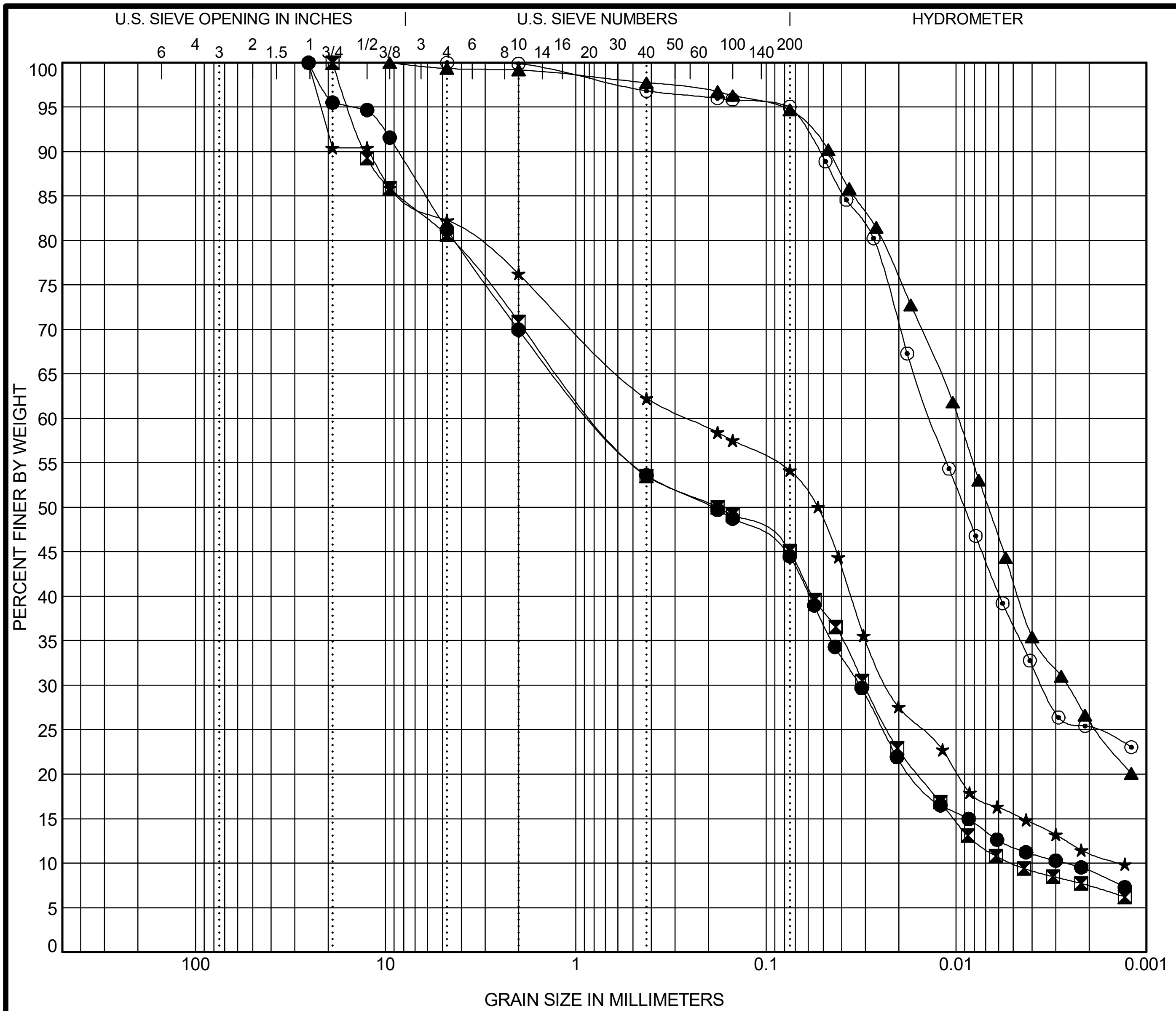
# APPENDIX B




| COBBLES | GRAVEL | SAND coarse | SAND fine | SILT AND CLAY |
|---|---|---|---|---|

| Specimen Identification | | IDH Classification | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|
| ● RRC-BSB-01#3 | 6.0 ft | **Gravelly Silty Loam** | 24 | 15 | 9 | 0.49 | 289.60 |
| ⧗ RRC-BSB-02#3 | 6.0 ft | **Gravelly Silty Loam** | 22 | 15 | 7 | 0.24 | 148.17 |
| ▲ RRC-BSB-03#1 | 16.0 ft | **Silty Clay Loam** | 69 | 32 | 37 | | |
| ★ RRC-BSB-06#2 | 3.5 ft | **Gravelly Silty Loam** | 25 | 16 | 9 | 1.50 | 187.98 |
| ⊙ RRC-BSB-06#6 | 13.5 ft | **Silty Clay Loam** | 33 | 16 | 17 | | |

| Specimen Identification | | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|
| ● RRC-BSB-01#3 | 6.0 ft | 25.4 | 0.778 | 0.032 | 0.003 | 30.0 | 25.7 | 35.1 | 9.1 |
| ⧗ RRC-BSB-02#3 | 6.0 ft | 19 | 0.757 | 0.03 | 0.005 | 29.1 | 26.0 | 37.4 | 7.4 |
| ▲ RRC-BSB-03#1 | 16.0 ft | 9.5 | 0.01 | 0.003 | | 0.8 | 4.6 | 68.5 | 26.1 |
| ★ RRC-BSB-06#2 | 3.5 ft | 25.4 | 0.256 | 0.023 | 0.001 | 23.7 | 22.3 | 42.8 | 11.2 |
| ⊙ RRC-BSB-06#6 | 13.5 ft | 4.75 | 0.014 | 0.004 | | 0.1 | 5.0 | 69.6 | 25.2 |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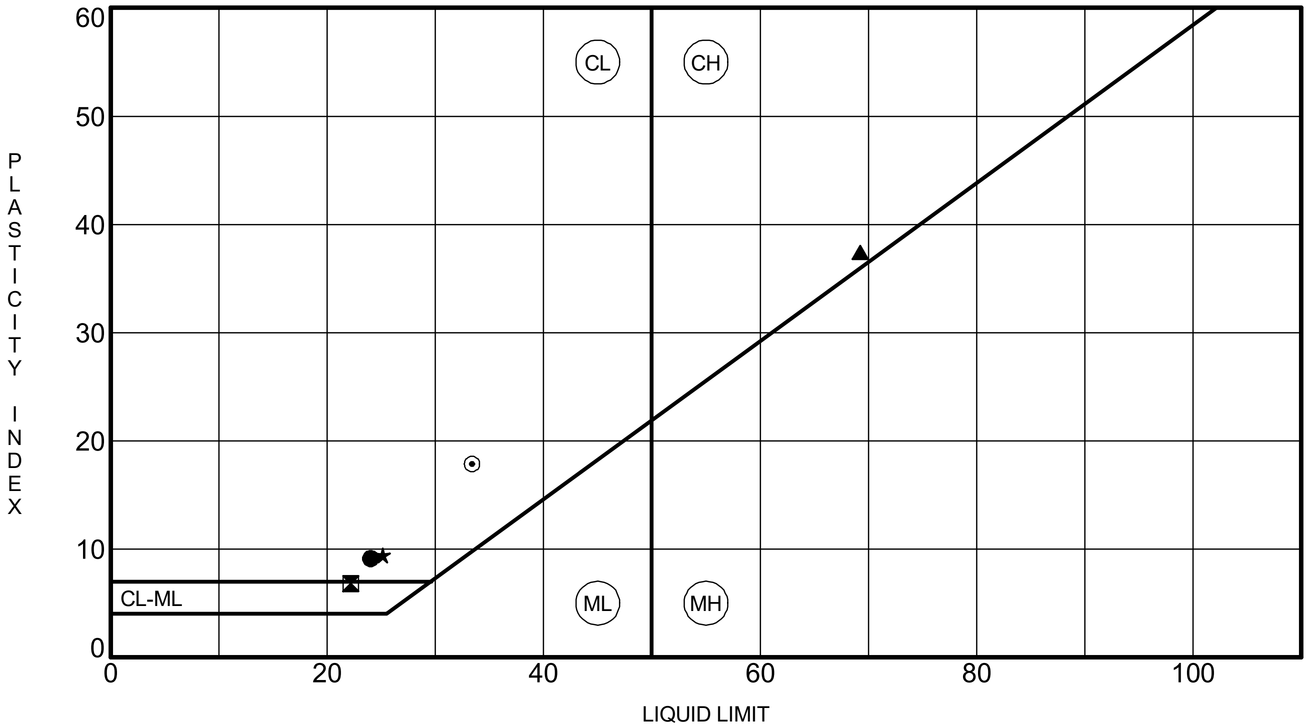
Lombard, IL 60148
Telephone: 630 953-9928
Fax: 630 953-9938


## GRAIN SIZE DISTRIBUTION

Project: I-80 Reconstruction, Ridge Road to Houbolt Road

Location: Will County, Illinois

Number: 255-39-01




| | Specimen Identification | | LL | PL | PI | Fines | IDH Classification |
|---|---|---|---|---|---|---|---|
| ● | **RRC-BSB-01#3** | **6.0 ft** | **24** | **15** | **9** | **44** | **Gravelly Silty Loam** |
| ⧗ | **RRC-BSB-02#3** | **6.0 ft** | **22** | **15** | **7** | **45** | **Gravelly Silty Loam** |
| ▲ | **RRC-BSB-03#1** | **16.0 ft** | **69** | **32** | **37** | **95** | **Silty Clay Loam** |
| ★ | **RRC-BSB-06#2** | **3.5 ft** | **25** | **16** | **9** | **54** | **Gravelly Silty Loam** |
| ⊙ | **RRC-BSB-06#6** | **13.5 ft** | **33** | **16** | **17** | **95** | **Silty Clay Loam** |

