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August 2016

Geotechnical Investigation Report: TCAAP Redevelopment Area (Amended August 2016)

Prepared for:

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I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.

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1.0 Project Information

1.1 Background

Ramsey County and the City of Arden Hills have formed a partnership to redevelop approximately 427 acres at the former Twin Cities Army Ammunition Plant (TCAAP). The project site is located in Ramsey County, Minnesota predominately within the limits of the City of Arden Hills. The site is located within portions of Sections 9 and 16, Township 30 North, Range 23 West of the 5th Principal Meridian, (the Site). The Site is bounded by U.S. Interstate Highway 35W on the west, Minnesota State Aid Highway (CSAH) 96 to the south and U.S. Highway 10 to the Southwest. The Arden Hills Army Training Site (AHATS) bounds the Site to the east (see Figure 1A).

An infrastructure improvement plan was developed by Kimley-Horn and Associates, Inc. (Kimley-Horn) to prepare the site for the proposed redevelopment. In support of this plan, Wenck Associates, Inc. (Wenck) was retained to prepare a geotechnical drilling and testing program to evaluate the subsurface conditions in proposed infrastructure improvement areas. A request for quote was completed and presented to Ramsey County to issue for contractor bidding. Bids were received and a contract was subsequently awarded to Northern Technologies, Inc. (NTI) to complete the geotechnical field investigation, sample collection, and sample testing.

A geotechnical report was completed and submitted to Kimley-Horn in January 2016. Following completion of that report, additional geotechnical site investigation and analysis requested for two pedestrian bridge sites, a sheet pile weir outlet structure, and a retaining wall. An investigation was performed at these locations including geotechnical drilling, sampling and testing. This amended report includes the site investigation and analysis results and geotechnical recommendations for the additional project sites. A site layout map showing the soil boring locations is included as Figure 1B.

Previous investigations at the site include a preliminary geotechnical investigation conducted by American Engineering Testing, Inc. and Braun Intertec Corporation for Ryan Companies US, Inc. in 2007. In that investigation, 219 soil borings were conducted on a 500-foot grid across the site to evaluate the general suitability of the site for redevelopment. Information from an interim report of that investigation entitled "*Interim Report-Preliminary Geotechnical Evaluation"* provides an overview of site geology, groundwater conditions, and unsuitable soils. A copy of the report was provided by Ramsey County and is included in Appendix A. The report is referenced herein to supplement the area-specific data collected for this investigation.

2.0 Spine Road Bridge

2.1 Proposed Design Understanding

It is our understanding that the proposed Spine Road Bridge will be constructed using an arch bridge design supported by a pile foundation. The design loads on this foundation were provided by the structural design engineer (Kimley-Horn) as follows:

- Vertical Reaction: 66 kips/foot
- Horizontal Reaction (Outward): 90 kips/foot
- Allowable Horizontal Movement (Outward): 0.84 inches

2.2 Soil Borings

Four soil borings (BR-600, BR-601, BR-602 and BR-603) were completed at the proposed Spine Road Bridge site in the locations shown in Figure 2. Boreholes were advanced with hollow stem auger methods using a 3.25-inch I.D. auger. Standard Penetration Testing was completed at 2.5-foot intervals to a depth of 20-feet below ground surface, then at 5-foot intervals until the end of each boring. All four soil borings were completed to a depth of approximately 75 feet below ground surface.

2.2.1 Southeast Abutment Subsurface Conditions

Borehole locations BR-600 and BR-601 represent the approximate extents of the proposed southeast abutment. In general, materials encountered below the southeast abutment location included varying amounts of fill and undisturbed alluvial and swamp deposit soils composed of fine-grained, medium dense silty sand with organics to depth of approximately 14.5 feet below the surface. A peat layer was encountered in BR-601 from 12.0- 14.5 feet. This material was underlain by sediments composed of medium-dense silty to clayey sand to a depth of approximately 35 feet below ground surface. A medium stiff to very stiff sandy lean clay material was encountered at 35 feet and continued to a depth ranging from 65-76 feet. Very dense poorly graded sand with some gravel was encountered below the sandy lean clay in both borings and continued to termination depth.

Groundwater was encountered at a depth of approximately 17 feet below the surface in BR-600 and approximately 4 feet below the surface in BR-601. The high groundwater elevation in BR-601 is apparently due to water collecting in the organic fill material encountered above the native soils. Heaving sands were also noted at the termination depth in BR-601.

2.2.2 Northwest Abutment Subsurface Conditions

Borehole locations BR-602 and BR-603 represent the approximate extents of the proposed northwest abutment. Materials encountered in these boring generally consisted of a shallow topsoil layer followed alternating layers of silty sand and sandy clay fluvial sediments to an approximate depth of 24 feet. This was underlain by stiff lean clay with sand to a depth of approximately 60 feet. A layer of stiff to very stiff silty lean clay was encountered from approximately 60 -70 feet below the surface, followed by very dense poorly graded sand with gravel to termination depth.

Groundwater was encountered in BR-602 at a depth of 17 feet below the surface, and in BR-603 at a depth of approximately 12 feet. All boreholes were grouted to the surface upon completion. A complete description of materials encountered is given on the boring logs included in Appendix B.

2.3 Sample Collection and Laboratory Testing

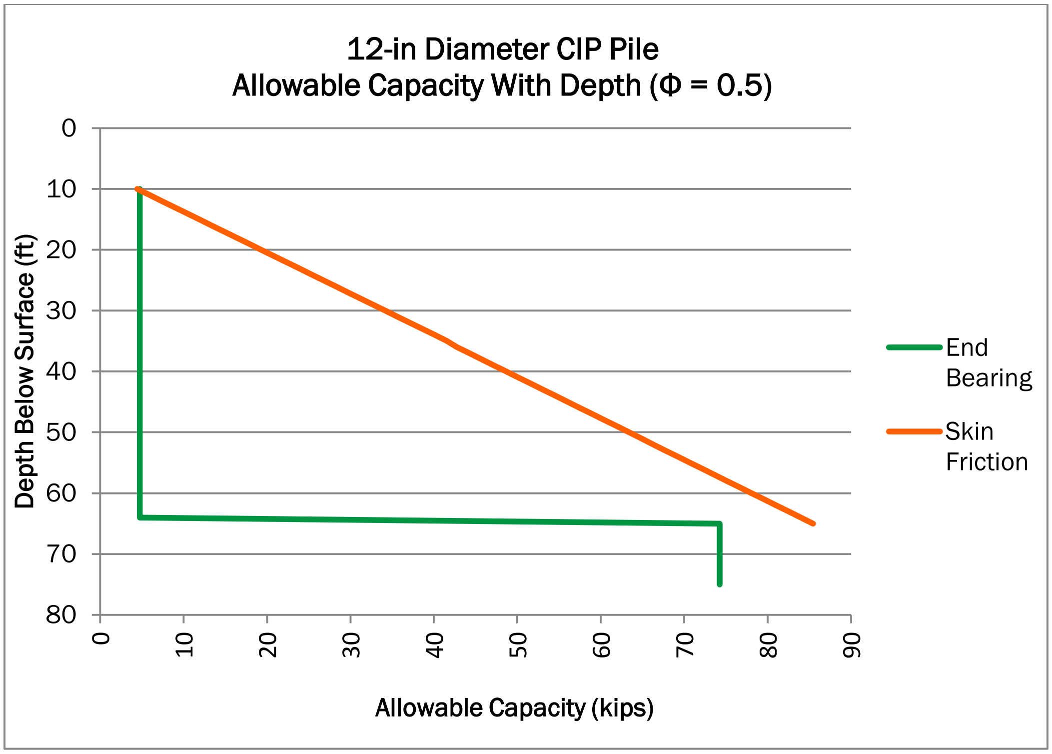
Shelby Tube samples of the clayey sand and sandy lean clay layers were collected at various depths in soil borings BR-600, BR-601 and BR-602. Selected samples were delivered to a soils testing laboratory and tested for the following:

- Atterberg Limits
- Moisture Content
- Mechanical Sieve Analysis
- Dry Density
- Tri-axial Compression Testing (CU with pore pressure measurements)

Soils from the samples were classified according to the Unified Soil Classification System using the test results. Summary reports of lab test results are given in Appendix C.

2.4 Pile Capacity Evaluation

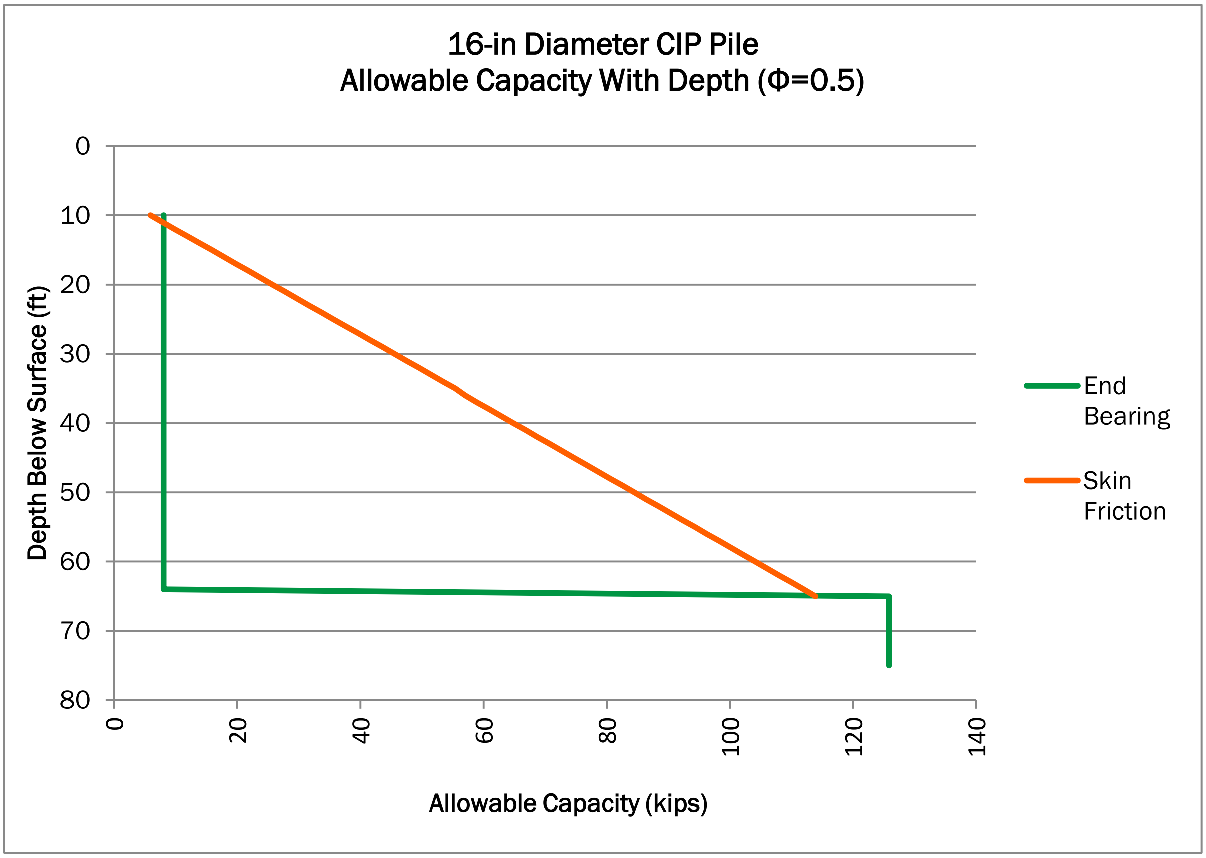
The proposed design calls for the installation of pile foundations to support the structure. An evaluation of CIP pile capacity was performed by calculating the ultimate end bearing and skin friction using information from boring log BR-600 and the soil sample data collected from the site. The top 10 feet of soil was neglected in the capacity analysis. An LRFD resistance factor (Φ =0.5) was used to determine the allowable capacity values (per MnDOT MPF12 for CIP piles). The results of the evaluation are illustrated in the chart below:



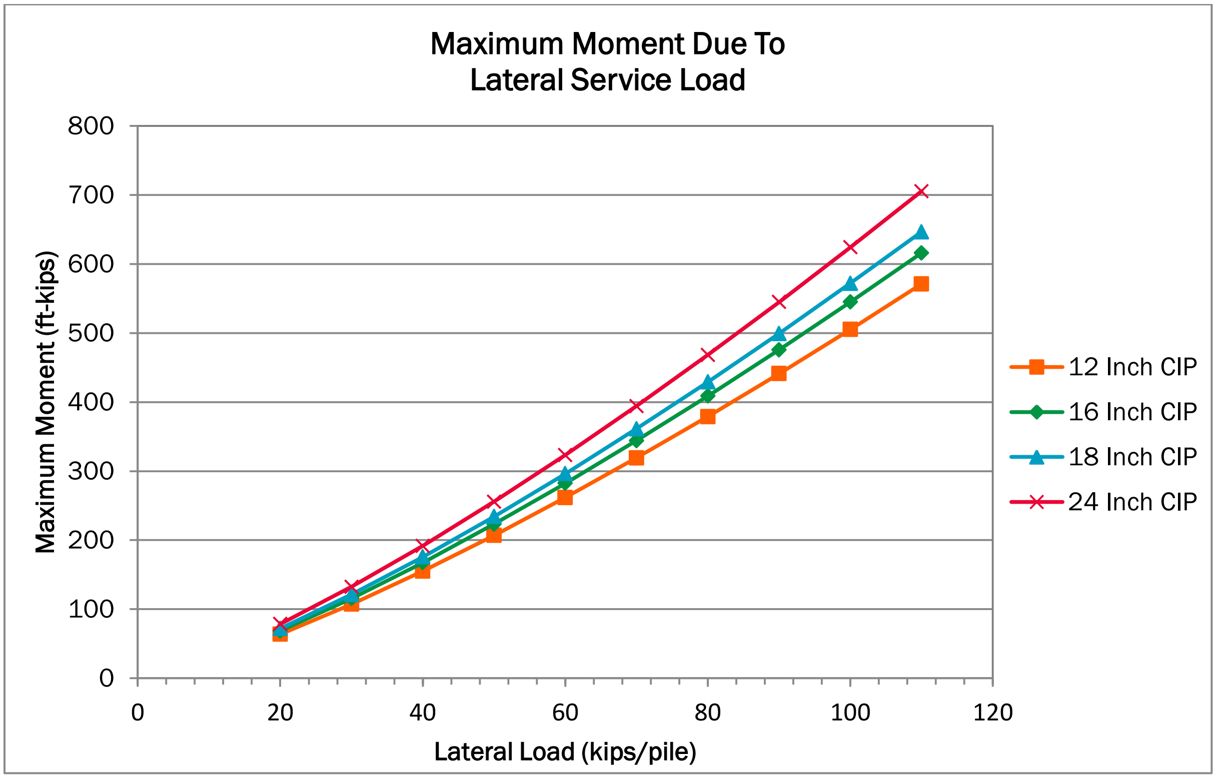
The drilled shaft capacity calculations indicate that the allowable end bearing capacity of a 12-inch diameter CIP pile is approximately 5 kips in the clayey sediments ranging from 10-65 feet below ground surface. This is likely conservative because there is typically some increase in soil strength with depth, as indicated by the N-values observed in boring logs BR-601, BR-602, and BR-603. However, there was little to no consistent increase in N-values with depth in the clayey sediments shown in BR-600. Therefore, the most conservative case was evaluated.

