COLLIN COUNTY

Office of the Purchasing Agent
2300 Bloomdale Road
Suite 3160
McKinney, Texas 75071
www.collincountytx.gov

ADDENDUM No. Two (2)

Construction, Collin County Walking Trail, Pedestrian Bridge Placement
IFB No. 2013-094

Effective: May 7, 2013

You are hereby directed to make changes to the Invitation for Bid in accordance with the attached information:

ADD: GeoTech Report

ADD: Project Access Locations

ADD: Questions and Answers to Date

Please note all other terms, conditions, specifications drawings, etc. remain unchanged.

Sincerely,
Michalyn Rains CPPO, CPPB
Purchasing Agent

GEOTECHNICAL EXPLORATION
on

HIKE AND BIKE TRAIL
AND PEDESTRIAN BRIDGE
COLLIN COUNTY ADMINISTRATION COMPLEX
Off Bloomdale Road
McKinney, Texas
ALPHA Report No. G100667

Prepared for:

CROSS ENGINEERING CONSULTANTS, INC.
106 W. Louisiana
McKinney, Texas 75069
Attention: Mr. Bill Perman
August 6, 2010

Prepared By:

ALPHA TESTING, INC.
2209 Wisconsin Street, Suite 100
Dallas, Texas 75229

August 6, 2010

Cross Engineering Consultants, Inc.
106 W. Louisiana
McKinney, Texas 75069
Attention: Mr. Bill Perman

Re: Geotechnical Exploration
Hike and Bike Trail and
Pedestrian Bridge
Collin County Administration Complex
Off Bloomdale Road
McKinney, Texas
ALPHA Report No. G100667

Attached is the report of the geotechnical exploration performed for the project referenced above. This study was authorized by Jon David Cross on June 17, 2010 and performed in accordance with ALPHA Proposal No. 26204 Revised dated June 16, 2010.

This report contains results of field explorations and laboratory testing and an engineering interpretation of these with respect to available project characteristics. The results and analyses were used to develop recommendations to aid design and construction of bridge foundations and pavement.

ALPHA TESTING, INC. appreciates the opportunity to be of service on this project. If we can be of further assistance, such as providing materials testing services during construction, please contact our office.

Sincerely,

ALPHA TESTING, INC.

Richard J White for/

David E. Schledorn, P.E.
Senior Geotechnical Engineer

STATE OF TEXAS
THEODORE A. JANISH
82418
LICENSED
PROFESSIONAL ENGINEER
8/6/10

Theodore A. (Tony) Janish, P.E.
Principal

DES/TAJ/des
Copies: (2) Client

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On

ALPHA REPORT NO. G100667

1.0 PURPOSE AND SCOPE

The purpose of this geotechnical exploration is to evaluate some of the physical and engineering properties of subsurface materials at the subject site with respect to formulation of appropriate geotechnical design parameters for the proposed bridge foundations and pavement. The field exploration was accomplished by securing subsurface samples from widely spaced test borings performed across the expanse of the site. Engineering analyses were performed from results of the field exploration and results of laboratory tests performed on representative samples.

Also included are general comments pertaining to reasonably anticipated construction problems and recommendations concerning earthwork and quality control testing during construction. This information can be used to evaluate subsurface conditions and to aid in ascertaining construction meets project specifications.

Recommendations provided in this report were developed from information obtained in test borings depicting subsurface conditions only at the specific boring locations and at the particular time designated on the logs. Subsurface conditions at other locations may differ from those observed at the boring locations. The scope of work may not fully define the variability of subsurface materials that is present on the site.

The nature and extent of variations between borings may not become evident until construction. If significant variations then appear evident, our office should be contacted to re-evaluate our recommendations after performing on-site observations and possibly other tests.

2.0 PROJECT CHARACTERISTICS

It is proposed to construct a new hike and bike trail and pedestrian bridge located generally northeast of the intersection of Bloomfield Road and Community Boulevard in McKinney, Texas. A site plan illustrating the general outline of the site is provided as Figure 1, the Boring Location Plan, in the Appendix of this report. At the time the field exploration was performed the site was grassy with some trees.

According to the Plan and Profile sheets prepared by Cross Engineering Consultants (Project No. 10006; Date: 5/20/10; Sheet Nos. C2, C3 and C4 of 6), the site slopes generally from east and west downward toward the creek (generally extending in a north/south direction at the eastern portion of the site) with a maximum change in surface elevation of about 38 ft (Elev. 633 to 595). Based on the proposed top of pavement elevation for the hike and bike trail and the existing topography at this site, cuts and fills of up to about 2 ft will be required except at the east side of the bridge where up to about 4 ft of fill will be required. The pedestrian bridge is planned at Elev. 600.50. It is anticipated the new pedestrian bridge structure will be supported using drilled pier foundations. We understand maximum pier loads will be 40 kips.

3.0 FIELD EXPLORATION

Subsurface conditions on the site were explored by drilling six (6) test borings in general accordance with ASTM D 420 to a depth of up to 30 ft using standard rotary drilling equipment. Borings 1 and 2 are associated with the pedestrian bridge and extended to a depth of 30 ft. Borings 3 through 6 are associated with the hike and bike trail and extended to a depth of 10 ft. The approximate locations of each test boring are shown on the Boring Location Plan, Figure 1, enclosed in the Appendix of this report. Details of drilling and sampling operations are briefly summarized in Methods of Field Exploration, Section A-1 of the Appendix.

Subsurface types encountered during the field exploration are presented on Log of Boring sheets included in the Appendix of this report. The boring logs contain our Field Technician's and Engineer's interpretation of conditions believed to exist between actual samples retrieved. Therefore, these boring logs contain both factual and interpretive information. Lines delineating subsurface strata on the boring logs are approximate and the actual transition between strata may be gradual.