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# ATTERBERG LIMITS' RESULTS

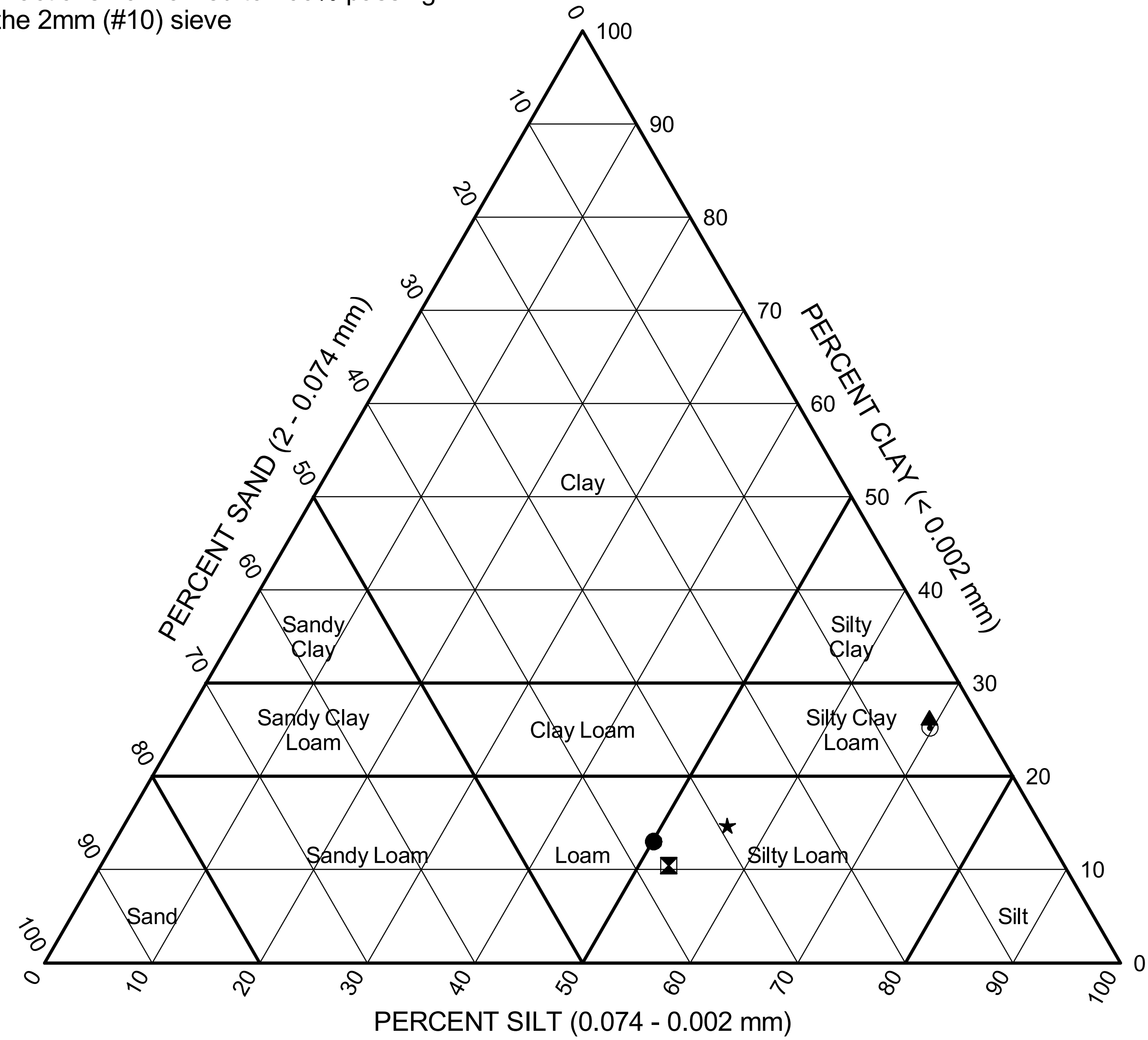
Project: I-80 Reconstruction, Ridge Road to Houbolt Road

Location: Will County, Illinois

Number: 255-39-01

WEI ATTERBERG LIMITS IDH 2553901.GPJ US LAB.GDT 7/15/22

Fractions normalized to 100% passing the 2mm (#10) sieve




| | Sample | Depth (ft) | Sand (%) | Silt (%) | Clay (%) | Classification | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | IL DOT | AASHTO | ASTM |
| ● | RRC-BSB-01#3 | 6.0 | 36.7 | 50.1 | 13.0 | Gravelly Silty Loam | A-4 (1) | SC |
| ⧗ | RRC-BSB-02#3 | 6.0 | 36.7 | 52.8 | 10.4 | Gravelly Silty Loam | A-4 (0) | SC-SM |
| ▲ | RRC-BSB-03#1 | 16.0 | 4.6 | 69.1 | 26.3 | Silty Clay Loam | A-7-5 (42) | CH |
| ★ | RRC-BSB-06#2 | 3.5 | 29.2 | 56.1 | 14.7 | Gravelly Silty Loam | A-4 (2) | CL |
| ⊙ | RRC-BSB-06#6 | 13.5 | 5.0 | 69.7 | 25.2 | Silty Clay Loam | A-6 (16) | CL |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

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# IDH Textural Classification Chart

Project: I-80 Reconstruction, Ridge Road to Houbolt Road

Location: Will County, Illinois

Number: 255-39-01

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# ORGANIC CONTENT in SOILS by LOSS on IGNITION

## ASTM D 2974, Method C

**Client:** Stantec
**Project:** I-80
**WEI Job:** 255-39-01
**Type/Condition:** SS
**Testing Furnace Temp °C.:** 440

**Analyst Name:** F. Bozga
**Date Received:** 3/31/2021
**Date Tested:** 8/31/2021

| **Sample No./ Depth** | RRC-BSB-03 S#1 (16-17.5 ft) | | | | |
|---|---|---|---|---|---|
| **Sample Description** | **Silty Clay Loam** | | | | |
| **wet soil + tare** | 65.66 | | | | |
| **Dry Soil + Tare** | 54.46 | | | | |
| **Tare Mass** | 42.31 | | | | |
| **w (%)** | 92 | | | | |
| **Dry Soil + Tare** | 54.46 | | | | |
| **Ash+ Tare** | 53.1 | | | | |
| **Tare Mass** | 42.31 | | | | |
| **Ash Content (%)** | 89 | | | | |
| **Organic Content (%)** | 11.2 | | | | |

Prepeared By:____________________________________________________

Reviwed By:_____________________________________________________

AASHTO ACCREDITATION PROGRAM

# Unconfined Compressive Strength of Intact Rock Core Specimens

**Project:** I-80 Reconstruction

**Client:** Stantec

**WEI Job No.:** 255-39-01

| Field Sample ID | Run # | Depth (ft) | Location | Sample Description | Length (in) | | Diameter (in) | Total Load (lbs) | Total Pressure (psi) | Fracture Type* | Break Date | Tested By | Area ($in^2$) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Before Capping | After Capping | | | | | | | |
| RRC-BSB-03 | 1 | 23.0 | Joliet, IL | Dolostone | 4.17 | NA | 2.06 | 38960 | 11678 | 3 | 8/31/21 | MAC | 3.33 |
| RRC-BSB-04 | 1 | 21.0 | Joliet, IL | Dolostone | 4.02 | NA | 2.06 | 24880 | 7494 | 3 | 8/31/21 | MAC | 3.32 |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |

*** Fracture Types:**

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;
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.

Prepared by:____________________________

Checked by: ____________________________

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Lombard, Illinois 60148
Phone (630) 953-9928
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# SHELBY TUBE DESCRIPTION

**WEI JOB #** 255-39-01
**ANALYST:** M. Ciapas
**PROJECT:** **I-80 Reconstruction**
**EXTRUSION DATE:** 9/1/2021
**CLIENT:** **Stantec**

**BORING ID:** RRC-BSB-01
**SAMPLE ID:** ST-1
**SAMPLE INTERVAL:** 6-8 ft.

**Length of Recovery** 5"

**Soil Description**

| | | |
|---|---|---|
| | Tare Mass | 31.61 |
| TOP | Wet + tare | 196.1 |
| | Dry + tare | 187.84 |
| | | 5% |

5" Brown and Black Gravelly SILTY LOAM to LOAM

* Penetrometer Reading
Moisture content taken in Hatched area

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# APPENDIX C

Run #1


0 6 inches

Boring RRC-BSB-01:
Run #1, 20.0 to 22.0 feet, RECOVERY=100%, RQD=0%
Run #2, 22.0 to 30.0 feet, RECOVERY=58%, RQD=0%

| | | |
|---|---|---|
| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | APPENDIX C-1 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
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| FOR STANTEC | | 255-39-01 |

Run #3


0 6 inches

Boring RRC-BSB-01:
Run #3, 30.0 to 37.0 feet, RECOVERY=99%, RQD=10%

| | | |
|---|---|---|
| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | APPENDIX C-2 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
| Wang Engineering | | 1145 N. Main Street<br>Lombard, IL 60148<br>www.wangeng.com |
| FOR STANTEC | | 255-39-01 |

Run #4


0 6 inches

Boring RRC-BSB-01:
Run #4, 37.0 to 42.0 feet, RECOVERY=55%, RQD=15%
Run #5, 42.0 to 46.0 feet, RECOVERY=100%, RQD=17%

| | | |
|---|---|---|
| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | APPENDIX C-3 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
| Wang Engineering | | 1145 N. Main Street<br>Lombard, IL 60148<br>www.wangeng.com |
| FOR STANTEC | | 255-39-01 |

Run #1


0 6 inches

Run #2


0 6 inches

Boring RRC-BSB-02:
Run #1, 20.0 to 27.0 feet, RECOVERY=18%, RQD=0%
Boring RRC-BSB-02B:
Run #2, 27.0 to 34.0 feet, RECOVERY=31%, RQD=0%
Run #3, 34.0 to 38.0 feet, RECOVERY=79%, RQD=0%

| | | |
|---|---|---|
| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | APPENDIX C-4 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
| Wang Engineering | | 1145 N. Main Street<br>Lombard, IL 60148<br>www.wangeng.com |
| FOR STANTEC | | 255-39-01 |

Run #4


Boring RRC-BSB-02B:
Run #4, 38.0 to 40.5 feet, RECOVERY=97%, RQD=30%
Run #5, 40.5 to 44.0 feet, RECOVERY=50%, RQD=0%

| | | |
|---|---|---|
| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | APPENDIX C-5 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
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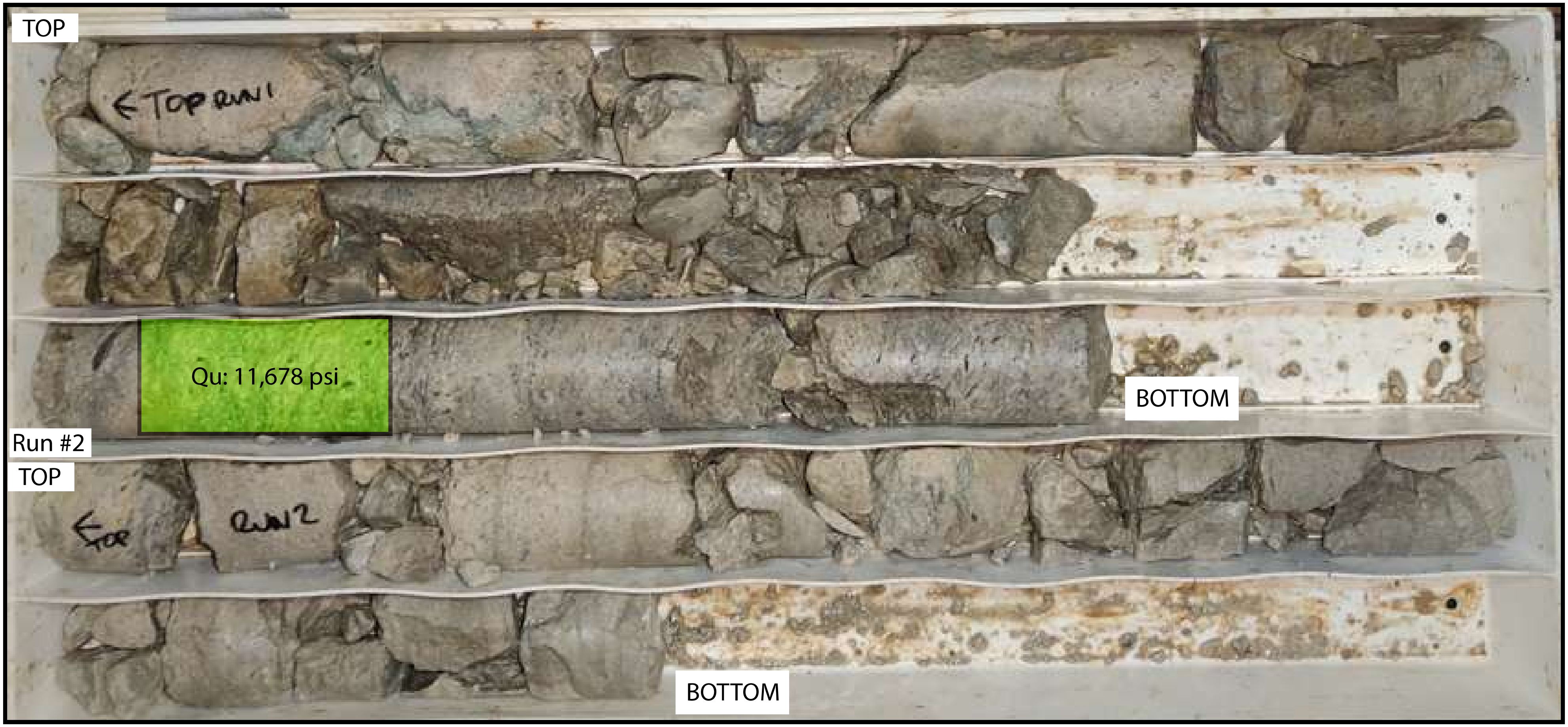
| FOR STANTEC | | 255-39-01 |