At approximately 65 feet, the calculated allowable end bearing capacity increases to approximately 74 kips as dense sand with gravel is encountered. Allowable skin friction increases at a rate of approximately 1.5 kips/ft over the interval from 10-65 feet below surface. According to the analysis a 12-inch CIP pile bearing on the dense sand and gravel strata at 65 feet below surface will provide an allowable vertical load capacity of approximately 155-160 kips.

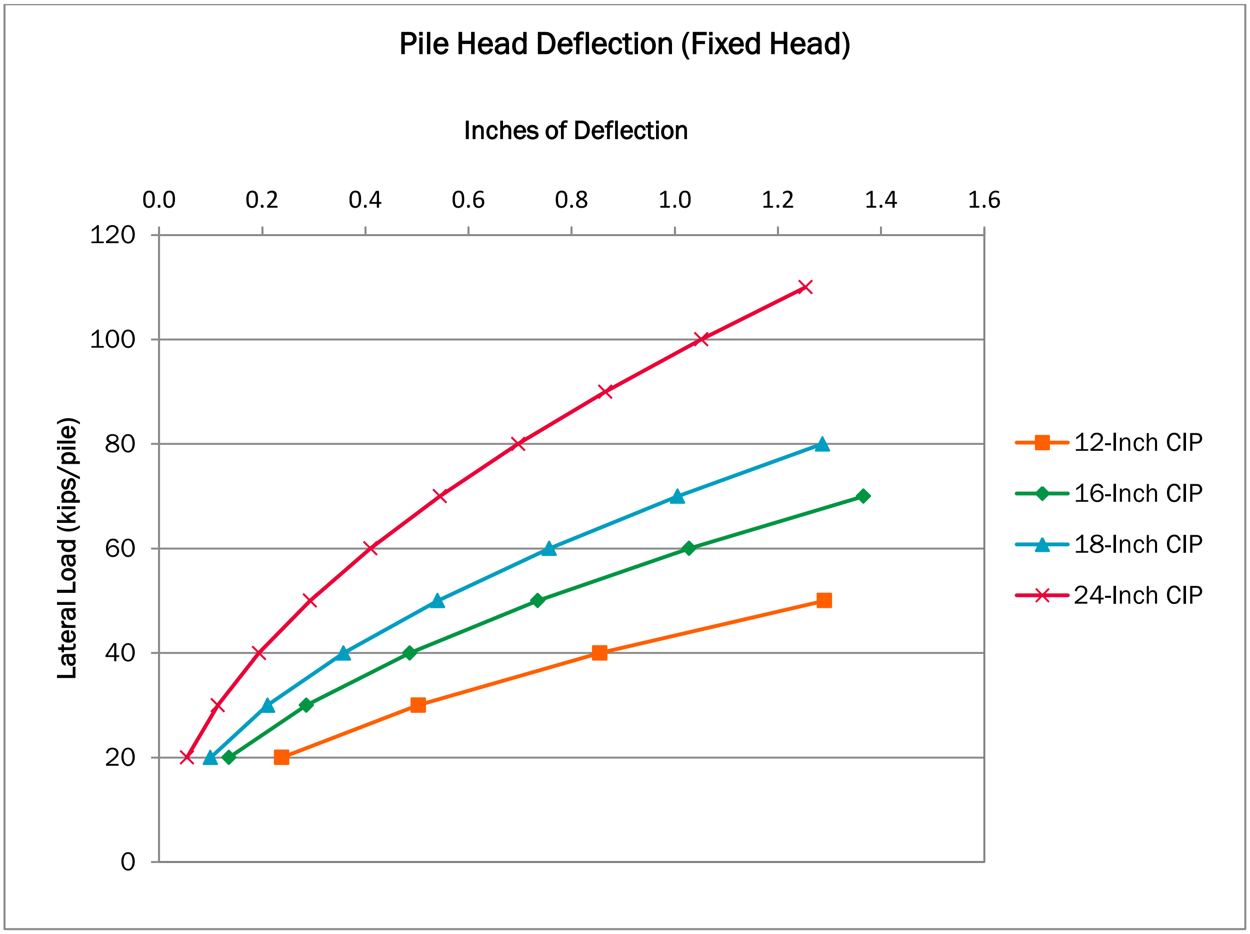
A 16-inch CIP pile was also evaluated. As shown in the chart below, a 16-inch CIP pile bearing on the dense sand and gravel strata at 65 feet below surface will provide an allowable vertical load capacity of approximately 125 kips due to end bearing resistance and 114 kips due to skin friction. The total vertical load capacity is approximately 240 kips.



An analysis of lateral capacity of the proposed piles was performed using a software program called LpileCLM2.0 Version1. The analysis included an evaluation of maximum moment and pile head deflection for various piles sizes subject to a range of loads. These analyses were performed without applying load or resistance factors and represent the ultimate expected values. The results of the maximum moment analysis are shown in the chart below:



The pile head deflection for various pile sizes was also evaluated and the results are shown in the chart below:



A resistance factor of Φ = 0.5 is recommended for use in LRFD pile design (per MnDOT MPF12 for CIP piles).

2.5 Lateral Earth Pressure

Based on the soil properties, we recommend the following coefficients of earth pressure for design purposes:

- Active: 0.42
- At Rest: 0.50
- Passive: 2.37

2.6 Shallow Foundation Bearing Capacity

If shallow foundations are needed for this area, we estimate that shallow foundations bearing on suitable or corrected soils may be designed for an allowable net bearing pressure of approximately 2500 psf. A coefficient of friction of 0.5 is estimated between the bottom of shallow foundations and suitable base grade soils.

2.7 Soil Correction

The boring logs indicated uncontrolled fill and unsuitable organic soils from the surface to a depth of approximately 14.5 feet below the proposed southeast abutment area. Fill depths of up to 5 feet below the surface where encountered in the northwest abutment area. It is recommended that all uncontrolled fill and organic containing soils be excavated and replaced with suitable controlled fill. The following minimum excavation depths are recommended:

Boring Location	Surface Elevation (ft)	Water Level Elevation (ft)	Minimum Excavation Depth (ft)	Excavation Bottom Elevation (ft)
BR-600	884.7	867.4	14.5	870.2
BR-601	884.8	880.8	14.5	870.3
BR-602	884.4	867.4	5.0	879.4
BR-603	883.3	871.3	4.5	878.8

Excavations for foundation elements such as bridge abutments should extend laterally beyond the edges of the proposed foundation. This extension distance should equal the vertical depth of fill needed to attain foundation base grade (1:1 lateral oversize). Suitable controlled fill material should consist of a free draining graded aggregate material free from frozen soil, organics, vegetation, debris, rocks larger than three inches in diameter. Fill material placed below abutment areas should be placed in maximum eight-inch lifts and compacted to a minimum of 98% Standard Proctor dry density to within three feet of the base grade elevation. The final three feet of aggregate fill should be compacted to 100% Standard Proctor dry density.

Foundation excavations in areas where soil correction has taken place should be inspected by the project geotechnical engineer or competent representative prior to the installation of aggregate base to ensure suitable material exists at the base grade elevation. Unsuitable or soft soils found at base grade elevation in soil corrected areas should be undercut a minimum of 24 inches and backfilled to base grade elevation with a well-graded aggregate material. The aggregate material should be compacted to 100% Standard Proctor dry density.

2.8 Excavation

The stability of excavation side slopes is dependent on soil strength, site geometry, moisture content, and surcharge load from excavated soils and equipment. The Contractor is solely responsible for assessing the stability and executing underground utility and project excavations using safe methods. The Contractor is also responsible for naming the "competent individual" as per Subpart P of 29 CFR 1926.6 (Federal Register - OSHA).

Excavation depths and sidewall inclinations should not exceed those specified in local, state or federal regulations. Excavations may need to be widened and sloped, or temporarily braced, to maintain or develop a safe work environment. Temporary shoring must be designed in accordance with applicable regulatory requirements.

Slopes created by placed fill material should not exceed 3H:1V. No continuous slope face should exceed 20 feet in height. Slopes required to exceed 20 feet in height should be benched a minimum of 6 feet horizontally for every 20 feet of height to reduce the continuous slope length.

2.9 Dewatering

Groundwater was encountered as shallow as 4-feet below the surface in the project area during completion of the subsurface investigation soil borings. Dewatering will likely be required to keep excavations dry. It is recommended that groundwater be lowered to a level at least 3 feet below the bottom of any planned excavation.

Depending on the depth of the planned excavation, the required dewatering effort may be substantial. A well point or installed well dewatering system may be necessary to reduce the groundwater elevation to the required level. A groundwater cutoff wall created by installation of a grout curtain or sheet pile wall may also be considered if site constraints limit the size of the excavation area. The grout curtain or sheet pile wall would likely need to extend to a depth of up to 35 feet to encounter the less permeably sandy lean clay layer beneath the site. Additional soil borings or CPT soundings should be conducted along the proposed grout curtain or sheet pile alignment to verify required depths if this method is to be used.

Dewatering activities at the site may be subject to the requirements of an MPCA approved Response Action Plan (RAP) and Construction Contingency Plan (CCP). These plans should be reviewed for any site-specific requirements before dewatering activities begin.

2.10 Trenching and Backfill

If utility trenches are needed for the project, they should be backfilled with non-organic suitable soils placed in eight-inch maximum depth loose lifts. Stockpiled site soils classified as clayey sand (SC), lean sandy clay (CL), silty sand (SM), and poorly graded sand (SP) are suitable for this use. Frozen soils will not be considered suitable for backfill. The utility trench backfill should be compacted sufficiently to minimize future settlement of green areas and areas that may receive pavement or structures. It is recommended that trench fill soils be compacted as follows:

- No less than 90% of the Standard Proctor maximum dry density to three feet below top of subgrade elevation
- No less than 95% of the Standard Proctor maximum dry density from three feet below top of subgrade elevation to top of subgrade elevation for green areas
- No less than 98% of the Standard Proctor maximum dry density from three feet below top of subgrade elevation to top of subgrade elevation for areas which may receive pavement or will provide foundation support

3.0 Spine Road Alignment

3.1 Proposed Design Understanding

It is our understanding that the Spine Road will be the main thoroughfare of the proposed development area and is anticipated to accommodate moderate to heavy vehicle loads. The alignment investigated includes a secondary roadway north of the proposed round-about known as the Thumb Road (See Figure 3). References to the Spine Road alignment in this report include both roadways.

3.2 Soil Borings

A total of 26 soil borings (SR-200 through SR-225) were completed along the Spine Road alignment in the locations shown in Figure 3. Boreholes were advanced with hollow stem auger methods using a 3.25-inch I.D. auger. Standard Penetration Testing was completed at 2.5-foot intervals to the end of each boring at a nominal depth of 20-feet below ground surface. In some cases, the soil borings could not be completed at the staked location due to accessibility issues. Off-set direction and distances are indicated on the soil boring logs where an off-set was required. Soil boring logs for these boreholes are included in Appendix D.

A review of the soil boring logs indicates that the subsurface along the alignment generally consisted of varying depths of top soil and fill material underlain by glacial, alluvial and occasional swamp deposits. Fill material was composed of poorly graded sand, silty sand, and clayey sand. Glacial deposits consisted of clayey sand, lean sandy clay, and silty clay and were generally encountered underlying the fill material in the south half of the alignment. The alluvial deposits consisted of poorly graded sand and silty sand and were generally encountered underlying the fill material in the north half of the alignment. Localized pockets of swamp deposits (soft, silty clay and peat) were also encountered along the alignment (soil borings SR-200 and SR-218).

Groundwater observations were recorded during drilling on each boring log. Groundwater was not encountered in most soil borings on the south half of the alignment in the clayey sand and lean sandy clay deposits. Groundwater was encountered in the poorly graded sand and silty sand deposits in the north half of the alignment at depths ranging from 7 to 18 feet below ground surface.

3.3 Sample Collection and Laboratory Testing

Representative samples of the clayey soil units encountered were collected for laboratory analysis. Selected samples tested for the following:

- Atterberg Limits
- Moisture Content
- Mechanical Sieve Analysis

The test results were used to confirm the field classifications of soils encountered during the investigation and to provide additional characterization of the clayey soils present at the site.

Tests performed on samples of the clayey sand (SC) and sandy lean clay (CL) soils indicated that they are low to medium plasticity soils and are considered inactive with regard to shrink-swell potential. The results for the samples tested are shown on the soil boring logs in Appendix D.

3.4 Soil Correction

The boring logs indicated varying depths of topsoil, uncontrolled fill material, and swamp deposits along the proposed road alignment. The topsoil and swamp deposits are unsuitable for road subgrade material and it is recommended that they be fully excavated and replaced with suitable controlled fill wherever they are encountered during site grading. Some swamp deposits may be too deep to be removed entirely. Any buried swamp deposits which are proposed to be left in place should be reviewed and evaluated on a case-by-case basis by the project geotechnical engineer prior to placing backfill material. In general, any swamp deposits which will remain in place along the Spine Road alignment should be separated from the top of subgrade elevation by a minimum of 5 feet of compacted controlled fill.

Much of the existing fill material is of good quality. This material may be excavated approximately three feet below subgrade elevation and re-used as controlled fill. The following minimum soil correction depths are estimated along the road alignment based on conditions encountered during the geotechnical investigation:

Boring Location	Soil Boring Surface Elevation (ft)	Groundwater Elevation Encountered (ft)	Estimated Minimum Excavation Depth (ft)	Estimated Excavation Bottom Elevation (ft)
SR-200	925.18	911	5.0	920
SR-201	951.10	--	3.0	948
SR-202	955.31	--	1.5	954
SR-203	956.01	--	3.0	953
SR-204	965.60	--	0.0	--
SR-205	951.02	--	0.5	950
SR-206	942.32	--	3.0	939
SR-207	938.51	--	3.0	935
SR-208	935.64	--	3.0	931
SR-209	911.04	--	3.0	908
SR-210	913.64	909	2.0	911
SR-211	907.43	--	3.0	904
SR-212	894.22	880	3.5	890
SR-213	890.76	879	3.0	887
SR-214	890.28	881	1.5	888
SR-215	889.66	883	3.0	886
SR-216	887.29	880	1.5	885
SR-217	886.05	879	0.0	--
SR-218	883.67	869	5.0	879
SR-219	898.15	--	3.0	895
SR-220	884.33	--	3.0	881
SR-221	892.07	877	3.0	889
SR-222	897.17	880	3.0	894
SR-223	898.48	881	3.0	895
SR-224	899.93	884	3.0	897
SR-225	898.97	--	0.0	--

3.5 Excavation

Excavations for the road subgrade soil correction should extend laterally beyond the edges of the proposed base aggregate. This extension distance should equal the vertical depth of fill needed to attain base grade (1:1 lateral oversize). Soil correction excavation areas should be inspected by the project geotechnical engineer or competent representative prior to the installation of controlled fill to ensure suitable material exists at the base of the excavation.

