GEOTECHNICAL INVESTIGATION REPORT

Gateway Business Park
City of Beloit, Wisconsin

for the

City of Beloit

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Prepared by

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INTRODUCTION

This report has been prepared to aid in the evaluation and design of the proposed roadways, utility support, and foundation support for the Gateway Business Park in the City of Beloit, Wisconsin. This geotechnical investigation was performed in accordance with HNTB's Design Engineering Services Contract with the City of Beloit. The scope of services included a subsurface investigation, field and laboratory testing and a geotechnical engineering analysis. The primary purpose of this geotechnical investigation was to determine the stratigraphy and physical properties of the soils underlying the project site, particularly the strength and deformation characteristics so that satisfactory and economical pavement, utility, and foundation designs may be developed. An addendum to this report will be provided with recommendations for the proposed stormwater ponds.

Information that could affect construction and earthwork activities, such as groundwater levels, was also obtained. The analysis and recommendations presented in this report are based in part on our interpretation of the subsurface information collected at the test boring locations indicated on the boring location plan. The report does not reflect variations in subsurface conditions which may exist between or beyond these borings. Variations in soil conditions should be expected between the borings, the nature and extent of which might not become evident until construction is undertaken. Additional subsurface investigation is required for any future building construction to accurately assess the subsurface conditions and type of construction on that specific site.

If variations are encountered and/or the scope of the project is altered, HNTB should be consulted for additional recommendations. This report is intended for geotechnical purposes only; however, petroleum odors or soil staining were noted in test borings SB-103, SB-107, SB-153, SB-186 and SB-195. A Phase II Environmental Site Investigation is recommended to evaluate if contamination is present at these locations.

SITE AND PROJECT DESCRIPTION

The project site location is identified on Figure 1 in Appendix I. The Gateway Business Park encompasses approximately 455 acres. The project is being developed adjacent to the Cranston Road Extension and Gateway Boulevard project, which is being constructed by WisDOT. The Gateway Business Park project consists of the following:

- Construction of proposed Gateway Boulevard north of Cranston Road and adjacent cul-de-sacs;
- Construction of new sanitary and storm sewers and connection to existing sanitary sewer systems;
- Construction of storm water detention basins and drainage swales; and
- Development of a site grading plan for future building development.

The project will be conducted in two phases. This geotechnical investigation incorporates recommendations for both phases and design considerations for the Gateway Business Park. The results from the Cranston Frontage Road Investigation are also discussed and included in this investigation.

LOCAL TOPOGRAPHY

The USGS 7.5-minute series topographic survey map dated 1961 (photorevised 1971 and 1976) of the Shopiere Quadrangle, Rock County, Wisconsin was reviewed. The project site is located in Section 28, the East ¼ of Section 29, the East ½ of Section 32, and the Northwest ¼ of Section 33 in Range 13 East, Township 1 North. The project site location is illustrated on the topographic map on Figure 1 in Appendix I.

On the northern half of the project site, the topography is generally sloping steeply down from the east to the west, southwest, and northwest towards Spring Brook Creek and its tributaries. On the southern half of the project site, the topography is characterized by steeply rolling hills, which diminish in the westward direction. The surface elevations range from approximately 810 feet, mean sea level (MSL) to approximately 940 feet, MSL.

The most significant topographic features include the elevated hills on the eastern portion of the project site, the low lying areas near Spring Brook Creek west of the site, Turtle Creek west of the site, and a drainage swale on the southeast portion of the site. Of the low-lying areas, Spring Brook Creek is located approximately 500 to 2000 feet west of the project site and has an elevation of approximately 800 feet, MSL. Turtle Creek is located approximately 6000 feet west of the project site with an elevation of approximately 800 feet, MSL. Both Spring Brook Creek and Turtle Creek flow to the west and southwest. The drainage swale southeast of the site flows south from an approximate elevation of 880 feet, MSL to 800 feet, MSL.

LOCAL GEOLOGY & HYDROGEOLOGY

The most significant feature of the bedrock surface in Rock County is the ancestral Rock River valley, which has been filled with glacial outwash to a depth of at least 396 feet below the present land surface. The project site is located over the eastern slope of the ancestral river valley. The outwash deposits in proximity to the project area range in thickness from approximately 50 to 150 feet and overlie bedrock comprised of the Platteville, Decorah and Galena formations, undifferentiated. The bedrock elevations range from 750 feet, MSL on the western portion of the site to 900 feet, MSL on the eastern portion of the site. The high point of 900 feet, MSL is a bedrock hump that is located in the North ½ of the Northeast ¼ of Section 28, the Northwest ¼ of Section 27, and the Southwest ¼ of Section 22. The bedrock appears to be as shallow as 20 feet below existing grade. However, this high point in the bedrock appears to be just east outside of the project area.

East of the Rock River, the primary source of water for domestic and stock wells is the Platteville, Decorah and Galena formations, undifferentiated. Wells have been developed in the outwash deposits along the Rock River for municipal and industrial use. These outwash deposits have been valuable because of their saturated thickness, high permeability, and storage capacity.

In Rock County, both the groundwater table and artesian conditions are considered to be one groundwater unit due to the close interconnection. One piezometric map represents the entire county. In the vicinity of the project site, the piezometric surface appears to range from approximately 760 to 800 feet, MSL. Groundwater appears to flow to the southwest and west towards the Rock River and Turtle Creek. The groundwater is charged primarily by precipitation and the infiltration from Turtle Creek as it flows through the Rock River valley.

The most significant drainage feature near the project site is Turtle Creek. Turtle Creek drains the southeastern part of the county and is located approximately 20 feet above the general groundwater body surface. As the Turtle Creek flows through the Rock River valley, the creek becomes an influent stream losing water to the groundwater system.

LOCAL SOIL CONDITIONS

The US Department of Agriculture Soil Conservation Service classified the soil types in the project area as part of the Plano-Warsaw-Dresden and Pecatonica-Ogle-Durand associations. The Plano-Warsaw-Dresden association is comprised of well-drained and moderately well-drained soils that have a subsoil varying from silty clay loam to sandy clay loam over stratified sand and gravel. The Pecatonica-Ogle-Durand association is comprised of well-drained and moderately well-drained soils that have a subsoil varying from silty clay loam to sandy clay loam over sandy loam glacial till.

Soils along the project corridor consist of the following:

- Durand silt loam
- Elburn silt loam
- Flagg silt loam
- Griswold loam
- Kidder silt loam
- Mahalasville silt loam
- Pecatonica silt loam
- Plano silt loam
- Ringwood silt loam
- Rockton silt loam
- Rotamer loam
- Sogn loam
- Wauconda loam

The specific soil types along the project corridor are shown on Figure 2 in Appendix I. The properties of these soil types are summarized in Figure 3 in Appendix I.

SITE EXPLORATION

Between January 26 and March 14, 2001, Wisconsin Soil Testing of Butler, Wisconsin advanced a total of 214 borings within the project site. Boring locations are shown on the Boring Location Map on Figure 4 in Appendix I. The scope, methods employed and the results of the subsurface exploration are described in the following sections:

Scope

A total of 220 borings (B-1 through B-220) were proposed to be advanced at approximately 300-foot intervals in a grid pattern within the project site. Eight of the borings (B-113, B-123, B-172, B-173, B-181, B-182, B-185, and B-219) were eliminated. The 212 completed borings were advanced to depths of 10, 15, or 25 feet below existing grade. In addition, three of the completed borings (B-12, B-190, and B-206) were advanced from the original termination depth of 15 feet below existing grade to 40 feet below existing grade.

Two additional borings (B-221 and B-222) were performed near two locations where the sanitary sewer is proposed to be jacked underneath Interstate Highway 90 (IH 90). One boring (B-221) was performed on west side of IH 90 across from the WisDOT Tourist Center and Rest Area. The other boring (B-222) was performed on the west side IH 90 south of the Union Pacific Railroad (UPRR). Boring depths and elevations are noted on the boring logs presented in Appendix II.

R.H. Batterman & Company surveyed the ground surface elevations at the boring locations. This data was supplied to Wisconsin Soil Testing for inclusion on the boring logs.

Water levels were recorded in all borings while drilling. Water levels were recorded at 24 or 48 hours after completion of those borings which encountered water. This was done to provide groundwater level data representative of the actual conditions. The groundwater observations are recorded on the boring logs. After the groundwater readings were completed, boreholes were backfilled with bentonite as required by the Wisconsin Administrative Code, section NR 141.