Run #1




0 6 inches

Boring RRC-BSB-03:
Run #1, 20.0 to 28.5 feet, RECOVERY=57%, RQD=10%
Run #2, 28.5 to 33.5 feet, RECOVERY=55%, RQD=0%

| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
|---|---|---|
| SCALE: GRAPHICAL | APPENDIX C-6 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
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Run #3


0 6 inches

Boring RRC-BSB-03:
Run #3, 33.5 to 37.0 feet, RECOVERY=90%, RQD=0%

| | | |
|---|---|---|
| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | APPENDIX C-7 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
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Run #1


0 6 inches

Boring RRC-BSB-04:
Run #1, 21.0 to 28.0 feet, RECOVERY=100%, RQD=10%

| | | |
|---|---|---|
| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | APPENDIX C-8 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
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Run #2


0 6 inches

Boring RRC-BSB-04:
Run #2, 28.0 to 33.3 feet, RECOVERY=100%, RQD=8%
Run #3, 33.3 to 36.0 feet, RECOVERY=100%, RQD=41%

| | | |
|---|---|---|
| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | APPENDIX C-9 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
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Run #1


0 6 inches

Boring RRC-BSB-05:
Run #1, 20.0 to 27.5 feet, RECOVERY=98%, RQD=17%

| | | |
|---|---|---|
| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | APPENDIX C-10 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
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| FOR STANTEC | | 255-39-01 |

Run #2


0 6 inches

Boring RRC-BSB-05:
Run #2, 27.5 to 35.0 feet, RECOVERY=98%, RQD=14%

| | | |
|---|---|---|
| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | APPENDIX C-11 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
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| FOR STANTEC | | 255-39-01 |

Run #1


0 6 inches

Boring RRC-BSB-06:
Run #1, 21.0 to 29.0 feet, RECOVERY=63%, RQD=4%

| | | |
|---|---|---|
| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | APPENDIX C-12 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
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| FOR STANTEC | | 255-39-01 |

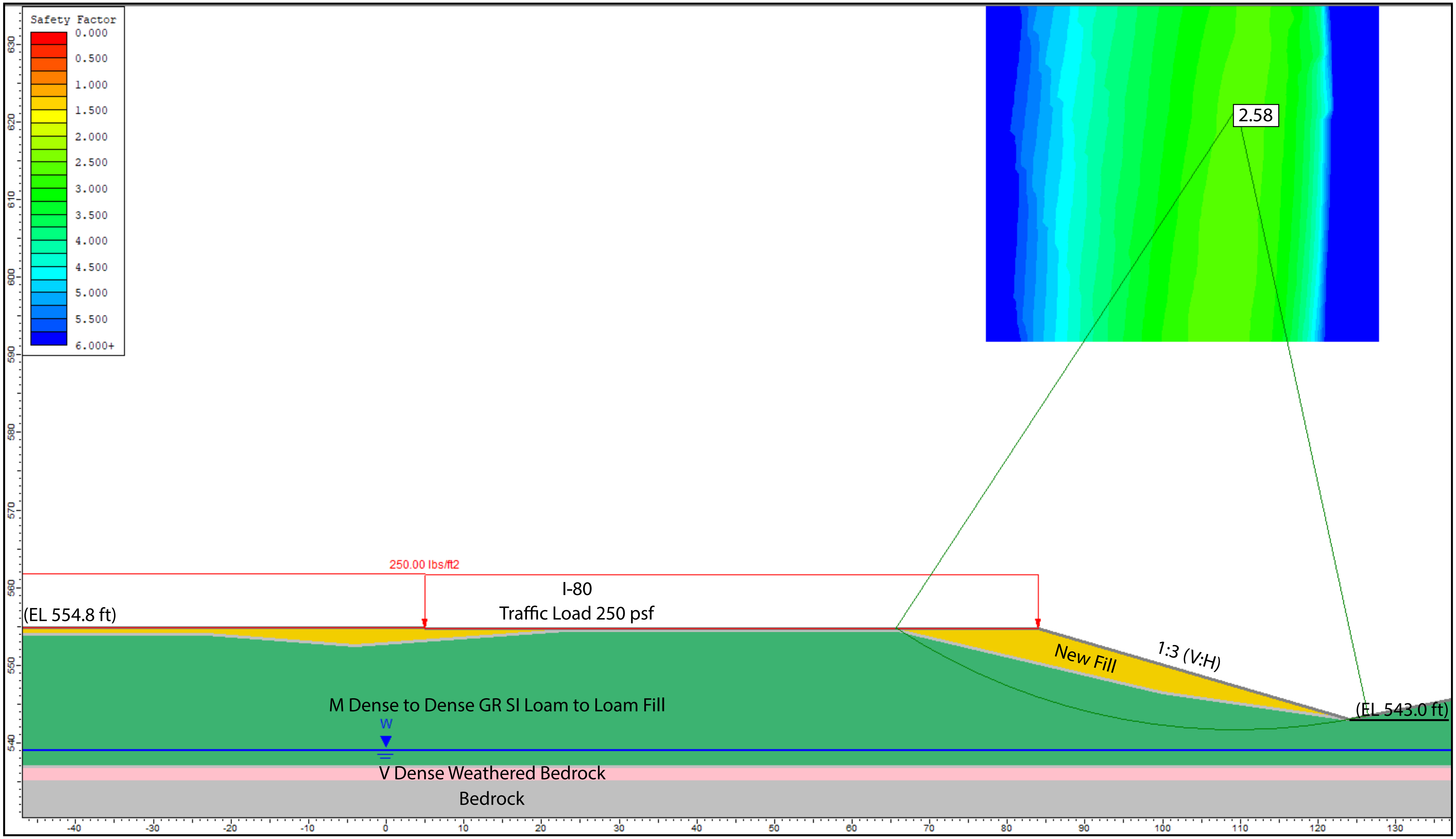
Run #2


0 6 inches

Boring RRC-BSB-06:
Run #2, 29.0 to 36.0 feet, RECOVERY=95%, RQD=4%

| | | |
|---|---|---|
| BEDROCK CORE: I-80 RECONSTRUCTION, I-80 BRIDGE OVER ROCK RUN CREEK, WILL COUNTY, ILLINOIS | | |
| SCALE: GRAPHICAL | APPENDIX C-13 | DRAWN BY: J. Bensen<br>CHECKED BY: A. Hamad |
| Wang Engineering | | 1145 N. Main Street<br>Lombard, IL 60148<br>www.wangeng.com |
| FOR STANTEC | | 255-39-01 |

# APPENDIX D




Undrained Analysis, West Approach, Station 386+00.00, Ref Boring: RRC-BSB-01 and RRC-BSB-02

| Layer ID | Description | Total Unit Weight (pcf) | Undrained Cohesion (psf) | Undrained Friction Angle (degrees) |
|---|---|---|---|---|
| 1 | New Fill | 125 | 1000 | 0 |
| 2 | M Dense to Dense GR SI Loam to Loam Fill | 115 | 0 | 30 |
| 3 | V Dense Weathered Bedrock | 120 | 0 | 36 |

GLOBAL STABILITY: I-80 BRIDGE OVER ROCK RUN CREEK; I-80 RECONSTRUCTION FROM EAST OF RIDGE RD TO HOUBOLT RD, WILL COUNTY, ILLINOIS

SCALE: GRAPHICAL

APPENDIX D-1

DRAWN BY: D You
CHECKED BY: A. Hamad

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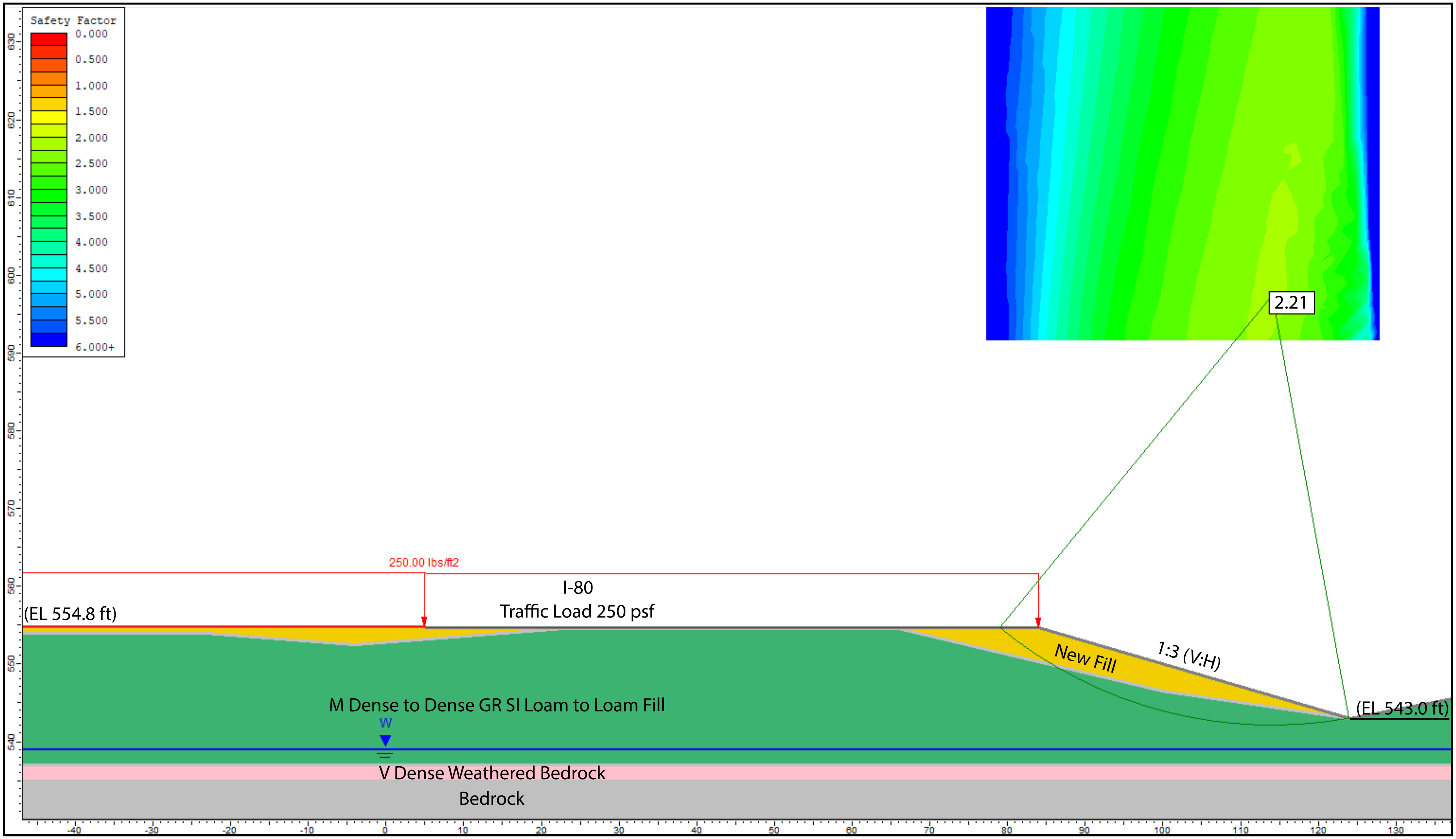
FOR STANTEC

