Madison Parks Division

210 Martin Luther King, Jr. Blvd., Room 104
Madison, WI 53703
608-266-4711 ● cityofmadison.com/parks

November 28, 2017

NOTICE OF ADDENDUM
ADDENDUM 1

CONTRACT NO.8062

Revise and amend the contract document(s) for the above project as stated in this addendum, otherwise, the original document shall remain in effect.

General

1. The geotechnical exploration report for the project has been amended to include a short discussion about potential installation challenges of helical piers on page 7 of the amended report. Other deep foundation types, such as micropiles, are permissible. Driven piles will not be acceptable due to noise issues.

Specifications

2. SECTION 102.9 BIDDER'S UNDERSTANDING
   Omit: SECTION 102.9 BIDDER'S UNDERSTANDING
   Add: SECTION 102.9 BIDDER'S UNDERSTANDING

   In the preparation of Drawings and Specifications, Strand Associates, Inc.® relied upon the following reports of explorations and tests of subsurface conditions at the Site which are attached at the end of the SPECIAL PROVISIONS:

   Report dated **November 17, 2017**, prepared by CGC, Inc., of Madison, Wisconsin, titled: Geotechnical Exploration Report–Proposed Bridge Replacements Vilas Park Island, Madison, Wisconsin, consisting of 23 pages.

   The technical data in the above report, upon which Contractor may rely, consists of boring methods, level of subsurface water, boring logs, laboratory test methods and results, and boring locations all as of the date made.

   City accepts no responsibility for accuracy of the soil data or water level information. Soil borings and report, included with these Contract Documents, were not obtained for the purposes of designing excavations and trenches. Soils information was used by Strand Associates, Inc.® for design purposes of new structures only. Contractor shall assure itself by personal examination as to subsurface conditions and shall provide its own investigations and make its own assumptions to comply with OSHA and any other applicable laws and regulations regarding excavation and trenching requirements.

3. SECTION 105.12 COOPERATION BY CONTRACTOR, last paragraph.
   Add: 'Swallow and other migratory birds' nests have been observed on or under the existing bridge. All active nests (when eggs or young are present) of migratory birds are protected under the federal Migratory Bird Treaty Act. The nesting season for swallows and other birds is usually between May 1 and August 30. The Contractor shall either prevent active nests from becoming established, or apply for a depredation permit from the US Fish and Wildlife Service for work that may disturb or destroy active nests. The need for a permit may be avoided by removing the existing bridge structure prior to nest occupation by birds, or clearing nests from all structures before the nests become active in early spring. As a last resort, the Contractor shall prevent birds from nesting by installing a suitable netting device on the remaining structure prior to nesting activity. The cost for preventing nesting and/or permitting shall be incidental to Bid Items 90005 and 90006 Removing Old Structure Over Waterway.'

4. BID ITEM 90008 – HELICAL PILES, DELIVERED AND INSTALLED
   Omit: BID ITEM 90008 – HELICAL PILES, DELIVERED AND INSTALLED
   Add: BID ITEM 90008-DEEP FOUNDATIONS – DESIGNED, DELIVERED AND INSTALLED. Per attached.

5. Contract 8062 Attachment A – Geotechnical Exploration Report.
   Omit: Contract 8062 Attachment A – Geotechnical Exploration Report, dated March 7, 2017.
   Add: Contract 8062 Attachment A – Geotechnical Exploration Report - REVISED, dated November 27, 2017. Per attached.

<u>Plans</u>

6. All Sheets.
   Replace: At all locations on the drawings where the word 'helical' appears, replace it with 'deep foundation'.

7. Sheet 15.
   Omit: full sheet.
   Add: Sheet 15, revision dated 11-27-17. Per attached.

<u>Bid Tab</u>

8. BID ITEM 90008 – HELICAL PILES, DELIVERED AND INSTALLED
   Omit: Bid item.
   Add: BID ITEM 90008-DEEP FOUNDATIONS – DESIGNED, DELIVERED AND INSTALLED.

Please acknowledge this addendum on page E1 of the contract documents and/or in Section E: Bidder's Acknowledgement on Bid Express.

Electronic version of these documents can be found on the Bid Express web site at:

http://www.bidexpress.com

If you are unable to download plan revisions associated with the addendum, please contact the Engineering office at 608-266-4751 receive the material by another route.

Kay H. Rutledge

for Eric Knepp, Parks Superintendent

BID ITEM 90008–DEEP FOUNDATIONS–DESIGNED, DELIVERED AND INSTALLED

A. Description. The work consists of designing, delivering and installing deep foundation supports for the bridge abutments. The specific type(s) of deep foundations used shall be determined by the contractor based on site conditions and the foundation loads shown on the drawings. Acceptable deep foundation types include, but are not limited to, helical piles and micropiles. The type of deep foundation used shall be suitable for the site conditions. The deep foundation system shall be designed for a minimum service life of 75 years.

B.1 Materials. If a deep foundation type other than micropiles or helical piles is used, materials used shall be determined by the deep foundation designer.

B.1.A Micropiles. Micropile materials shall be designed by the deep foundation designer.

B.1.B Helical Piles. Helical piles shall be by Foundation Supportworks, Inc., 12330 Cary Circle, Omaha, NE 68128, or equal. Manufacturer of helical piles shall have at least five years of production experience manufacturing helical piles and have documentation that manufacturer's helical piles have been used successfully in at least five engineered construction projects within the last three years.

Helical piles subject to compression loading shall be hollow round shaft. Helical piles subject to tension loading shall be solid square or round shaft or hollow round shaft. Size of piles shall be determined by the designer/manufacturer based on the specific project conditions. Pile shaft sections shall be in full, direct contact within couplings so as to remove coupling bolts and coupling welds from the "in-service" axial load path. Pile shafts and couplings shall have a fit-up tolerance of 1/16-inch or less. Helix plates shall meet the following geometry and spacing criteria to minimize soil disturbance:

1. True helix-shaped plates that are normal to the shaft such that the leading and trailing edges are within 1/4-inch of parallel.

2. Helix pitch is 3 inches ($\pm$ 1/4-inch).

3. All helix plates have the same pitch.

4. Helix plates have circular edge geometry.

5. Helix spacing along the shaft shall be between 2.4 and 3.6 times the helix diameter.

6. Helix plates are arranged along the shaft such that they all theoretically track the same path as the proceeding plate.

Central steel shaft of the lead and extension sections shall be a hollow steel structural section meeting ASTM A500 Grade B or C. Shaft coupling shall be factory welded to the extension shaft and be a hollow steel structural section meeting ASTM A513 Type 5. Helix plates shall be factory welded to the lead or extension shaft sections and shall be structural steel plate material meeting ASTM A572 Grade 50. Brackets shall be structural steel plate material meeting ASTM A572 Grade 50 or ASTM A36. All steel components shall receive a hot-dipped galvanized finish system in accordance with ASTM A123 after fabrication.

All hardware shall conform to ASTM A325 and shall be hot-dipped galvanized in accordance with ASTM A153.

B.2 Design and Performance Requirements. Deep foundations shall be designed to support the compressive or tensile load(s) as shown on the drawings.

B.2.A Micropiles. The design shall include pile design and pile-footing connection design. The design shall conform with applicable provisions of accepted industry practice.

B.2.B Helical Piles. The overall length, helix configuration and minimum torsional resistance of a helical pile shall be such that the required capacity is developed by the helix plate(s) in an appropriate bearing stratum.

All structural steel pile components shall be designed within the limits provided by the American Institute of Steel Construction (AISC) Specification for Structural Steel Buildings (AISC-360) using Allowable Stress Design (ASD) method of analysis. Product testing in accordance with ICC-ES Acceptance Criteria 358 may also be considered as an acceptable means of establishing system capacities.

Except where noted otherwise on the drawings, all piles shall be installed to provide an ultimate torque-correlated capacity based on an ASD analysis using a minimum factor of safety of 2.5 applied to the service or nominal loading.

The required ultimate torque-correlated capacity shall be verified at each pile location by monitoring and recording the final installation torque and applying default torque correlations per ICC-ES AC358.