3.6 Dewatering

Groundwater was generally encountered in the silty sand material in the northern half of the proposed road alignment during this investigation. However, during the preliminary investigation conducted in 2007, groundwater was encountered site wide, including the area along the southern half of the proposed road alignment. A site-wide groundwater contour map was produced based on those observations and included in the *Interim Report-Preliminary Geotechnical Evaluation, (AET and Braun, 2007)* attached in Appendix A.

Based on groundwater observations from this investigation and the 2007 preliminary investigation by AET and Braun, groundwater may be encountered during excavation for soil correction activities along the Spine Road alignment. It is recommended that groundwater be lowered to a level at least 3 feet below the bottom of any planned excavation to allow dry placement of controlled fill.

Dewatering activities at the site may be subject to the requirements of an MPCA approved Response Action Plan (RAP) and Construction Contingency Plan (CCP). These plans should be reviewed for any site-specific requirements before dewatering activities begin.

3.7 Controlled Fill

Controlled fill suitable for subgrade backfill in soil correction areas along the proposed road alignment should consist of material free of high plasticity clays, silt, organics, vegetation, debris, and rocks larger than three inches in diameter. Stockpiled site soils classified as clayey sand (SC), lean sandy clay (CL), silty sand (SM), and poorly graded sand (SP) are suitable for subgrade backfill when placed and compacted as controlled fill.

The base of the excavation should be scarified and re-compacted prior to placement of controlled fill material. Controlled fill material should be moisture conditioned to within +/- 3% of optimum moisture content and placed in maximum eight-inch lifts. Controlled fill should be compacted to a minimum of 95% of the Standard Proctor dry density to within three feet of the base grade elevation. The final three feet of controlled fill placed for subgrade backfill should be compacted to a minimum of 98% of the Standard Proctor dry density.

3.8 Estimated Subgrade R-Value

Table 5-3.3a of the MNDOT Pavement Manual indicates that typical R-values for non-plastic sands and sandy loam soils range from 30-70, depending on the fines content. Assuming subgrade soils along the road alignment are corrected with suitable compacted controlled fill as described above, an average R-value of 50 may be used for design purposes. However, it is recommended that samples of proposed subgrade backfill material stockpiled during site grading operations be collected and tested in a soils laboratory to verify the final design R-value.

4.0 Rice Creek Re-Meander

4.1 Proposed Design Understanding

A portion of Rice Creek will be re-meandered to improve the alignment of the proposed Spine Road Bridge crossing the creek. The re-meander will involve excavation and placement of soil to re-locate a section of Rice Creek.

4.2 Soil Borings

A total of 4 soil borings (RC-500 through RC-503) were completed in the proposed Rice Creek re-meander alignment in the locations shown in Figure 4. Boreholes were advanced with hollow stem auger methods using a 3.25-inch I.D. auger. Standard Penetration Testing was completed at 2.5-foot intervals to the end of each boring at a nominal depth of 20-feet below ground surface. Soil borings RC-502 and RC-503 could not be completed at the staked location due to accessibility issues. These two boreholes were off-set approximately 15 feet west of the staked location. Soil boring logs for the Rice Creek boreholes are included in Appendix E.

A review of the soil boring logs indicates that the subsurface in the proposed re-meander alignment generally consisted of 1.5 to 4.5 feet of fill underlain by alluvial and swamp deposits. Fill material was composed of silty sand and gravel. The alluvial deposits consisted of silty sands with some organic material and thin peat layers. Sandy lean clay was encountered at 15 feet below the surface in borehole RC-501. Groundwater was encountered in all four of the boreholes and ranged in elevation from 875 ft. to 879 ft.

4.3 Sample Collection and Laboratory Testing

Samples of the clayey soil encountered in RC-501 and silty sand with trace organics encountered in RC-503 were collected for laboratory analysis and tested for the following:

- Atterberg Limits
- Moisture Content
- Mechanical Sieve Analysis

During testing, it was found that the silty sand material from RC-503 was non-cohesive and Atterberg Limit tests were not performed. However, the organic content of this material was determined to be 2.6%.

Tests performed on the sandy clay sample from RC-501 indicated that it is a low to medium plasticity clay soil and inactive with regard to shrink-swell potential. The results for the samples tested are shown on the soil boring logs in Appendix E.

4.4 Excavation

Soil excavated from the area should be stripped of vegetation and topsoil and sorted according to suitability for reuse. Inorganic soils suitable for controlled fill can be stockpiled for backfill material. Organic soils may be stockpiled for potential topsoil use during site restoration. Excavated areas that will receive controlled fill should be inspected by the project geotechnical engineer or competent representative prior to the installation of controlled fill to ensure suitable material exists at the base of the excavation.

4.5 Dewatering

Based on groundwater observations in the soil borings, groundwater will likely be encountered during excavation of the new creek channel. However, because the work will largely involve excavation and shaping of existing soil rather than placement of controlled fill, it is anticipated that adequate dewatering can be achieved by directing surface water away from work areas and pumping from sumps as needed.

Dewatering activities at the site may be subject to the requirements of an MPCA approved Response Action Plan (RAP) and Construction Contingency Plan (CCP). These plans should be reviewed for any site-specific requirements before dewatering activities begin.

4.6 Controlled Fill

If controlled fill is needed for streambank construction suitable embankment material should consist of mineral soil free of high plasticity clays, silt, organics, vegetation, debris, and rocks larger than three inches in diameter. Stockpiled site soils classified as clayey sand (SC), lean sandy clay (CL), silty sand (SM), and poorly graded sand (SP) are suitable for this use.

The prepared subgrade in areas to receive controlled fill should be inspected by the project geotechnical engineer or qualified representative to verify suitability of the surface to receive fill. The subgrade surface should be scarified and re-compacted prior to placement of controlled fill material. Controlled fill material should be moisture conditioned to within +/- 3% of optimum moisture content and placed in maximum eight-inch lifts. Controlled fill placed for embankment construction should be compacted to a minimum of 95% of the Standard Proctor dry density.

Material suitable for placement as general fill in non-structural areas such as constructed wetlands and green areas may consist of common excavation material free of vegetation, debris, and large rocks. This material may be placed in 1-2 foot lifts and receive quality compaction equivalent to approximately 90% Standard Proctor dry density.

5.0 Water Main

5.1 Proposed Design Understanding

A new water main is proposed to extend potable water supply to parts of the development area. The alignment was divided into two areas: A short section near a proposed water tower near the southern end of the site, and a longer alignment located in the western portion of the site.

5.2 Soil Borings

A total of 11 soil borings (WM-400 through WM-410) were completed along the proposed water main alignment in the locations shown in Figure 5. Boreholes were advanced with hollow stem auger methods using a 3.25-inch I.D. auger. Standard Penetration Testing was completed at 2.5-foot intervals to the end of each boring at a nominal depth of 15-feet below ground surface. Some of the boreholes could not be completed at the staked location due to accessibility issues. These boreholes were off-set from the staked location as indicated on the boring logs. Soil boring logs for the Rice Creek boreholes are included in Appendix F.

A review of the soil boring logs indicates that the subsurface in the proposed water main alignment generally consisted of a thin topsoil layer and 0 to 6.0 feet of fill underlain by alluvial, glacial and swamp deposits. Fill material was composed of silty sand and poorly graded sand with silt and clay. The alluvial deposits consisted of silty sands and poorly graded sand with silt. Glacial deposits included clayey sand and sandy lean clay. A 9.5 thick layer of peaty swamp deposits was encountered in borehole WM-406.

Groundwater was not encountered in soil borings WM-400 and WM-401. However, groundwater was encountered in the remaining soil borings ranging in elevation from approximately 903 ft. on the south end of the alignment to 880 ft. on the north end of the alignment.

5.3 Sample Collection and Laboratory Testing

Samples of silty sand, clayey sand, sandy lean clay, and poorly graded sand with silt were collected for laboratory analysis. Samples were tested for the following as applicable:

- Atterberg Limits
- Moisture Content
- Mechanical Sieve Analysis

Tests performed on the sandy lean clay and clayey sand materials indicated that they are low to medium plasticity clay soils and inactive with regard to shrink-swell potential. The results for the samples tested are shown on the soil boring logs in Appendix F.

5.4 Trench Excavation

It is anticipated that the water main piping will be installed using open trench methods. Excavation depths and sidewall inclinations should not exceed those specified in local, state or federal regulations. Given the sandy, non-cohesive nature of much of the soils encountered along the proposed alignment, excavations may need to be widened and sloped, or temporarily braced to maintain or develop a safe work environment. Temporary shoring must be designed in accordance with applicable regulatory requirements.

The stability of excavation side slopes is dependent on soil strength, site geometry, moisture content, and surcharge load from excavated soils and equipment. The Contractor is solely responsible for assessing the stability and executing underground utility and project excavations using safe methods. The Contractor is also responsible for naming the "competent individual" as per Subpart P of 29 CFR 1926.6 (Federal Register - OSHA).

The sandy and clayey mineral soils encountered in the borehole locations along the alignment are generally suitable for pipe support. However, organic swamp deposits were encountered in WM-406. These soils are unsuitable for pipe support and should be removed and replaced with suitable controlled fill.

5.5 Dewatering

Groundwater was encountered as shallow as 5-feet below the surface along the water main alignment during completion of the subsurface investigation soil borings. Groundwater was generally found in non-cohesive sandy soils which may become unstable when unconfined if the groundwater is not controlled to an elevation below the excavation. Dewatering along the water main trench alignment will likely require a well point dewatering system in addition to sumps located in the excavation. It is recommended that groundwater be lowered to a level at least 3 feet below the bottom of the water main excavation.

Dewatering activities at the site may be subject to the requirements of an MPCA approved Response Action Plan (RAP) and Construction Contingency Plan (CCP). These plans should be reviewed for any site-specific requirements before dewatering activities begin.

5.6 Pipe Bedding

The existing silty sand and poorly graded sand with silt soils are suitable for pipe support and no additional pipe bedding is necessary where these soils are encountered at the pipe invert. However, in areas where clayey soils are encountered at the proposed pipe invert, granular bedding material should be used. Stockpiled site soils composed of silty sand (SM), and poorly graded sand (SP) are suitable for this use.

5.7 Backfill and Compaction

The water main trench should be backfilled with non-organic suitable soils placed in eight-inch maximum depth loose lifts. Stockpiled site soils classified as clayey sand (SC), lean sandy clay (CL), silty sand (SM), and poorly graded sand (SP) are suitable for this use. The backfill should be compacted sufficiently to minimize future settlement of green areas and areas that may receive pavement or structures. It is recommended that trench fill soils be compacted as follows:

- No less than 90% of the Standard Proctor maximum dry density to three feet below top of subgrade elevation
- No less than 95% of the Standard Proctor maximum dry density from three feet below top of subgrade elevation to top of subgrade elevation for green areas

- No less than 98% of the Standard Proctor maximum dry density from three feet below top of subgrade elevation to top of subgrade elevation for areas which may receive pavement or will provide foundation support

6.0 Natural Resources Corridor

6.1 Proposed Design Understanding

The natural resources corridor will consist of constructed wetlands, storm water infiltration features, and green areas. The area will be accessible by various walking trails constructed throughout the site. Ponds in the corridor will be constructed with approximately 2 feet of compacted controlled fill, overlain by 1 foot of un-compacted topsoil material.

6.2 Soil Borings

A total of 34 soil borings (NR-100 through NR-133) were completed in the proposed natural resources corridor in the locations shown in Figure 6. Boreholes were advanced with hollow stem auger methods using a 3.25-inch I.D. auger. Standard Penetration Testing was completed at 2.5-foot intervals to the end of each boring at a nominal depth of 20-feet below ground surface. Some soil borings could not be completed at the staked location due to accessibility issues. These boreholes were off-set as indicated on the soil boring logs. Soil boring logs for the natural resources corridor boreholes are included in Appendix G.

A review of the soil boring logs indicates that the subsurface in the natural resources corridor is similar to other areas of the site. Soils encountered generally consisted of thin layers of topsoil and/or fill ranging from 0 to 7 feet in thickness. Fill material was composed of silty sand, silty sand with gravel, and clayey sand. The fill, where present, was underlain by alluvial, glacial and swamp deposits. The alluvial deposits consisted of silty sands, poorly graded sand with gravel. Glacial sediments encountered included clayey sand, sandy lean clay, and lean clay with sand. Swamp deposits of peat material were encountered in various locations across the corridor.

Groundwater was encountered in approximately half of the boreholes in the natural resources corridor and ranged in elevation from 875 ft. to 879 ft.

6.3 Sample Collection and Laboratory Testing

Representative samples of clayey and sandy soil units encountered were collected for laboratory analysis and testing. However, upon review of the soil boring logs, it was determined that the soils encountered were already well characterized by testing performed on samples collected in other development areas surrounding the natural resources corridor. As a result, no testing was requested on samples collected from this area. Samples have been retained for future testing, should it be deemed necessary.

6.4 Infiltration Testing

Infiltration testing was performed to assess the infiltration capacity of soils in the natural resources corridor. Infiltration testing was performed according to the *Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer (ASTM D3385)*.

Tests were performed in 9 locations as shown on Figure 6. The near surface soils in this area generally consist of poorly graded sand with silt and silty sand fill as shown in the water main soil boring logs (WM-402 to WM-410), spine road boring logs (SR-211 to SR-

217), and natural resources corridor boring log NR-133. However, there are areas of near surface clayey soils present as well, as indicated in natural resources corridor boring log NR-132. Graphs of the infiltration rate test results are included in Appendix H.

6.5 Soil Correction

It is recommended that topsoil be removed from the pond construction site prior to site grading. Once the site has been graded to the proposed subgrade elevation, suitable soils found at the base of the excavation may be lightly scarified and recompacted in preparation to receive controlled fill. If soft soil or organic soils such as topsoil or swamp deposits are encountered at subgrade elevation, it is recommended that they be over-excavated a minimum of one foot and replaced with suitable controlled fill.

6.6 Excavation

It is anticipated that soil excavation activities in this area will consist of grading and shaping to create the proposed wetlands and surface water ponds. Soil excavation operations should include stripping of vegetation and topsoil and sorting excess materials according to suitability for reuse. Inorganic soils suitable for controlled fill can be stockpiled for backfill material. Organic soils may be stockpiled for potential topsoil use during site restoration. Excavated areas that will receive controlled fill should be inspected by the project geotechnical engineer or competent representative prior to the installation of controlled fill to ensure suitable material exists at the base of the excavation.

6.7 Dewatering

Based on groundwater observations in the soil borings, groundwater will likely be encountered during excavation of the pond areas. However, because the work will largely involve excavation and shaping of existing soil rather than placement of controlled fill, it is anticipated that adequate dewatering can be achieved by directing surface water away from work areas and pumping from sumps as needed.

Dewatering activities at the site may be subject to the requirements of an MPCA approved Response Action Plan (RAP) and Construction Contingency Plan (CCP). These plans should be reviewed for any site-specific requirements before dewatering activities begin.

6.8 Controlled Fill

Suitable controlled fill material should consist of mineral soil free of high plasticity clays, silt, organics, vegetation, debris, and rocks larger than three inches in diameter. Stockpiled site soils classified as clayey sand (SC), lean sandy clay (CL), silty sand (SM), and poorly graded sand (SP) are suitable for this use.