Methods

The soil borings were performed by Wisconsin Soil Testing using a using a CME 55 drill rig mounted on an all-terrain vehicle. Borings were advanced using 2.25-inch hollow stem augers. Remolded, disturbed soil samples were collected at 2.5- or 5-foot intervals using a 2 inch O.D. split barrel sampler in accordance with ASTM Method D 1586. Borings advanced to 10 feet below existing grade were sampled in 2.5-foot intervals. Borings advanced to 15 feet below existing grade were sampled at 2.5-foot intervals to 10 feet below existing grade and then at 5-foot intervals to 15 feet below exiting grade. Borings advanced to 20 or 25 feet below existing grade were sampled at 5-foot intervals.

Using this sampling procedure, a split barrel sampler is driven into the soil a distance of 18 inches using a 140-pound hammer falling 30 inches. The value of the Standard Penetration Resistance is obtained by counting the number of blows completed by the hammer over the final 12 inches of driving. This value provides a qualitative indication of the in-place relative density of cohesionless soil. The indication is qualitative only, since many factors can significantly affect the Standard Penetration Resistance value. Direct correlation of the results obtained by drill crews using different rigs, drilling procedures and hammer-rod-spoon assemblies should not be made.

Visual classification of soil samples was conducted according to the Unified Soil Classification System (USCS) ASTM D 2488. A portion of each recovered sample was placed in an airtight glass jar and returned to the laboratory for further testing.

The following tests were performed on selected samples:

- Water Content Determination (ASTM D 2216) was conducted on selected samples;
- Wet and Dry Unit Weight Determinations were performed on selected Shelby tube samples; and
- Particle Size Distribution Testing (ASTM D 422) was conducted on selected samples.

Water Content determination measures the percentage of water contained in the mass of soil or rock. This percentage helps to identify where the water table may be located. It is also compared to the Atterberg Limits to identify the consistency of the soil.

Dry unit weight determines the density or weight per volume of the soil or rock. These results are representative of one of the physical properties of soil. These results can be used in conjunction with other test results such as the water content to determine additional properties of the soil.

The results of the particle size distribution test are used to determine the soil classification. This test separates the selected soil sample by grain size. This grain size distribution is then compared to and confirms the given USCS classification. The results of the particle size distribution test are also used for qualifying soils for acceptable fill materials. Laboratory results are shown on the boring logs and presented in Appendix III.

Results

Current Investigation

Areas A through H

The project site was divided into the following areas. Area A is located from the southern project boundary to Colley Road and from the western project boundary to Gateway Boulevard. Area B is located from Gateway Boulevard to the eastern project boundary and from the southern project boundary to Colley Road.

Area C is located between IH 90 and Gateway Boulevard and from Colley Road to Cranston Road. Area D is located between Gateway Boulevard, RL 200, and Colley Road. Area E is located from RL 200 to the eastern project boundary and from Colley Road to north of B-181. Area F is located from IH 90 to Gateway Boulevard and from Cranston Road to north of B-179. Area G is located from the western project boundary and Gateway Boulevard and from north of B-179 through B-181 to north of B-215 through B-220. Area H is located from Gateway Boulevard to the eastern project boundary and from north of B-179 through B-181 to north of B-215 through B-220. The divisions of the project site are presented on the Boring Location Map on Figure 4 in Appendix I.

Surface elevations ranged from 810.2 feet, MSL to 934.5 feet, MSL. In general, the project site decreases in elevation from east to west. The highest point is located on the northeast corner of the project site in Area H. From the highest point, the topography steeply descends either south to southwest towards a depression in Area F or north towards borings B-218 and B-219.

In Areas D, E, and F, the topography is characterized by hills that descend gently southwest to a depression in Area F and west to a depression in Area C. In Area F, the depression is located north of the intersection of Gateway Boulevard and Cranston Road. In Area C, the depression is located near the southern portion of Area adjacent to Colley Road and IH 90.

In Areas A and B located south of Colley Road, the topography is characterized by hills that descend gradually to the west. Over the entire site in the north – south direction, the topography is steeply rolling from the hilltops to the drainage pathways.

The soils on site generally consisted of 0 to 24 inches of topsoil over 0 to 6 feet of medium stiff to stiff silty clay. The clay contained a trace of fine-grained sand and organics. The liquid limit ranged between 19 and 40 with an average of 34 and a median of 38. The plasticity index ranged from 5 to 17 with an average of 14 and a median of 16. The moist unit weight ranged from 99 to 146 pounds per cubic foot (pcf) with an average of 126 pcf and a median of 123 pcf. Moisture contents ranged from 7.8 and 30.0% with an average of 21.0% and a median of 24.2%. The lab results are presented in Appendix III. The consistency of clay for each boring is illustrated on Figure 5 in Appendix I.

Over the majority of the site, the clay was underlain by a medium dense to dense yellowish-brown to brown silty sand. The silty sand was fine- to coarse-grained and contained varying amounts of fine to coarse gravel. The silty sand extended to the termination depth of the borings; however, the majority of the borings indicated that the soil density increased with depth.

At approximately 10 to 15 feet below existing grade, the silty sand became very dense. Cobbles were noted in several borings at depths where the blow counts were greater than 50 blows per foot. Refusal was noted in eight of the borings, which were located on the eastern half of the project site. The elevation of refusal ranged from 826 to 914.2 feet, MSL. The higher elevations correspond to the borings drilled in the northeastern portion of the site.

To determine if auger refusal was due to the presence of bedrock or boulders, three of the borings were extended to 40 feet below existing grade. In all the extended borings, the auger did not hit refusal. The silty sand was very dense and extended to the termination of the borings. Some cobbles were also noted. The density of sand underlying the clay for each boring is illustrated on Figure 6 in Appendix I. The density of sand located at depths greater than 10 feet below existing grade for each boring is shown on Figure 7 in Appendix I.

A total of 20 samples were selected for particle size distribution testing. The sieve results indicated that the percent passing the No. 4 sieve (P4) ranged from 47 to 97% with an average of 77% and median of 83%. Note that 15 of 20 sieves had a P4 greater than 70%. The five samples with P4 less than 70% were very dense silty sands located greater than 8 feet below existing grade.

The percent passing the No. 10 (P10) ranged from 29 to 91% with an average of 62% and a median of 69%. The percent passing the No. 200 (P200) ranged from 3 to 45% with an average of 27% and a median of 31%. Moisture contents ranged from 2.0 to 15.5% with and average and median of 7.6%. These sieve results confirmed that the soil was primarily sand and silt with varying amounts of gravel.

Exceptions to the previously described soil profile were encountered in Areas C, D, and E. In Areas C and D, some of the borings indicated that the brown silty clay was underlain by a reddish-brown clayey sand prior to encountering the silty sand. The clayey sand was medium dense and fine- to medium-grained. The sand also contained some silt and traces of fine gravel. These borings indicating the clayey sand were located in or adjacent to the depression identified in Area C.

In Area D, two borings (B-144 and B-145) indicated medium stiff to stiff black organic silty clay. The silty clay contained traces of fine-grained sand. The organic silty clay extended to 5 to 6 feet below existing grade. These borings were located adjacent to a depression identified in Area F.

In Area A, water was encountered 13.8 feet below existing grade at 803.7 feet, MSL in B-49. In Area C, water was encountered 3.5 feet below existing grade at 811.1 feet, MSL in B-95. In Area D, water was encountered between 5.5 to 8.0 feet below existing grade or 814.0 to 820.6 feet, MSL in B-144 and B-145. Contractors should be prepared if water is encountered during utility and wall excavations.

IH 90 and WisDOT Tourist Center Crossing

Boring B-221 was performed west of IH 90 and south of the WisDOT Tourist Center. The soil consisted of 4 feet of soft, black, organic sandy clay. The clay contained some fine-grained sand and little silt. The organic sandy clay was underlain by medium dense, brown, silty fine-grained sand. The sand contained traces of clay and fine gravel. The sand was wet and extended to 9 feet below existing grade.

The silty sand was underlain by grayish-brown sand. The sand was dense to very dense and fine- to medium- grained. The sand contained traces of silt and some gravel. The soil was moist, and cobbles were noted at approximately 20 feet below existing grade. Blow counts exceeded 50 blows per foot between 10 and 24 feet below existing grade. No groundwater was encountered after the boring was completed.

IH 90 and UPRR Crossing

Boring B-222 was performed west of IH 90 and south of the UPRR. The soil consisted of 12 inches of topsoil over approximately 14 feet of possible fill. The fill was medium dense to dense and black to dark brown in color. The fill consisted of fine- to coarse-grained sand, black sandy clay with organics, and some fine- to coarse-grained gravel.

The fill was underlain by very dense to hard brown to brownish-yellow sandy gravel. The gravel contained some fine- to coarse-grained sand and some silt. The gravel was saturated and had blow counts greater than 50 blows per foot. Cobbles were noted at 19.5 feet below existing grade. Auger refusal was noted at 19.5 and 18.0 feet below existing grade after two attempts at different locations. Groundwater was encountered at 14 feet below existing grade while drilling.