4.0 LABORATORY TESTS

Selected samples of the subsurface materials were tested in the laboratory to evaluate their engineering properties as a basis in providing recommendations for foundation design and earthwork construction. A brief description of testing procedures used in the laboratory can be found in Methods of Laboratory Testing, Section B-1 of the Appendix. Individual test results are presented on Log of Boring sheets or on summary data sheets also enclosed in the Appendix.

5.0 GENERAL SUBSURFACE CONDITIONS

Within the 30-ft maximum depth explored at the site, subsurface materials consist generally of clay and silty clay. At Boring 5, tan weathered shaly limestone was encountered at a depth of about 5 ft and extended to boring termination. The upper 2 ft of clay encountered in Boring 2 is considered to be possible fill and the clays encountered at Boring 5 are considered fill based on visual examination of the samples obtained.

Most of the subsurface materials are relatively impermeable and are anticipated to have a slow response to water movement. Therefore, several days of observation will be required to evaluate actual groundwater levels within the depths explored. Also, the groundwater level at the site is anticipated to fluctuate seasonally depending on the amount of rainfall, prevailing weather conditions and subsurface drainage characteristics.

During field explorations, groundwater was noted on drilling tools and in the open boreholes upon completion at depths of about 19 ft and 16 ft at Borings 1 and 2, respectively. Groundwater was not encountered at the other shallower borings. It is common to detect shallower seasonal groundwater within the fill soils, from natural fractures within the clayey matrix, near the soil/rock (shaly limestone) interface or from fractures in the rock, particularly during or after periods of precipitation. If more detailed groundwater information is required, monitoring wells or piezometers can be installed.

Further details concerning subsurface materials and conditions encountered can be obtained from the Log of Boring sheets provided in the Appendix of this report.

6.0 DESIGN RECOMMENDATIONS

The following design recommendations were developed on the basis of the previously described Project Characteristics (Section 2.0) and General Subsurface Conditions (Section 5.0). If project criteria should change our office should conduct a review to determine if modifications to the recommendations are required. Further, it is recommended our office be provided with a copy of the final plans and specifications for review prior to construction.

6.1 Drilled and Underreamed Pier Foundation System

Our findings indicate the proposed pedestrian bridge could be supported using a system of drilled and underreamed piers. It is recommended these piers bear in very stiff to hard clay at least 15 ft below existing grade or final grade, whichever is deeper. Some field adjustments in the depth of the piers may be required in some areas to maintain the bottom of the piers above groundwater seepage encountered near the bearing depth in Borings 1 and 2. Adjustments in the depths of the piers should be observed in the field by ALPHA personnel. Piers can be dimensioned using a net allowable end bearing pressure of 4 kips per sq ft and no skin friction component of resistance. The above bearing capacity contains a factor of safety of at least 3 considering a general bearing capacity failure. Normal elastic settlement of piers under loading is estimated to be less than about 1 inch.

Each pier shaft should be reinforced with suitable tension steel over its entire length to adequately resist potential uplift (tensile) forces due to potential soil swell (soil-to-pier adhesion) along the shaft, from post construction heave and other uplift forces applied by structural loadings. The magnitude of uplift adhesion due to soil swell along the pier shaft cannot be defined accurately and can vary according to the actual in-place moisture content of the soils during construction. It is estimated this uplift adhesion will not exceed about 1.8 kips per sq ft. This soil adhesion is approximated to act uniformly over the upper 12 ft of the pier shaft.

The uplift force due to swelling of active clays should be resisted by the underreamed portion of the pier. The underreamed portion should be at least 2 and not exceeding 3 times the diameter of the shaft. The minimum clear spacing between edges of adjacent piers should be at least one (1) underream diameter, based on the larger underream.

Lateral loads imposed on pier foundations for the structures can be resisted by passive resistance in the underlying clays. An *allowable uniform passive resistance* of 2 ksf can be considered for the clay soils. This allowable passive pressure value has a factor of safety of at least 2. Further, the above lateral resistance value should be applied uniformly over the projected face of the drilled pier shaft. The lateral resistance of the portion of the pier shafts within 6 ft of final grade should be neglected.

All grade beams or pier caps should be formed and not cast in earthen trenches. Grade beams and pier caps should be formed with a nominal 6-inch void at the bottom. Commercially available cardboard box forms (cartons) are made for this purpose. The cardboard cartons should extend the full length and width of the grade beams. Prior to concrete placement, the cartons should be inspected to verify they are firm, properly placed, and capable of supporting wet concrete. Some type of permanent soil retainer, such as pre-cast concrete panels, must be provided to prevent soils adjacent to grade beams or pier caps from sloughing into the void space at the bottom of the structural elements. Additionally, backfill soils placed adjacent to grade beams must be compacted as outlined in Section 7.3 of this report.

6.2 Lateral Earth Pressure

Abutment walls and associated wing walls should be designed to resist the expected lateral earth pressures. The magnitude of lateral earth pressure against abutment walls is dependent on the method of backfill placement, type of backfill soil, drainage provisions, and type of wall (rigid or yielding) after placement of the backfill. Experience demonstrates when a wall is held rigidly against horizontal movement (restrained at the top), the lateral pressure (at-rest lateral earth pressure) against the wall is greater than the normally assumed active pressure. Yielding walls (rotation at the top of the wall on the order of 0.1 to 0.4 percent of the wall height) can be designed for active earth pressures (k_a) but rigid walls should be designed for higher at-rest lateral earth pressures (k_o). Walls should be designed using the equivalent fluid pressures provided in Tables A and B, considering a triangular distribution and assuming either a horizontal ground surface extending backward from the top of the wall (Table A) or the ground surface extending backward from the top of the wall that is sloped upward not steeper than 4 (horizontal) to 1 (vertical) – Table B. The equivalent fluid pressures provided do not include a factor of safety.