The prepared subgrade in areas to receive controlled fill should be inspected by the project geotechnical engineer or qualified representative to verify suitability of the surface to receive fill. The subgrade surface should be scarified and re-compacted prior to placement of controlled fill material. Controlled fill material should be moisture conditioned to within +/- 3% of optimum moisture content and placed in maximum eight-inch lifts. Controlled fill placed for embankment construction should be compacted to a minimum of 95% of the Standard Proctor dry density.

Material suitable for placement as general fill in non-structural areas may consist of common excavation material from the site that is free of vegetation, debris, and large rocks. This material may be placed in 1-2 foot lifts and receive quality compaction equivalent to approximately 90% Standard Proctor dry density.

Slopes created by placed fill material should not exceed 3H:1V. No continuous slope face should exceed 20 feet in height. Slopes required to exceed 20 feet in height should be benched a minimum of 6 feet horizontally for every 20 feet of height to reduce the continuous slope length.

7.0 Regional Trail

7.1 Proposed Design Understanding

A walking trail named the Rice Creek Regional Trail will be constructed throughout the development area for recreational use. The trail is anticipated to be asphalt paved and constructed to support light duty maintenance vehicles.

A total of 10 soil borings (TR-300 through TR-309) were completed along the eastern portion of the proposed trail alignment in the locations shown in Figure 7. Other portions of the trail wind through areas which have already been characterized by soil borings. Boreholes were advanced with hollow stem auger methods using a 3.25-inch I.D. auger. Standard Penetration Testing was completed at 2.5-foot intervals to the end of each boring at a nominal depth of 12-feet below ground surface. Soil borings TR-303, TR-304, and TR-305 could not be completed at the staked location due to accessibility issues. These three boreholes were off-set as indicated on the soil boring log. Soil boring logs for the Regional Trail boreholes are included in Appendix I.

A review of the soil boring logs indicates that the subsurface along the trail alignment was similar to other areas of the site and generally consisted of varying depths of topsoil and/or fill underlain by poorly graded sand with silt/gravel, silty sand, clayey sand, and sandy lean clay. A 1-foot thick swamp deposit of peat was encountered at 7 feet below the surface in soil boring TR-308.

Groundwater was encountered in boring locations TR-305 to TR-309. The elevation of groundwater encountered ranged from approximately 927 ft. in TR-305 to 885 ft. in TR-308.

7.2 Sample Collection and Laboratory Testing

Representative soil samples were collected for laboratory analysis and tested for the following as applicable:

- Atterberg Limits
- Moisture Content
- Mechanical Sieve Analysis

Tests performed on clayey sand samples indicated that it is a low to medium plasticity clay soil and inactive with regard to shrink-swell potential. The results for the samples tested are shown on the soil boring logs in Appendix I.

7.3 Soil Correction

The boring logs indicated varying depths of topsoil, uncontrolled fill material, and swamp deposits along the proposed trail alignment. The topsoil and swamp deposits are unsuitable for trail subgrade material. However, the existing fill material, native sands and sandy clays are generally suitable for trail subgrade material if they are placed in a controlled manner.

It is recommended that the soil beneath the trail construction area be excavated to an elevation of 1 foot below the top of base grade elevation. Suitable soils found at the base of the excavation may be scarified and recompacted in preparation to receive controlled fill. If soft soil or organic soils such as topsoil or swamp deposits are encountered at the base of the excavation, it is recommended that they be over-excavated a minimum of two feet below top of subgrade elevation and replaced with suitable controlled fill.

7.4 Excavation

Excavations for the trail subgrade soil correction should extend laterally beyond the edges of the proposed base aggregate. This extension distance should equal the vertical depth of fill needed to attain base grade (1:1 lateral oversize). Soil correction excavation areas should be inspected by the project geotechnical engineer or competent representative prior to the installation of controlled fill to ensure suitable material exists at the base of the excavation.

7.5 Dewatering

Groundwater was generally encountered 4 to 12 feet below the ground surface in the soil borings along the trail alignment. If groundwater is encountered during trail construction, it is recommended that it be lowered to a level at least 1 foot below the bottom of the trail excavation to allow dry placement of controlled fill. This can likely be accomplished by pumping from sumps placed as needed along the alignment.

Dewatering activities at the site may be subject to the requirements of an MPCA approved Response Action Plan (RAP) and Construction Contingency Plan (CCP). These plans should be reviewed for any site-specific requirements before dewatering activities begin.

7.6 Controlled Fill

Controlled fill for trail subgrade will consist of mineral soil free of high plasticity clays, silt, organics, vegetation, debris, and rocks larger than three inches in diameter. Stockpiled site soils classified as clayey sand (SC), lean sandy clay (CL), silty sand (SM), and poorly graded sand (SP) are suitable for this use to within 1 foot of top of subgrade elevation. Stockpiled soil classified as silty sand (SM), and poorly graded sand (SP) are suitable use as controlled from 1 foot below top of subgrade elevation to top of subgrade elevation.

The prepared subgrade in areas to receive controlled fill should be inspected by the project geotechnical engineer or qualified representative to verify suitability of the surface to receive fill. The subgrade surface should be scarified and re-compacted prior to placement of controlled fill material. Controlled fill material should be moisture conditioned to within +/- 3% of optimum moisture content and placed in maximum eight-inch lifts. Controlled fill should be compacted to a minimum of 98% of the Standard Proctor dry density.

7.7 Estimated Subgrade R-Value

As discussed in Section 3.8, Table 5-3.3a of the MNDOT Pavement Manual indicates that typical R-values for non-plastic sands and sandy loam soils range from 30-70, depending on the fines content. Assuming subgrade soils along the trail alignment are corrected with suitable compacted controlled fill, an average R-value of 50 may be used for design purposes. However, it is recommended that samples of proposed subgrade backfill material stockpiled during site grading operations be collected and tested in a soils laboratory to verify the final design R-value.

8.0 Town and Creek Development Area

8.1 Proposed Design Understanding

The Town and Creek Development Area will include potential residential and commercial development. Construction in this area is expected to include shallow foundations, utilities, and green spaces.

8.2 Soil Borings

A total of 26 soil borings (DE-800 through DE-825) were completed in the proposed development area in the locations shown in Figure 8. Boreholes were advanced with hollow stem auger methods using a 3.25-inch I.D. auger. Standard Penetration Testing was completed at 2.5-foot intervals to the end of each boring at a nominal depth of 20-feet below ground surface. Soil borings DE-802, DE-803 and DE-804 could not be completed at the staked location due to accessibility issues. These boreholes were off-set approximately as shown on each boring log. Soil boring logs for the Town and Creek Development Area boreholes are included in Appendix J.

A review of the soil boring logs indicates that the subsurface in the development area was similar to other areas of the site and generally consisted of varying depths of topsoil and/or fill material underlain by poorly graded sand with silt/gravel, silty sand, clayey sand, and sandy lean clay. Swamp deposits of peat were encountered in soil borings DE-818, DE-819, and DE-820.

8.3 Soil Correction

The boring logs indicated varying depths of topsoil, uncontrolled fill material, and organic swamp deposits in the development area. The topsoil and swamp deposits are unsuitable for foundation subgrade material. However, the existing fill material, native sands and sandy clays are generally suitable for subgrade material if they are recompacted and placed as controlled fill.

The subgrade requirements for site specific foundations should be evaluated once their type and location are known. However, in general, it is recommended that the soil beneath shallow foundations be excavated to an elevation of 3 foot below the top of base grade elevation. Suitable soils found at the base of the excavation may be scarified and recompacted in preparation to receive controlled fill. If soft soil or organic soils such as topsoil or swamp deposits are encountered at the base of the excavation, it is recommended that they be removed and replaced with suitable controlled fill. Swamp deposits were encountered in soil borings DE-818, DE-819, and DE-820.

Suitable controlled fill material should consist of a free draining graded aggregate material free from frozen soil, organics, vegetation, debris, rocks larger than three inches in diameter. Stockpiled site soils classified as clayey sand (SC), lean sandy clay (CL), silty sand (SM), and poorly graded sand (SP) are suitable for this use from the bottom of excavations to within 3 feet of top of subgrade elevation. Stockpiled soil classified as silty sand (SM), and poorly graded sand (SP) are suitable use as controlled from 3 feet below top of subgrade elevation to top of subgrade elevation.

The prepared subgrade in areas to receive controlled fill should be inspected by the project geotechnical engineer or qualified representative to verify suitability of the surface to receive fill. Controlled fill material should be moisture conditioned to within +/- 3% of optimum moisture content and placed in maximum eight-inch lifts. Controlled fill in foundation areas should be compacted to a minimum of 95% Standard Proctor dry density to within three feet of the base grade elevation. The final three feet of fill should be compacted to a minimum of 98% Standard Proctor dry density.

8.4 Bearing Capacity

We estimate that shallow foundations bearing on suitable or corrected soils may be designed for an allowable net bearing pressure of approximately 2500 psf. A coefficient of friction of 0.5 is estimated between the bottom of shallow foundations and suitable base grade soils.

8.5 Excavation

The stability of excavation side slopes is dependent on soil strength, site geometry, moisture content, and surcharge load from excavated soils and equipment. The Contractor is solely responsible for assessing the stability and executing underground utility and project excavations using safe methods. The Contractor is also responsible for naming the “competent individual” as per Subpart P of 29 CFR 1926.6 (Federal Register - OSHA).

Excavations for foundation elements should extend laterally beyond the edges of the proposed foundation. This extension distance should equal the vertical depth of fill needed to attain foundation base grade (1:1 lateral oversize). Excavation depths and sidewall inclinations should not exceed those specified in local, state or federal regulations. Excavations may need to be widened and sloped, or temporarily braced, to maintain or develop a safe work environment. Temporary shoring must be designed in accordance with applicable regulatory requirements.

8.6 Dewatering

Groundwater was encountered as shallow as 3.5 feet and as deep as 14.5 feet below the surface in the project area during completion of the subsurface investigation soil borings. Groundwater found in non-cohesive sandy soils may become unstable when unconfined if the groundwater is not controlled to an elevation below the excavation. It is recommended that groundwater be lowered to a level at least 3 feet below the bottom of foundation excavations to allow proper subgrade preparation.

Dewatering, if necessary, will likely be achieved pumping from sumps in the excavation area. However, if deep excavations are required in saturated areas, a well-point dewatering system may be required. Each excavation should be evaluated individually to assess the dewatering methods needed.

Dewatering activities at the site may be subject to the requirements of an MPCA approved Response Action Plan (RAP) and Construction Contingency Plan (CCP). These plans should be reviewed for any site-specific requirements before dewatering activities begin.

8.7 Trenching and Backfill

Utility trenches excavated in the development area should be constructed following the excavation stability recommendations in Section 8.5 and the dewatering recommendations in Section 8.6. Where sewer and water services will be installed, the existing silty sand and poorly graded sand with silt soils are suitable for pipe support and no additional pipe bedding is necessary where these soils are encountered at the pipe invert. However, in areas where clayey soils are encountered at the proposed pipe invert, granular bedding material should be used. Stockpiled site soils composed of silty sand (SM), and poorly graded sand (SP) are suitable for this use.

Trenches should be backfilled with non-organic suitable soils placed in eight-inch maximum depth loose lifts. Stockpiled site soils classified as clayey sand (SC), lean sandy clay (CL), silty sand (SM), and poorly graded sand (SP) are suitable for this use. Frozen soils will not be considered suitable for backfill. The utility trench backfill should be compacted sufficiently to minimize future settlement of green areas and areas that may receive pavement or structures. It is recommended that trench fill soils be compacted as follows:

- No less than 90% of the Standard Proctor maximum dry density to three feet below top of subgrade elevation
- No less than 95% of the Standard Proctor maximum dry density from three feet below top of subgrade elevation to top of subgrade elevation for green areas
- No less than 98% of the Standard Proctor maximum dry density from three feet below top of subgrade elevation to top of subgrade elevation for areas which may receive pavement or will provide foundation support

9.0 Pedestrian Bridges

9.1 Proposed Design Understanding

It is our understanding that the proposed pedestrian bridges will be constructed using a truss design. The bridge abutments will be supported by a cast-in-place (CIP) pile foundation. The anticipated foundation design loads were provided by the structural design engineer (Kimley-Horn) as follows:

- Factored Design Load (Q): 50 tons/pile
- Resistance Factor (Φ): 0.5
- Required Nominal Pile Bearing Resistance (R_n): 100 tons/pile

9.2 Soil Borings

Four soil borings (PB-1, PB-2, PB-3 and PB-4) were completed; one at each of the proposed pedestrian bridge abutment locations shown in Figure 9. Boreholes were advanced with hollow stem auger methods using a 3.25-inch I.D. auger. Standard Penetration Testing was completed at 2.5-foot intervals to a depth of 20-feet below ground surface, then at 5-foot intervals until the end of each boring. All four soil borings were completed to a depth of approximately 75 feet below ground surface.

9.2.1 South Pedestrian Bridge Subsurface Conditions

Borehole locations PB-1 and PB-2 represent the abutment locations of the south pedestrian bridge. In general, similar materials were encountered in both abutment locations. Top soil was encountered from the surface to depths ranging from 1.1 to 1.3 feet below ground surface. Fill material composed of medium dense sand with silt was encountered below the top soil to approximately 7-8 feet below ground surface. Non-cohesive native sand with silt and silty sand was encountered below the fill material to a depth of approximately 24.5 feet in PB-1 and 19.5 feet in PB-2 and ranged in density from medium dense to loose. This material was underlain by medium-stiff sandy lean clay in PB-1 and medium dense clayey sand in PB-2 to the bottom of each borehole.

Groundwater was encountered at a depth of approximately 17 feet below the surface in PB-1 and approximately 12 feet below the surface in PB-2.

9.2.2 North Pedestrian Bridge Subsurface Conditions

Borehole locations PB-3 and PB-4 represent the abutment locations of the north pedestrian bridge. In general, similar materials were encountered in both abutment locations. Top soil was encountered from the surface to depths ranging from 0.7 to 1.4 feet below ground surface. Fill material composed of medium dense sand with silt was encountered below the top soil to approximately 4.5 feet below ground surface in location PB-3. Fill material was not encountered in PB-4, but clayey sand was encountered below the topsoil to a depth of 7 feet below ground surface. Non-cohesive native sand with silt and silty sand was encountered in both locations below the fill material and clayey sand to a depth of approximately 24.5 feet in PB-3 and 14.5 feet in PB-4 and ranged in density from medium dense to loose. This material was underlain by stiff sandy lean clay and lean clay to a depth of approximately 69.5 feet in both boring locations. Poorly graded sand with silt and clayey sand/sandy lean clay was encountered below the clay to the end of the borehole in each location.

Groundwater was encountered at a depth of approximately 12 feet below the surface in PB-3 and approximately 7 feet below the surface in PB-4. The pedestrian bridge boreholes were grouted to the surface upon completion. A complete description of materials encountered at each abutment location is given on the boring logs included in Appendix K.