255-39-01
KE225039




Drained Analysis, West Approach, Station 386+00.00, Ref Boring: RRC-BSB-01 and RRC-BSB-02

| Layer ID | Description | Total Unit Weight (pcf) | Drained Cohesion (psf) | Drained Friction Angle (degrees) |
|---|---|---|---|---|
| 1 | New Fill | 125 | 100 | 30 |
| 2 | M Dense to Dense GR SI Loam to Loam Fill | 115 | 0 | 30 |
| 3 | V Dense Weathered Bedrock | 120 | 0 | 36 |

GLOBAL STABILITY: I-80 BRIDGE OVER ROCK RUN CREEK; I-80 RECONSTRUCTION FROM EAST OF RIDGE RD TO HOUBOLT RD, WILL COUNTY, ILLINOIS

SCALE: GRAPHICAL

APPENDIX D-2

DRAWN BY: D You
CHECKED BY: A. Hamad

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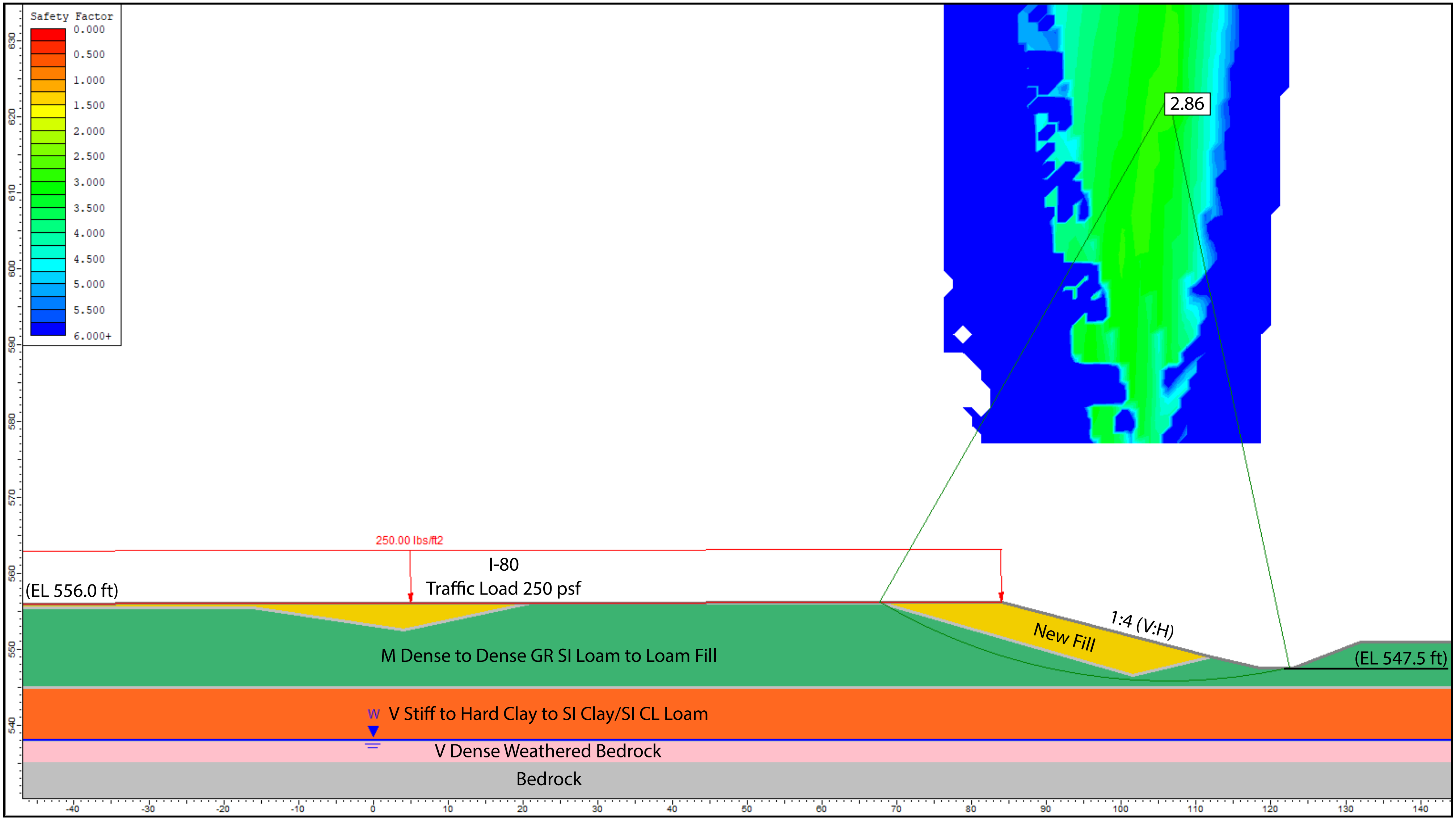
FOR STANTEC

255-39-01
KE225039




Undrained Analysis, East Approach, Station 389+00.00, Ref Boring: RRC-BSB-05 and RRC-BSB-06

| Layer ID | Description | Total Unit Weight (pcf) | Undrained Cohesion (psf) | Undrained Friction Angle (degrees) |
|---|---|---|---|---|
| 1 | New Fill | 125 | 1000 | 0 |
| 2 | M Dense to Dense SI Loam to CL Loam | 115 | 0 | 30 |
| 3 | V Stiff to Hard Clay to SI Clay/SI CL Loam | 120 | 3500 | 0 |
| 4 | V Dense Weathered Bedrock | 120 | 0 | 36 |

GLOBAL STABILITY: I-80 BRIDGE OVER ROCK RUN CREEK; I-80 RECONSTRUCTION FROM EAST OF RIDGE RD TO HOUBOLT RD, WILL COUNTY, ILLINOIS

SCALE: GRAPHICAL | APPENDIX D-3 | DRAWN BY: D You | CHECKED BY: A. Hamad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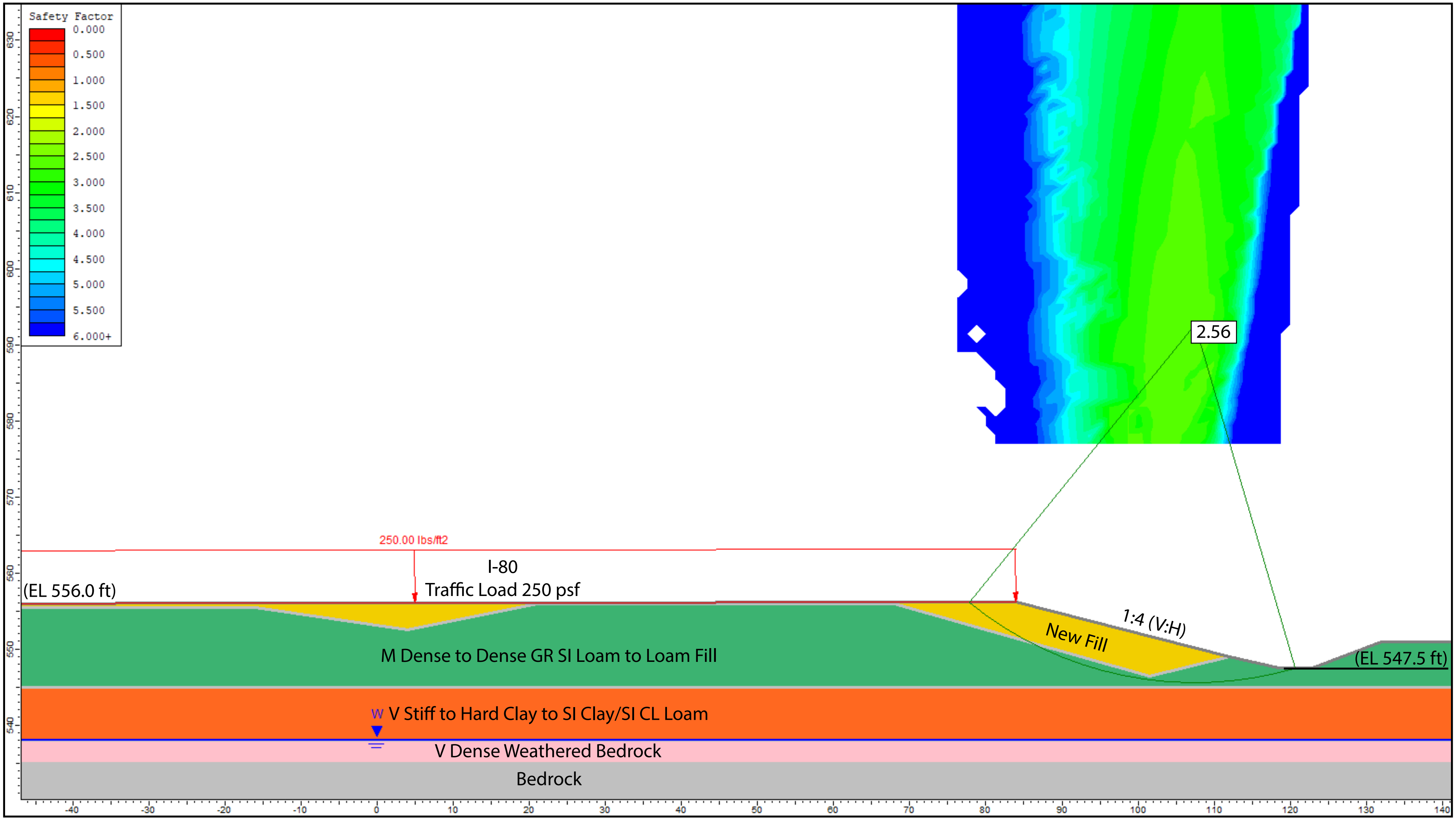
FOR STANTEC | 255-39-01 | KE225039