Subsurface water levels should be expected to fluctuate seasonally and annually from the readings noted in the borings. The time of year that the borings were performed and the history of precipitation prior to drilling should be known when using the water readings presented in this report to extrapolate water levels at other points in time.

To aid in the design of the proposed roadways, the borings located on or adjacent to proposed roadway alignments were specifically evaluated for each proposed roadway. Descriptions of these soils are discussed in the following sections.

Gateway Boulevard

The alignment of Gateway Boulevard begins at the Wisconsin / Illinois state line, south of the project and continues north across the project site to Cranston Road (Station 4+00). Gateway Boulevard then curves to the east and intersects with Town Hall Road. Design recommendations for the portion of Gateway Boulevard between Cranston Road and the state line were included in a report prepared for WisDOT in January 1999. The scope of this investigation includes Gateway Boulevard north of Cranston Road.

According to the boring logs adjacent to or within the proposed roadway, approximately 0 to 24 inches of clayey topsoil overlaid 1 to 6 feet of medium stiff to stiff silty clay. The clay was dark brown to brown in color and contained traces of fine-grained sand and organics. At the south end of Gateway Boulevard, the silty clay was organic to a depth of 6 feet, as indicated on the logs for borings B-144 through B-146.

One sample of silty clay collected near the proposed corridor was tested for Atterberg Limits. The liquid limit was 40 and the plasticity index was 17. The pocket penetrometer readings ranged from 1.0 to 3.25 tons per square foot (tsf). Four Shelby tubes were collected in the silty clay layer to determine unit weight. The moist unit weight ranged from 119 to 143 pcf with an average of approximately 134 pcf. Moisture contents ranged from 10 to 30%.

The brown clay was underlain by silty sand. The density of the sand increased in depth. The silty sand became very dense to hard at approximately 3 to 6 feet below the clay layer. The silty sand was fine- to coarse-grained and contained varying amounts of fine to coarse gravel and traces of clay. Cobbles were noted in several borings at depths where the blow counts were greater than 50 blows per foot. The silty sand extended to the termination depths of the borings. Sieve results from two samples adjacent to the roadway indicated P10 values of 63 and 85%, P40 values of 51 and 73%, and P200 values of 32 and 45%. Moisture contents were 8.0 and 11.5%.

Groundwater was not encountered in any of the borings along the Gateway Boulevard alignment, except in a depression near Station 4+06 where borings B-144 and B-145 were located. In this depression, water levels were reported at 5.5 and 8.0 feet below existing grade at 24 hours after completion of the borings. This corresponds to water level elevation ranging from 814.1 to 820.6 feet, MSL.

The piezometric map of Rock County indicated that the piezometric surface is approximately 780 to 790 feet, MSL in that location. Turtle Creek, located approximately 6000 feet west of the site, has an approximate water elevation of 800 feet, MSL.

The existing topography is characterized by steeply rolling hills. Surface elevations along the Gateway Boulevard alignment ranged from approximately 852 feet, MSL near Cranston Road to 822 feet, MSL near Station 4+05.5 to 926 feet, MSL near the Station 4+32.5 by Water Tower Access Road to 850 feet, MSL near Station 4+43.

Reference Line 200 (RL 200)

Reference Line 200 is a proposed roadway that extends from approximately 1800 feet south of Colley Road (Station 2+00) to approximately 700 feet northwest of Gateway Boulevard (Station 2+38). In future phases, the proposed roadway alignment will continue north, then curve to the east to intersect with Gateway Boulevard at approximately Station 2+38.

According to the soil boring logs, approximately 0 to 24 inches of clayey topsoil was underlain by a 1- to 6-foot layer of medium stiff to stiff silty clay. The clay was dark brown to brown and contained traces of fine-grained sand and organics.

One sample collected adjacent to the proposed roadway was tested for Atterberg Limits. The liquid limit was 38, and the plasticity limit was 16. The pocket penetrometer readings ranged from 1.0 to 4.0 tsf. Eight Shelby tubes were collected to determine the unit weight of the silty clay. The moist unit weights ranged from 111 to 131 pcf with an average of approximately 122 pcf. The moisture contents ranged from 14 to 26%.

Underlying the clay was a fine- to coarse-grained silty sand that increased in density with depth. The silty sand became very dense at approximately 4 to 10 feet below the clay layer. The silty sand was fine- to coarse-grained and contained varying amounts of fine to coarse gravel. Cobbles were noted in the majority of the borings at depths where the blow counts were greater than 50 blows per foot. The silty sand extended to the termination depths of the borings.

Moisture contents ranged from 4.5 to 7.5%. Sieve results from four samples tested in borings adjacent to the roadway indicated P10 values ranged from 55 to 97% with an average of 75%. The P40 values ranged from 34 to 91% with an average of 60%. The P200 values ranged from 15 to 35% with an average of 24%. The lower percentages corresponded to the deeper samples containing more gravel and cobbles. Groundwater was not encountered in borings along the proposed roadway.

Surface elevations indicated the existing topography is gently rolling. Surface elevations ranged from approximately 849 feet, MSL near the south end of the roadway to 892 feet, MSL near Station 2+11 to 856 feet, MSL at Colley Road. From Colley Road, surface elevations were approximately 872 feet, MSL near Station 2+27 to 836 feet, MSL in the depression adjacent to Gateway Boulevard to 878 feet, MSL near Station 2+40.

Water Tower Access Road (RL 300)

Water Tower Access Road or RL 300 is located on the north end of the project corridor and interests with Gateway Boulevard at Station 4+32. According to the soil boring logs, approximately 0 to 6 inches of clayey topsoil overlaid a 1.5- to 5-foot layer of medium stiff to stiff silty clay. The clay was dark brown to brown and contained traces of fine-grained sand and organics. The pocket penetrometer readings ranged from 2.75 to over 4.5 tsf. Three Shelby tubes were collected to determine the unit weight of the silty clay. The moist unit weights ranged from 98 to 125 pcf. Moisture contents ranged from 24.2 to 26.2%.

The clay was underlain by a fine- to coarse-grained silty sand. The sand increased in density with depth. The silty sand became very dense at approximately 2 to 8 feet below the clay layer. The silty sand was fine- to coarse-grained and contained varying amounts of fine to coarse gravel. Cobbles were noted in the majority of the borings at depths where the blow counts were greater than 50 blows per foot.

Auger refusal due to rock or boulders was noted in borings B-195, B-204, and B-213 at depths of 9.5, 12.0, and 12.0 feet below existing grade, respectively. To determine whether auger refusal was due to bedrock or boulders, these borings were offset and drilled to 40 feet below existing grade. The silty sand extended to the termination depths of the borings.

Sieve results from three samples indicated that the P10 values ranged from 83 to 88%, the P40 values ranged from 69 to 72%, and the P200 values ranged from 30 to 41%. The moisture contents ranged 8.0 to 12.5%.

Groundwater was not encountered in any of the borings along the proposed roadway. Surface elevations indicated that the existing topography consists of steeply rolling hills from west to east. Surface elevations ranged from approximately 886 feet, MSL near the west end of the roadway, to 936 feet, MSL near the east end of the roadway.

Reference Line 400 (RL 400) – Cul de Sac

Reference Line 400 is located on the south end of the project corridor and intersects with Gateway Boulevard, approximately 2400 feet north of the CTH P. According to the soil boring logs, approximately 0 to 18 inches of clayey topsoil overlaid a 1- to 6.5-foot layer of medium stiff to stiff silty clay. The clay was dark brown to brown and contained traces of fine- to coarse-grained sand and organics.

One clay sample collected adjacent to the proposed roadway was tested for Atterberg Limits. The liquid limit was 39 and the plasticity index was 17. The pocket penetrometer readings ranged from 2.25 to 2.75 tsf. Two Shelby tubes were collected adjacent to the proposed roadway to determine the unit weight of the silty clay. The moist unit weights were 120 and 124 pcf. The moisture contents were 21.9 and 27.0%.

Underlying the clay was a fine- to coarse-grained silty sand that increased in density with depth. The silty sand was very dense at approximately 2 to 7 feet below the clay layer. The silty sand was fine- to coarse-grained and contained varying amounts of fine to coarse gravel. Some to traces of cobbles were noted in the majority of the borings at depths where the blow counts were greater than 50 blows per foot. The silty sand extended to the termination depths of the borings.

Groundwater was not encountered in the borings along the proposed roadway. Surface elevations indicated that the existing topography was steeply rolling from west to east. Surface elevations approximately ranged from 817 feet, MSL near the west end of the roadway, to 876 feet, MSL near the east end of the roadway.