9.3 Sample Collection and Laboratory Testing

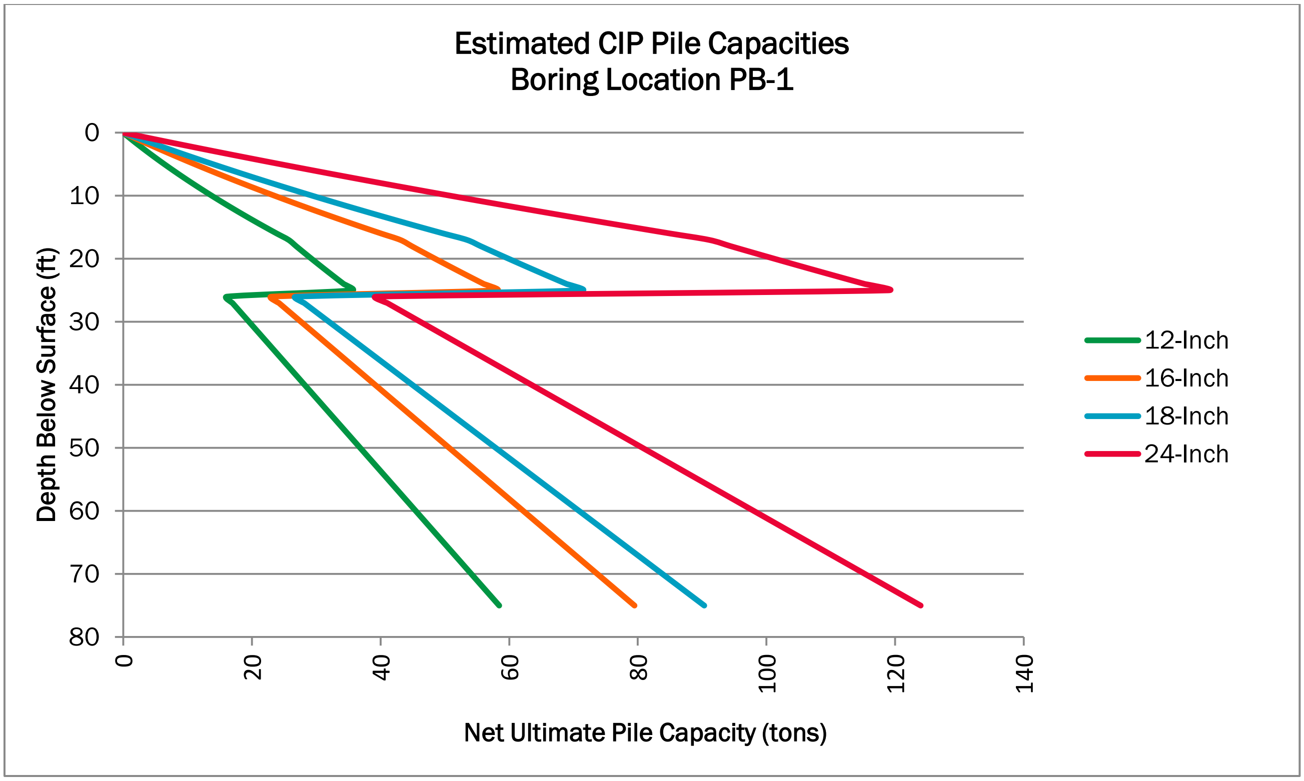
Shelby Tube samples of the clayey sand and sandy lean clay layers were collected at various depths in the pedestrian bridge soil borings. Selected samples were delivered to a soils testing laboratory and tested for the following:

- Atterberg Limits
- Moisture Content
- Mechanical Sieve Analysis
- Tri-axial Compression Testing (CU with pore pressure measurements)

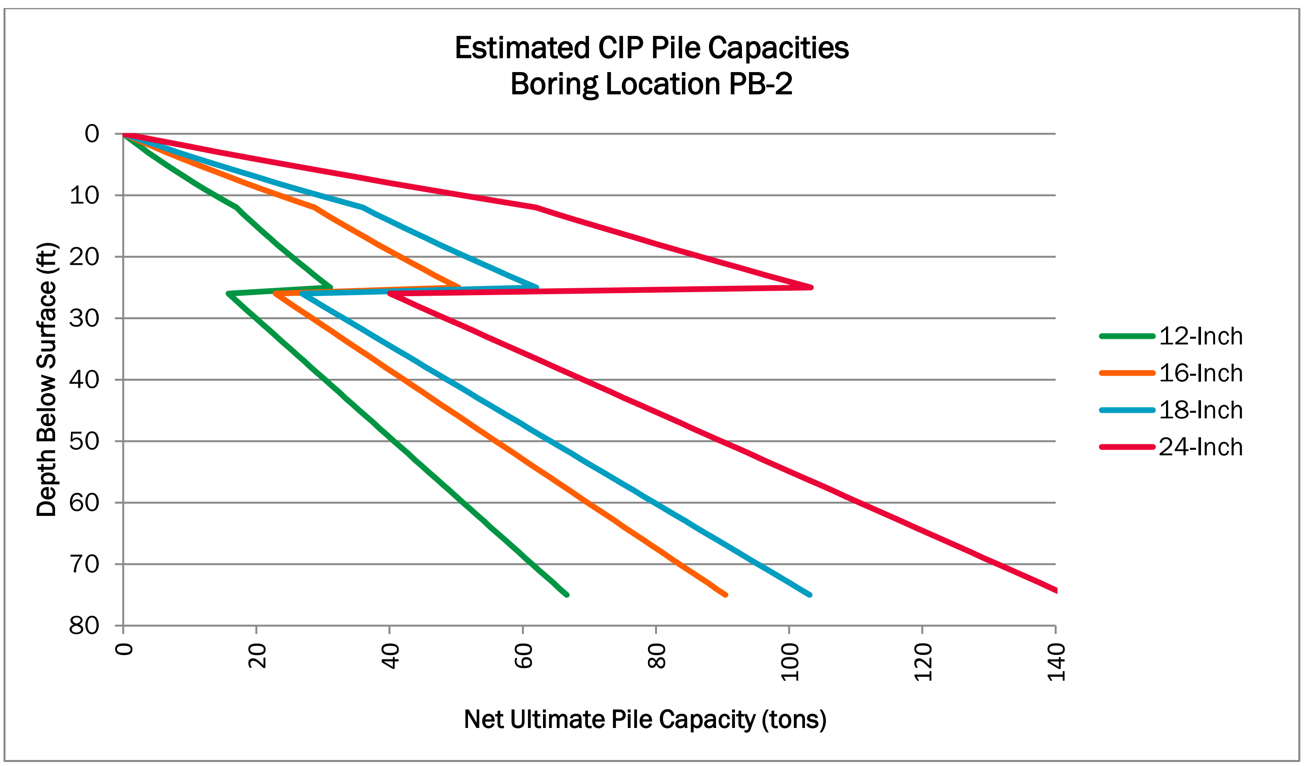
Soils from the samples were classified according to the Unified Soil Classification System using the test results. Summary reports of lab test results are given in Appendix L.

9.4 Pile Capacity Evaluation

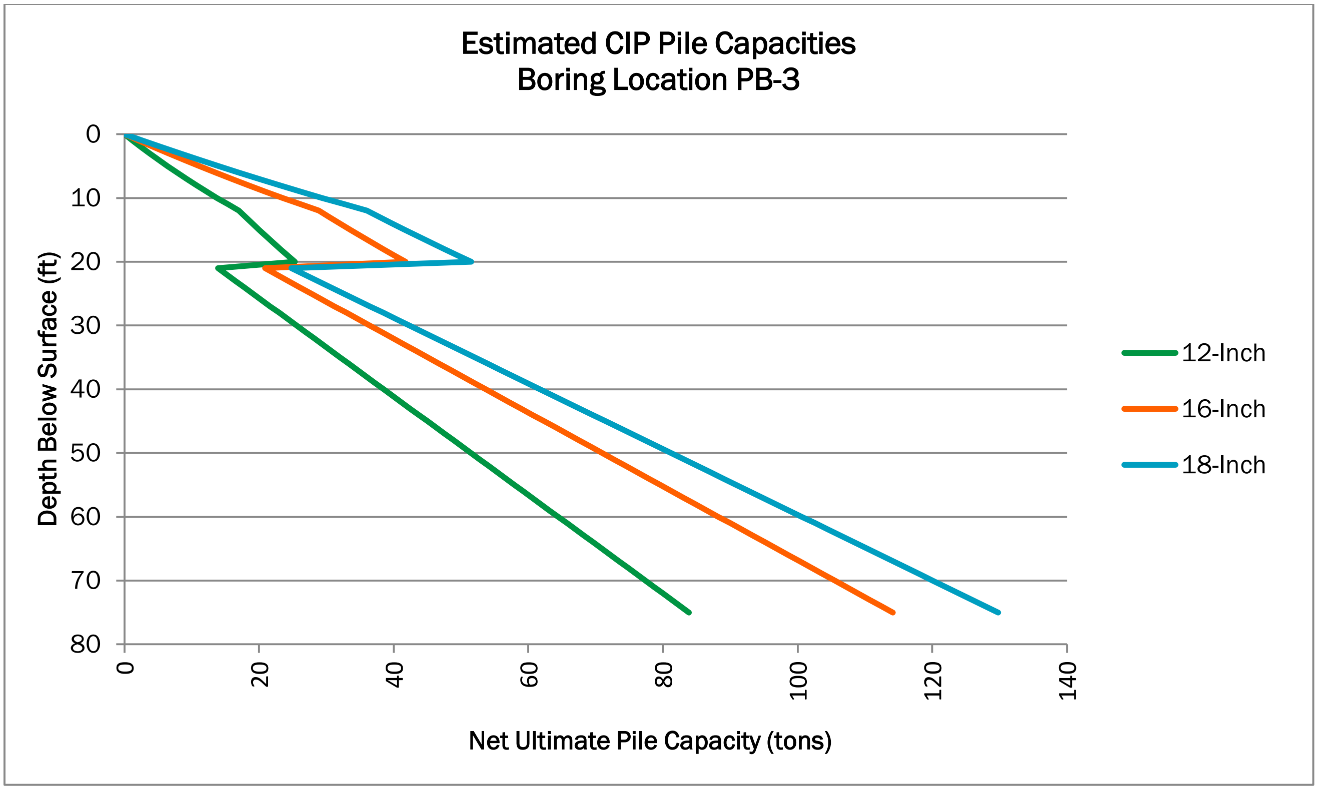
The proposed pedestrian bridge design calls for the installation of pile foundations to support the abutments. An evaluation of CIP pile capacity for different diameter piles was performed for each abutment location. The net ultimate pile capacities were determined by calculating the net ultimate end bearing and skin friction using information from the soil boring logs for each abutment location and the soil sample data collected from the site. The results of the evaluation are illustrated for each abutment location in the charts below:

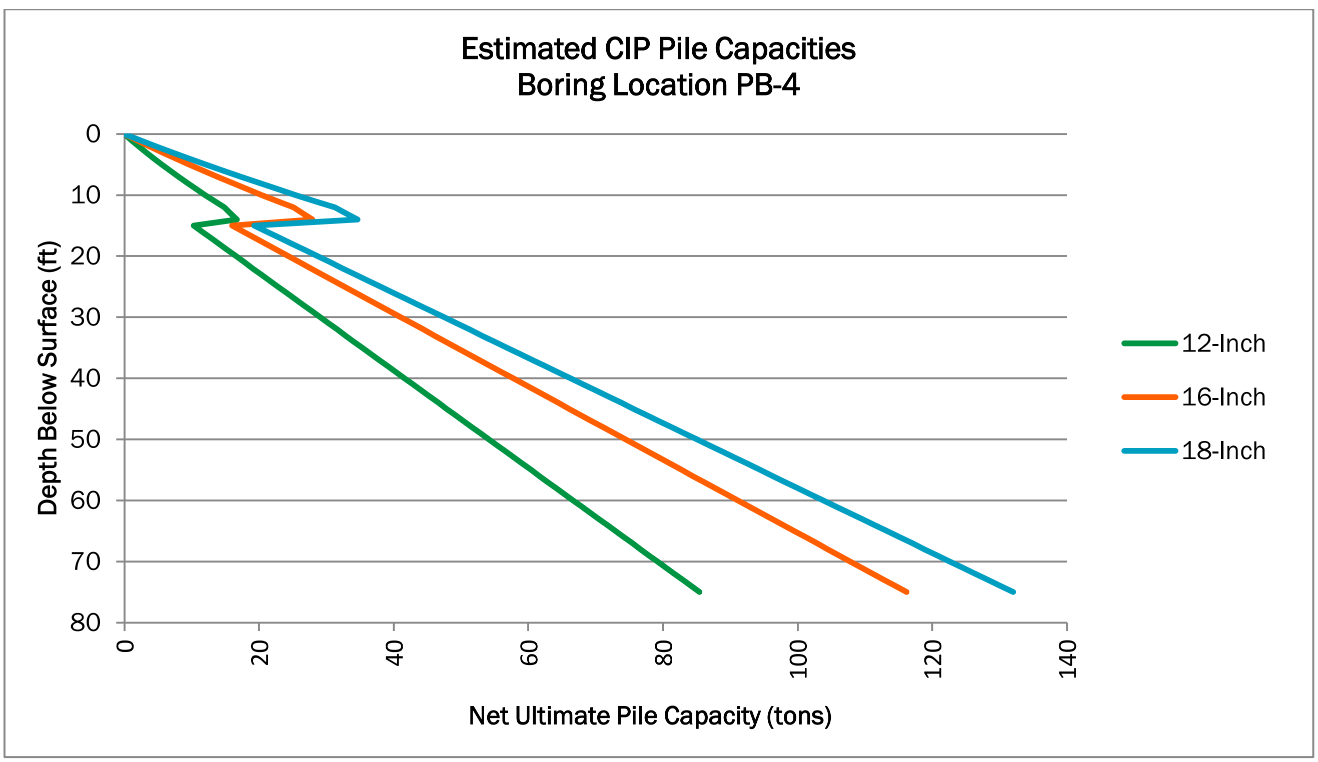


Estimated CIP Pile Capacities
Boring Location PB-1
Depth Below Surface (ft)
0
10
20
30
40
50
60
70
80
0
20
40
60
80
100
120
140
Net Ultimate Pile Capacity (tons)
12-Inch
16-Inch
18-Inch
24-Inch



Estimated CIP Pile Capacities
Boring Location PB-2
Depth Below Surface (ft)
0
10
20
30
40
50
60
70
80
0
20
40
60
80
100
120
140
Net Ultimate Pile Capacity (tons)
12-Inch
16-Inch
18-Inch
24-Inch


Estimated CIP Pile Capacities
Boring Location PB-3
Depth Below Surface (ft)
0
10
20
30
40
50
60
70
80
0
20
40
60
80
100
120
140
Net Ultimate Pile Capacity (tons)
12-Inch
16-Inch
18-Inch


Estimated CIP Pile Capacities
Boring Location PB-4
Depth Below Surface (ft)
0
10
20
30
40
50
60
70
80
0
20
40
60
80
100
120
140
Net Ultimate Pile Capacity (tons)
12-Inch
16-Inch
18-Inch

The capacity calculations indicate that CIP piles constructed to a minimum embedment depth of approximately 75 feet can provide the required nominal bearing resistance of 100 tons/pile. However, the minimum pile diameter required to meet the specified nominal pile bearing resistance varies by abutment location. The following minimum CIP pile diameters are recommended for each abutment location:

- PB-1: 24-Inch
- PB-2: 18-Inch
- PB-3: 16-Inch
- PB-4: 16-Inch

9.5 Soil Correction

The boring logs indicated uncontrolled fill underlain by soft silty sand soils with traces of organic content to depths of up to 19.5 feet below ground surface in the proposed pedestrian bridge abutment areas. The depths of these soils are summarized for each bridge abutment soil boring location as follows:

Boring Location	Surface Elevation (ft)	Water Level Elevation (ft)	Soft Soil Depth (ft)	Bottom Elevation (ft)
PB-1	883.2	886.2	19.5	863.7
PB-2	883.7	871.7	19.5	864.2
PB-3	887.5	865.5	17.0	860.5
PB-4	878.0	871.0	12.0	866.0

It is recommended that these soft soils be excavated a minimum of three feet below the proposed bridge abutment base elevation and replaced with a free-draining aggregate fill. The base of the excavation should be covered with a geotextile filter fabric equivalent to MNDOT Type 4 non-woven geotextile. Free draining aggregate material such as 1" clear crushed rock may be placed and compacted over the geotextile to serve as a working surface during abutment pile construction. The aggregate material can be re-compacted and remain in place following pile installation and serve as the base material for the final bridge abutment.