TABLE A LATERAL EARTH PRESSURE			
Material	Condition	Equivalent Fluid Pressure, pcf	
		Drained	Undrained including Hydrostatic Pressure
Free Draining Granular Soil $\Phi=32^{\circ}$, γ_T =125 pcf	At-Rest, k_o=0.47	59	92
	Active, k_a=0.31	39	82
On-site Clayey Soil, $\Phi=12^{\circ}$, γ_T =125 pcf	At-Rest, k_o=0.8	--	112
	Active, k_a=0.7	--	106

TABLE B LATERAL EARTH PRESSURE Ground surface extending backward from the top of the wall is sloped upward at a maximum 4 (horizontal) to 1 (vertical) or flatter			
Material	Condition	Equivalent Fluid Pressure, pcf	
		Drained	Undrained including Hydrostatic Pressure
Free Draining Granular Soil $\Phi=32^{\circ}$, γ_T =125 pcf	At-Rest, k_o=0.6	78	103
	Active, k_a=0.36	47	87
On-site Clayey Soil, $\Phi=12^{\circ}$, γ_T =125 pcf	At-Rest, k_o=1.0	--	125
	Active, k_a=0.85	--	116

Note: Free Draining Granular Backfill
This material should be a non-plastic, clean, relatively well-graded granular soil consisting of either a sand or a sand and gravel mixture (less than 5 percent finer than the No. 200 sieve size). To reduce surface water seepage into the free draining backfill, the top 1-ft of the backfill should consist of on-site clay soil with a plasticity index of at least 25.

The free draining granular backfill (if used) should extend outward at least 2 ft from the base of the wall and then extend upward on a 1 (horizontal) to 2 (vertical) slope. The free draining granular backfill should be separated from the adjacent native soils using a filter fabric (Mirafi 140N, or equivalent) to prevent intrusion of native soils into the free draining granular backfill.

Complete drainage of the free draining granular backfill could be provided to prevent the development of hydrostatic pressures behind the wall. A typical drainage system could consist of perforated (slotted) PVC pipes placed in filter trenches excavated parallel to the base of the walls for their entire length. Septic field drain pipe is **not** acceptable for this purpose. The drain pipes should be positioned at a depth lower than the bottom elevation of the wall and should also be wrapped with filter fabric (Mirafi 140N, or equivalent). A drainage system is beneficial regardless of the type of backfill used behind the wall. As a minimum weep holes should be provided for freestanding walls.

The effects of surcharge loading at the surface near the retaining walls must also be considered, including sloped backfill steeper than 1 vertical to 5 horizontal. The surcharge load should be multiplied by the applicable coefficient of earth pressure (from the table above), with the resulting pressure applied as a uniform lateral pressure over the full height of the wall.

Lightweight, hand-controlled vibrating plate compactors are recommended for compaction of backfill adjacent to walls to reduce the possibility of increases in lateral pressures due to over-compaction. Heavy compaction equipment should not be operated near the walls. Also, compaction of backfill soils behind walls should not exceed 100 percent standard Proctor maximum dry density (ASTM D 698) to further limit lateral earth pressures against walls.

6.3 Foundation Support For Abutment/Wing Walls

Drilled and underreamed piers as discussed earlier in Section 6.1 should be used to support any abutment or wing walls.

6.4 Seismic Considerations

The Site Class for seismic design is based on several factors that include soil profile (soil or rock), shear wave velocity, and strength, averaged over a depth of 100 ft. Since our boring did not extend to 100-foot depths, we based our determinations on the assumption that the subsurface materials below the bottom of the boring were similar to those encountered at the termination depths of the deepest boring. Based on Section 1613.5.2 of the 2006 International Building Code, we recommend using Site Class D (stiff soil profile) for seismic design at this site.

6.5 Concrete Pavement (Hike and Bike Trail)

We understand the proposed hike and bike trail will be constructed with Portland cement concrete (PCC) pavement. Clayey soils encountered at the borings along the planned hike and bike trail, or similar materials used as fill for site grading will probably constitute the subgrade for the planned hike and bike trail. To permit correlation between information from the test borings and actual subgrade conditions exposed during construction, a qualified Geotechnical Engineer should be retained to provide subgrade monitoring and testing during construction. If there is any change in project criteria, the recommendations contained in this report should be reviewed by our office.

After final subgrade elevation along the hike and bike trail is achieved, subgrade preparation as discussed in Section 7.1 of this report should be followed. After subgrade preparation, the exposed surface of the pavement subgrade soils should be scarified to a depth of at least 6 inches. Then, the scarified soils should be compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698) and within the range of 1 percentage point below to 3 percentage points above the material's optimum moisture content.

Pavement can then consist of at least 5 inches of adequately reinforced concrete. The above pavement section should be capable of sustaining wheel loads of occasional emergency, patrol and light-weight maintenance vehicles. Portland-cement concrete should have a minimum compressive strength of 3,000 lbs per sq inch (psi) at 28 days. Concrete should be designed with 5 $\pm$ 1 percent entrained air. Joints in concrete paving should not exceed 15 ft. Reinforcing steel should consist of No. 3 bars placed at 18 inches on-center in two directions.

6.6 Drainage

Adequate drainage should be provided to reduce seasonal variations in moisture content of foundation soils. All pavement and embankments should be sloped to prevent ponding of water around the pier foundations. Final grades within 10 ft of the embankment, pavement and piers should be adjusted to slope away at a minimum slope of 1 percent.