Reference Line 500 (RL 500)– Cul-de-Sac

Reference Line 500 is located on the south end of the project corridor and interests with Gateway Boulevard, approximately 3600 feet north of CTH P. According to the soil boring logs, approximately 6 to 24 inches of clayey topsoil was underlain by a 2- to 7.5-foot layer of medium stiff to stiff silty clay. The clay was dark brown to brown and contained traces of fine- grained sand and organics. The pocket penetrometer readings ranged from 1.0 to over 4.5 tsf.

The clay was underlain by a fine- to coarse-grained silty sand. The density of the sand increased with depth. The silty sand became very dense at approximately 2 to 8 feet below the clay layer. The silty sand was fine- to coarse-grained and contained varying amounts of fine to coarse gravel. Cobbles were noted in the majority of the borings at depths where the blow counts were greater than 50 blows per foot. The silty sand extended to the termination depths of the borings.

Some exceptions were noted in several borings. In B-52, located in the east cul-de-sac on RL 500, the very dense silty sand was underlain by very stiff, brownish-gray, sandy clay. The pocket penetrometer readings for the clay ranged from 4.0 to 4.25 tsf. This was not characteristic of the overall project site.

In B-53, the brown silty clay contained gray and black mottles. The silty clay was underlain by a dark brown sandy silt, which contained very plastic clay lumps and traces of organics. The sandy silt was underlain by the characteristic silty sand.

Groundwater was not encountered in the borings along the proposed roadway except in B-49. Boring B-49 is located in a depression adjacent the proposed RL 500 and has an elevation of 817.6 feet, MSL. Water was encountered 13.8 feet below existing grade or 803.7 feet, MSL 24 hours after completion of the boring. The piezometric map for Rock County indicated that the piezometric surface is approximately at 770 to 780 feet, MSL. Turtle Creek, which is located approximately 6000 feet west of the site, has a groundwater elevation of approximately 800 feet, MSL.

Surface elevations indicated that the existing topography is gently rolling from west to east. Surface elevations ranged from approximately 826 feet, MSL near the west end of the roadway to 822 feet, MSL approximately 700 feet west of Gateway Boulevard to 833 feet, MSL near the east end of the roadway.

Reference Line 600 (RL 600) – Cul de Sac

Reference Line 600 is located on the south end of the project site and intersects with RL 200 at approximately Station 2+28. According to the soil boring logs, approximately 6 to 18 inches of clayey topsoil is underlain by a 3- to 5-foot layer of medium stiff to stiff silty clay. The clay was dark brown to brown and contained traces of fine- to coarse-grained sand and organics.

The pocket penetrometer readings in the silty clay were approximately 3.0 tsf. One Shelby tube was collected adjacent to the proposed roadway to determine the unit weight of the clay. The moist unit weight was 146 pcf. The moisture content was 8.6%.

Underlying the clay was a fine- to coarse-grained silty sand that increased in density with depth. The silty sand became very dense at approximately 5 to 9 feet below the clay layer. The silty sand was fine- to coarse-grained and contained varying amounts of fine to coarse gravel. Cobbles were noted in the majority of the borings at depths where the blow counts were greater than 50 blows per foot. The silty sand extended to the termination depths of the borings.

One sample adjacent to the proposed roadway was selected for particle size distribution testing. The sieve results indicated the P10 value was 83%, the P40 value was 68%, and the P200 value was 32%.

Groundwater was not encountered in the borings adjacent to RL 600. Surface elevations indicated that the existing topography is steeply rolling from west to east. Surface elevations ranged from approximately 866 feet, MSL near the intersection with RL 200 to 896 feet, MSL near the east end of the roadway.

Previous Investigations

Excerpts for previous investigations are presented in Appendix V.

Structure B-53-48/51 at Union Pacific Railroad & IH 90

The geotechnical investigation for structure B-53-48/51, at the intersection of the Union Pacific Railroad and IH 90 was completed in March 1958, prior to bridge construction. The structure is located approximately 1600 feet northwest of the project site. Three soil borings were advanced from 805.5 to 785 feet, MSL, and one boring was advanced from 805.5 to 791.5 feet, MSL.

At depths ranging from 805 feet, MSL to approximately 800 feet, MSL, a loose, dark brown clayey silt and sand was encountered. Blow counts toward the bottom of the silt and sand layer ranged from 14 to 25 blows per foot. Organic matter and traces of sand were noted in the surface layer. At approximately 800 feet, MSL, a dense, brown, silty and gravelly sand layer extended to 785 feet, MSL. In three of the four borings, the sand was very dense and contained sandstone fragments from 790 feet, MSL to 785 feet, MSL. Blow counts for the very dense sand layer ranged from 46 to 200 blows per foot. The higher blow counts were noted toward the bottom of the soil boring.

Cranston Road Extension

The previous investigation for the proposed Cranston Frontage Road in 1999 included a total of twenty-six borings, which were completed along the proposed Cranston Road extension, and along Gateway Boulevard between Cranston Road and the Wisconsin / Illinois state line. The soil boring locations are also presented on the boring location map on Figure 4 in Appendix I and in Appendix V. Soil borings B-1 through B-4 were located along the Cranston Road, and borings B-15 through B-26 were located along the centerline of the proposed Gateway Boulevard. Boring depths extended to 20 feet below existing grade in soil borings B-1 through B-3 and 10 feet below existing grade in soil borings B-4 and B-15 through B-26.

At the intersection of Spring Brook Creek and the proposed Cranston Road, soil boring pairs B-5 through B-8 were performed on each bank of the Spring Brook Creek. These borings extended approximately to 25 feet below existing grade. At the intersection of IH 90 and the proposed Cranston Road, soil boring pairs B-9, B-10, B-13 and B-14 were performed either side of the IH 90. Soil boring pair B-11 and B-12 was performed in the median of IH 90. Boring depths extended to 40 feet below existing grade. Boring logs and laboratory results are presented in Appendix V.

Surface elevations along the project corridor range from 793.8 feet, MSL at test boring B-4 located approximately 450 feet north of the Wisconsin/Illinois line at the south end of the Gateway Boulevard corridor, to 856.7 feet, MSL at test boring B-16 located approximately 900 feet north of Colley Road. Elevations west of IH 90 range from 800.2 feet at test boring B-3 located about 300 feet east of the Spring Brook Creek, to 810.7 feet at test boring B-1, near the Union Pacific Railroad at the west end of the Cranston Road extension. Surface elevations are indicated on the soil boring logs presented in Appendix V.

On the west end of the Cranston Road extension, the soil boring logs for test borings B-1, B-2, B-5, and B-6 consisted of primarily 1 foot of topsoil over approximately 3 feet of silty clay or sandy clay. The consistency of the soil was medium stiff to stiff, based on standard penetration resistance values. A typical sample of the native clay deposits had a liquid limit of 40 and a plasticity index of 13.

The clay layer was underlain by medium dense to dense sand and gravel. The standard penetration resistance values may have been elevated due to the presence of occasional cobbles in this layer. Moisture contents varied from 19% to 2%, with the higher moisture contents occurring in the top 4 feet.

Located in a low area east of Spring Brook Creek, test borings B-3, B-7, and B-8 indicated various soil layers that differed from the other test borings. Creek deposits may have accounted for this discrepancy of the top layer in test borings B-7 and B-8. However, below the top layer of these borings was a medium dense silty gravel with fine to coarse-grained sand and occasional cobbles. Test boring B-3 was located in a different soil type and consisted of 6 feet of stiff organic clay underlain by loose to dense clayey sand with silt. Moisture contents ranged from 25.5% to 6.5% with the higher moisture contents occurring in the top 7 feet.

Located near IH 90, test borings B-9 through B-14 indicated a foot of topsoil underlain by layers of medium dense to dense to very dense silty sand and clayey sand ranging from 26 to 40 feet deep. A typical sample of the silty sand and clayey sand had a liquid limit of 38 and a plasticity index of about 17. Below the silty and clayey sand layers were very dense, fine grained to coarse grained sand and silty sand layers. Moisture contents varied from 15% to 4.5% with the higher moisture contents occurring at the bottom of the borings.

The subsurface soil encountered at test borings B-4 and B-15 through B-26, located along the Gateway Boulevard alignment, consisted of primarily 1 to 2 feet of topsoil underlain by layer of soft to medium stiff silty clay with traces of organic material. As the test borings continued south, the depth of the silty clay layer increased and ranged from 2 to 6.5 feet. A typical sample of the native clay deposits had a liquid limit of 35 and a plasticity index of 15. The clay layer was underlain by a loose to medium dense fine-grained silty sand or sand layer. Moisture contents varied from 34% to 5.5% with the higher moisture contents occurring in the top 3 to 4 feet.