Except where noted otherwise on the drawings, all tension anchors shall be installed to provide a minimum factor of safety against ultimate pullout resistance of 3, a maximum axial deflection at nominal tension load of 0.5 inches, and must satisfy the deflection criteria as stated on the plans or drawings. Pre-tensioning anchors is an acceptable and common means of reducing deflection at service loads.

The pile design shall take into account group efficiency from pile spacing, pile buckling potential, soil stratification, and strain compatibility issues.

B.3 Qualifications of Installing Contractor and Designer. The installing contractor and pile designer shall submit to the City the following documentation prior to starting work. Work shall not begin until all the submittals have been received and approved by the City. All costs associated with incomplete or unacceptable submittals shall be the responsibility of the Contractor.

Evidence of installing contractor’s competence in the installation of proposed deep foundations shall be provided to the City’s satisfaction and shall include the following:

1. If helical piles are used, pile manufacturer’s certificate of competency for the installation of helical piles.

2. A list of at least three projects completed within the previous three years wherein the installing contractor installed proposed deep foundations similar in size and

scope to this project. Such list to include names and phone numbers of those project representatives who can verify the installing contractor's participation in those projects.

Evidence of deep foundation designer's competence shall be provided to the City's satisfaction and shall include the following:

1. Registration as a Professional Engineer in the State of Wisconsin.

2. If helical piles are used, recommendation from the pile manufacturer or manufacturer's representative.

B.4 Submittals. At a minimum, submit the following for review by City prior to installation:

1. Qualifications of deep foundation installer and designer.

2. Design calculations stamped by a professional engineer licensed in the State of Wisconsin.

3. Design drawings stamped by a professional engineer licensed in the State of Wisconsin.

B.4.A Micropiles. Submit qualifications, design calculations, design drawings, and product information for proposed deep foundation system.

B.4.B Helical Piles. Submit the following helical pile design documents for review by City prior to installation:

1. Certification from the pile designer that the proposed piles meet the requirements of this specification.

2. Qualifications of the manufacturer, installing contractor and pile designer per Sections B.1 and B.3.

3. Design calculations stamped by a professional engineer licensed in the State of Wisconsin.

4. Product designations for helical lead and extension sections and all ancillary products to be supplied at each helical pile location.

5. Individual pile nominal loads, factors of safety, and required ultimate torque correlated capacities, where applicable.

6. Individual pile loading requirements (if any).

7. Manufacturer's published allowable system capacities for the proposed pile assemblies, including load transfer devices.

8. Calculated mechanical and theoretical geotechnical capacities of the proposed piles.

9. Minimum pile termination torque requirements.

10. Maximum estimated installation torque and allowable installation torque rating of the proposed piles.

11. Minimum and/or maximum embedment lengths or other site-specific embedment depth requirements as may be appropriate for the site soil profiles.

12. Inclination angle and location tolerance requirements.

13. Copies of certified calibration reports for torque measuring equipment to be used on the project. The calibrations shall have been performed within one year of the proposed helical pile installation starting date or as recommended by the equipment manufacturer.

C.1 Construction Methods. Deep foundations shall be installed in accordance with deep foundation designer requirements.

C.1.A Micropiles. A record shall be kept of each pile and shall include as a minimum:

1. Length of pile installed.

2. Depth to rock.

3. Length of rock socket.

4. Theoretical grout volume.

5. Actual grout volume for primary and regrouting.

6. Conditions encountered during drilling.

7. Date and time of installation.

8. Pile number or location description.

C.1.B Helical Piles. Helical piles shall be installed in the locations indicated on the drawings and in accordance with the manufacturer's instructions. Helical piles shall be installed within 3 inches of the indicated drawing location. Helical pile shaft alignment shall be within 2 degrees of the inclination angle shown on the drawings. Depth of piles shall be sufficient to obtain the required working loads in compression, as determined by installation torque readings. Cut off tops of piles and anchor to new foundations with brackets. Top elevation of the helical piles shall be within 2 inches of the design vertical elevation.

C.2 Helical Pile Installation. Installing contractor shall give City minimum 24-hour notice prior to start of pile installation. The helical pile installation technique shall be such that it is consistent with the geotechnical, logistical, environmental, and load carrying conditions of the project. The lead section shall be positioned at the appropriate site survey stake location as determined from the drawings. The helical pile sections shall be advanced into the soil in a continuous manner at a rate of rotation less than 25 revolutions per minute (rpm). Sufficient crowd shall be applied to advance the helical pile sections at a rate approximately equal to the pitch of the helix plate per revolution. The rate of rotation and magnitude of down pressure shall be adjusted for different soil conditions and depths. The magnitude of down pressure shall exceed the amount of torque required to install the pile. Extension sections shall be provided to obtain the required minimum overall length and minimum torsional resistance required.

C.3 Helical Pile Termination Criteria. The minimum final torsional resistance and any required pile length and embedment depth criteria, as specified by the helical pile designer, must be satisfied prior to terminating the pile installation. In the event any helical pile fails to meet these production quality control termination criteria, the following remedies may be suitable, if authorized by the City:

1. If the installation fails to meet the minimum torsional resistance criterion at the minimum embedment length as determined by the pile designer:
    a. Continue the installation to greater depths until the torsional resistance criterion is met, provided that, if a maximum length constraint is applicable, continued installation does not exceed said maximum length constraint, or
    b. Demonstrate acceptable pile performance through pile load or proof testing, or
    c. Replace the pile with one having a different helix plate configuration. The replacement pile must not exceed any applicable maximum embedment length criteria and be embedded to a length that places the last helix plate at least equal to its own diameter beyond the depth of the first helix plate of the replaced pile and meet the minimum torsional resistance criterion or pass load or proof testing.

2. If the torsional resistance during installation reaches the helical pile's allowable torque rating prior to satisfaction of the minimum embedment length criterion:
    a. Terminate the installation at the depth obtained if approved by City, or
    b. Replace the pile with one having a shaft with a higher torsional strength rating. The replacement pile must be installed to satisfy the minimum embedment length criterion. It must also be embedded to a length that places the last helix plate at least equal to its own diameter beyond the depth of the first helix plate of the replaced pile without exceeding any applicable maximum embedment length requirements and it must meet the minimum final torsional resistance criterion, or
    c. Replace the pile with one having a different helix plate configuration. The replacement pile must be installed to satisfy the minimum embedment length criterion. It must also be embedded to a length that places the last helix plate at least equal to its own diameter beyond the depth of the first helix plate of the replaced pile without exceeding any applicable maximum embedment length requirements, and it must meet the minimum final torsional resistance criterion.

3. If the installation reaches a specified maximum embedment length, as determined

by the pile designer, without achieving the minimum torsional resistance criterion:

a. If approved by City, remove and reinstall the pile at a position at least three times the diameter of the largest helix plate away from the initial location. Original embedment length and torsional resistance criteria must be met. The pile repositioning may require the installation of additional helical piles with nominal loads adjusted for these spacing changes, or
b. Demonstrate acceptable pile performance through pile load testing, or
c. De-rate the load capacity of the helical pile based on default or site-specific torque correlation factors and install additional piles as necessary.

4. Replace the pile with one having a different helix plate configuration. The replacement pile must be installed to satisfy the minimum and/or maximum embedment length criterion and it must meet the minimum final torsional resistance criterion.

5. If a helical anchor fails to meet acceptance criteria in a performance or proof test:
   a. Install the anchor to a greater depth and installation torque and re-test provided that, if a maximum embedment length constraint is applicable, continued installation will not exceed said maximum length constraint, or
   b. Replace the anchor with one having more and/or larger helix plates. It must be embedded to a length that places its last helix at least three times its own diameter beyond the position of the first helix of the replaced pile without exceeding any applicable maximum embedment length requirements. This replacement pile must be re-tested, or
   c. If approved by the City, de-rate the load capacity of the helical anchor and install additional anchors. Additional anchors must be installed at positions that are at least three times the diameter of the largest helix away from any other anchor locations and are approved by the City. Anchors installed in cohesive soils shall not be spaced closer than four helix diameters.