7.0 GENERAL CONSTRUCTION PROCEDURES AND RECOMMENDATIONS

Variations in subsurface conditions could be encountered during construction. To permit correlation between test boring data and actual subsurface conditions encountered during construction, it is recommended a registered Professional Engineering firm be retained to observe construction procedures and materials.

Some construction problems, particularly degree or magnitude, cannot be anticipated until the course of construction. The recommendations offered in the following paragraphs are intended not to limit or preclude other conceivable solutions, but rather to provide our observations based on our experience and understanding of the project characteristics and subsurface conditions encountered in the borings.

7.1 Site Preparation and Grading

All areas supporting the pavement and areas to receive new fill should be properly prepared.

After completion of the necessary stripping, clearing, and excavating and prior to placing any required fill, the exposed subgrade should be carefully evaluated by probing and testing. Any undesirable material (organic material, wet, soft, or loose soil) still in place should be removed.

The exposed subgrade should be further evaluated by proof-rolling with a heavy pneumatic tired roller, loaded dump truck or similar equipment weighing approximately 10 tons to check for pockets of soft or loose material hidden beneath a thin crust of possibly better soil.

Proof-rolling procedures should be observed routinely by a Professional Engineer, or his designated representative.

Any undesirable material (organic material, wet, soft, or loose soil) exposed should be removed and replaced with well-compacted material as outlined in Section 7.3.

Prior to placement of any fill, the exposed subgrade should then be scarified to a minimum depth of 6 inches and recompacted as outlined in Section 7.3.

The root systems from existing trees at this site will have dried and desiccated the surrounding clay soils, resulting in soil with near-maximum swell potential. Clay soils surrounding tree root mats in flatwork areas should be removed to a depth of 3 ft and compacted in-place with moisture and density control as described in Section 7.3 of this report, below.

If fill is to be placed on existing slopes (natural or constructed) steeper than six horizontal to one vertical (6:1), the fill materials should be benched into the existing slopes in such a manner as to provide a minimum bench width of five (5) feet. This should provide a good contact between the existing soils and new fill materials, reduce potential sliding planes and allow relatively horizontal lift placements.

Slope stability analysis of embankments (natural or constructed) was not within the scope of this study.

All excavations should be braced or cut at stable slopes in accordance with Occupational Safety and Health Administration (OSHA) requirements.

Due to the nature of clayey soils found near the surface at the borings, traffic of heavy equipment (including heavy compaction equipment) may create pumping and general deterioration of shallow soils. Therefore, some construction difficulties should be anticipated during periods when these soils are saturated.

7.2 Foundation Excavations

All foundation excavations should be monitored to verify foundations bear on suitable material. The bearing stratum exposed in the base of all foundation excavations should be protected against any detrimental change in conditions. Surface runoff water should be drained away from excavations and not allowed to collect. All concrete for foundations should be placed as soon as practical after the excavation is made. Underreamed piers should be excavated and concrete placed the same day.

Prolonged exposure of the bearing surface to air or water will result in changes in strength and compressibility of the bearing stratum. Therefore, if delays occur, underreamed pier excavations should be slightly deepened and cleaned, in order to provide a fresh bearing surface.

All pier shafts should be at least 1.5-ft in diameter to facilitate clean-out of the base and proper monitoring. Concrete placed in pier holes should be directed through a tremie, hopper, or equivalent. Placement of concrete should be vertical through the center of the shaft without hitting the sides of the pier or reinforcement to reduce the possibility of segregation of aggregates. Concrete placed in piers should have a minimum slump of 5 inches (but not greater than 7 inches) to avoid potential honey-combing.

Observations during pier drilling should include, but not necessarily be limited to, the following items:

Verification of proper bearing strata and consistency of subsurface stratification with regard to boring logs,

Confirmation the minimum required penetration into the bearing strata is achieved,

Complete removal of cuttings from bottom of pier holes,

Proper handling of any observed water seepage and sloughing of subsurface materials,

No more than 2 inches of standing water should be permitted in the bottom of pier holes prior to placing concrete, and

Verification of pier diameter, underream size and steel reinforcement.

Groundwater seepage was encountered at depths of 16 ft and 19 ft at Borings 1 and 2. However, it is common to detect shallower seasonal groundwater particularly during or after periods of precipitation. The clays at the foundation bearing level could be prone to collapse during construction of the underreams. Immediate placement of concrete after constructing the underream and/or the use of submersible pumps may be adequate to control seepage. Some field adjustments in the depth of the piers may be required in some areas to maintain the bottom of the piers above any groundwater seepage. Adjustments in the depths of the piers should be observed in the field by ALPHA personnel.

Temporary casing may be useful in controlling water seepage occurring from fractures in the native clay soils. As casing is extracted, care should be taken to maintain a positive head of plastic concrete and minimize the potential for intrusion of water seepage. It is recommended a separate bid item be provided for casing on the contractors' bid schedule.

7.3 Fill Compaction

Clay soils with a plasticity index equal to or greater than 25 should be compacted to a dry density between 93 and 98 percent of standard Proctor maximum dry density (ASTM D 698). The compacted moisture content of the clays during placement should be within the range of 2 to 6 percentage points above optimum.

Clayey materials with a plasticity index below 25 should be compacted to a dry density of at least 95 percent of standard Proctor maximum dry density (ASTM D 698) and within the range of 1 percentage point below to 3 percentage points above the material's optimum moisture content.

Clayey materials used as fill should be processed and the largest particle or clod should be less than 6 inches prior to compaction.

In cases where either mass fills or utility lines are more than 10 ft deep, the fill/backfill below 10 ft should be compacted to at least 100 percent of standard Proctor maximum dry density (ASTM D-698) and within 2 percentage points of the material's optimum moisture content. The portion of the fill/backfill shallower than 10 ft should be compacted as outlined above.