Detailed descriptions of the soils encountered are shown on the boring logs in Appendix V. Laboratory data sheets are included in Appendix V following the boring logs.

In test borings B-1 through B-26, water levels were measured at 24 and 48 hours after the completion of the borings. Water level measurements are included on the boring logs. Most of the soil borings were dry at and after the time of drilling. However, water was present in 5 of 26 borings measured at 48 hours after completion. Water level measurements collected at 48 hours after completion ranged from 12 to 29 below existing grade at 786.1 to 797.8 feet, MSL in those boreholes that contained water. These boreholes were located near the Spring Brook Creek and IH 90.

Structure B-53-52/53 at Colley Road & IH 90

The geotechnical investigation for Structure B-53-52/53 at Colley Road and IH 90 was completed in April 1958, prior to bridge construction. The structure is located on the western edge of the project site. Two soil borings were completed from 823 feet, MSL to 785 feet, MSL and from 815 feet, MSL to 790 feet, MSL. From 823 feet, MSL to approximately 817 feet, MSL, medium dense, brown, clayey and sandy silt was encountered with gravel. At 815 feet, MSL, medium dense to very dense, brown, silty and gravelly sand extended to about 785 feet, MSL. The blow counts for the very dense layer ranged from 19 to 160 blows per foot. Water levels or observations were not noted.

WisDOT Tourist Center and Rest Area

A geotechnical investigation was completed for the WisDOT Tourist Center and Rest Area 22 on IH 90. The rest area is located at the southwest corner of the project site. Twelve soil borings were completed. The majority of the borings were performed from approximately 808 feet, MSL to approximately 800 to 795 feet, MSL. One to three feet of topsoil was encountered at the surface of most borings. Underlying the topsoil, the soil profile consisted of clay overlaying damp to moist sand. The blow counts at the lower end of the sand layers ranged from 14 to 46 blows per foot. Groundwater was not encountered in the borings.

The liquid limit and the plasticity index were determined in five clay samples. The liquid limits ranged from 28 to 50. The plasticity indices ranged from 15 to 24. The unconfined compressive strength was determined in five borings and ranged from 0.75 to 1.9 tsf. The dry density of the clay was determined in four borings and ranged from 95 to 113 pcf. The wet density of the clay ranged from 121 to 132 pcf.

Structure B-53-204 at County Highway (CTH) P & IH 90

In 1996, a geotechnical investigation was completed for Structure B-53-204 at CTH P and IH 90. The bridge over IH 90 is located approximately 1800 feet south of the project corridor. Three soil borings were advanced to approximately 35 to 40 feet below existing grade.

Silt and sand were encountered to five feet below existing grade. Silty sand and gravel extended to the termination depths of the borings. The blow counts for the sand and gravel layers ranged from 20 to 110 blows per foot. Water levels were not noted in the borings.

Wisconsin Department of Natural Resources (WDNR) Well Construction Reports

Well logs were reviewed for five wells located between ½ mile to one mile north of the project. Bedrock was encountered at 15, 20, 41, 60, and 60 feet below existing grade. Clay and a mixture of clay and gravel overlaid the bedrock in this area. The static water levels in the area were 40, 50, 50, 80 and 90 feet below existing grade. The elevations were not noted on the well logs. The bedrock typically was limestone over dolomite and sandstone.

In 1993, one well was advanced one mile east of the project site. Clay and gravel overlaid the limestone bedrock. Limestone was encountered at 84 feet below existing grade and extended to 165 feet below existing grade. The static water level was 90 feet below existing grade.

In 1996, two wells were constructed for the WisDOT Tourist Center and Rest Area on IH 90. The rest area in located at the southwest corner of the project site. In one well, bedrock was encountered at 115 feet below existing grade. The bedrock encountered was composed of a layer of limerock, 45 feet thick. The limerock overlaid five feet of shaley limerock and 70 feet of sandrock.

In the second well, limerock bedrock was encountered at 130 feet below existing grade. The limerock layer was 35 feet thick and overlaid 5 feet of shaley limerock. A 75-foot thick layer of sandrock was encountered below the shaley limerock layer. The static water level at the rest area was between 55 and 56 feet below existing grade.

Well logs were reviewed for five wells located northeast of the project corridor. Clay and gravel overlaid the limestone bedrock. Bedrock was encountered at 41, 68, 89, 101, and 131 feet below existing grade. The static water levels were recorded at 110, 120, 135, 60 and 84 feet below existing grade.

PROJECT DESIGN RECOMMENDATIONS

All project design recommendations are based on the assumption that all construction work, including grading, subgrade preparation, fill placement and concrete work will be performed in accordance with the appropriate section of the WisDOT "Standard Specifications for Highway and Structure Construction," current edition.

Earthwork

The proposed right of way passes over land that has historically been used for agricultural purposes. Removal of all topsoil and organic matter is recommended for all areas that will receive fill or be paved. Prior to placing the granular base course, it is recommended that the subgrade be compacted using the methods described in Part II of the WisDOT "Standard Specification for Highway and Structure Construction", current edition. The compaction should be performed under the direction and observation of the project engineer.

Any areas exhibiting pumping, excessive yielding, or deflection greater than 1 inch should be excavated, removed and replaced with suitable granular backfill material. Areas of overexcavation and subgrade stabilization should be expected if subgrade is disturbed by construction traffic or wet weather conditions. An additional 6 to 12 inches of subsoil is recommended to be removed and replaced with structural compacted fill. Temporary haul roads for construction traffic are recommended to minimize the need for overexcavation of disturbed subsoils.

The thickness of topsoil removal for the project site was estimated based upon the information shown on the boring logs. The thickness of topsoil along proposed roadway alignments was determined where the vertical alignment was situated on existing grade or in fill areas.

Estimation of Topsoil Removal

Area	A	B	C	D	E	F	G	H
Range (in.)	4 – 24	0 – 24	3 – 20	4 – 20	0 – 24	2 – 24	0 – 10	0
Average (in.)	11 - 12	7 – 8	11 - 12	11 - 12	9 - 10	11 - 12	2 - 3	0

Roadway	Gateway	RL 300	RL 200	RL 400	RL 500	RL 600
Range (in.)	0 - 24	0 – 6	0 – 24	0 – 18	6 – 24	6 – 18
Average (in.)	9	1	10	11	12	11

The subsurface investigation indicated areas of medium stiff, organic silty clays in Area D near the RL 200 roadway and near Gateway Boulevard and RL 200 intersection. For optimal performance of new pavement or structures, these organic subsoils should be cut and removed where they occur under the proposed roadbase or proposed building pads. No other areas of organic soils were encountered.

Fill or backfill for foundations and walls are described in their appropriate sections. Fill or backfill required for pavement, utility trenches, and general site grading should consist of a nonfrozen, inorganic natural soil that is free of debris and has a liquid limit less than 40 and a plasticity index of less than 20.

The subgrade soils are generally composed of silty clay and silty sand materials, containing varying amounts of gravel. The silty clay soils can be used as backfill for embankments, general site grading, and utility trenches. If the clays are reused as fill material, the volume bulk shrinkage will range from 20 to 25%. The silty sand soil can be used as backfill for pavement and foundation subgrade, drainage ditches, general site grading, and utility trenches. If the sands are reused as fill material, it is estimated that volume bulk shrinkage for these soils will range from 15 to 20%.

The natural water content of the soil should be within 2% of the optimum water content determined by an ASTM-D1557 Modified Proctor Density test. Fill and backfill material should be compacted to a minimum of 92% of Modified Proctor Density. The recommended degree of compaction may be difficult to obtain in soil that is too wet or too dry. Natural water content may need adjustment by sprinkling or by scarifying and aerating to facilitate compaction.

The in-situ sands are expected to stand temporarily on slopes of 2 horizontal to 1 vertical (2:1) during excavation. They are not stable at slopes steeper than 2:1. The in-situ clays are expected to stand temporarily at slopes of on a slope of 1:1. Contractors should take care to shore open sides of utility trenches and wall excavations in accordance with current OSHA provisions.

Groundwater is not expected to be encountered in topographic high areas of the project site area. However, there are several low areas that have been identified where groundwater may be encountered in excavations. These areas identified as topographic depressions are located in Areas A, C, and D.

In Area A, water was encountered at 13.8 feet below existing grade or 803.7 feet, MSL in B-49. In Area C, water was encountered at 3.5 feet below existing grade at 811.1 feet, MSL in B-95. In Area D, water was encountered between 5.5 to 8.0 feet below existing grade or 814.0 to 820.6 feet, MSL in B-144 and B-145. Contractors should be prepared to temporarily dewater from open sumps if groundwater or accumulated surface water is encountered during utility trench and wall excavations.