Excavations for foundation elements such as bridge abutments should extend laterally beyond the edges of the proposed foundation. This extension distance should equal the vertical depth of fill needed to attain foundation base grade (1:1 lateral oversize). Fill material placed below abutment areas should be placed in maximum eight-inch lifts and compacted to a minimum of 98% Standard Proctor dry density to within three feet of the base grade elevation. The final three feet of aggregate fill should be compacted to 100% Standard Proctor dry density.

Foundation excavations in areas where soil correction has taken place should be inspected by the project geotechnical engineer or competent representative prior to the installation of aggregate base to ensure suitable material exists at the base grade elevation. Unsuitable or soft soils found at base grade elevation in soil corrected areas should be undercut a minimum of 24 inches and backfilled to base grade elevation with a well-graded aggregate material. The aggregate material should be compacted to 100% Standard Proctor dry density.

9.6 Lateral Earth Pressure

The pedestrian bridge foundation design is not governed by lateral loads and no design lateral loads were specified. Therefore, lateral pile capacity was not analyzed for the pedestrian bridges. However, lateral earth pressure should be considered as part of the bridge abutment designs. It is recommended that the bridge abutments be backfilled using suitable site soils classified as SP, SP-SM, and SM according to the USCS soil classification system. Based on this backfill composition, we recommend the following coefficients of earth pressure for design purposes:

- Active: 0.42
- At Rest: 0.50
- Passive: 2.37

9.7 Shallow Foundation Bearing Capacity

If shallow foundations are included in the design for the pedestrian bridges area, we estimate that shallow foundations bearing on suitable or corrected soils may be designed for an allowable net bearing pressure of approximately 2000 psf. A coefficient of friction of 0.5 is estimated between the bottom of shallow foundations and suitable base grade soils.

9.8 Excavation

The stability of excavation side slopes is dependent on soil strength, site geometry, moisture content, and surcharge load from excavated soils and equipment. The Contractor is solely responsible for assessing the stability and executing underground utility and project excavations using safe methods. The Contractor is also responsible for naming the "competent individual" as per Subpart P of 29 CFR 1926.6 (Federal Register - OSHA).

Excavation depths and sidewall inclinations should not exceed those specified in local, state or federal regulations. Excavations may need to be widened and sloped, or temporarily braced, to maintain or develop a safe work environment. Temporary shoring must be designed in accordance with applicable regulatory requirements.

Slopes created by placed fill material should not exceed 3H:1V. No continuous slope face should exceed 20 feet in height. Slopes required to exceed 20 feet in height should be benched a minimum of 6 feet horizontally for every 20 feet of height to reduce the continuous slope length.

9.9 Dewatering

Groundwater was encountered at depths of 12-17 feet below the surface in the southern pedestrian bridge soil borings (PB-1 and PB-2) and 7-12 feet below ground surface in the north pedestrian bridge soil borings (PB-3 and PB-4). Dewatering may be required to keep the abutment construction areas dry. It is recommended that groundwater be lowered to a level at least 3 feet below the bottom of any planned excavation.

It is anticipated that the abutment excavations may be dewatered by pumping from sumps placed in the excavation area. If sumps are unable to control groundwater levels in the excavation, a well point or installed well dewatering system may be necessary to reduce the groundwater elevation to the required level.

Dewatering activities at the site may be subject to the requirements of an MPCA approved Response Action Plan (RAP) and Construction Contingency Plan (CCP). These plans should be reviewed for any site-specific requirements before dewatering activities begin.

9.10 Trenching and Backfill

If utility trenches are needed for this project area, they should be backfilled with non-organic suitable soils placed in eight-inch maximum depth loose lifts. Stockpiled site soils classified as clayey sand (SC), lean sandy clay (CL), silty sand (SM), and poorly graded sand (SP) are suitable for this use. Frozen soils will not be considered suitable for backfill. The utility trench backfill should be compacted sufficiently to minimize future settlement of green areas and areas that may receive pavement or structures. It is recommended that trench fill soils be compacted as follows:

- No less than 90% of the Standard Proctor maximum dry density to three feet below top of subgrade elevation
- No less than 95% of the Standard Proctor maximum dry density from three feet below top of subgrade elevation to top of subgrade elevation for green areas
- No less than 98% of the Standard Proctor maximum dry density from three feet below top of subgrade elevation to top of subgrade elevation for areas which may receive pavement or will provide foundation support

10.0 Sheet Pile Surface Water Control Structure

10.1 Proposed Design Understanding

A surface water pond outlet structure is proposed near the south pedestrian bridge location. The structure will be a low-head weir constructed using sheet pile and will retain a maximum water depth of 4 feet on the pond side.

10.2 Soil Borings

Subsurface information for the sheet pile weir location was taken from south pedestrian bridge soil borings PB-1 and PB-2. The subsurface conditions encountered in these soil borings is summarized in Section 9.2.1. A complete description of the materials encountered is given on the boring logs included in Appendix K.

10.3 Sheet Pile Evaluation

The sheet pile weir was evaluated to estimate the minimum sheet pile embedment depth, maximum bending moment, and required section modulus. The minimum embedment depth is used to determine the total length of sheet pile needed along the alignment to retain the design head and provide rotational stability. The maximum bending moment is used to determine the minimum sheet pile cross sectional rigidity needed, indicated by the section modulus. As a result of the evaluation, the following values are recommended:

- Minimum Embedment Depth: 12 ft
- Maximum Bending Moment: 3500 lb-ft/ft
- Minimum Section Modulus: 2 in^3/ft of wall

These values are un-factored and appropriate design factors should be applied for use in design specification.

11.0 Retaining Wall

11.1 Proposed Design Understanding

A retaining wall is proposed for construction adjacent to the proposed Regional Trail and new Rice Creek alignment. The proposed retaining wall design entails a large gravity block system supported by a shallow foundation. The wall will extend over 200 feet in length with a maximum height of approximately 20 feet. Based on preliminary drawings of the site layout, the front slope will be approximately flat and the back slope behind the top of the wall will be approximately 10 degrees above horizontal. The Spine Road will be located approximately 20 feet behind the top of the retaining wall.

11.2 Soil Borings

Three soil borings (RW-1, RW-2, RW-3) were completed along the proposed retaining wall alignment as shown on Figure 9. Boreholes were advanced with hollow stem auger methods using a 3.25-inch I.D. auger. Standard Penetration Testing was completed at 2.5-foot intervals to a termination depth of approximately 30-feet below ground surface.

In soil boring location RW-1, top soil was encountered from the surface to 1.3 feet below ground surface. Fill material composed of medium dense sand with silt was encountered below the top soil to approximately 7 feet below ground surface. Native sand with silt was encountered below the fill material to a depth of approximately 19.5 feet with a more clayey interval encountered from 12-14.5 feet. This material was underlain by medium-stiff sandy lean clay to the bottom of the borehole. Groundwater was encountered at a depth of approximately 4.5 feet below the surface in RW-1.

The subsurface conditions were similar in both RW-2 and RW-3. Topsoil was encountered from 1-1.5 feet below ground surface, followed by fill material composed of poorly graded sand with silt to depths ranging from 4.5-7 feet. The fill material was underlain by native clayey sand to the bottom of the borehole in both locations. Groundwater was encountered at 4.5 feet below ground surface in both in both RW-2 and RW-3. A complete description of materials encountered at each retaining wall soil boring location is given on the boring logs included in Appendix M.

11.3 Sample Collection and Laboratory Testing

Shelby Tube samples of the clayey sand and sandy lean clay layers were collected at various depths in the pedestrian bridge soil borings. Selected samples were delivered to a soils testing laboratory and tested for the following:

- Atterberg Limits
- Moisture Content
- Mechanical Sieve Analysis
- Tri-axial Compression Testing (CU with pore pressure measurements)

Soils from the samples were classified according to the Unified Soil Classification System using the test results. Based on the SPT and laboratory testing, the following soil engineering properties are recommended for retaining wall design:

Material Type	γ_{moist} (pcf)	Φ' (Degrees)	c' (psf)	C_u (psf)	LL	PL	PI	P_{200}	w
Existing Silty Sand Fill Material	115	30	0	--	--	--	--	--	--
Native Sand with Silt (SP-SM)	110	29	0	--	--	--	--	7	25
Native Clayey Sand (SC)	125	28	0	900	27	12	15	42	17
Sandy Lean Clay (CL)	125	28	0	900	27	12	15	42	17

The design parameters shown in the table above for soils classified as clayey sand (SC) and sandy lean clay (CL) are the same. The grain size analysis for these materials indicated that they were both clayey sand (SC). However, the triaxial compression strength test result report for the same soil sample identifies the material as sandy lean clay (CL). For geotechnical purposes, they are the same material. Summary reports of the lab test results are given in Appendix N.

11.4 Retaining Wall Evaluation

Based on the proposed design described in Section 11.1, the retained soil will consist of a combination of silty sand fill material and native sand with silt (SP-SM). The retaining wall foundation material will consist of clayey sand (SC)/sandy lean clay (CL). A preliminary evaluation of the large gravity block wall design was completed using the segmental retaining wall design program, SRWall, version 3.22, produced by the National Concrete Masonry Association. The program indicated potential poor retaining wall performance with regard to overturning, sliding, bearing capacity, and facing stability.

We recommend that a reinforced segmental retaining wall design be considered. An analysis was performed in which a high tensile strength geogrid reinforcement layer was installed between each course of block in the gravity wall design. The analysis indicated that the geogrid reinforcement can provide acceptable factors of safety with respect to overturning, sliding, and bearing capacity. The analysis also indicated that geogrid reinforcement would reduce lateral earth pressures resulting in acceptable factors of safety for facing stability. The final design should also be evaluated for global stability. This analysis was not performed as part of this report due to the unknowns regarding final wall design. Wenck can complete this analysis on the final retaining wall design cross section.

Based on our analysis, we recommend the following general retaining wall design features:

- Geogrid reinforcement consisting of high tensile strength polyester geogrid material (Such as Miragrid 7XT or equal) with a maximum vertical spacing of 2 feet.
- The minimum reinforcement length will depend on the wall design configuration. However, our analysis indicated a minimum geogrid length of 13.5 feet per layer.
- An infill soil composed of USCS classification poorly graded sand (SP) or similar well-draining uniform granular material should be used in the geogrid reinforced area.
- Infill soils should be compacted in maximum 8-inch lifts to no less than 98% of the Standard Proctor maximum dry density

- The design should include a minimum 8-inch block leveling pad composed of compacted MNDOT Class 5 aggregate with MNDOT Type V geotextile underlayment; or a concrete footing with a minimum 6 inches Class 5 aggregate base. The leveling pad should be sized according to the allowable subgrade bearing capacity discussed in Section 11.6.
- A minimum 12" drainage layer of ¾-inch clear crushed aggregate should be placed between the back of the retaining wall block and the reinforced soil. The drainage layer should include a 4-inch perforated PVC drainage pipe with outlets spaced a maximum of 50 feet apart.

11.5 Soil Correction

The boring logs indicated varying depths of topsoil underlain by uncontrolled fill material and natural soil deposits. Topsoil should be removed from the retaining wall construction area. The silty sand fill soil beneath shallow foundations should be excavated a minimum of 3 feet below the retaining wall base grade elevation. Suitable soils found at the base of the excavation may be scarified and re-compacted in preparation to receive controlled fill. If soft soil is encountered at the base of the excavation, it is recommended that they be over-excavated an additional two feet and replaced with suitable controlled fill. If organic soils or swamp deposits are encountered at the base of the excavation, they should be removed entirely and replaced with suitable controlled fill.

Suitable controlled fill material should consist of a free draining graded aggregate material free from frozen soil, organics, vegetation, debris, rocks larger than three inches in diameter. Stockpiled site soils classified as clayey sand (SC), lean sandy clay (CL), silty sand (SM), and poorly graded sand (SP) are suitable for this use from the bottom of excavations to within 3 feet of top of subgrade elevation. Stockpiled soil classified as silty sand (SM), and poorly graded sand (SP) are suitable use as controlled from 3 feet below top of subgrade elevation to top of subgrade elevation.

The prepared subgrade in areas to receive controlled fill should be inspected by the project geotechnical engineer or qualified representative to verify suitability of the surface to receive fill. Controlled fill material should be moisture conditioned to within +/- 3% of optimum moisture content and placed in maximum eight-inch lifts. Controlled fill in foundation areas should be compacted to a minimum of 95% Standard Proctor dry density to within three feet of the base grade elevation. The final three feet of fill should be compacted to a minimum of 98% Standard Proctor dry density.

11.6 Bearing Capacity

We estimate that shallow foundations bearing on suitable or corrected soils may be designed for an allowable net bearing pressure of approximately 1500 psf in the retaining wall area. A coefficient of friction of 0.5 is estimated between the bottom of shallow foundations and suitable base grade soils.

11.7 Lateral Earth Pressure

Based on the soil properties, we recommend the following coefficients of earth pressure for design purposes:

- Active: 0.42

- At Rest: 0.50
- Passive: 2.37

11.8 Excavation

The stability of excavation side slopes is dependent on soil strength, site geometry, moisture content, and surcharge load from excavated soils and equipment. The Contractor is solely responsible for assessing the stability and executing underground utility and project excavations using safe methods. The Contractor is also responsible for naming the "competent individual" as per Subpart P of 29 CFR 1926.6 (Federal Register - OSHA).

Excavations for foundation elements should extend laterally beyond the edges of the proposed foundation. This extension distance should equal the vertical depth of fill needed to attain foundation base grade (1:1 lateral oversize). Excavation depths and sidewall inclinations should not exceed those specified in local, state or federal regulations. Excavations may need to be widened and sloped, or temporarily braced, to maintain or develop a safe work environment. Temporary shoring must be designed in accordance with applicable regulatory requirements.