Compaction should be accomplished by placing fill in about 8-inch thick loose lifts and compacting each lift to at least the specified minimum dry density. Field density and moisture content tests should be performed on each lift. As a guide, one test per 5,000 sq ft or greater per lift may be used. Utility trench backfill should be tested at a rate of one test per lift per each 300 lineal feet of trench.

7.4 Groundwater

Groundwater seepage was encountered at depths of 16 ft and 19 ft at Borings 1 and 2. From our experience with similar soils, groundwater seepage could be encountered at shallower depths in excavations for abutments, pier caps, grade beams, foundations, utility conduits and other general excavations. The risk of encountering seepage increases with depth of excavation and during or after periods of precipitation. Standard sump pits and pumping may be adequate to control seepage on a local basis.

8.0 LIMITATIONS

Professional services provided in this geotechnical exploration have been performed, findings obtained, and recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. The scope of services provided herein does not include an environmental assessment of the site or investigation for the presence or absence of hazardous materials in the soil, surface water or groundwater.

ALPHA TESTING, INC. is not responsible for conclusions, opinions or recommendations made by others based on this data. Information contained in this report is intended for exclusive use of the Client (and their design representatives) and design of specific structures outlined in Section 2.0. Recommendations presented in this report should not be used for design of any other structures except those specifically described in this report. Further, subsurface conditions can change with passage of time. Recommendations contained herein are not considered applicable for an extended period of time after the completion date of this report. It is recommended our office be contacted for a review of the contents of this report for construction commencing more than one (1) year after completion of this report.

Recommendations provided in this report are based on our understanding of information provided by the Client about characteristics of the project. If the Client notes any deviation from the facts about project characteristics, our office should be contacted immediately since this may materially alter the recommendations. Further, ALPHA TESTING, INC. is not responsible for damages resulting from workmanship of designers or contractors and it is recommended the Owner retain qualified personnel, such as a Geotechnical Engineering firm, to verify construction is performed in accordance with plans and specifications.

APPENDIX

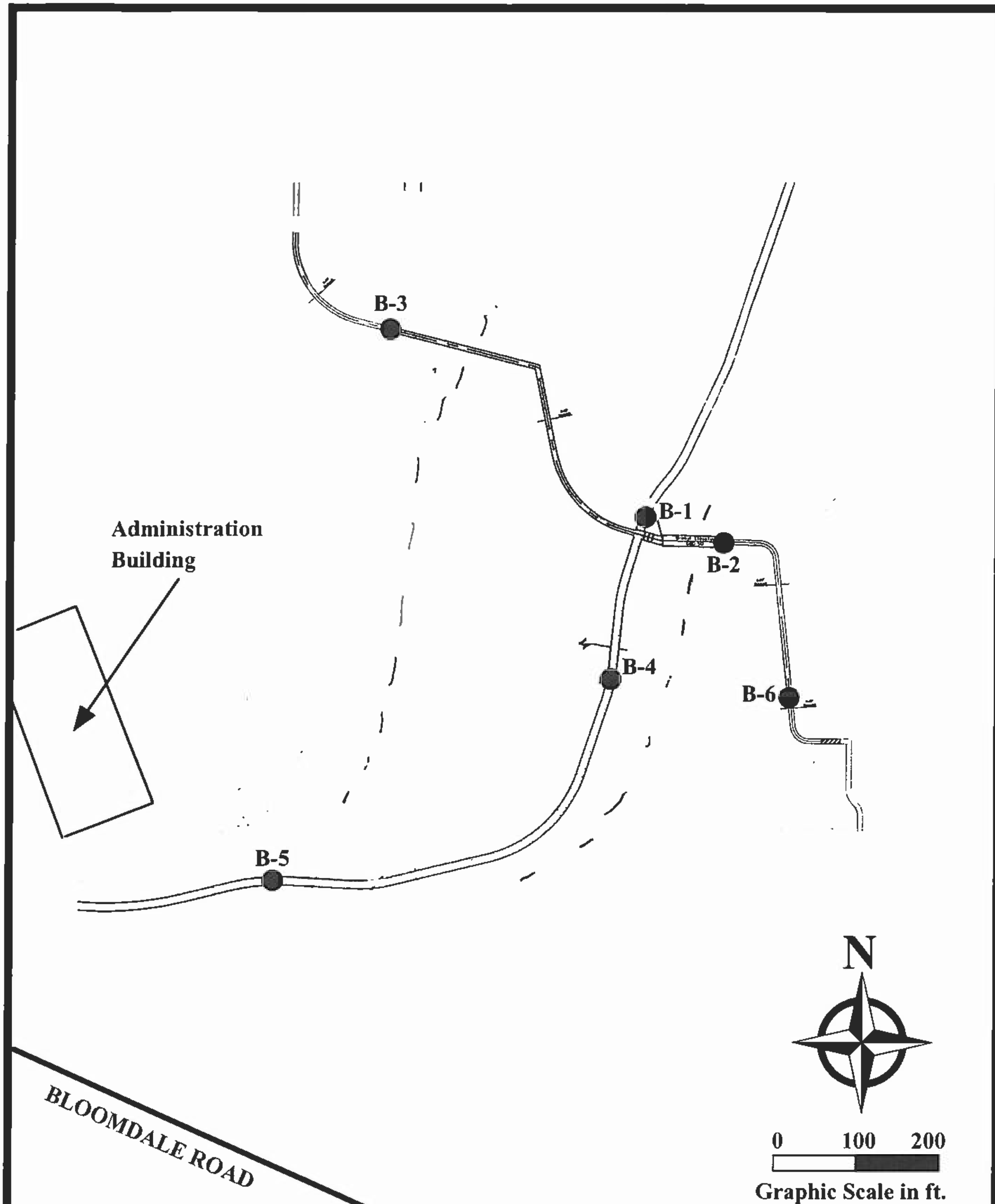
A-1 METHODS OF FIELD EXPLORATION

Using standard rotary drilling equipment, a total of six (6) test borings were performed for this geotechnical exploration at the approximate locations shown on the Boring Location Plan, Figure 1. The test boring locations were staked by either pacing or taping and estimating right angles from landmarks which could be identified in the field and as shown on the site plans provided during this study. The location of test borings shown on the Boring Location Plans is considered accurate only to the degree implied by the method used to locate the borings.