If wet weather conditions produce accumulation or ponding of surface water, accumulated water should be removed immediately. The topography of the site lends itself to be susceptible to erosion. The use of silt fences is recommended to minimize the movement of eroded materials off-site. Periodic maintenance is recommended to remove accumulated sediment and replace damaged fences if necessary.

Pavement Design Parameters

The pavement subgrade should be prepared using the earthwork procedures described above. Variations in soil conditions will be encountered between the test boring locations. Therefore, prior to placing the granular base course, it is recommended that the subgrade be test rolled in all areas. The equipment to be used for test rolling should be a tri-axle dump truck fully loaded to within 2 tons of the vehicle legal load limit. The testing should be performed in multiple passes under the direction and observation of the project engineer. Any areas exhibiting pumping, excessive yielding, or deflection greater than 1 inch should be excavated, removed and replaced with suitable granular backfill material.

For estimating purposes, the need for excavation below subgrade (EBS) was approximated for areas where the pavement will be placed on existing grade. The need for EBS is approximately 15 to 20% for Gateway Boulevard, 5 to 10% for Water Tower Access Road, 15 to 20% for RL 200, 10 to 15% along RL 400, 10 to 15% for RL 500, and 5 to 10% for RL 600.

The use of a geotextile beneath the aggregate base course is not anticipated to be required on this project. However, if subgrade preparation is performed in wet weather, there may be significant losses of granular material if the base course is installed over a saturated clay subgrade. At the option of the contractor, a geotextile may be used to facilitate construction operations.

If a geotextile is used, it should be non-woven and needle-punched, installed in accordance with the manufacturer's specifications, and possess the following minimum physical properties:

GEOTEXTILE PROPERTIES	
Grab Strength	300 kg
Puncture Strength	100 kg
Trapezoidal Tear Strength	100 kg
Mullen Burst Strength	1500kPa

The key pavement parameters are based on the assumption that all of the pavement subgrade would consist of primarily silty clay subgrade soils and silty sand subsoils or on silty sand subgrade soils and subsoils. Due to the steeply rolling topography, the proposed alignment is located in both cut and fill areas.

For estimating purposes, the percentage of pavement resting on silty clay versus silty sand was estimated using information from soil borings on or adjacent to the roadway. Note that variations in soil conditions will be encountered between borings and the estimations are within ± 5%.

Estimation of the Type of Pavement Subgrade

Roadway	Gateway	RL 300	RL 200	RL 400	RL 500	RL 600
Silty Clay Subgrade	82%	57%	59%	67%	100%	68%
Silty Sand Subgrade	18%	43%	41%	33%	0%	32%

The location and type of soil subgrade anticipated is illustrated on Figures 5 and 6 in Appendix I. Note that these estimations and key pavement parameters were prepared from the proposed alignments as of the date of this report. Any changes in the alignment are not reflected in these estimations or parameters. The key pavement design parameters are summarized on the following table.

Key Pavement Parameters

DGI	AASHTO	UCS	R	CBR	K (pci)	S
PROFILE: CLAY SUBGRADE OVER SILTY SAND SUBSOIL						
Subgrade Soil - Organic Silty Clay (F-4)						
17	A-7	OL	25	4	125	3.5
Subgrade Soil - Silty Clay (F-3)						
11	A-6	CL	35	7	165	4.5
Subsoil – Silty Sand (F-2)						
5	A-2-7	SC-SM	42	10	200	5
PROFILE: SILTY SAND SUBGRADE OVER SILTY SAND SUBSOIL						
Subgrade & Subsoil – Silty Sand (F-2)						
2	A-2-4	SM	61	25	275	5.5

The values given above represent ranges based on soil classifications and have not been verified by actual CBR nor plate bearing tests.

Notes:

DGI=Design Group Index
AASHTO = AASHTO Soil Classification
UCS = Unified Soil Classification
S = Soil Support Group Index
R = Resistance Value
CBR = California Bearing Ratio
K = Modulus of Subgrade Reaction

The in-situ organic clay is expected to stand temporarily on slopes of 2:1 vertical and have a volume bulk shrinkage in the range of 20 to 25%. The in-situ silty clay is expected to stand temporarily on slopes of 1:1 vertical and estimated to have a volume bulk shrinkage in the range of 20 to 25%. The in-situ silty sand can stand temporarily on slopes of 2:1 vertical. The volume bulk shrinkage is estimated in the range of 15 to 20%.

Foundation Design

The subsurface investigation indicated that the majority of the site is suitable for building construction; however, there were several areas that are not recommended for building construction. These areas are located in the topographic lows of the project site where surface water runoff accumulates and where soft and compressible subsoils were encountered. In Area D and F, a depression is located north of the intersection of Gateway Boulevard and Cranston Road. Soft, black organic soils were encountered to depths of 6 feet below existing grade in B-144 and B-145 near the depression. In Area C, a topographic low is located near the southern portion of Area adjacent to Colley Road and IH 90.

These topographic depressions are not recommended for building construction due the number of long term problems. The location is prone to poor drainage and heavy inflow of surface water, which causes saturation of the subsoil and may cause the building to settle excessively. Improper drainage can also cause lateral movement of basement walls.

The borings were performed approximately 300 feet apart. The analysis and recommendations are based on our interpretation of the subsurface information collected at the test boring locations. This report does not reflect variations in subsurface conditions, which exist between or beyond these borings. Variations in soil conditions should be expected between the borings, the nature and extent of which might not become evident until construction is undertaken.

This subsurface investigation program was designed to assess whether the site is suitable for building construction. Recommendations in this report address only general building construction concerns. Additional subsurface investigation is required in all cases for any future building construction to accurately assess the subsurface conditions and type of construction on that specific site. The information in this report is to be used only for comparative purposes and preliminary cost estimates.

Construction activites associated with foundations should be performed under the direction and observation of geotechnical or project engineer experienced in building construction to verify that all specifications are met.

Shallow Foundations

The subsurface investigation indicated that subsoils are generally suitable for conventional strip and spread footings for one to three story buildings having column loads under 75 tons, for slabs on grade pre-engineered for parking, or for box culverts. When dry or damp, at or below their natural moisture contents, the subsoils have adequate strength to support foundations. These soils decrease in strength or become more compressible if they become saturated or disturbed by construction activities. It will be necessary to undercut topsoil, organic soils, and disturbed clay soils and replace them with engineered fill prior to construction of foundations.

If wet weather conditions are encountered, accumulated water must be removed immediately and clayey subsoils must be stabilized or removed and replaced with granular compacted fill. An additional 6 to 12 inches of clayey subsoil is recommended to be removed.

Footings should be placed on a minimum of 12 inches of suitable bearing material or structural compacted granular backfill. The silty sands encountered on-site are suitable for subgrade material. The natural water content of the soil should be within 2% of the optimum water content determined by an ASTM-D1557 Modified Proctor Density test. Fill and backfill material should be compacted to a minimum of 92% of Modified Proctor Density. The recommended degree of compaction may be difficult to obtain in soil that is too wet or too dry. Natural water content may need adjustment by sprinkling or by scarifying and aerating to facilitate compaction.

Footings are recommended to be placed a minimum of 4.5 feet below final site grades to prevent frost heave. To prevent eccentric or unbalanced loading on shallow foundations, the footings should be at least six inches wider on either side of the wall that they support regardless of net soil pressures.

In estimating the net allowable bearing capacities, factor of safety of 3.0 was used, and the building pads were assumed to follow the recommended site grading preparations. The bearing capacities also do not rely on pocket penetrometer readings, as they are performed on disturbed samples that are not representative of the actual soil conditions.

If spread footings, slabs on grade, or box culverts are the selected foundation types and placed on silty clay, the net allowable bearing capacities are estimated on the following table. Calculations are presented in Appendix IV.

Estimated Net Allowable Bearing Capacity

	Strip Footings		
Consistency of Clay	**Medium Stiff**	**Stiff**	**Very Stiff**
Local Shear	1,300 psf	2,500 psf	3,800 psf
	Square Footings		
Consistency of Clay	**Medium Stiff**	**Stiff**	**Very Stiff**
Local Shear	1,600 psf	3,300 psf	4,900 psf

The consistency of silty clay encountered in each boring is illustrated on Figure 7 in Appendix I. Medium stiff clays have blow counts ranging from 6 to 8 blows per foot. Stiff clays have blows counts ranging from 9 to 18 blows per foot. Very stiff clays have blow counts ranging from 19 to 30 blows per foot.

It is not recommended to estimate bearing capacities from pocket penetrometer readings. This field test is performed on disturbed clay samples, and the readings do not represent actual soil conditions.