6. If a helical pile fails a production quality control criterion as described in this Section or for any reason other than described in this Section, any proposed remedy must be approved by the City prior to initiating its implementation at the project site.

Submit copies of individual helical pile installation records within 24 hours after each installation is completed. Formal copies shall be submitted within 30 days following the completion of the helical pile installation. These installation records shall include the following information:

1. Date and time of installation.

2. Location of helical pile and pile identification number.

3. Installed helical pile model and configuration.

4. Termination depth, pile head depth, and length of installed pile.

5. Actual inclination of the pile.

6. Final torsional resistance.

7. Calculated working load capacity based on final torsional resistance.

8. Comments pertaining to interruptions, obstructions, or other relevant information.

C.3 Helical Pile Field Compression Load Testing. If field compression load testing is done, the installing contractor shall furnish all labor, equipment and pre-production helical piles necessary to accomplish the testing as shown in the approved pile design documentation. Installing Contractor shall apply the specified loads for the specified durations and record the specified data, for the specified number of piles. No deviations from the test plan(s) will be allowed without explicit approval in writing from the City. Pile testing shall be in general accordance with the ASTM D1143 quick test method and the following criteria:

1. Failure criteria shall be in accordance with AC358 and is when plunging occurs or when the net deflection exceeds 10% of the average helix plate diameter, whichever occurs first.

2. An alignment load equal to 5% of the anticipated failure load or maximum anticipated test load may be applied prior to the start of the test to take out slack in the load test frame.

3. Loading increments shall be performed at 5% of the anticipated failure load or maximum anticipated test load with a minimum hold time of 4 minutes at each increment.

4. Upon completion of the maximum test load hold increment, the pile shall be unloaded in 5 to 10 even increments with minimum hold times of 4 minutes at each increment.

Installing contractor shall provide the City copies of raw field test data within 24 hours after the completion of each load test. Formal test reports shall be submitted within 30 days following test completion. Formal test reports shall include the following information:

1. Name of project and Installing Contractor’s representative(s) present during load testing.

2. Name of manufacturer’s representative(s) present during load testing, if any.

3. Name of third party test agency and personnel present during load testing, if any.

4. Date, time, duration and type of the load test.

5. Unique test identifier and map showing the test pile location.

6. Pile model and installation information including shaft type, helix configuration, lead and extension section quantities and lengths, final pile tip depth, installation

date, total test pile length and final termination torque.

7. Calibration records for applicable pile installation and test equipment.

8. Tabulated test results including cumulative pile head movement, loading increments and hold times.

9. Plots showing load versus deflection for each loading/unloading interval.

C.4 Helical Pile Field Pre-Tensioning for Tension Anchors. The installing contractor shall furnish all labor, equipment and materials necessary to accomplish the pre-tensioning as shown in the approved anchor design documentation. Installing contractor shall apply the specified loads for the specified durations and record the specified data, for the specified number of anchors. Anchor testing shall be in general accordance with ASTM D3689. Limit the maximum test load to 1.33 times the design load.

Installing contractor shall provide the City copies of raw field test data or reports within 24 hours after completion of each tension anchor pre-tension. Formal test reports shall be submitted within 30 days following completion of anchors. Formal test reports shall include the following information:

1. Name of project and installing contractor.

2. Name of Installing Contractor’s supervisor during installation.

3. Type of test.

4. Date, time, and duration of test.

5. Unique identifier and location of helical anchor test.

6. Description of calibrated testing equipment and test set-up.

7. Actual helical anchor type and configuration.

8. Steps and duration of each load increment.

9. Cumulative anchor-head movement at each load step.

D. Method of Measurement. The City will measure the Deep Foundations-Designed, Delivered, and Installed bid item by a single lump sum unit.

E. Basis of Payment. The bid item will be paid for at the contract unit price. Payment is full compensation for designing, fabricating, delivering, and installing a complete deep foundation system as specified. Payment includes compression load testing of compression anchors, as needed; and for pre-tensioning of tension anchors. Changes or modifications to foundation type during construction, as required to satisfy the specification requirements, are the responsibility of the contractor.

CGC, Inc.

Construction • Geotechnical
Consulting Engineering/Testing

Attachment A

November 27, 2017
C17051-4

Mr. Michael Sturm
City of Madison – Parks Division
210 Martin Luther King, Jr. Boulevard, Room 104
Madison, WI 53703

Re: Geotechnical Exploration Report - REVISED
Proposed Bridge Replacements
Vilas Park Island
Madison, Wisconsin

Dear Mr. Sturm:

Construction • Geotechnical Consultants, Inc. (CGC) *is providing a revised geotechnical report for the proposed replacement bridge project described above to update the helical pier foundation recommendations. Other parts of the report are not changed.* The purpose of this exploration program was to evaluate the subsurface conditions within the proposed construction area and to provide geotechnical recommendations regarding bridge foundation design and construction. An electronic copy of the *revised* report is being sent to you and Keith Behrend at Strand Associates, and we can provide a paper copy upon request.

PROJECT AND SITE DESCRIPTIONS

We understand that two existing timber-framed pedestrian bridges which connect the Vilas Park Island to the surrounding park areas will be replaced. The timber-pile supported bridges, which have an average clear width of 7.5 ft, are located on the north and south sides of the island in the shallow Vilas Park lagoon. The current bridge spans are approximately 73 ft and 100 ft at the north and south bridges, respectively. It is our understanding that the replacement bridges will be constructed at the same general locations and utilize existing approach paths and retaining walls. The new structures will be pile supported and have cast-in-place concrete decks with clear widths of 12 ft. In order to maintain site aesthetics and minimize site disturbance to the park, it is our understanding that existing timber piles may potentially remain in-place. However, the existing timber piles, which support the abutments, will need to be evaluated and depending on their condition, may need to be supplemented with new piles installed adjacent to the existing timber piles. Alternatively, the existing timber piles may need to be replaced with new steel driven piles constructed following removal of the existing abutments.

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FAX: 608/288-7887

Although the bridges will generally experience lighter loads associated with pedestrian and bicycle traffic, occasional maintenance vehicles and infrequent emergency vehicles may also travel on the bridges. It is our understanding that the load capacity of the new bridges has not been determined, and that the City of Madison is considering designs which accommodate two alternative maximum loading criteria: 1. All City maintenance vehicles, with a maximum load of 50,000 lbs (e.g., tandem-axle dump truck), and 2. All maintenance vehicles excluding the tandem-axle dump truck, with a maximum load of 28,000 lbs (e.g., clam truck).

SUBSURFACE CONDITIONS

Subsurface conditions in the vicinity of the proposed bridge abutments were explored by drilling four Standard Penetration Test (SPT) borings to depths of 24 to 39 ft below existing site grades. Note that the borings were originally planned to extend to depths of 50 to 100 ft, but were stopped short after extending the borings at least 10 to 25 ft into dense to very dense, apparent sandstone bedrock, which has also been encountered in borings completed at Henry Vilas Zoo. The borings were drilled by Badger State Drilling (under subcontract to CGC) on February 13 and 14, 2017 using an ATV-mounted D-50 drill-rig equipped with hollow-stem augers and an automatic SPT hammer. The boring locations and planned depths were selected by CGC in consultation with the City, and were located in the field by CGC based off a map provided by the City. Ground surface elevations at the boring locations were estimated by CGC using an online mapping tool (DCiMap) and should be considered approximate (± 1 ft). The boring locations are shown in plan on the Soil Boring Location Exhibit attached in Appendix A.

The subsurface profile encountered at the boring locations was fairly similar and can be described by the following strata, in descending order:

- 6 to 12 in. of ***topsoil/topsoil fill***; over
- About 3 to 8 ft of very loose to loose ***silt***, with the upper 3 to 5 ft being described as ***organic silt*** in each of the borings; underlain by
- About 5 to 8 ft of clayey soils, including medium stiff to stiff ***lean clay*** and medium dense ***clayey sand***; followed by
- About 5 ft of loose to medium dense ***sand*** with significant silt content in Borings 1 and 2; over
- Apparent dense to very dense ***weathered to increasingly competent sandstone bedrock***.