Drained Analysis, East Approach, Station 389+00.00, Ref Boring: RRC-BSB-05 and RRC-BSB-06

| Layer ID | Description | Total Unit Weight (pcf) | Drained Cohesion (psf) | Drained Friction Angle (degrees) |
|---|---|---|---|---|
| 1 | New Fill | 125 | 100 | 30 |
| 2 | M Dense to Dense SI Loam to CL Loam | 115 | 0 | 30 |
| 3 | V Stiff to Hard Clay to SI Clay/SI CL Loam | 120 | 100 | 32 |
| 4 | V Dense Weathered Bedrock | 120 | 0 | 36 |

GLOBAL STABILITY: I-80 BRIDGE OVER ROCK RUN CREEK; I-80 RECONSTRUCTION FROM EAST OF RIDGE RD TO HOUBOLT RD, WILL COUNTY, ILLINOIS

SCALE: GRAPHICAL

APPENDIX D-4

DRAWN BY: D You
CHECKED BY: A. Hamad


Wang Engineering
A Terracon Company
1145 N. Main Street
Lombard, IL 60148
www.wangeng.com


FOR STANTEC

255-39-01
KE225039

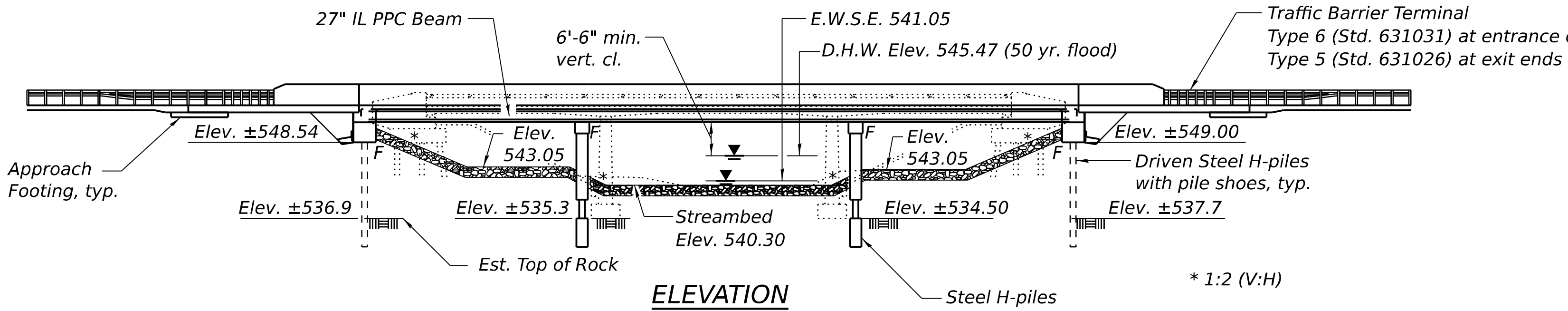
1145 North Main Street
Lombard, Illinois 60148
Phone (630) 953-9928
www.wangeng.com

# APPENDIX E

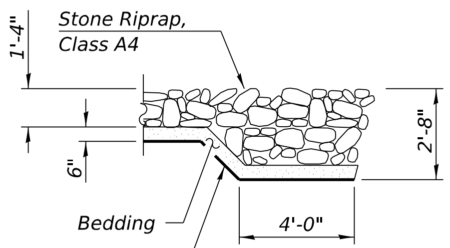
Benchmark: (BM 13) 3½" DOH brass disc on headwall at south east corner of SN 099-0046. Elev. 558.009.

Existing Structures: SN 099-0046 E.B. and SN 099-0047 W.B. Built in 1963 as F.A.I. Rte.80, Section 99-2B, at Sta. 94+40.00. Existing dual structures each consist of 3-span continuous reinforced concrete haunched slab supported by cast-in-place reinforced concrete stub abutments and solid pier stems. 104'-9" Back to Back abutments, 44'-0" out-to-out deck, with no skew. Structures to be removed and replaced. Traffic to be maintained utilizing staged construction.

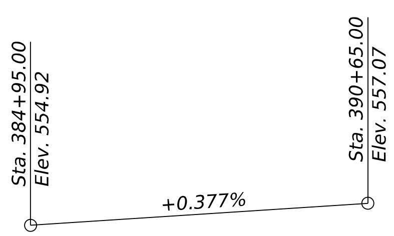
No Salvage.




## ELEVATION




## SECTION A-A



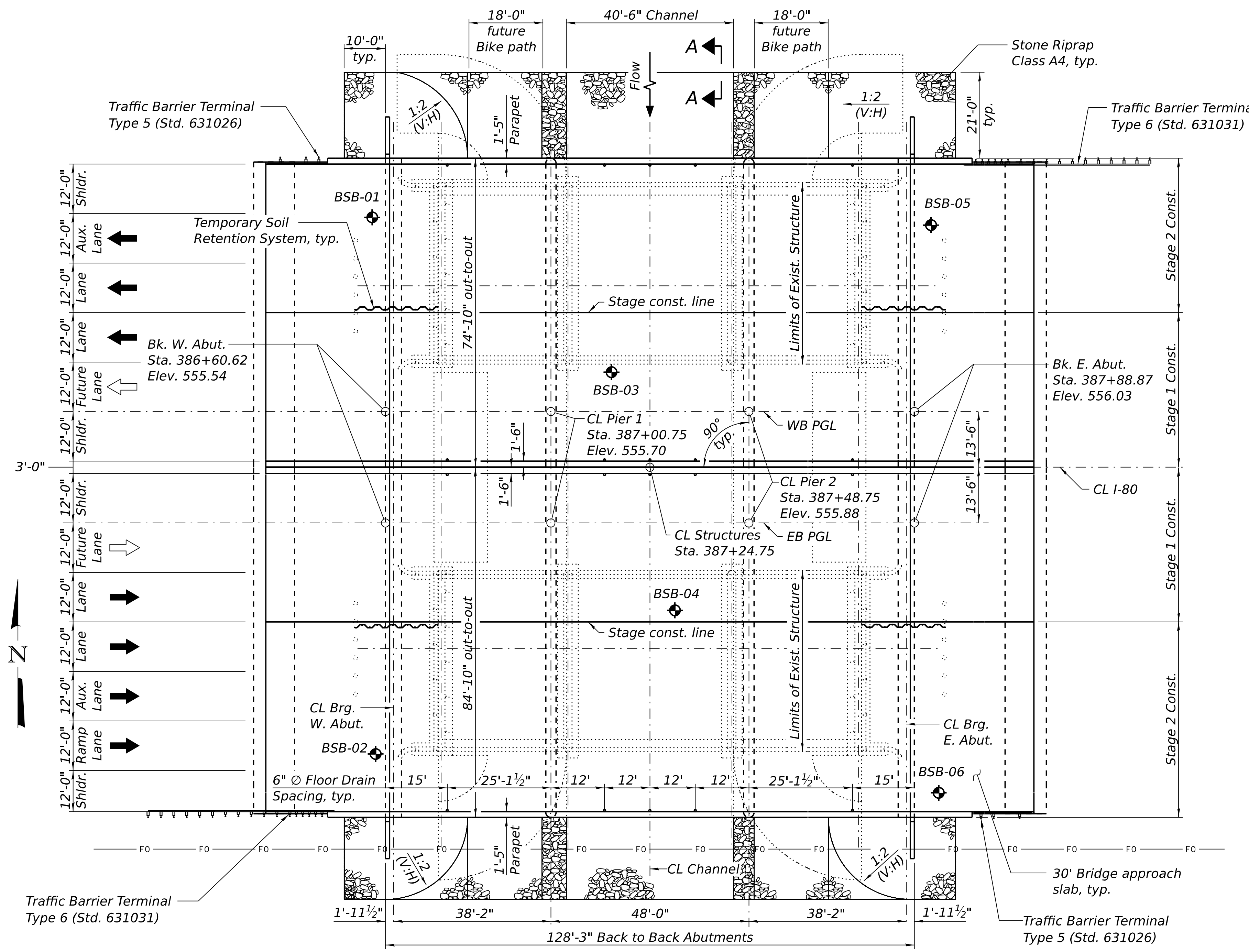

## PROFILE GRADE

(along EB and WB I-80 PGLs)

## LEGEND

Soil Boring

Existing Fiber Optic Cable




## PLAN

## HIGHWAY CLASSIFICATION

FAI Rte. 80 - I-80
Functional Class: Interstate
ADT: 65,200 (2019); 93,900 (2040)
ADTT: 15,170 (2019); 21,840 (2040)
DHV: 11,270 (2040)
Design Speed: 70 m.p.h.
Posted Speed: 65 m.p.h.
Two-Way Traffic
Directional Distribution: 50:50

## LOADING HL-93

Allow 50#/sq. ft. for future wearing surface.

## DESIGN SPECIFICATIONS

2020 AASHTO LRFD Bridge Design Specifications, 9th Edition

## DESIGN STRESSES

### FIELD UNITS

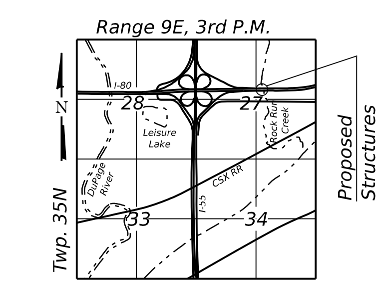
f'c = 3,500 psi
f'c = 4,000 psi (Superstructure)
fy = 60,000 psi (Reinforcement)
fy = 50,000 psi (Piles)

### PRECAST PRESTRESSED UNITS

f'ci = 6,500 psi
f'c = 8,500 psi
fpu = 270,000 psi (0.6" ∅ low lax strands)
fpbt = 202,300 psi (0.6" ∅ low lax strands)

## SEISMIC DATA

Seismic Performance Zone (SPZ) = 1
Design Spectral Acceleration at 1.0 sec. (SD1) = 0.068g
Design Spectral Acceleration at 0.2 sec. (SDS) = 0.127g
Soil Site Class = C




## LOCATION SKETCH

## GENERAL PLAN AND ELEVATION
## I-80 OVER ROCK RUN CREEK
## F.A.I. ROUTE 80 - SEC. 2021-154-R
## WILL COUNTY
## STA. 387+24.75
## STRUCTURE NO. 099-8318 (EB)
## STRUCTURE NO. 099-8319 (WB)

MODEL: Default
FILE NAME: pw://transystems-pw.bentley.com:transyscorp-pw1-hosted/Documents/Projects_2018/CH401/401180022/01-Stantec/CAD/ML-04_62R28/04-Sheets/04-Structures/099-8318 & 8319 I-80 over Rock Run/TSL/0998318&8319-R2R28-TSL-001.dgn