Relatively undisturbed samples of the cohesive subsurface materials were obtained by hydraulically pressing 3-inch O.D. thin-wall sampling tubes into the underlying soils at selected depths (ASTM D 1587). These samples were removed from the sampling tubes in the field and examined visually. One representative portion of each sample was sealed in a plastic bag for use in future visual examinations and possible testing in the laboratory.

The Texas Cone Penetration (TCP) test was used to assess the apparent in-place strength characteristics of the rock type materials. The TCP test consists of a 3-inch diameter steel cone driven by a 170-pound hammer dropped 24 inches (340 ft-pounds of energy) and is the basis for TxDOT strength correlations. Depending on the resistance (strength) of the materials, either the number of blows of the hammer required to provide 12 inches of penetration, or the inches of penetration of the cone due to 100 blows of the hammer are recorded on the field logs and are shown on the Log of Boring sheets as "TX Cone" (reference: TxDOT Test Method TEX 132-E).

Logs of all borings are included in the Appendix of this report. The logs show visual descriptions of subsurface strata encountered using the Unified Soil Classification System. Sampling information, pertinent field data, and field observations are also included. Samples not consumed by testing will be retained in our laboratory for at least 30 days and then discarded unless the Client requests otherwise.



Geotechnical Exploration
Hike and Bike Trail and Pedestrian Bridge
Collin Co. Administration Complex
Off Bloomdale Road
McKinney, Texas
Alpha Project No. G100667
August 6, 2010

ALPHA TESTING
WHERE IT ALL BEGINS

Boring Location Plan
Figure 1

B-1 METHODS OF LABORATORY TESTING

Representative samples were examined and classified by a qualified member of the Geotechnical Division and the boring logs were edited as necessary. To aid in classifying the subsurface materials and to determine the general engineering characteristics, natural moisture content tests (ASTM D 2216) and Atterberg-limit tests (ASTM D 4318) were performed on selected samples. In addition, pocket-penetrometer tests and unconfined compression strength tests (ASTM D 2166) were conducted on selected soil samples to evaluate the soil shear strength. Results of all laboratory tests described above are provided on the accompanying Log of Boring sheets.

ALPHA TESTING
WHERE IT ALL BEGINS

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www.alphatesting.com

LOG OF BORING NO.: 1

Sheet 1 of 1

PROJECT NO.: G100667

Client: CROSS ENGINEERING CONSULTANTS, INC.
Project: HIKE & BIKE TRAIL AND PEDESTRIAN BRIDGE
Start Date: 7/15/2010 **End Date:** 7/15/2010
Drilling Method: CONTINUOUS FLIGHT AUGER

Location: MCKINNEY, TEXAS
Surface Elevation:
West:
North:
Hammer Drop (lbs / in):

GROUND WATER OBSERVATIONS
- ▽ On Rods (ft): 19
- ▼ After Drilling (ft): 19
- ▼ After ____ Hours (ft):

Depth, feet	MATERIAL DESCRIPTION	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft,in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
0 – 2.0	Dark Brown CLAY with calcareous nodules				4.5							
2.0 – 21.0	Brown CLAY with calcareous nodules				4.5				16			
					4.5				17			
					4.0				19	50	19	31
					3.5				21			
					3.5				22			
	▼				3.0	1.5		97	32			
21.0 – 30.0	Light Brown SILTY CLAY				1.0							
					1.0							
30.0	TEST BORING TERMINATED AT 30 FT											

ALPHA TESTING

WHERE IT ALL BEGINS

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LOG OF BORING NO.: 2

PROJECT NO.: G100667

Client: CROSS ENGINEERING CONSULTANTS, INC.
Location: MCKINNEY, TEXAS
Project: HIKE & BIKE TRAIL AND PEDESTRIAN BRIDGE
Surface Elevation:
Start Date: 7/15/2010 **End Date:** 7/15/2010
West:
Drilling Method: CONTINUOUS FLIGHT AUGER
North:
Hammer Drop (lbs / in):

GROUND WATER OBSERVATIONS
- On Rods (ft): 16
- After Drilling (ft): 16
- After ____ Hours (ft):

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft,in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
0–2.0		Dark Brown and Brown CLAY -possible fill (2.0)				4.5							
2.0–4.0		Dark Brown CLAY (4.0)				4.5				20	56	22	34
4.0–30.0		Brown CLAY with calcareous nodules				4.5				14			
						4.5				15			
						4.5				17	50	18	32
						2.5	2.8		101	24			
						2.0							
						1.5							
		(30.0)				1.5							
		TEST BORING TERMINATED AT 30 FT											

ALPHA TESTING

WHERE IT ALL BEGINS

2209 Wisconsin Street
Suite 100
Dallas, Texas
75229
Phone: 972-620-8911
Fax: 972-241-5835
www.alphatesting.com

LOG OF BORING NO.: 3

Sheet 1 of 1

PROJECT NO.: G100667

Client: CROSS ENGINEERING CONSULTANTS, INC.
Project: HIKE & BIKE TRAIL AND PEDESTRIAN BRIDGE
Start Date: 1/3/1900 End Date: 7/15/2010
Drilling Method: CONTINUOUS FLIGHT AUGER