Groundwater was noted at depths of 3.5 to 6 ft below existing grades, which corresponds to approximately EL 843 to 845 ft, during drilling. As a reference, it is our understanding the normal controlled surface water elevation of nearby Lake Wingra is approximately 847.7 ft. Groundwater levels can be expected to fluctuate with seasonal variations in precipitation, infiltration, evapotranspiration, the water level in Lake Wingra, as well as other factors. A more detailed description of the site soil and groundwater conditions is presented on the Soil Boring Logs attached in Appendix B.

DISCUSSION AND RECOMMENDATIONS

As mentioned above, in order to minimize site disturbance and to maintain site aesthetics, it is our understanding that the quality and structural integrity of the existing timber piles will be evaluated prior to design of the new bridges. Depending on the quality of the existing timber piles, and the capacity required for the new structures, the existing timber piles may remain in-place and be supplemented by new driven piles at each abutment. If the quality of the timber piles are not suitable for re-use, new abutments also supported by driven piles will be constructed.

Subject to the limitations discussed below and based on the subsurface exploration, it is our opinion that the proposed bridges can be supported by either: 1) The existing abutments (pending timber pile evaluation by others indicate satisfactory condition and capacity) supplemented with steel HP pile or helical piers installed through the shallow soft/loose soils to bear within the apparent sandstone bedrock underlying the site; or 2) New HP pile or helical piers supporting abutments constructed following removal of the existing abutments. Note that if the condition and capacity of the existing timber piles are determined to be satisfactory such that the existing abutments may remain and be supplemented with additional new piles constructed adjacent to the existing abutments, helical piers may prove the more feasible foundation support alternative because smaller installation equipment would likely reduce site disturbance and lower anticipated mobilization costs compared to driven piles. *Note that CGC did not complete an analysis of the existing timber piles. We recommend that the structural integrity of the existing timber piles and abutments be reviewed/evaluated by a licensed structural engineer during the design phase of this project.*

The following subsections provide our recommendations for design/construction or driven pile and helical pier bridge foundation support alternatives. *The revised report includes some additional considerations regarding the use of helical pier foundations.* Additional information regarding the conclusions and recommendations presented in this report are discussed in Appendix C,

1. <u>Foundation Recommendation Alternatives</u>

A. Driven Piles

In our opinion, steel HP-section piles (WisDOT *Standard Specifications*, Section 550) will likely be the preferred driven pile type for this project where piles will be driven into bedrock, and these piles are expected to encounter driving refusal (and the required driving resistance) within the very dense weathered sandstone bedrock layer. The top of the bedrock layer was observed at approximately 12 to 22 ft below existing site grades in the borings, with very dense conditions typically encountered below 25 ft in Borings 1 and 2 (north), with slightly shallower very dense conditions observed in Borings 3 and 4 (south), at between about 15 and 20 ft. HP piles driven to practical refusal are expected to drive approximately 5 to 10 ft into the very dense bedrock. Therefore, the estimated depth for HP10X42 piles to develop a maximum ultimate driving resistance of 180 tons is about 30 to 35 ft below existing grades (approximately EL 814 to 819 ft) in Borings 1 and 2, and 20 to 25 ft below existing grade (approximately EL 825 to 830 ft) in Borings 3 and 4. If a higher capacity is required, HP12X53 are anticipated to drive to similar depths as HP10X42 piles, but have a higher maximum driving resistance of 220 tons due to the larger pile section.

The driving criteria for production piling should be established by the modified Gates formula, as discussed in the WisDOT *Bridge Manual*, Chapter 11. Using the modified Gates Formula, HP10x42 piles driven to a resistance of 180 tons will have a *factored* axial compression resistance of 90 tons, assuming a resistance factor (ϕ_{dyn}) of 0.5 (see Section 11.3.1.18.2). Similarly, HP12x53 piles driven to a maximum driving resistance of 220 tons will have a *factored* axial compression resistance of 110 tons. If dynamic or static pile load testing is completed, the resistance factor can be increased (effectively increasing the available load carrying capacity of the piles), but because the piles will be driven to refusal within the sandstone bedrock layer, we do not anticipate that the moderate to significant expense associated with pile load testing will make economic sense for this project. It may also be the case that the bridge loads are light enough such that the maximum load per pile allowed by WisDOT may not be required for this project, as has been the case for other recreational path bridges in Madison. In which case, the actual required driving resistance should be sated on the plans. We can provide additional consultation on load testing, if needed.

Based on our past experience completing drivability analyses and within piles driven to refusal within weathered bedrock, HP10X42 piles can generally be driven to refusal (or the required driving resistance) with an appropriately sized pile hammer without overstressing the piles. However, we recommend that a drivability analysis be completed by the pile driving contractor prior to construction to check that the selected pile type, cushion and hammer are compatible and do not result in the pile being overstressed. We recommend including rock tips on the ends of the HP piles to reduce the potential for damage to piles driven into bedrock: rock tips also assist in the piles driving straighter.

Other pertinent pile design parameters include the following:

- For adequate frost protection, we recommend that the abutment pile caps be founded at least 4 ft below finish grade. A minimum embedment depth of 2.5 ft is recommended for sill abutments, per *WisDOT Bridge Manual*.

- It is recommended that the minimum spacing between individual piles be no less than 2.5 ft or 2.5 times the pile diameter, whichever is greater. WisDOT recommends a maximum pile spacing of 8 ft. During driving, heaving and/or lateral displacements of driven piles may occur during subsequent nearby pile driving operations. Therefore, it is important that horizontal and vertical alignment checks be performed during pile driving operations. Piles that heave more than 0.25 in. vertically must be reseated. However, heaving is generally not a concern with HP piling since they are considered non-displacement piles.

- To minimize pile driving problems, new embankment fill material in the vicinity of the abutments or wing walls should not contain cobbles or boulders.

- Appropriate scour protection should be provided to prevent soil eroding from the below the abutments (and around the piles) in the event of high water events.

B. Helical Piers

Depending on the maximum loading of the new structures, as well as the quality of the existing timber piles at each structure, helical piers may be a more feasible alternative to provide supplemental bridge abutment support. Helical piers are a proprietary product that involves the design and installation of a deep foundation system which transfers loads through upper loose or soft soil zones to bear within more suitable bearing strata. At this site, helical piers can be designed to extend through the existing organic and softer clay soils to bear within the underlying very dense weathered sandstone bedrock, using the soil parameters summarized in Table 1. Helical piers can often be installed using lightweight, mobile equipment (e.g., skidsteer or mini-excavator with special attachments) and are often used in applications where there is limited space available, similar to this site. In addition, because the mobilization cost will likely be much lower than for driven piles (which generally require a crawler crane), helical piers may prove to be the more economical foundation support alternative.

Helical pier capacity will vary depending on the number and size of helices, depth of installation and bearing stratum. Using the computer program HeliCAP V2.0 (produced by Chance/Hubbell) we estimated the capacity of a two-helix (8 in./10 in.) helical pier installed to different depths at the two bridge locations. In general, we estimate that *ultimate* helical pier capacities (in compression) ranging from about 20 to 60 kips (potentially higher with heavy-duty helical piers) can be developed for helical piers installed to depths of 15 to 30 ft below existing site grades, within the higher end in the range of ultimate capacities developed within deeper, very dense sandstone bedrock. Based on the very dense nature of the sandstone bedrock, smaller diameter helix configurations (e.g., 8 and 10 in.) may be required to allow for the pier to penetrate into the strata to achieve required capacity. However, it should be noted that within deeper, more competent sandstone bedrock layers, the helical pier capacity at higher loads will likely be limited by the structural capacity of the helices and not from the geotechnical capacity of the soils/bedrock. *The helical pier depths and capacities should be considered approximate and since helical piers are proprietary (with a multitude of variables), the helical pier installer should determine the helix configuration and depth necessary to satisfy project requirements.*

TABLE 1 - Soil Parameters for Analysis of Helical Foundations
Vilas Park Replacement Bridges
Vilas Park, Madison, WI