LIN ENGINEERING,LTD.
Consulting Engineers
Springfield, Illinois

| | | | |
|---|---|---|---|
| USER NAME = | DESIGNED - CZ | REVISED - | STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION |
| | CHECKED - MTH | REVISED - | |
| PLOT SCALE = | DRAWN - LAV | REVISED - | |
| PLOT DATE = | CHECKED - MTH | REVISED - | |

SHEET SG-1 OF 3 SHEETS

| F.A.I. RTE. | SECTION | COUNTY | TOTAL SHEETS | SHEET NO. |
|---|---|---|---|---|
| 80 | 2021-154-R | WILL | 3 | 272 |
| | | CONTRACT NO. 62R28 | | |
| | | ILLINOIS | FED. AID PROJECT | |

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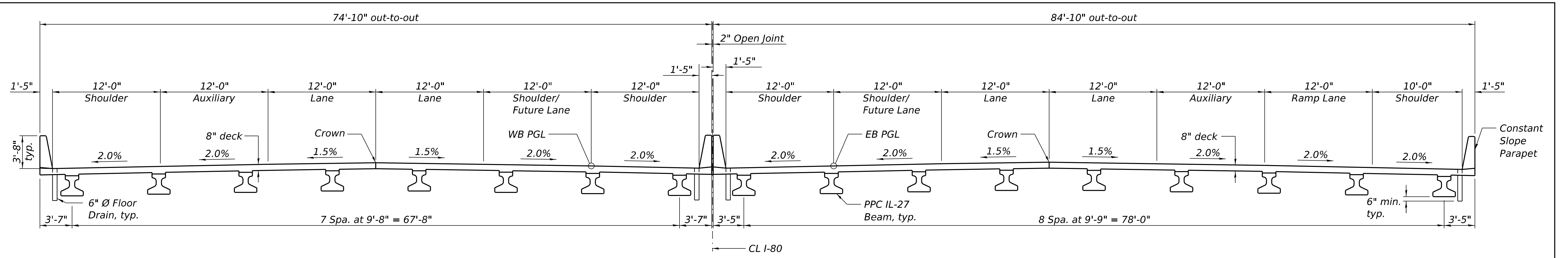
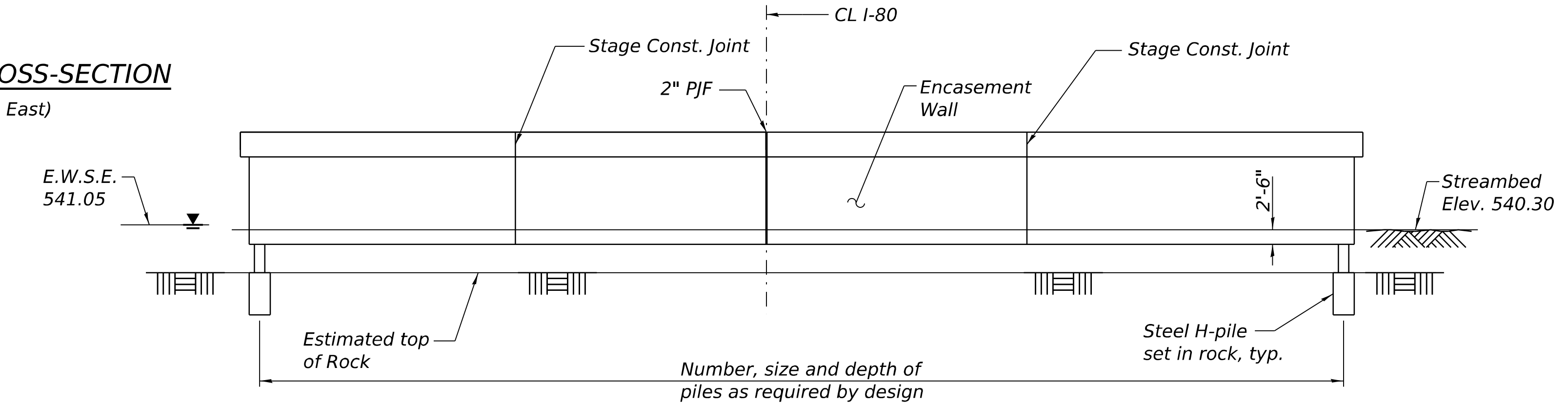
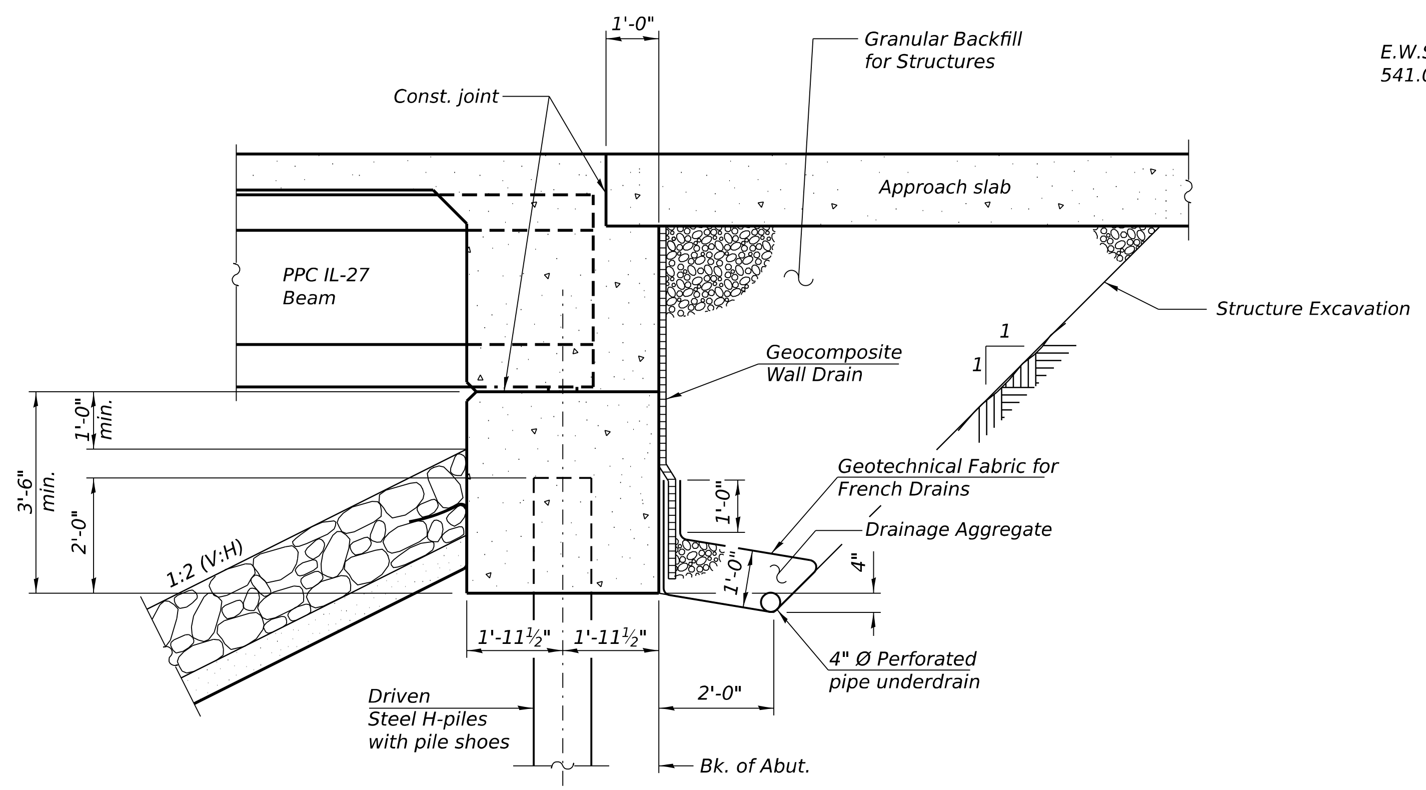


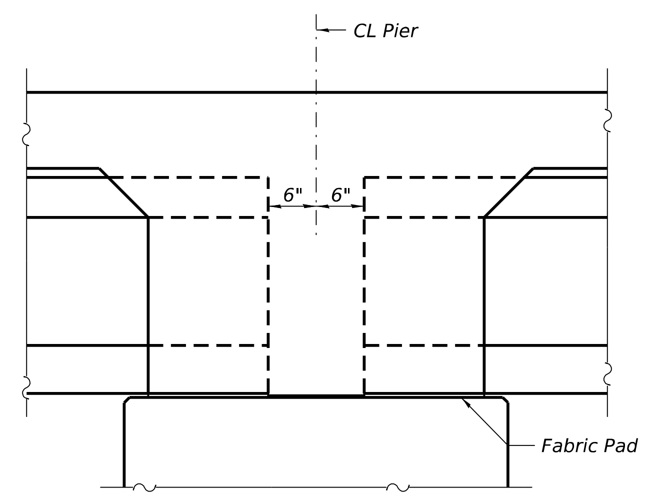

## PROPOSED CROSS-SECTION
(Looking East)




## PIER SKETCH
(Looking East)




## SECTION THRU INTEGRAL ABUTMENT




## SECTION AT PIER

## WATERWAY INFORMATION

Drainage Area = 14.1 mi.²
Exist. Low Grade Elev. 554.50 @ Sta. 384+24.57
Prop. Low Grade Elev. 555.42 @ Sta. 383+78.74

| Flood | Freq. Yr. | Q C.F.S. | Opening Ft² Exist. | Opening Ft² Prop. | Nat. H.W.E. | Head - Ft. Exist. | Head - Ft. Prop. | Headwater El. Exist. | Headwater El. Prop. |
|---|---|---|---|---|---|---|---|---|---|
| | 10 | 567 | 281.7 | 345.7 | 545.50 | 0.00 | 0.00 | 545.50 | 545.50 |
| Design | 50 | 1,103 | 415.6 | 502.7 | 547.47 | 0.00 | 0.00 | 547.47 | 547.47 |
| Base | 100 | 1,460 | 500.2 | 680.8 | 548.61 | 0.00 | 0.00 | 548.61 | 548.61 |
| Max. Calc. | 500 | 2,795 | 787.4 | 1141.1 | 552.06 | 0.00 | 0.09 | 552.06 | 552.15 |

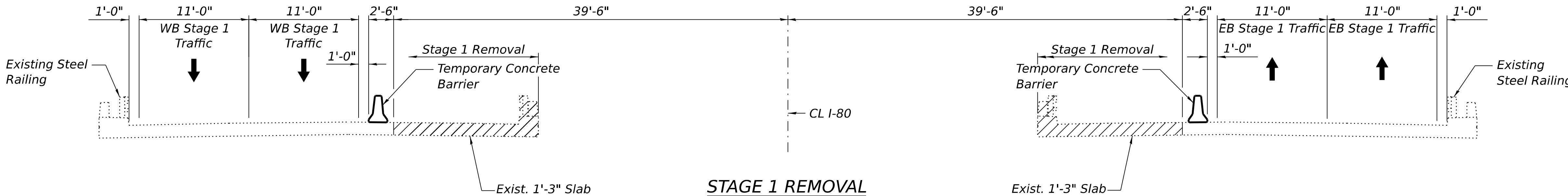
10 year velocity through existing bridge = 2.01 fps
10 year velocity through prop. bridge = 1.64 fps