To determine bearing capacities for sand, the width of the shallow foundation, the type of footing, and the soil density must be known. The estimated net allowable bearing capacities for various widths, footing types, footing depths, and soil types are shown in the following tables.

Strip Footings at 4.5 feet below Existing Grade

Width (feet)	2 – 3	4 - 5	6 – 10	11 - 15
Medium Dense Sand	1,700 psf	2,100 psf	2,400 psf	3,300 psf
Dense Sand	2,100 psf	2,500 psf	2,900 psf	3,900 psf
Very Dense Sand	2,700 psf	3,300 psf	3,800 psf	5,100 psf

Square Footings at 4.5 feet below Existing Grade

Width (feet)	3 - 5	6 - 10	11 - 15	16 – 20
Medium Dense Sand	1,800 psf	2,200 psf	2,900 psf	3,600 psf
Dense Sand	2,100 psf	2,600 psf	3,500 psf	4,300 psf
Very Dense Sand	2,800 psf	3,500 psf	4,500 psf	5,600 psf

Strip Footings at 9 feet Below Existing Grade

Width (feet)	2 – 3	4 - 5	6 - 10	11 - 15
Medium Dense Sand	3,100 psf	3,400 psf	3,800 psf	4,600 psf
Dense Sand	3,700 psf	4,100 psf	4,500 psf	5,600 psf
Very Dense Sand	4,900 psf	5,500 psf	6,000 psf	7,300 psf

Square Footings at 9 feet Below Existing Grade

Width (feet)	3 - 5	6 - 10	11 - 15	16 – 20
Medium Dense Sand	3,200 psf	3,600 psf	4,300 psf	5,000 psf
Dense Sand	3,800 psf	4,300 psf	5,100 psf	5,900 psf
Very Dense Sand	5,000 psf	5,700 psf	6,700 psf	7,800 psf

The lower estimated capacities are assigned to square footings. Rectangular footings have bearing capacities ranging between those of square and strip footings. These capacities are taken from tables and graphs presented in Appendix IV. The first set of tables is for footings placed at 4.5 feet below existing grade. The second set of tables is for footings placed at 9 feet below existing grade. Assumptions included using a factor of safety of 3.0 and a minimum width of 2 feet for strip footings and 3 feet for square footings.

Medium dense sands have blow counts ranging from 10 to 15 blows per foot. Dense sands have blow counts ranging from 15 to 30 blows per foot. Very dense sands have blow counts greater than 30 blows per foot. The density of sands encountered in the top 10 feet of each boring is illustrated on Figure 8 in Appendix I. The density of sands encountered at depths greater than 10 feet for each boring is illustrated on Figure 9 in Appendix I.

Contractors should take care to shore open sides of wall excavations in accordance with current OSHA provisions. Wet weather conditions and the presence of groundwater can cause the subsurface soils to become unstable and cave in. Contractors should expect the presence of surface water from wet weather and runoff and take precautions to dewater the wall excavations. The presence of groundwater was encountered in four borings and was discussed in the Earthwork section in this report.

Backfill used to fill excavations surrounding basements or walls should consist of compacted granular soil that is non-frozen, inorganic and free of debris and extend at least 18 inches from the exterior of the wall. The backfill should consist of WisDOT Grade 1 granular backfill. The backfill material should have the following properties: 100% percent passing the No. 4 sieve, less than 75% passing the No. 40 sieve, less than 15% passing No. 100 sieve, and less than 8% passing the No. 200 sieve. According the lab results, the silty sands encountered on site are ***not*** recommended to be used for backfill of walls or basement excavations, as they contained too much silt.

The natural water content of the soil should be within 2% of the optimum moisture as determined by ASTM-D1557 Modified Proctor Density tests. The recommended degree of compaction may be difficult to obtain in soil too wet or too dry. Natural water content may need adjustment by sprinkling or by scarifying to facilitate compaction. Backfill material should be compacted to a minimum of 90% modified Proctor Density and placed in no more than 8-inch lifts in the manner away from the wall.

Lightweight construction equipment is recommended for the compaction of backfill around the basement walls. Heavy equipment may increase lateral earth pressures against the wall and cause failure. Temporary wall bracing during construction and backfilling is recommended.

Backfill material as specified is adequate for drainage of water and minimizing the water pressure acting to push the wall inward. For design of temporary subsurface walls, the lateral earth pressure was calculated to be 70 pounds per vertical foot of wall per foot of width. For design of permanent subsurface walls, lateral earth pressure was calculated to be 85 pounds per vertical foot of wall per foot of width.

These recommendations assume that the maximum basement wall height is approximately 13 feet below grade. The building pad should be at an elevation of at least 5 feet above the water table. The computation assumes that the specified backfill recommendations were followed so that perched water does not accumulate within the backfill material. The computation also includes two feet of soil surcharge. Calculations for lateral earth pressures are presented in Appendix IV.

To reduce the potential for water pressure buildup against subsurface walls due to perched water accumulation within the backfill material surrounding the walls and beneath the floor slab, continuous drain tiles or equivalent subsurface drainage system are recommended. Drain tiles should be placed along the base of both interior and exterior of perimeter footings on at least 12 inches of compacted granular backfill material. Drain tiles should be at least 6 inches in diameter and be connected to a sump pump, storm sewer, or drainage swale sloping away from the building.

Drain tiles should be surrounded by a minimum of 6 inches of base course cover on each side and then wrapped with a geotextile fabric. The geotextile fabric helps prevent soil particles from entering and void spaces of the base course. In order to insure drainage and discharge of water, the open ends of the drain tiles should be covered with a lightweight fabric "sock". The "sock" prevents debris from entering and clogging the drain tiles.

Placing a minimum 12-inch clay cap, sidewalk or pavement above the backfill material along the walls that slope away from the building is also recommended to limit the intrusion and accumulation of water

These recommendations apply only to subsurface basement walls and not intended for retaining walls. A qualified geotechnical engineer should be contacted to design retaining walls. Retaining walls should be checked against sliding, overturning, bearing capacity, eccentricity, settlement, and global stability. Soil borings should be evaluated to determine what soil type the retaining wall will rest upon and what soil type is being retained. Recommended soil parameters for the design of retaining walls include the following:

Estimated Soil Parameters for Design of Retaining Walls

	Moist Unit Weight	Angle of Internal Friction	Cohesion
Soil Type	(pcf)	(degrees)	(psf)
Organic Silty Clay	115	0	200
Medium Stiff Silty Clay	120	0	1000
Stiff Silty Clay	125	0	2000
Very Stiff Silty Clay	130	0	3000
Medium Dense Silty Sand	125	30	0
Dense Silty Sand	130	32	0
Very Dense Silty Sand	135	35	0

This investigation did not test for soil parameters or specify building loads used in estimating settlement. Settlement may occur in areas of organic subsoils, saturated clay subgrade or loose sand subgrade if not removed or exposed to construction traffic and building loads. All fill, topsoil, and organic matter are recommended to be removed if expected to placed on by fill or building pad.

Areas of soft to medium stiff silty clays were encountered in depressions and drainage ways. The consistency of the clay for each boring is illustrated on Figure 7 in Appendix I. The density of sand for each boring is illustrated on Figure 8 in Appendix I. Additional investigation is recommended to estimate the amount of settlement, which may occur on specific building sites.

The borings were performed approximately 300 feet apart. The analysis and recommendations are based on our interpretation of the subsurface information collected at the test boring locations. This report does not reflect variations in subsurface conditions, which exist between or beyond these borings.

The borings were performed approximately 300 feet apart. The analysis and recommendations are based on our interpretation of the subsurface information collected at the test boring locations. This report does not reflect variations in subsurface conditions, which exist between or beyond these borings.
Variations in soil conditions should be expected between the borings, the nature and extent of which might not become evident until construction is undertaken. Therefore, additional subsurface investigation is required in all cases for any future building construction to accurately assess the subsurface conditions and type of construction on that specific site. The information in this report is to be used only for comparative purposes and preliminary cost estimates.

Deep Foundations

Driven pile foundations are recommended for buildings which require greater bearing capacities than those recommended for footings. The borings performed for the Gateway Business Park were not advanced to a sufficient depth to determine pile capacities. Therefore, pile recommendations are taken from the Cranston Road Bridge and Approaches Geotechnical Investigation. The pile capacities for the Cranston Road project were determined from borings B-9 through B-14. A driven pile foundation is recommended for structures such as bridges or large industrial facilities. Driven piles on this site will derive their load bearing capacity from a combination of skin friction and end bearing. The allowable axial capacity of each pile will be dependent on the pile section and the depth to which it is driven.