Location: MCKINNEY, TEXAS
Surface Elevation:
West:
North:
Hammer Drop (lbs / in):

GROUND WATER OBSERVATIONS
- On Rods (ft): NONE
- After Drilling (ft): DRY
- After ___ Hours (ft):

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft,in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
0		Brown CLAY with calcareous deposits											
						4.5							
						4.5				14			
5						4.5				17			
						4.5							
10		10.0				4.5							
		TEST BORING TERMINATED AT 10 FT											
15													
20													
25													
30													
35													

ALPHA TESTING
WHERE IT ALL BEGINS

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75229
Phone: 972-620-8911
Fax: 972-241-5835
www.alphatesting.com

LOG OF BORING NO.: 4

Sheet 1 of 1

PROJECT NO.: G100667

Client: CROSS ENGINEERING CONSULTANTS, INC.
Location: MCKINNEY, TEXAS
Project: HIKE & BIKE TRAIL AND PEDESTRIAN BRIDGE
Surface Elevation:
Start Date: 1/4/1900 End Date: 7/15/2010
West:
Drilling Method: CONTINUOUS FLIGHT AUGER
North:
Hammer Drop (lbs / In):

GROUND WATER OBSERVATIONS
On Rods (ft): NONE
After Drilling (ft): DRY
After ___ Hours (ft):

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft,in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
		Dark Brown CLAY 2.0											
		Brown CLAY with calcareous nodules					5.4		113	15			
5										17			
10		10.0											
		TEST BORING TERMINATED AT 10 FT											
15													
20													
25													
30													
35													

ALPHA TESTING

WHERE IT ALL BEGINS

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75229
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Fax: 972-241-5835
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LOG OF BORING NO.: 5

Sheet 1 of 1

PROJECT NO.: G100667

Client: CROSS ENGINEERING CONSULTANTS, INC.
Location: MCKINNEY, TEXAS
Project: HIKE & BIKE TRAIL AND PEDESTRIAN BRIDGE
Surface Elevation:
Start Date: 1/5/1900 **End Date:** 7/15/2010
West:
Drilling Method: CONTINUOUS FLIGHT AUGER
North:
Hammer Drop (lbs / in): 170 / 24

GROUND WATER OBSERVATIONS

- On Rods (ft): NONE
- After Drilling (ft): DRY
- After ____ Hours (ft):

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft,in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
0–4.0	▨	Brown CLAY	■			4.5+	4.9		112				
						4.5+				12	53	18	35
4.0–10.0		Tan Weathered SHALY LIMESTONE with gray limestone seams	◣		100/ 1.5"					13			
			◣		100/ 1.25"								

TEST BORING TERMINATED AT 10 FT

Depth scale: 5, 10, 15, 20, 25, 30, 35

ALPHA TESTING
WHERE IT ALL BEGINS

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Suite 100
Dallas, Texas
75229
Phone: 972-620-8911
Fax: 972-241-5835
www.alphatesting.com

LOG OF BORING NO.: 6

PROJECT NO.: G100667

Client:	CROSS ENGINEERING CONSULTANTS, INC.	**Location:**	MCKINNEY, TEXAS
Project:	HIKE & BIKE TRAIL AND PEDESTRIAN BRIDGE	**Surface Elevation:**	
Start Date: 1/5/1900	**End Date:** 7/15/2010	**West:**	
Drilling Method:	CONTINUOUS FLIGHT AUGER	**North:**	
		Hammer Drop (lbs / in):	

GROUND WATER OBSERVATIONS
- On Rods (ft): NONE
- After Drilling (ft): DRY
- After ___ Hours (ft):

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft,in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
		Tan and Gray CLAY with limestone fragments and calcareous nodules-FILL				4.5+							
						4.5+				15			
5						4.5+				13	44	22	22
						4.5+							
						4.5+							
10		10.0											
		TEST BORING TERMINATED AT 10 FT											
15													
20													
25													
30													
35													

ALPHA TESTING

WHERE IT ALL BEGINS

KEY TO SOIL SYMBOLS AND CLASSIFICATIONS

SOIL & ROCK SYMBOLS

- (CH), High Plasticity CLAY
- (CL), Low Plasticity CLAY
- (SC), CLAYEY SAND
- (SP), Poorly Graded SAND
- (SW), Well Graded SAND
- (SM), SILTY SAND
- (ML), SILT
- (MH), Elastic SILT
- LIMESTONE
- SHALE / MARL
- SANDSTONE
- (GP), Poorly Graded GRAVEL
- (GW), Well Graded GRAVEL
- (GC), CLAYEY GRAVEL
- (GM), SILTY GRAVEL
- (OL), ORGANIC SILT
- (OH), ORGANIC CLAY
- FILL

SAMPLING SYMBOLS

- SHELBY TUBE (3" OD except where noted otherwise)
- SPLIT SPOON (2" OD except where noted otherwise)
- AUGER SAMPLE
- TEXAS CONE PENETRATION
- ROCK CORE (2" ID except where noted otherwise)

RELATIVE DENSITY OF COHESIONLESS SOILS (blows/ft)

VERY LOOSE	0 TO 4
LOOSE	5 TO 10
MEDIUM	11 TO 30
DENSE	31 TO 50
VERY DENSE	OVER 50

SHEAR STRENGTH OF COHESIVE SOILS (tsf)

VERY SOFT	LESS THAN 0.25
SOFT	0.25 TO 0.50
FIRM	0.50 TO 1.00
STIFF	1.00 TO 2.00
VERY STIFF	2.00 TO 4.00
HARD	OVER 4.00

RELATIVE DEGREE OF PLASTICITY (PI)