Soil Layer		Very Loose to Loose Silt and Medium Stiff to Stiff Clay	Loose to Medium Dense Sand	Dense to Very Dense Apparent Weathered Sandstone Bedrock
Boring 1	**Approximate Depth Below Existin Grade**	**0 to 12 ft**	**12 to 17 ft**	**Below 17 ft**
Boring 2		**0 to 16 ft**	**16 to 22 ft**	**Below 22 ft**
Boring 3		**0 to 14 ft**	**14 to 16.5 ft**	**Below 16.5 ft**
Boring 4		**0 to 8 ft**	**8 to 12 ft**	**Below 12 ft**
Estimated Soil Parameters (2)				
	Angle of internal friction, ϕ	**0**	**30**	**40**
	Cohesion (psf)	**1000**	**0**	**0**
	Total Unit Weight (pcf)	**115**	**120**	**125**
	Submerged unit weight (pcf)	**53**	**58**	**63**
Earth pressure coefficients (2)				
	Active, K_a	**1.0**	**0.33**	**0.22**
	Passive, K_p	**1.0**	**3.0**	**4.6**
Sand Strata				
	Constant of subgrade reaction, n_h (pci) (2, 3)	**-**	**40**	**125**
Clay/Silt Strata				
	Subgrade modulus of reaction, k_h (pci) (constant with depth)	**50 (static)**	**-**	**-**
L-Pile soil type		**Soft Clay**	**Sand**	**Sand**

Notes:

(1) Depths have been generalized to some extent. Refer to boring logs for detailed descriptions at each location.

(2) Values do not include a factor of safety (i.e., FS = 1.0)

(3) Where k_h =(n_h)(x) and x is the depth below ground surface.

The torque of the equipment installing the helical piers is generally correlated with axial capacity, although static load tests can also be completed to confirm the ultimate and allowable capacities. A minimum factor of safety of 2.0 to 2.5 is generally used for helical pier design. If a factor of safety of 2.0 is used to determine the allowable helical pier capacity, we recommend that one static load test be performed per bridge location to confirm the helical pier design satisfies the project requirements. The static load tests should be performed on piers installed to similar installation depths and torques as production piers. Additionally, the torque of each pier should be monitored during installation to document that each pier is torqued to the minimum torque established by the static load tests or empirical correlations to ultimate capacity. If static load tests are not performed, we recommend using a minimum factor of safety of at least 2.5 in determining the allowable capacity, and the installation torque of each pier should be monitored, which is empirically correlated to the ultimate capacity. *Since there are multiple proprietary helical pier systems, it is the responsibility of the contractor to determine that their selected helical pier configuration, installation procedures and termination criteria satisfy the project requirements.*

Other helical pier considerations include the following (**the last two bullet points form the basis of the revised report**):

- The helical pier installer should have provisions to deal with the presence of potential obstructions. If obstructions are encountered, removing obstructions with an excavator would be one method to deal with the obstructions. Using smaller diameter helix configuration may also assist in the installation process but may require deeper piers to develop capacity.

- The shallow organic and softer clay soils have relatively low lateral capacity. As such, round helical pier shafts, which have higher resistance to buckling, are recommended over square shafts. A buckling analysis should be completed to check that the pier shaft has adequate buckling resistance.

- The shallow organic soils may be slightly corrosive to steel, which may result in section loss over time. Therefore, we recommend either increasing the steel section of the shaft to accommodate potential section loss or covering the upper part of the shaft with an anti-corrosion coating to reduce the corrosion potential.

- If the existing timber piles are supplemented with helical piers (or driven piles), consideration should be given to the compatibility of the two different foundation system. Loads will tend to be attracted to the stiffer foundation elements, which may result in localized higher stresses in an abutment of mixed foundation elements. *We recommend that that the structural integrity of the existing piles be reviewed/evaluated by a licensed structural engineer.*

- *Due to the relatively dense nature of the sandstone bedrock, it is possible that the helical piers will "spin-off" during installation when the piers reach the upper portion of bedrock, particularly if the piers are battered. Smaller helical pier lead sections designed to penetrate dense soils and weathered bedrock will likely reduce (though not necessarily eliminate) the risk of "spin off". If "spin-off" occurs, torque correlation cannot be used to estimate pile capacity. For compression piers, it is recommended that if "spin-off" occurs, full scale on-site static load tests be performed to verify capacity. For tension piers, it is recommended that if "spin-off" occurs, a pullout test be performed to check pullout capacity or an alternate foundation type be used.*

- *Due to the possibility of helical pier "spin-off" during installation, other foundation types, such as micropiles, could also be considered to provide bridge abutment support. Provided that they are properly designed, the use of a micropile foundation support system could eliminate the risk that a helical pier foundation support system does not work due to "spin-off" potential of the helical piers during installation due to the dense sandstone bedrock.*

2. <u>Lateral Earth Pressures</u>

In accordance with WisDOT *Bridge Manual* procedures (Sections 12.4 and 12.8), wing walls should be designed as cantilever retaining walls extending from the abutments, and an equivalent fluid pressure of 40 psf per foot of depth and a 2 ft surcharge (240 psf) should be used in design. This recommendation is based on granular fill being used as backfill, as indicated in Section 210 of the WisDOT *Standard Specifications*. It is recommended that procedures for placement and compaction of backfill conform to those outlined in paragraph 207.3.6.2 (Standard Compaction) of the *Standard Specifications*. The wing wall design should include surcharge loads, if applicable.

CONSTRUCTION CONSIDERATIONS

Due to variations in weather, construction methods and other factors, specific construction problems are difficult to predict. Soil related difficulties that could be encountered on the site are discussed below:

- Earthwork construction during the early spring or late fall could be complicated as a result of wet weather and freezing temperatures. During cold weather, exposed subgrades should be protected from freezing before and after pile cap construction. Fill should never be placed while frozen or on frozen ground.

- Excavations extending greater than 4 ft in depth below the existing ground surface should be sloped in accordance with current OSHA standards.

- Based on observations made during the field exploration and depending on final abutment elevations, groundwater will likely be encountered during abutment excavation. Temporary cofferdams and dewatering inside the cofferdams will likely be required so that construction can occur "in the dry" during pile driving or helical pier installation and abutment construction. Additional water accumulating at the base of the excavation should be controlled and removed using pumps operating from filtered sump pits. A layer of clear stone at the bottom of the excavation may be useful for creating a working platform and also assist in dewatering efforts.

RECOMMENDED CONSTRUCTION MONITORING

The level of care exercised during site development will largely determine the quality of the foundations and pavement subgrades on the approaches. To check that earthwork and foundation construction proceeds in accordance with our recommendations, qualified construction inspectors should monitor the following operations:

- Pile driving observations;
- Abutment fill/backfill placement and compaction; and
- Concrete placement.

*** * * * ***

CGC, Inc.

We trust this report addresses your present needs. General limitations regarding the conclusions and opinions presented in this report are discussed in Appendix B. If you have any questions, please contact us.

Sincerely,

CGC, Inc.

Alex J. Bina, P.E.
Staff Engineer

David A. Staab, P.E., LEED AP
Consulting Professional

Encl:	Appendix A -	Subsurface Exploration
	Appendix B -	Soil Boring Location Exhibit
		Logs of Test Borings (4)
		Log of Test Boring-General Notes
		Unified Soil Classification System
	Appendix C -	Document Qualifications

APPENDIX A
SUBSURFACE EXPLORATION

APPENDIX A

SUBSURFACE EXPLORATION

Subsurface conditions in the vicinity of the proposed bridge abutments were explored by drilling four Standard Penetration Test (SPT) borings to depths of 24 to 39 ft below existing site grades. Note that the borings were originally planned to extend to depths of 50 to 100 ft, but were stopped short after extending the borings 10 to 25 ft into the dense to very dense, apparent sandstone bedrock. The borings were drilled by Badger State Drilling (under subcontract to CGC) on February 13 and 14, 2017 using an ATV-mounted D-50 drill-rig equipped with hollow-stem augers and an automatic SPT hammer. The boring locations and planned depths were selected by CGC in consultation with the City, and were located in the field by CGC based off a map provided by the City. Ground surface elevations at the boring locations were estimated by CGC using an online mapping tool (DCiMap) and should be considered approximate (± 1 ft). The boring locations are shown in plan on the Soil Boring Location Exhibit attached in Appendix A.