11.9 Dewatering

Groundwater was encountered at approximately 4.5 feet below ground surface in the retaining wall project area during completion of the subsurface investigation soil borings. Excavations in non-cohesive sandy soils may become unstable if the groundwater is not controlled to an elevation below the excavation. It is recommended that groundwater be lowered to a level at least 3 feet below the bottom of foundation excavations to allow proper subgrade preparation.

Dewatering, if necessary, will likely be achieved pumping from sumps placed in the excavation area. Dewatering activities at the site may be subject to the requirements of an MPCA approved Response Action Plan (RAP) and Construction Contingency Plan (CCP). These plans should be reviewed for any site-specific requirements before dewatering activities begin.

11.10 Trenching and Backfill

Trenches should be backfilled with non-organic suitable soils placed in eight-inch maximum depth loose lifts. Stockpiled site soils classified as clayey sand (SC), lean sandy clay (CL), silty sand (SM), and poorly graded sand (SP) are suitable for this use. Frozen soils will not be considered suitable for backfill. The utility trench backfill should be compacted sufficiently to minimize future settlement of green areas and areas that may receive pavement or structures. It is recommended that trench fill soils be compacted as follows:

- No less than 90% of the Standard Proctor maximum dry density to three feet below top of subgrade elevation
- No less than 95% of the Standard Proctor maximum dry density from three feet below top of subgrade elevation to top of subgrade elevation for green areas
- No less than 98% of the Standard Proctor maximum dry density from three feet below top of subgrade elevation to top of subgrade elevation for areas which may receive pavement or will provide foundation support

12.0 Qualifications

12.1 Variations in Subsurface Conditions

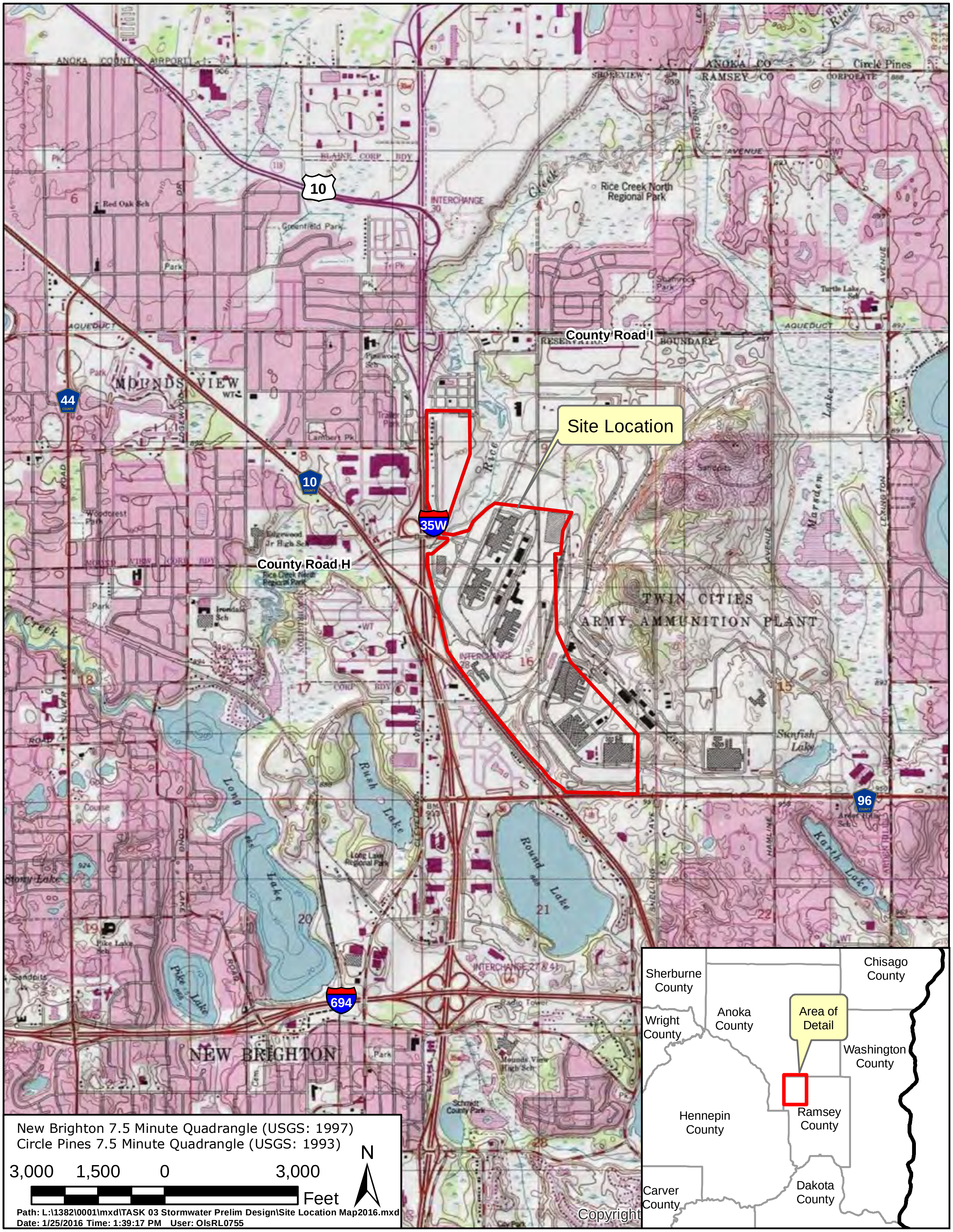
Our evaluation, analysis and recommendations were developed from a limited amount of subsurface information. It is not standard practice to collect soil samples continuously with depth, and therefore the interface between soil layers and their estimated thicknesses are inferred. Soil layer boundaries may also be gradual transitions, and can be expected to vary in depth, elevation and thickness away from the exploration locations.

Variations in subsurface conditions, including the location and presence of groundwater, determined between exploration locations may not be revealed until additional exploration work is completed, or construction commences. If any such variations are revealed, they should be evaluated by the project geotechnical engineer.

12.2 Standard of Care

In performing its services, Wenck Associates, Inc. used a degree of care and skill ordinarily exercised by similar professionals working under similar circumstances in the same general geographic area and at the same time. No warranty, express or implied, is made.

Figures



TCAAP REDEVELOPMENT SITE

Site Location Map

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Figure 1

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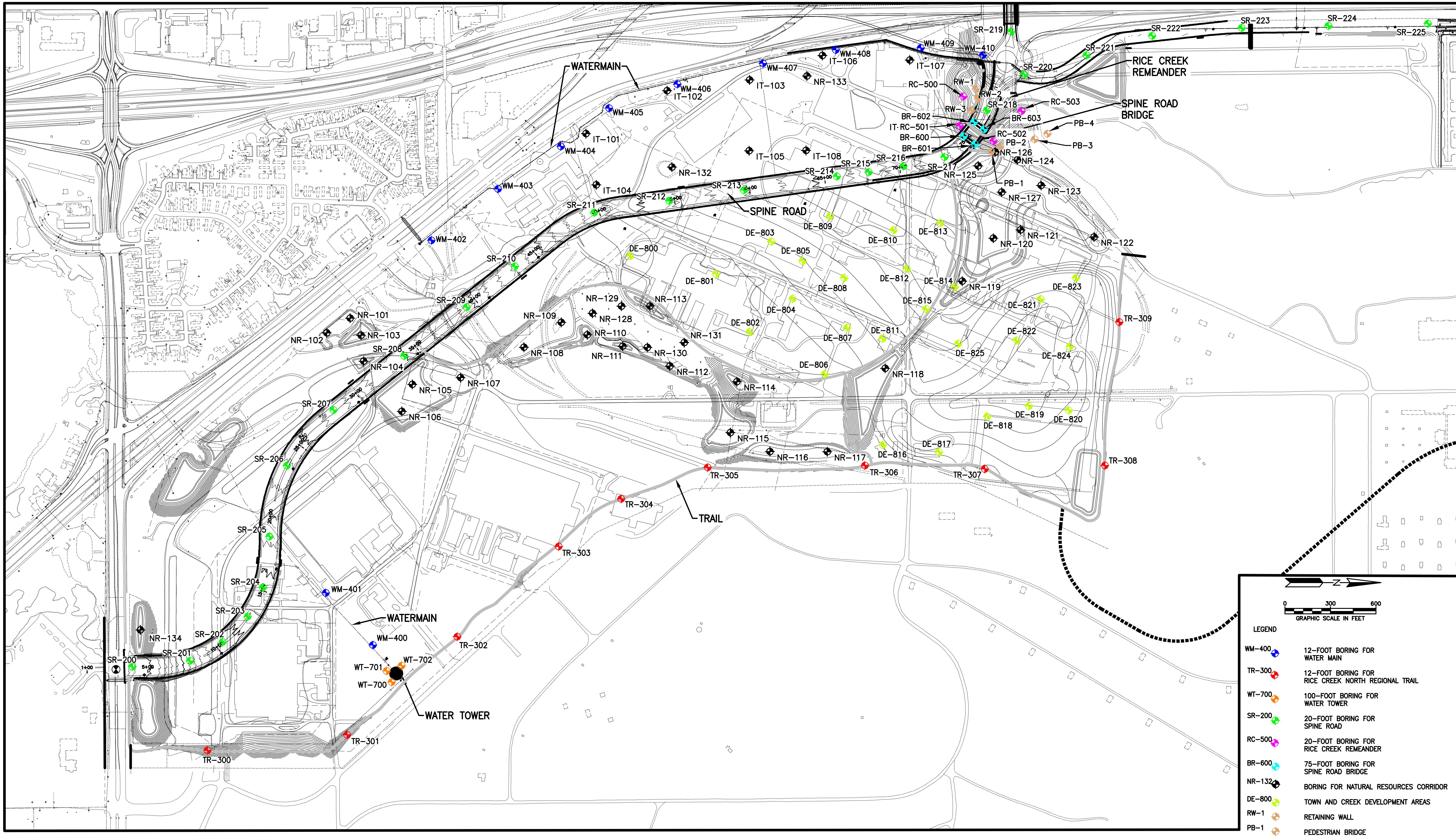

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PROJECT TITLE
GEOTECH REPORT

SHEET TITLE
SITE LAYOUT

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PROJECT NO.	SHEET NO.	REV NO.
1382-0001	FIGURE 1B	0

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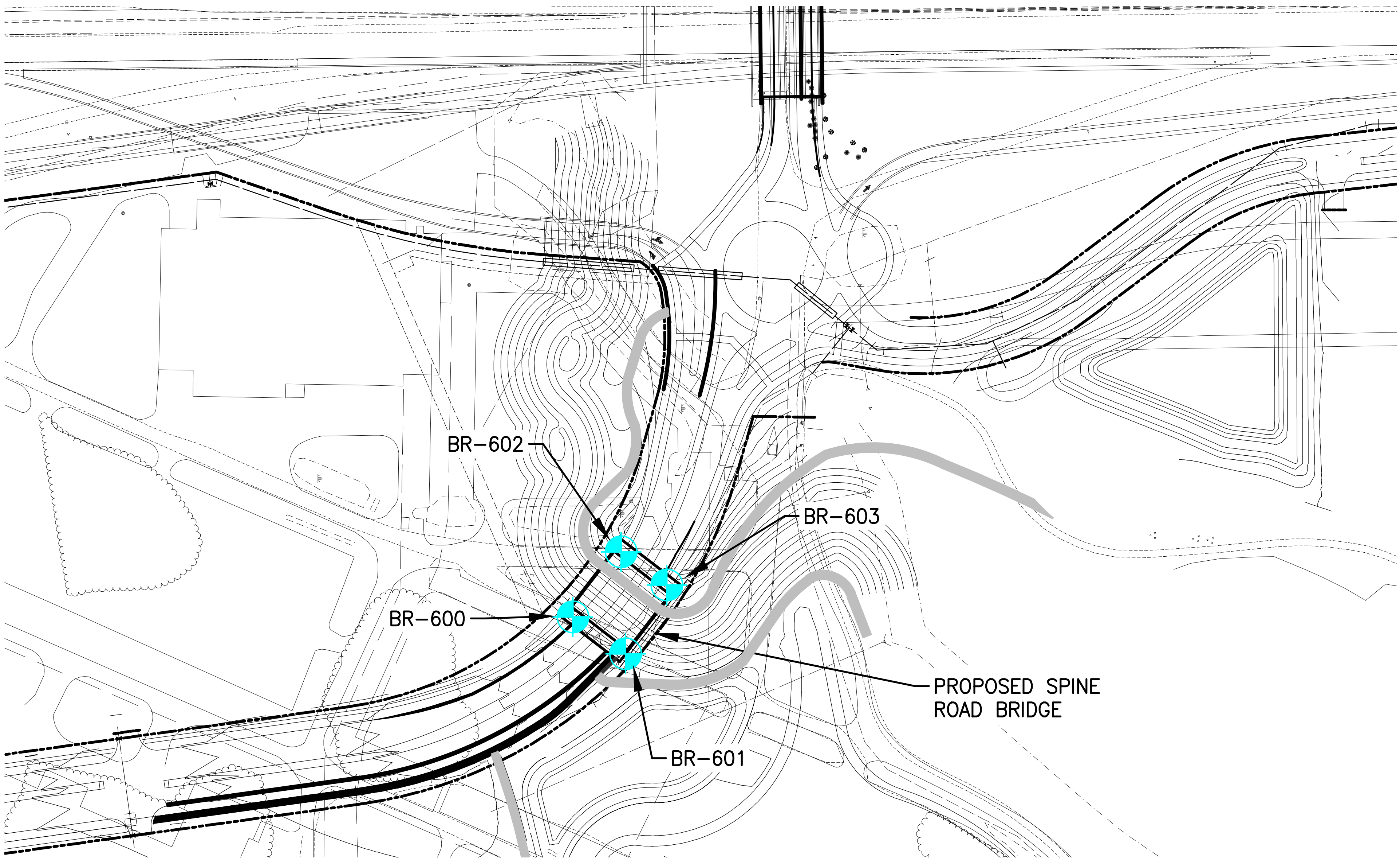