The most common pile section used by WisDOT is a 10-3/4 inch diameter pipe pile. This pile section, driven to a depth of 40 feet below the base of the pile cap, is estimated to achieve an allowable capacity in the range of 55 to 65 tons per pile using a factor of safety of 3.0. Capacity is derived from both skin friction and end bearing. Pile sections driven to greater depths will achieve higher allowable capacities. Estimated pile capacities are presented in Appendix IV.

Pile driving activities should be performed under the direction and observation of the geotechnical or project engineer experienced in pile driving to verify that all piles are driven to a sufficient depth and achieve their design capacity. Pile capacities should be calculated based on the energy of the hammer and the average penetration per blow of the hammer, according to the methods outlined in Part V of the WisDOT "Standard Specifications for Highway and Structure Construction", current edition.

If the pile capacity is achieved in the field with the specified lengths, no further driving shall be required. However, pile capacity is not expected to be reached within the specified lengths. It is recommended that the contractor stops driving piling at the specified length and allows the soil to "seize" around the piling, then re-tap all piling. Re-tapping should be considered only after a minimum of 48 hours has elapsed (for piles that did not reach pile capacity as calculated by a dynamic driving formula).

If re-tapping indicates that a pile has not reached capacity, the contractor should allow the pile to set up for an additional 48 hours before again attempting to re-tap the pile. If, after re-tapping the pile after 96 hours, the engineer shall determine if the bearing capacity is acceptable or whether additional piling shall be driven.

Gravel and cobbles were observed in soils encountered in the borings, which may cause damage to thin-wall pipe piles. A minimum wall thickness of 0.3125 inches is recommended. Increasing the wall thickness could reduce the potential for damage. In addition, a rock cross or wedge point is recommended to be welded on the end of each pile for ease of driving.

Displacement pipe piles are recommended and should be driven closed-ended and backfilled with concrete. The interior of the pile should be checked for dryness, straightness and general integrity before placing concrete. The concrete should have a minimum compressive strength of 3000 psi.

A minimum spacing of 4 pile diameters is recommended between pile centers to prevent damage to adjacent piles during driving, as well as to minimize heaving of adjacent piles. Pile caps should be located a minimum of 4.5 feet below final grade to minimize frost heave.

Due to the presence of sandy soil, reduction of individual pile capacity due to group action is not needed. The sandy subsoil densifies around the pile as the piles are driven. The driving action causes vibrations into the subsoil, which strengthens the soil around the pile. Therefore, group capacity will be the sum of individual pile capacities.

Utility Design Considerations

Sanitary Sewer, Water, & Storm Sewer Utility Design

Installation of sanitary sewer, water, and storm sewer utilities is proposed for the project site. Contractors should take care to shore open sides of utility trenches and wall excavations in accordance with current OSHA provisions. The in-situ clays are expected to stand temporarily 1:1. The in-situ sands are expected to stand temporarily on slopes of 2:1 vertical during excavation.

Assuming the sanitary sewers will be placed at depths greater than 10 feet below existing grade, the sewers are expected to rest on dense or very dense silty sand subsoil. This subsoil provides adequate strength for utility support. The estimated net allowable bearing capacity for sanitary sewer is approximately 4500 psf. If variations of expected subsoils are encountered, a qualified geotechnical engineer should be contacted to estimate net allowable bearing capacity of the soil.

Assuming water lines and storm sewers will be placed between 4 and 6 feet below existing grade, ~~it~~ they will rest on medium stiff to stiff silty clay subsoil or on medium dense to dense silty sand subsoil. These subsoils provide adequate strength for utility support. The net allowable bearing capacity is approximately 1300 psf for silty clay subsoil and 1700 psf for silty sand subsoil. If variations of expected subsoils are encountered, a qualified geotechnical engineer should be contacted to estimate net allowable bearing capacity of the soil.

Cobbles and boulders were encountered consistently across the site at depths greater than 10 feet below existing grade. Cobbles and boulders were noted where blow counts exceeded 50 and / or where the auger hit refusal. In locations where auger refusal was noted, borings were offset and extended to determine if refusal was due to the presence of bedrock or boulders. Bedrock was not encountered in the borings located within the project site.

According to regional geology, bedrock elevations range from 750 feet, MSL on the western portion of the site to 900 feet, MSL on the eastern portion of the site. The high point of 900 feet, MSL is a bedrock hump that is located in the North ½ of the Northeast ¼ of Section 28, the Northwest ¼ of Section 27, and the Southwest ¼ of Section 22. The bedrock appears to be as shallow as 20 feet below existing grade. However, this high point in the bedrock appears to be just east outside of the project area.

According to site visits, angular limestone cobbles and boulders were observed in the area located north of where the soil borings were completed. This area appears to be located in the Sogn loam soil type, which indicated dolomite at 12 to 60 inches below the surface. The soil type, Rockton loam, also indicated dolomite at 27 to 60 inches below the surface, but borings advanced in that soil type did not encounter bedrock. Additional investigation should be performed if the utility design extends outside of where borings were advanced.

Groundwater is not expected to be encountered in topographic high areas of the project site area. However, there are depressions and drainage ways that have been identified that are expected to encounter water. These areas are identified in Area A, Area C, and Area D.

In Area A, water was encountered 13.83 feet below existing grade or 803.72 feet, MSL in B-49. In Area C, water was encountered 3.5 to 6.0 feet below existing grade or 808.63 to 811.13 feet, MSL in B-95. In Area D, water was encountered between 5.5 to 8.0 feet below existing grade or 814.07 to 820.62 feet, MSL in B-144 and B-145. However, contractors should be prepared to dewater if groundwater or accumulated surface water is encountered in any location during utility excavations.

Pipe Jacking for the Sanitary Sewer

The sanitary sewer system in the business park will be connected to the existing City of Beloit sewer system west of IH 90 in two locations. Boring B-222 was performed in the first location where the sanitary sewer is proposed to be underneath IH 90 and the UPRR located approximately 1000 to 2000 feet northwest of the project site. Fill consisting of sand, clay and gravel was encountered in the top 14 feet of the boring. The fill was underlain by a very dense gravel with cobbles. The gravel extended until the auger hit refusal at 19.5 and 18 feet below existing grade after two attempts.

The boring logs from the WisDOT investigation for B-53-48/51 at the intersection of IH 90 and UPRR indicated very dense brown silty sand and sandstone fragments or sandy gravel. On the east side of IH 90, this very dense layer was encountered at approximately 790 feet, MSL. On the west side of IH 90, this very dense layer was encountered at approximately between 791 and 797 feet, MSL.

It is recommended that the sanitary sewer be jacked 10 to 15 feet below existing grade. Coarse gravel, cobbles, sandstone fragments and possibly boulders may be encountered at greater depths. Contractors should carefully select an installation method to accommodate these conditions.

Groundwater was not encountered in the boring after completion of the drilling. However, Spring Brook Creek is located near the area where the sanitary sewer is proposed to be jacked. The elevation of Spring Brook Creek in this location is approximately 800 feet, MSL. Therefore, contractors should be prepared to dewater excavations if groundwater or accumulated surface water is encountered.

Boring B-221 was advanced at the second location where the sanitary sewer is proposed to be jacked underneath IH 90 south of the WisDOT Tourist Center. The boring indicated organic sandy clay over medium silty sand to a depth of 9 feet below existing grade. A dense to very dense sand was then encountered with gravel and cobbles noted at 21 feet below existing grade. Boring logs from the WisDOT Tourist Center indicated clay over medium dense to dense silty sands with gravel.

It is recommended that the sanitary sewer be jacked 10 to 15 feet below existing grade. Coarse gravel, cobbles, sandstone fragments and possibly boulders may be encountered at greater depths. Contractors should carefully select an installation method to accommodate these conditions.

At the Tourist Center, groundwater was not encountered above 792 feet, MSL, the termination depth of the borings. It is not expected to encounter groundwater at this location; however, contractors should be prepared to dewater if surface water accumulates in excavations.

CONCLUSIONS

This report has been prepared for the exclusive use of the City of Beloit in order to assist engineers in the design and preparation of project plans and specification for site-specific final roadway and utility construction planning purposes. This report may contain insufficient or incomplete information for other construction uses or locations, even nearby in the same general area. Construction planning should be based on an assumption that subsurface conditions are variable and that the borings represent only a small fraction of the actual subsurface conditions.

Variations in subsoil type, consistency, level of saturation and changes in groundwater elevations should be expected and planned for by contractors.

The recommendations contained in this report represent our professional opinions. These opinions were arrived at in accordance with currently accepted engineering practices and no warranty is implied or intended. "Important Information About Your Geotechnical Engineering Report", prepared by ASFE, is presented in Appendix VI, and is considered an integral part of this report.

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