Soil samples were obtained at 2.5-foot intervals to a depth of 10 ft and at 5-foot intervals thereafter. The soils samples were obtained in general accordance with specifications for standard penetration testing, ASTM D 1586. The specific procedures used for drilling and sampling are described below:

1. Boring Procedures Between Samples

   The boring is extended downward, between samples, by a hollow stem auger. Before encountering groundwater, the drilling method is switched to mud rotary and the hole is advanced with a roller bit.

2. Standard Penetration Test and Split-Barrel Sampling of Soils
   (ASTM Designation: D 1586)

This method consists of driving a 2-inch outside diameter split barrel sampler using a 140-pound weight falling freely through a distance of 30 inches. The sampler is first seated 6 inches into the material to be sampled and then driven 12 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the log of borings and known as the Standard Penetration Resistance. Recovered samples are first classified as to texture by the driller.

Field screening of the soil samples for possible environmental contaminants was not conducted by the drillers, as environmental site assessment activities were not part of CGC's work scope. Upon completion of drilling, the borings were backfilled to satisfy WDNR requirements, and soil samples delivered to our laboratory for visual classification and laboratory testing. The soils were visually classified by a geotechnical engineer using the Unified Soil Classification System. The final logs prepared by the engineer and a description of the Unified Soil Classification System are presented in Appendix B.

APPENDIX B
SOIL BORING LOCATION EXHIBIT
LOGS OF TEST BORINGS (4)
LOG OF TEST BORING – GENERAL NOTES
UNIFIED SOIL CLASSIFICATION SYSTEM

Legend

Denotes Boring Location

Notes
1. Soil borings performed by Badger State Drilling in February 2017
2. Boring locations are approximate.

Scale: Reduced

Date: 2/2017	CGC, Inc.	**Soil Boring Location Exhibit** **Vilas Bridge Replacements** **Madison, WI**
Job No. C17051-4		

CGC Inc.

LOG OF TEST BORING

Project: Vilas Park Bridge Replacements

Location: Madison, Wisconsin

Boring No. 1
Surface Elevation (ft) 849 +/-
Job No. C17051-4
Sheet 1 of 1

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						8 in. TOPSOIL (Probable Fill)					
1		4	M	1		Very Loose, Gray Organic SILT, Trace Sand and Shels (OL - Possible Fill)					
2		14	W	9	5	Stiff, Gray Lean CLAY (CL)	(1.25)	17.6			
3		18	W	1			(1.5)	23.6			
4		16	W	13	10		(1.75)	23.5			
						Loose, Brown Fine to Medium SAND, Some Silt and Gravel, Scattered Cobbles/Boulders (SM)					
5		12	W	8	15						
						Apparent Weathered to Competent, Brown to Light Brown Sandstone Bedrock					
6		14	W	47	20						
7		16	W	33	25						
8		1	W	50/2"	30						
9		2	W	50/2"	35	End Boring at 34 ft Borehole backfilled with bentonite slurry and chips					
					40						
					45						

WATER LEVEL OBSERVATIONS

While Drilling ∇ 3.5' Upon Completion of Drilling ______
Time After Drilling
Depth to Water
Depth to Cave in

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

GENERAL NOTES

Start 2/14/17 End 2/14/17
Driller BSD Chief DB Rig D-50
Logger DD Editor ESF ATV
Drill Method 4.25" HSA to 5'; 3 7/8" RB with Mud to 34'; Autohammer

LOG OF TEST BORING

CGC Inc.

Project: Vilas Park Bridge Replacements

Location: Madison, Wisconsin

Boring No.: 2
Surface Elevation (ft): 849 +/-
Job No.: C17051-4
Sheet 1 of 1

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE No.	TYPE	Rec (in.)	Moist	N	Depth (ft)	VISUAL CLASSIFICATION and Remarks	qu (qa) (tsf)	W	LL	PL	LI
						8 in. TOPSOIL FILL					
1		18	M	2		Very Loose/Soft, Laminated Organic SILT and CLAY (OL - Probable Fill to 2 ft)	(0.5)				
2		18	M/W	2	5	Very Loose, Gray Organic SILT, Little to Some Shells and Sand (OL)					
3		18	W	4		Loose to Very Loose, Gray Sandy SILT (ML)					
4		18	W	8	10	Stiff, Gray Lean CLAY (CL)	(1.25)	22.2			
5		18	W	27	15	Medium Dense, Brown to Gray Clayey Fine to Medium SAND, Some Silt and Gravel, Scattered Cobbles (SC/SM)					
6		18	W	13	20	Medium Dense, Light Brown Silty Fine SAND, Little to Some Gravel (SM)					
7		18	W	33	25	Apparent Weathered to Competent, Brown to Light Brown Sandstone Bedrock					
8		3	W	50/3"	30	End Boring at 29 ft Borehole backfilled with bentonite slurry and chips					
					35						
					40						
					45						

WATER LEVEL OBSERVATIONS

While Drilling ∇ 6.0'   Upon Completion of Drilling ______
Time After Drilling ______
Depth to Water ______
Depth to Cave in ______

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

GENERAL NOTES

Start 2/13/17 End 2/13/17
Driller BSD Chief DB Rig D-50
Logger DD Editor ESF ATV
Drill Method 4.25" HSA to 5'; 3 7/8" RB with Mud to 29'; Autohammer

CGC Inc.

LOG OF TEST BORING

Project: Vilas Park Bridge Replacements

Location: Madison, Wisconsin

Boring No. 3
Surface Elevation (ft) 849 +/-
Job No. C17051-4
Sheet 1 of 1

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE No.	TYPE	Rec (in.)	Moist	N	Depth (ft)	VISUAL CLASSIFICATION and Remarks	qu (qa) (tsf)	W	LL	PL	LI
						6 in. TOPSOIL (Probable Fill)					
1		14	M	6		Loose to Very Loose, Gray Organic SILT, Little to Some Shells and Sand (OL) (Possible Fill to 3 ft)					
2		16	W	0	5			102.6			7.3
3		15	W	4		Loose to Very Loose, Gray Sandy SILT, Trace Shell Fragments (ML)		16.0			
4		18	W	10	10	Medium Stiff to Stiff/Medium Dense to Loose, Laminated Lean CLAY and SILT, Trace Sand (ML/CL)	(1.0)	21.8			
5		12	W	11	15	Medium Dense, Reddish-Brown Clayey Fine to Medium SAND, Some Silt and Gravel, Scattered Cobbles (SC/SM)	(1.0)				
6		14	W	78/10"	20	Apparent Weathered to Competent, Brown to Light Brown Sandstone Bedrock					
7		4	W	50/5"	25						
8		2	W	50/3"	30						
9		2	W	50/4"	35						
10		0	W	50/1"	40	End Boring at 39 ft Borehole backfilled with bentonite slurry and chips					
					45						

WATER LEVEL OBSERVATIONS

While Drilling ▽ 3.5' Upon Completion of Drilling ______
Time After Drilling
Depth to Water
Depth to Cave in

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

GENERAL NOTES

Start 2/13/17 End 2/13/17
Driller BSD Chief DB Rig D-50
Logger DD Editor ESF ATV
Drill Method 4.25" HSA to 5'; 3 7/8" RB with Mud to 40'; Autohammer

CGC Inc.