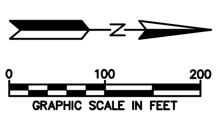

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PROJECT: BORING LOCATIONS

SHEET TITLE: SPINE ROAD BRIDGE BORING LOCATIONS

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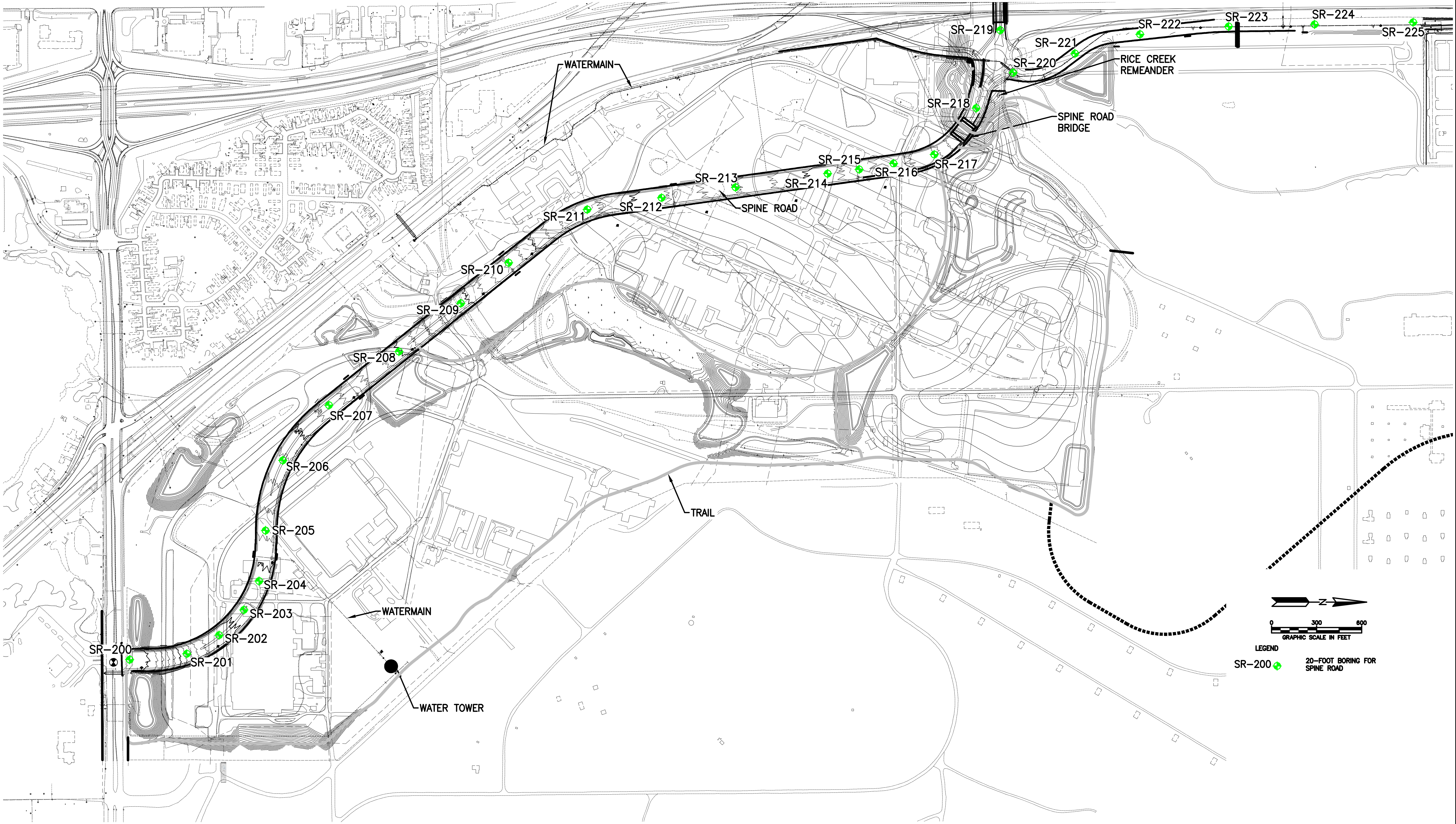
				JVB	JW	JW	JAN. 2016	1382-0001	FIGURE 2	A

SCALE: AS NOTED



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SHEET TITLE
SPINE ROAD BORING LOCATION MAP

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1382-0001	FIGURE 3	1

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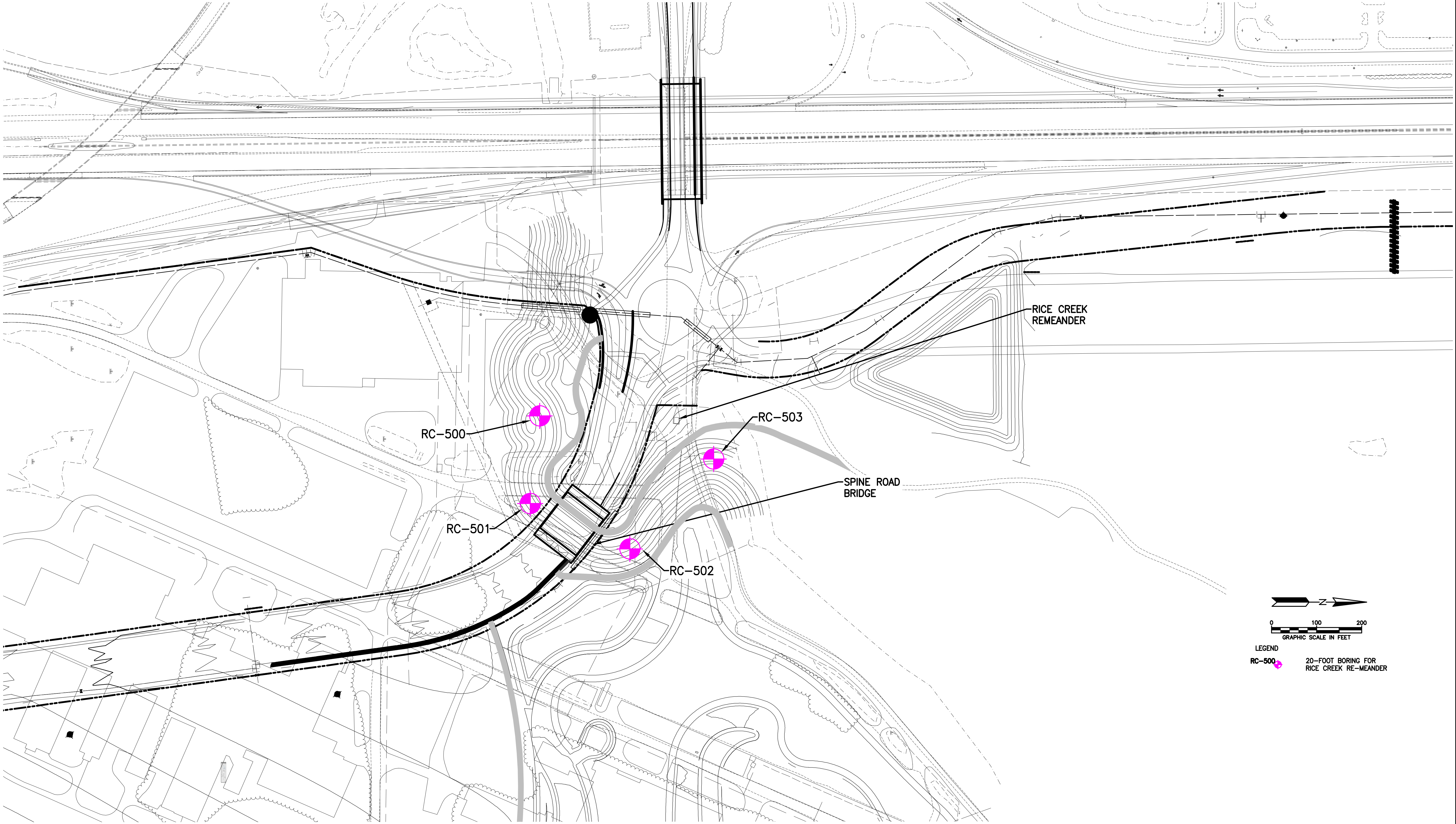

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SHEET TITLE
RICE CREEK RE–MEANDER
BOREHOLE LOCATION MAP

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PROJECT NO.	SHEET NO.	REV NO.
1382–0001	FIGURE 4	1

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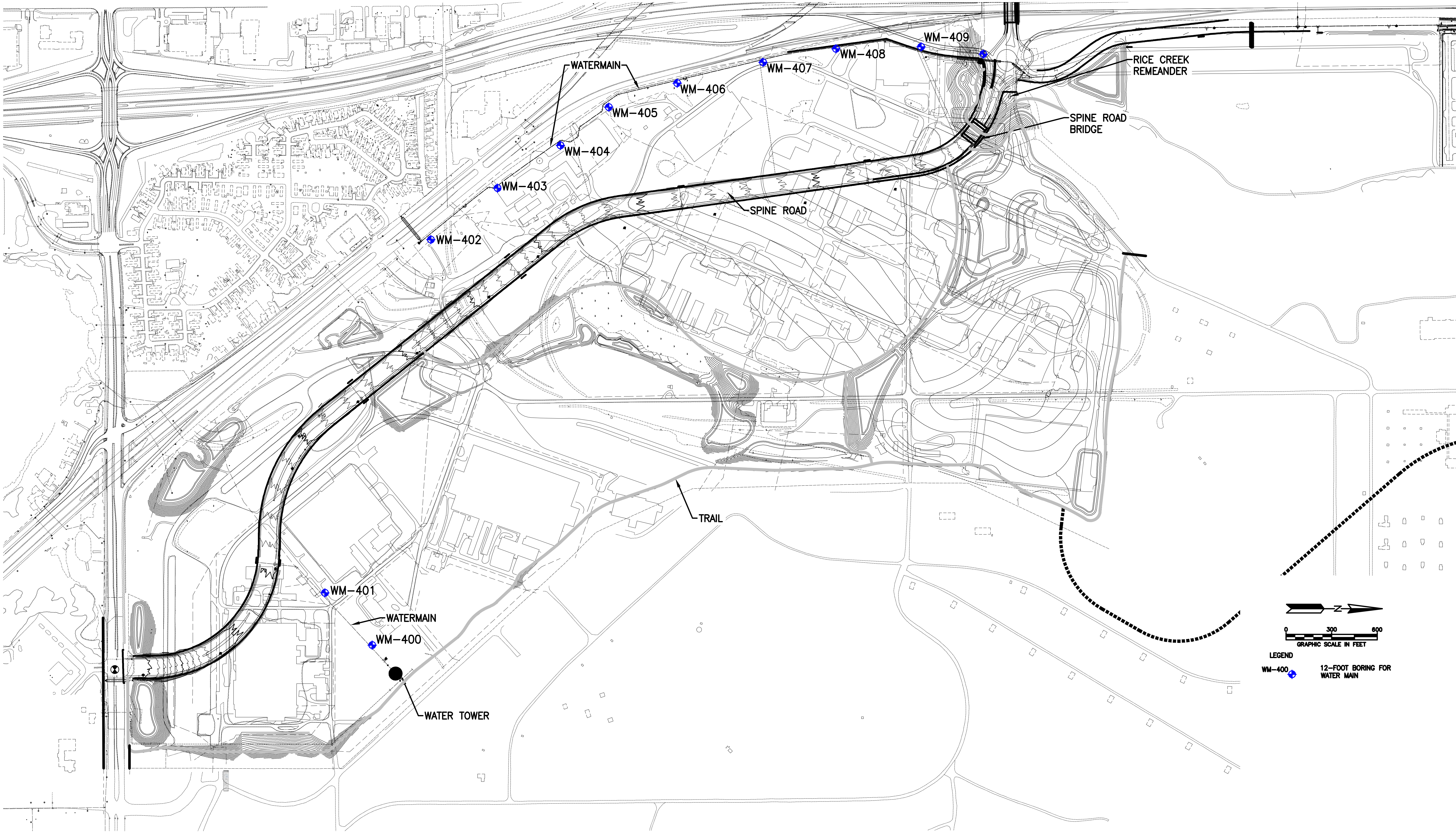

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WATER MAIN BOREHOLE LOCATION MAP

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1382-0001	FIGURE 5	1

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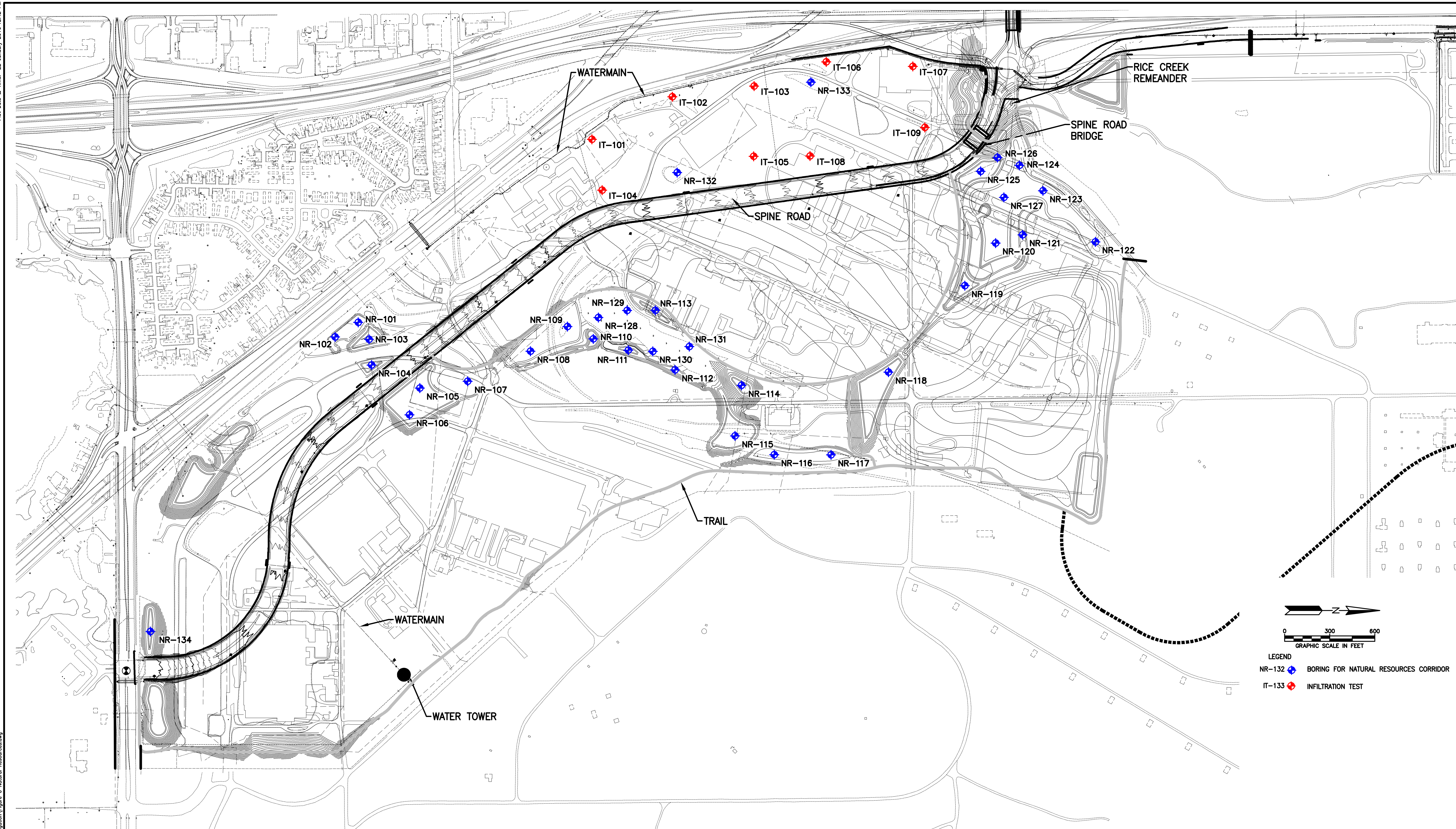

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PROJECT TITLE
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SHEET TITLE
NATURAL RESOURCES CORRIDOR BOREHOLE AND INFILTRATION TEST LOCATION MAP

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JVB	JW	TR	JANUARY 2016	AS NOTED

PROJECT NO.	SHEET NO.	REV NO.
1382-0001	FIGURE 6	1