## DESIGN SCOUR ELEVATION TABLE

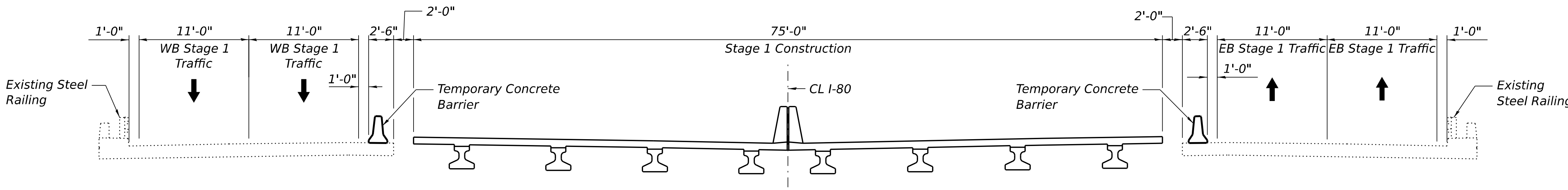
| Event / Limit State | Design Scour Elevations (ft.) W. Abut. | Pier 1 | Pier 2 | E. Abut. | Item 113 |
|---|---|---|---|---|---|
| Q100 | 548.54 | 531.01 | 531.01 | 549.00 | 5 |
| Q200 | 548.54 | 529.87 | 529.87 | 549.00 | |
| Design | 548.54 | 534.50 | 534.50 | 549.00 | |
| Check | 548.54 | 534.50 | 534.50 | 549.00 | |

## GENERAL DETAILS
## I-80 OVER ROCK RUN CREEK
## F.A.I. ROUTE 80 - SEC. 2021-154-R
## WILL COUNTY
## STA. 387+24.75
## STRUCTURE NO. 099-8318 (EB)
## STRUCTURE NO. 099-8319 (WB)

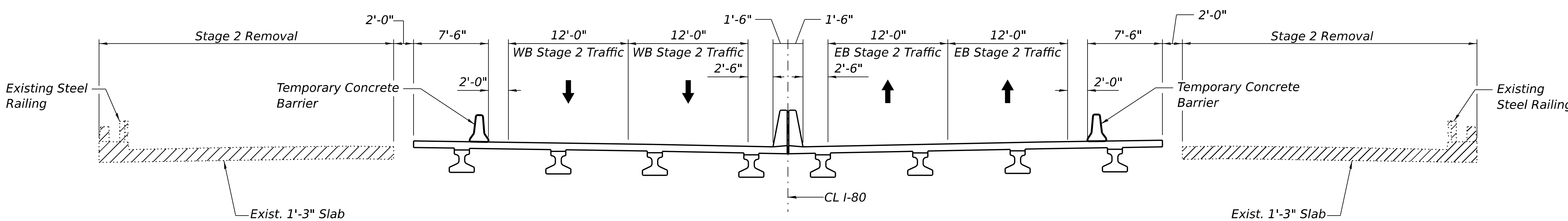
Notes:
Hatching represents limits of removal.
All sections are looking east.



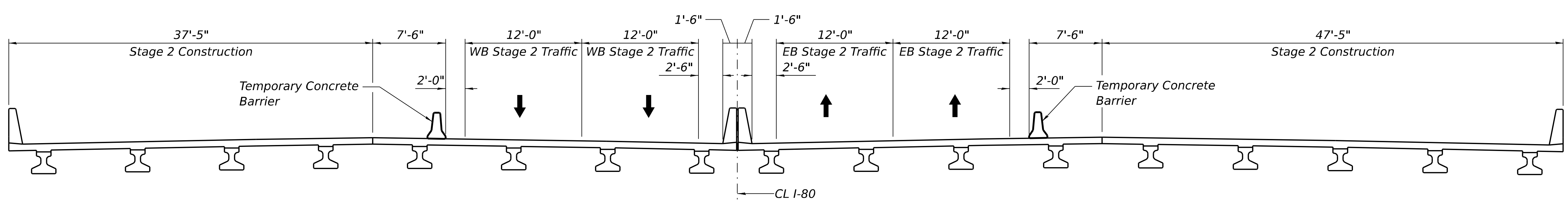

## STAGE 1 REMOVAL




## STAGE 1 CONSTRUCTION




## STAGE 2 REMOVAL




## STAGE 2 CONSTRUCTION

STAGE CONSTRUCTION DETAILS
I-80 OVER ROCK RUN CREEK
F.A.I. ROUTE 80 - SEC. 2021-154-R
WILL COUNTY
STA. 387+24.75
STRUCTURE NO. 099-8318 (EB)
STRUCTURE NO. 099-8319 (WB)

| LIN ENGINEERING,LTD. Consulting Engineers Springfield, Illinois | USER NAME = | DESIGNED - CZ | REVISED - | STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION |
|---|---|---|---|---|
| | | CHECKED - MTH | REVISED - | |
| | PLOT SCALE = | DRAWN - LAV | REVISED - | |
| | PLOT DATE = | CHECKED - MTH | REVISED - | |

SHEET SG-3 OF 3 SHEETS

| F.A.I. RTE. | SECTION | COUNTY | TOTAL SHEETS | SHEET NO. |
|---|---|---|---|---|
| 80 | 2021-154-R | WILL | 3 | 274 |

CONTRACT NO. 62R28

ILLINOIS FED. AID PROJECT

MODEL: Default
FILE NAME: pw://transystems-pw.bentley.com:transyscorp-pw1-hosted/Documents/Projects_2018/CH401/401180022/01-Stantec/CAD/ML-04_62R28/04-Sheets/04-Structures/099-8318 & 8319 I-80 over Rock Run/TSL/0998318&8319-R2R28-TSL-003.dgn

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Lombard, Illinois 60148
Phone (630) 953-9928
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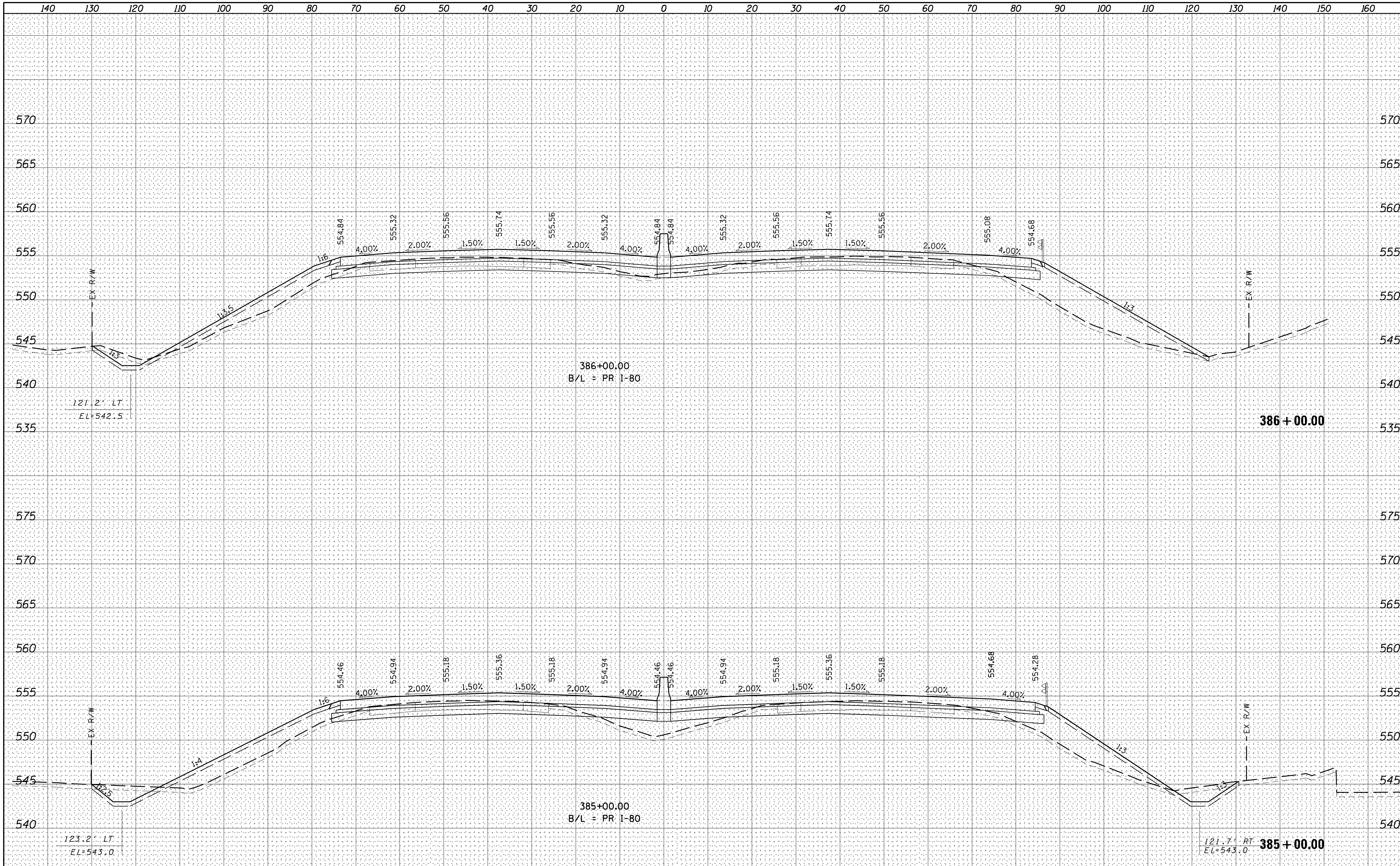
# APPENDIX F




FINAL SURVEY: NOTE BOOK | SURVEYED | PLOTTED | TEMPLATE | AREAS | AREAS CHECKED | NO. | BY | DATE

ORIGINAL SURVEY: NOTE BOOK | SURVEYED | PLOTTED | TEMPLATE | AREAS | AREAS CHECKED | NO. | BY | DATE

| | | | |
|---|---|---|---|
| FILE NAME = d146314-sht-ILR-I80-XSect.dgn | USER NAME = mostafan | DESIGNED - BAJ | REVISED - |
| | | DRAWN - BAJ | REVISED - |
| | PLOT SCALE = 20.0000 ' / in. | CHECKED - MAM | REVISED - |
| | PLOT DATE = 8/12/2020 | DATE - 10/11/2019 | REVISED - |