LOG OF TEST BORING

Project: Vilas Park Bridge Replacements

Location: Madison, Wisconsin

Boring No. 4
Surface Elevation (ft) 850 +/-
Job No. C17051-4
Sheet 1 of 1

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE No.	TYPE	Rec (in.)	Moist	N	Depth (ft)	VISUAL CLASSIFICATION and Remarks	qu (qa) (tsf)	W	LL	PL	LI
						12 in. TOPSOIL FILL					
1		10	M	4		FILL: Stiff, Brown Clay with Traces Sand and Topsoil	(1.5)				
2		14	W	2	▽ 5	Very Loose, Gray Organic SILT, Little to Some Shells and Sand (OL)		90.3			6.3
3		16	W	9		Stiff, Gray Lean CLAY (CL)		22.3			
4		16	W	11	10	Medium Dense, Brown to Gray Clayey Fine to Medium SAND, Some Silt and Gravel, Scattered Cobbles (SC/SM)					
5		4	W	50/5"	15	Apparent Weathered to Competent, Brown to Light Brown Sandstone Bedrock					
6		2	W	50/3"	20						
7		1	W	50/2"	25	End Boring at 24 ft due to rotary-bit refusal on presumed bedrock Borehole backfilled with bentonite slurry and chips					
					30						
					35						
					40						
					45						

WATER LEVEL OBSERVATIONS

While Drilling ▽ 3.5' Upon Completion of Drilling ______
Time After Drilling ______
Depth to Water ______ ▼
Depth to Cave in ______

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

GENERAL NOTES

Start 2/14/17 End 2/14/17
Driller BSD Chief DB Rig D-50
Logger DD Editor ESF ATV
Drill Method 4.25" HSA to 5'; 3 7/8" RB with Mud to 24'; Autohammer

CGC, Inc.

LOG OF TEST BORING

General Notes

DESCRIPTIVE SOIL CLASSIFICATION

Grain Size Terminology

Soil Fraction	Particle Size	U.S. Standard Sieve Size
Boulders	Larger than 12"	Larger than 12"
Cobbles	3" to 12"	3" to 12"
Gravel: Coarse	¾" to 3"	¾" to 3"
Fine	4.76 mm to ¾"	#4 to ¾"
Sand: Coarse	2.00 mm to 4.76 mm	#10 to #4
Medium	0.42 to mm to 2.00 mm	#40 to #10
Fine	0.074 mm to 0.42 mm	#200 to #40
Silt	0.005 mm to 0.074 mm	Smaller than #200
Clay	Smaller than 0.005 mm	Smaller than #200

Plasticity characteristics differentiate between silt and clay.

General Terminology

Physical Characteristics
Color, moisture, grain shape, fineness, etc.
Major Constituents
Clay, silt, sand, gravel
Structure
Laminated, varved, fibrous, stratified, cemented, fissured, etc.
Geologic Origin
Glacial, alluvial, eolian, residual, etc.

Relative Density

Term	"N" Value
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

Relative Proportions Of Cohesionless Soils

Proportional Term	Defining Range by Percentage of Weight
Trace	0% - 5%
Little	5% - 12%
Some	12% - 35%
And	35% - 50%

Consistency

Term	q_u-tons/sq. ft
Very Soft	0.0 to 0.25
Soft	0.25 to 0.50
Medium	0.50 to 1.0
Stiff	1.0 to 2.0
Very Stiff	2.0 to 4.0
Hard	Over 4.0

Organic Content by Combustion Method

Soil Description	Loss on Ignition
Non Organic	Less than 4%
Organic Silt/Clay	4 – 12%
Sedimentary Peat	12% - 50%
Fibrous and Woody Peat	More than 50%

Plasticity

Term	Plastic Index
None to Slight	0 - 4
Slight	5 - 7
Medium	8 - 22
High to Very High	Over 22

The penetration resistance, N, is the summation of the number of blows required to effect two successive 6" penetrations of the 2" split-barrel sampler. The sampler is driven with a 140 lb. weight falling 30" and is seated to a depth of 6" before commencing the standard penetration test.

SYMBOLS

Drilling and Sampling

CS – Continuous Sampling
RC – Rock Coring: Size AW, BW, NW, 2"W
RQD – Rock Quality Designation
RB – Rock Bit/Roller Bit
FT – Fish Tail
DC – Drove Casing
C – Casing: Size 2 ½", NW, 4", HW
CW – Clear Water
DM – Drilling Mud
HSA – Hollow Stem Auger
FA – Flight Auger
HA – Hand Auger
COA – Clean-Out Auger
SS - 2" Dia. Split-Barrel Sample
2ST – 2" Dia. Thin-Walled Tube Sample
3ST – 3" Dia. Thin-Walled Tube Sample
PT – 3" Dia. Piston Tube Sample
AS – Auger Sample
WS – Wash Sample
PTS – Peat Sample
PS – Pitcher Sample
NR – No Recovery
S – Sounding
PMT – Borehole Pressuremeter Test
VS – Vane Shear Test
WPT – Water Pressure Test

Laboratory Tests

q_a – Penetrometer Reading, tons/sq ft
q_a – Unconfined Strength, tons/sq ft
W – Moisture Content, %
LL – Liquid Limit, %
PL – Plastic Limit, %
SL – Shrinkage Limit, %
LI – Loss on Ignition
D – Dry Unit Weight, lbs/cu ft
pH – Measure of Soil Alkalinity or Acidity
FS – Free Swell, %

Water Level Measurement

▽- Water Level at Time Shown
NW – No Water Encountered
WD – While Drilling
BCR – Before Casing Removal
ACR – After Casing Removal
CW – Cave and Wet
CM – Caved and Moist

Note: Water level measurements shown on the boring logs represent conditions at the time indicated and may not reflect static levels, especially in cohesive soils.

CGC, Inc.

Madison - Milwaukee

Unified Soil Classification System

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

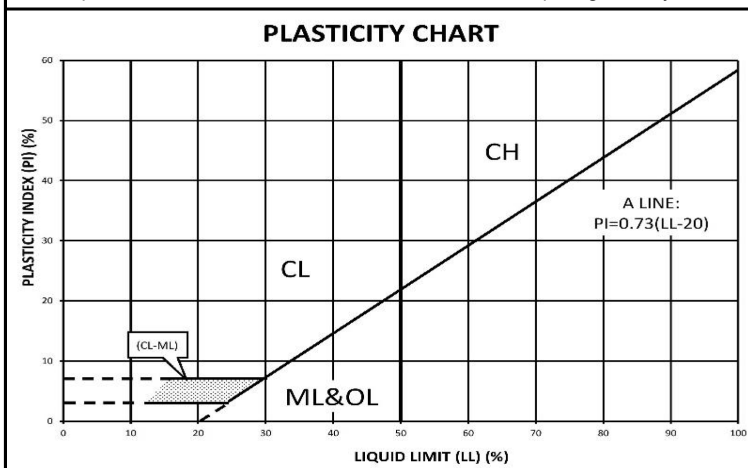
COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size)			
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravels (Less than 5% fines)		
		GW	Well-graded gravels, gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)		
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sands (Less than 5% fines)		
		SW	Well-graded sands, gravelly sands, little or no fines
		SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)		
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)			
SILTS AND CLAYS Liquid limit less than 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils

LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limts below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limts above "A" line or P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for GW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
SC	Atterberg limits above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent .... GW, GP, SW, SP
More than 12 percent .... GM, GC, SM, SC
5 to 12 percent .... Borderline cases requiring dual symbols



APPENDIX C

DOCUMENT QUALIFICATIONS

APPENDIX C
DOCUMENT QUALIFICATIONS

I. GENERAL RECOMMENDATIONS/LIMITATIONS

CGC, Inc. should be provided the opportunity for a general review of the final design and specifications to confirm that earthwork and foundation requirements have been properly interpreted in the design and specifications. CGC should be retained to provide soil engineering services during excavation and subgrade preparation. This will allow us to observe that construction proceeds in compliance with the design concepts, specifications and recommendations, and also will allow design changes to be made in the event that subsurface conditions differ from those anticipated prior to the start of construction. CGC does not assume responsibility for compliance with the recommendations in this report unless we are retained to provide construction testing and observation services.

This report has been prepared in accordance with generally accepted soil and foundation engineering practices and no other warranties are expressed or implied. The opinions and recommendations submitted in this report are based on interpretation of the subsurface information revealed by the test borings indicated on the location plan. The report does not reflect potential variations in subsurface conditions between or beyond these borings. Therefore, variations in soil conditions can be expected between the boring locations and fluctuations of groundwater levels may occur with time. The nature and extent of the variations may not become evident until construction.

II. IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes. While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. *No one except you* should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one - not even you* - should apply the report for any purpose or project except the one originally contemplated.

READ THE FULL REPORT

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, *do not rely on a geotechnical engineering report* that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes - even minor ones - and request an assessment of their impact. *CGC cannot accept responsibility or liability for problems that occur because our reports do not consider developments of which we were not informed.*

SUBSURFACE CONDITIONS CAN CHANGE

A geotechnical engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

MOST GEOTECHNICAL FINDINGS ARE PROFESSIONAL OPINION

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgement to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ - sometimes significantly - from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A REPORT'S RECOMMENDATIONS ARE NOT FINAL

Do not over-rely on the confirmation-dependent recommendations included in your report. *Those confirmation-dependent recommendations are not final*, because geotechnical engineers develop them principally from judgement and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *CGC cannot assume responsibility or liability for the report's confirmation-dependent recommendations if we do not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical engineering report. Confront that risk by having CGC participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

DO NOT REDRAW THE ENGINEER'S LOGS

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

GIVE CONSTRUCTORS A COMPLETE REPORT AND GUIDANCE

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

READ RESPONSIBILITY PROVISIONS CLOSELY

Some clients, design professionals, and constructors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineer's responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

ENVIRONMENTAL CONCERNS ARE NOT COVERED

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

OBTAIN PROFESSIONAL ASSISTANCE TO DEAL WITH MOLD

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

RELY ON YOUR GEOTECHNICAL ENGINEER FOR ADDITIONAL ASSISTANCE

Membership in the Geotechnical Business Council (GBC) of Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with CGC, a member of GBC, for more information.

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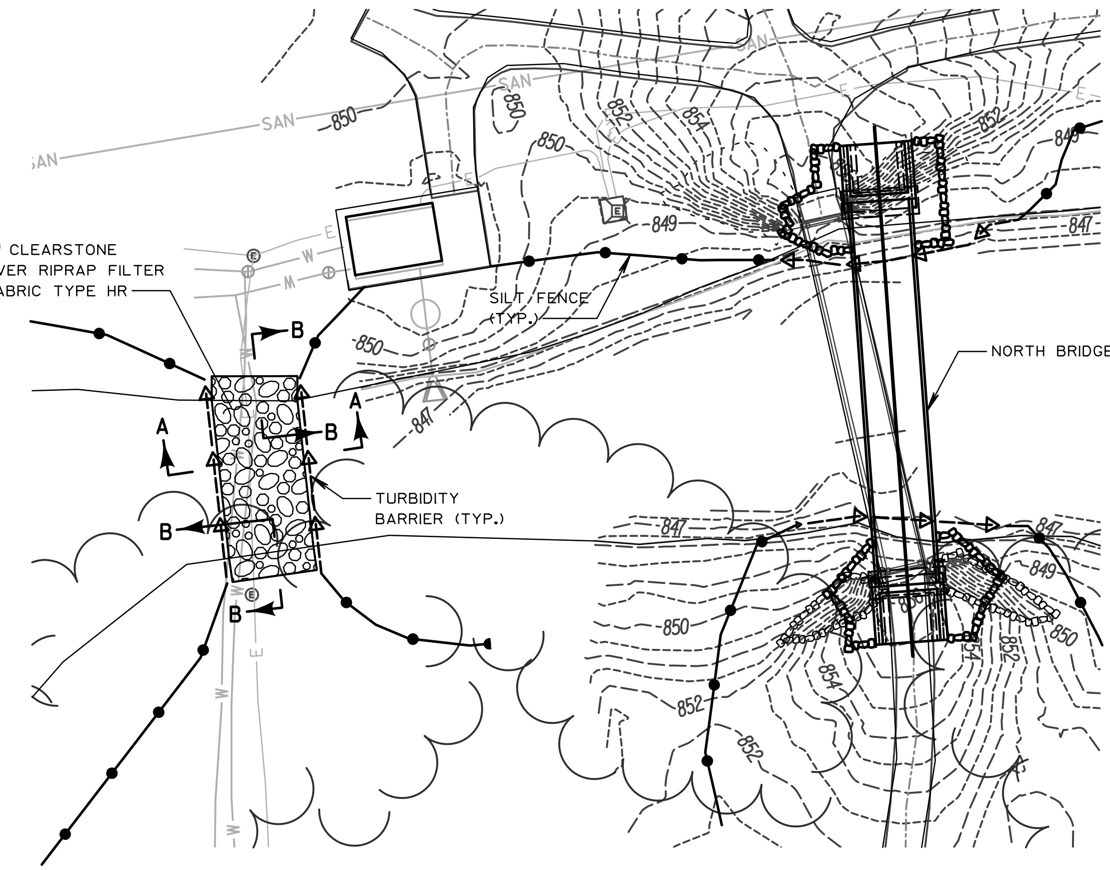
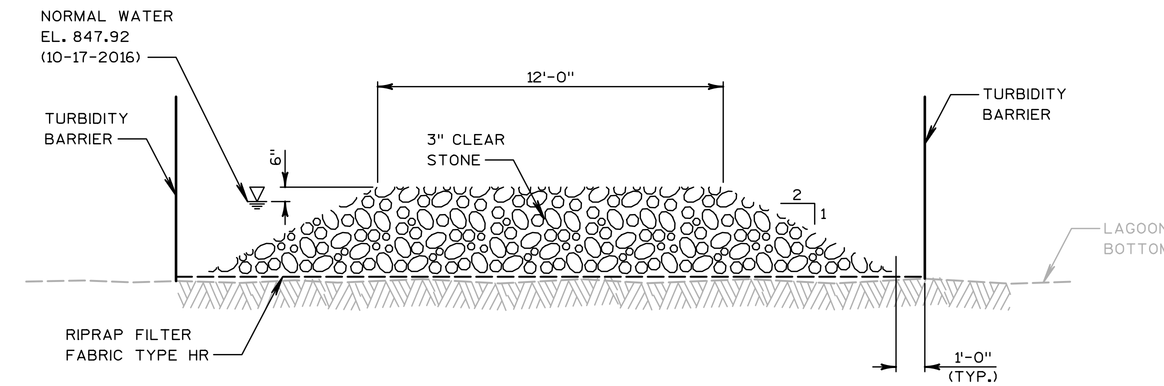
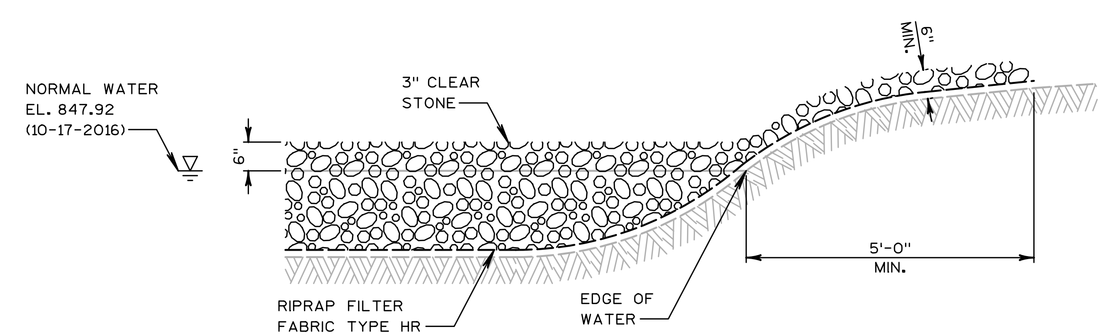
Geotechnical Business Council
of the Geoprofessional Business Association
8811 Colesville Road, Suite G 106
Silver Spring, MD 20910



TEMP FORD CROSSING PLAN



SECTION A-A



SECTION B-B

NO.	REVISIONS	DATE:
1	Addendum 1	11-27-17

TEMP. FORD CROSSING DETAILS

VILAS PARK - LAGOON BRIDGE REPLACEMENTS

CITY OF MADISON PARKS DIVISION
MADISON, WISCONSIN

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