**STATE OF ILLINOIS**
**DEPARTMENT OF TRANSPORTATION**

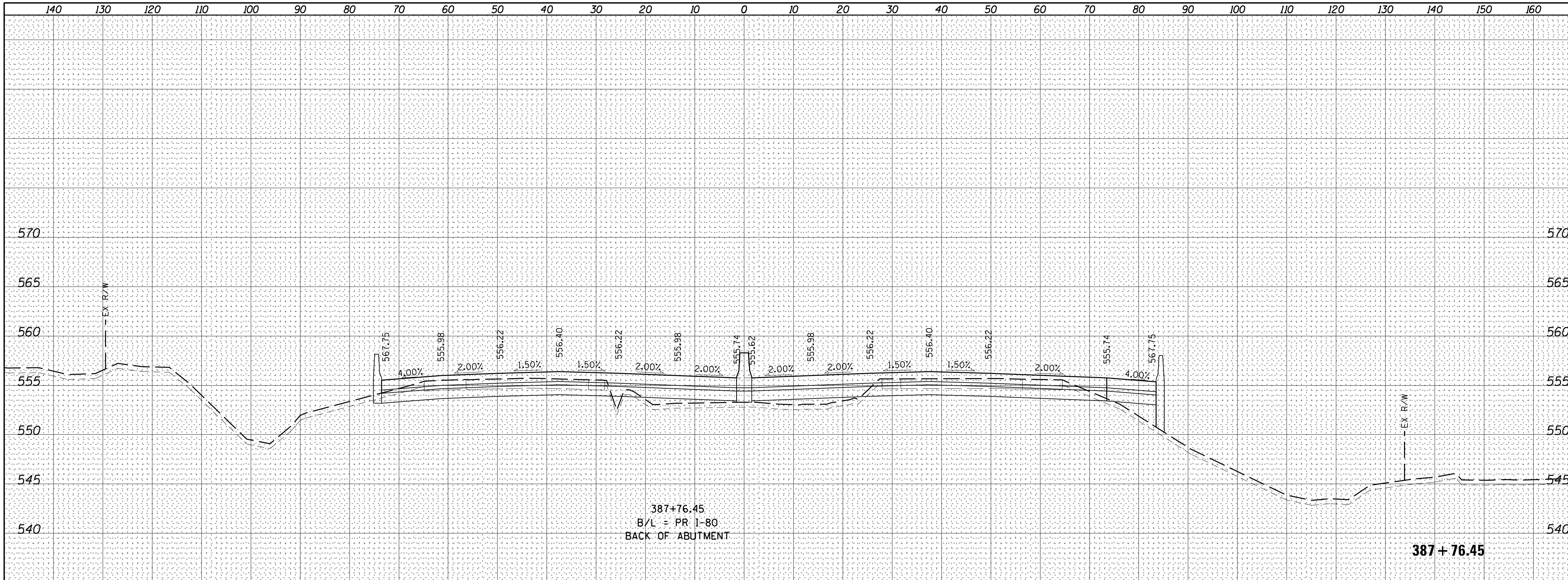
**ILR – I-80 FROM RIDGE ROAD TO US ROUTE 30**
**I-80 PROPOSED CROSS SECTIONS**

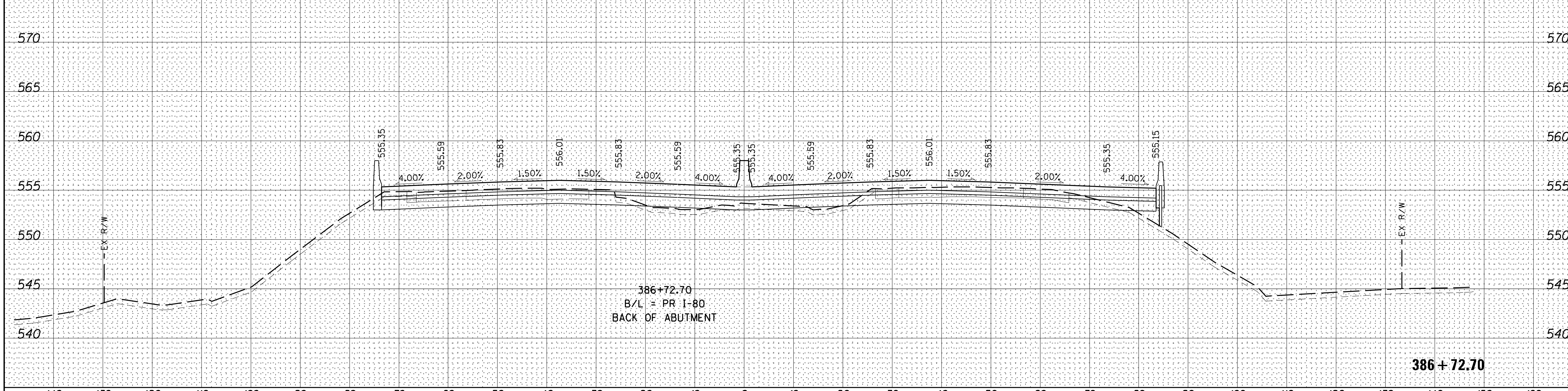
SCALE: 10 H : 5 V | SHEET 99 OF 352 SHEETS | STA. 385+00.00 TO STA. 386+00.00

| F.A. RTE. | SECTION | COUNTY | TOTAL SHEETS | SHEET NO. |
|---|---|---|---|---|
| | | | 352 | 99 |

CONTRACT NO.

ILLINOIS | FED. AID PROJECT




FINAL SURVEY — NOTE BOOK NO. / SURVEYED / PLOTTED / TEMPLATE / AREAS / AREAS CHECKED / BY / DATE

ORIGINAL SURVEY — NOTE BOOK NO. / SURVEYED / PLOTTED / TEMPLATE / AREAS / AREAS CHECKED / BY / DATE

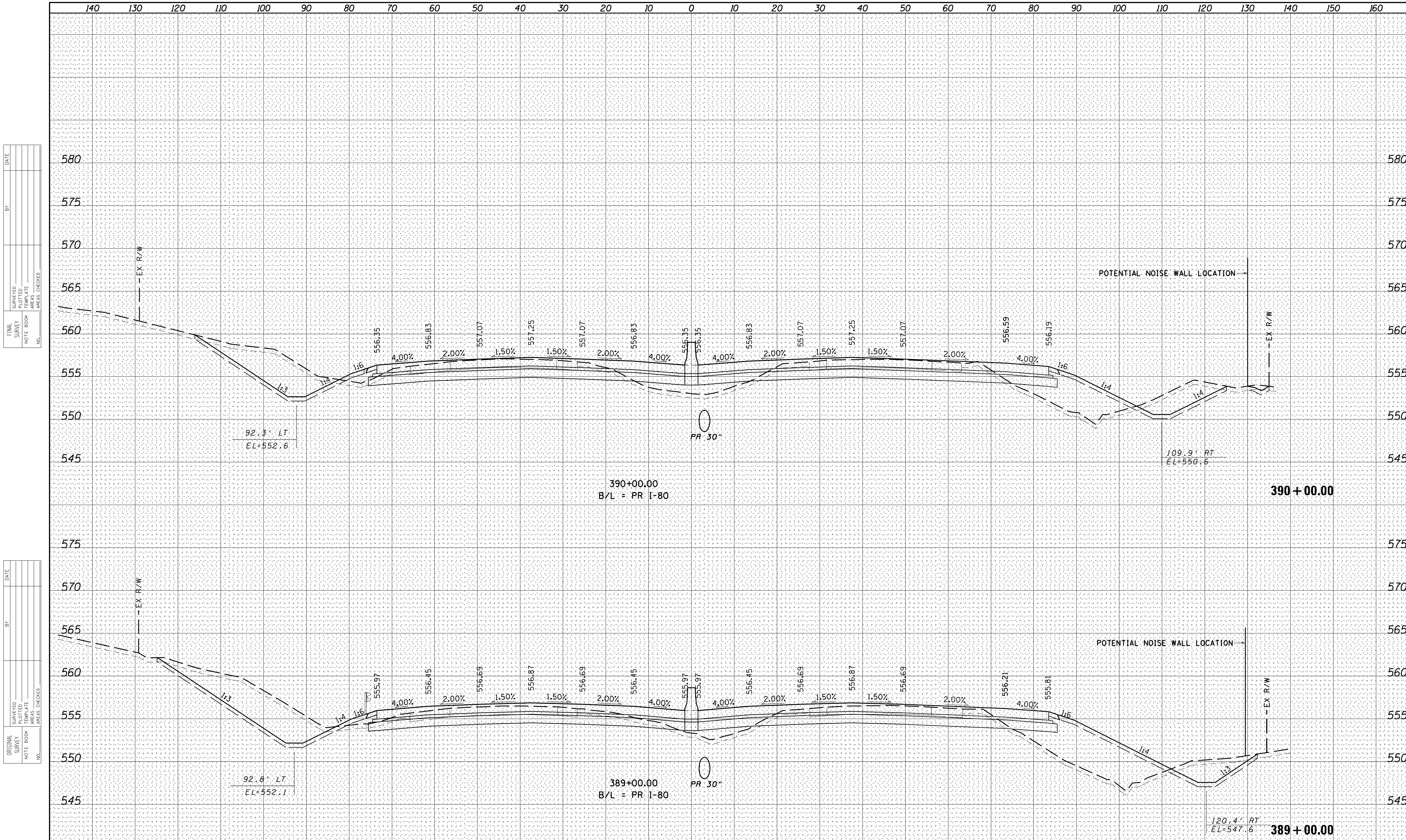
| | | | |
|---|---|---|---|
| FILE NAME = d146314-sht-ILR-I80-XSect.dgn | USER NAME = mostafan | DESIGNED - BAJ | REVISED - |
| | | DRAWN - BAJ | REVISED - |
| | PLOT SCALE = 20.0000 ' / in. | CHECKED - MAM | REVISED - |
| | PLOT DATE = 8/12/2020 | DATE - 10/11/2019 | REVISED - |

**STATE OF ILLINOIS**
**DEPARTMENT OF TRANSPORTATION**

**ILR – I-80 FROM RIDGE ROAD TO US ROUTE 30**
**I-80 PROPOSED CROSS SECTIONS**

SCALE: 10 H : 5 V | SHEET 100 OF 352 SHEETS | STA. 386+72.70 TO STA. 387+76.45

| F.A. RTE. | SECTION | COUNTY | TOTAL SHEETS | SHEET NO. |
|---|---|---|---|---|
| | | | 352 | 100 |
| | | CONTRACT NO. | | |
| | ILLINOIS FED. AID PROJECT | | | |




| | | | | |
|---|---|---|---|---|
| FILE NAME = d146314-sht-ILR-I80-XSect.dgn | USER NAME = mostafan | DESIGNED - BAJ | REVISED - | STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION |
| | | DRAWN - BAJ | REVISED - | |
| | PLOT SCALE = 20.0000 ' / in. | CHECKED - MAM | REVISED - | |
| | PLOT DATE = 8/12/2020 | DATE - 10/11/2019 | REVISED - | |

ILR – I-80 FROM RIDGE ROAD TO US ROUTE 30
I-80 PROPOSED CROSS SECTIONS

SCALE: 10 H : 5 V | SHEET 101 OF 352 SHEETS | STA. 389+00.00 TO STA. 390+00.00

| F.A. RTE. | SECTION | COUNTY | TOTAL SHEETS | SHEET NO. |
|---|---|---|---|---|
| | | | 352 | 101 |
| | | CONTRACT NO. | | |
| | ILLINOIS FED. AID PROJECT | | | |