LOW	4 TO 15
MEDIUM	16 TO 25
HIGH	26 TO 35
VERY HIGH	OVER 35

RELATIVE PROPORTIONS (%)

TRACE	1 TO 10
LITTLE	11 TO 20
SOME	21 TO 35
AND	36 TO 50

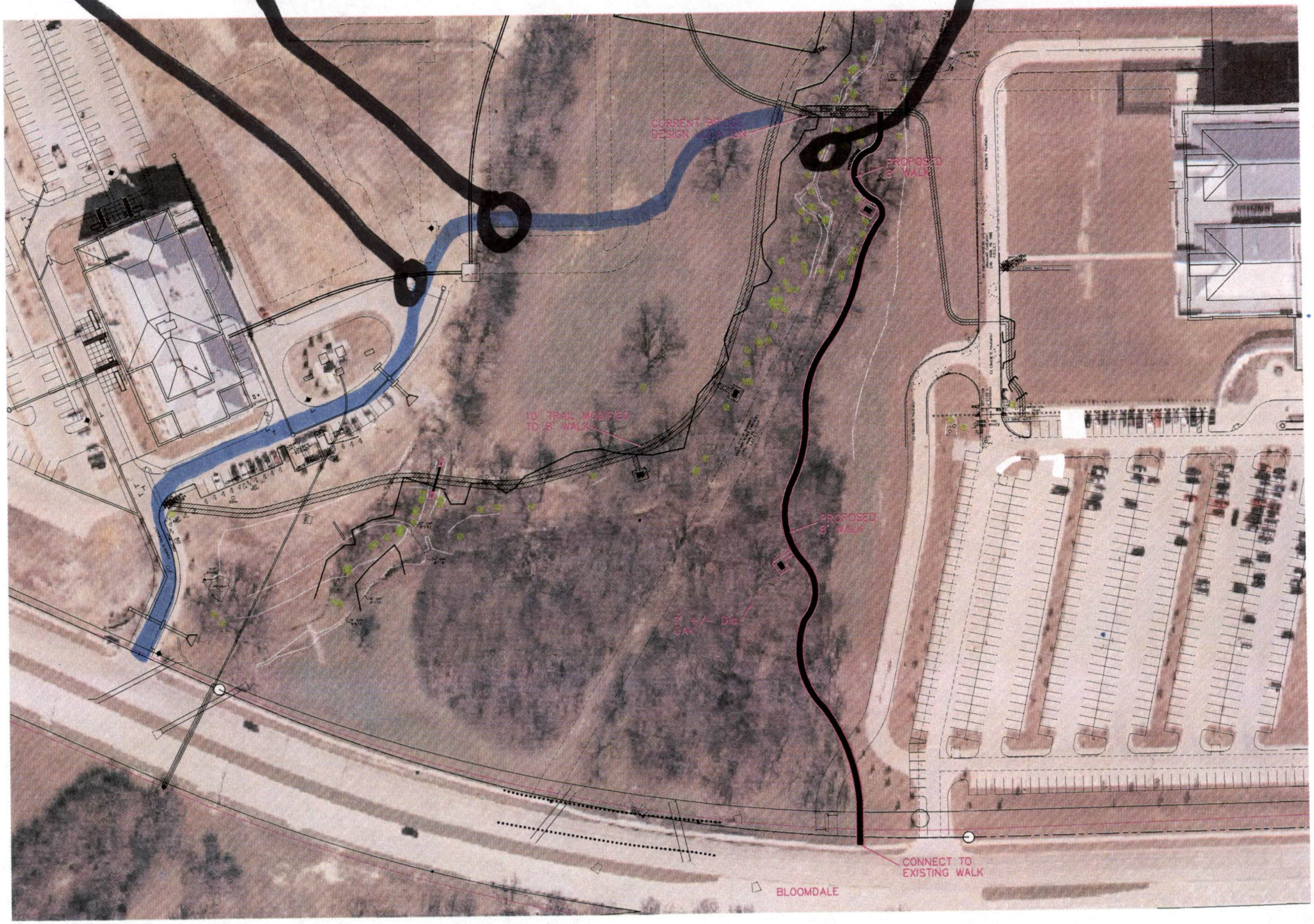
PARTICLE SIZE IDENTIFICATION (DIAMETER)

BOULDERS	8.0" OR LARGER
COBBLES	3.0" TO 8.0"
COARSE GRAVEL	0.75" TO 3.0"
FINE GRAVEL	5.0 mm TO 3.0"
COURSE SAND	2.0 mm TO 5.0 mm
MEDIUM SAND	0.4 mm TO 5.0 mm
FINE SAND	0.07 mm TO 0.4 mm
SILT	0.002 mm TO 0.07 mm
CLAY	LESS THAN 0.002 mm

Protect curb

Ravine crossing-fill with rock and trim trees as needed. Remove rock when complete.

Creek crossing-fill with rock and trim trees as needed. Remove rock when complete



Distance from curb to bridge approximately 600 ft.

Question and Answers for Bid #2013-094 - Construction, Collin County Walking Trail, Pedestrian Bridge Placement

OVERALL BID QUESTIONS

Question 1
What is the budget for this project **(Submitted: Apr 19, 2013 8:07:57 AM CDT)**
Answer

- $100,000 **(Answered: Apr 19, 2013 1:27:50 PM CDT)**

Question 2
Please provide project specifications. no document is uploaded on Bid sync.com thanks. **(Submitted: Apr 20, 2013 2:43:06 PM CDT)**
Answer

- There are 12 documents loaded to the bid including the project manual and plans. If you are unable to see them please contact bidsync. **(Answered: Apr 22, 2013 12:34:37 PM CDT)**

Question 3
Can a pre-manufactured bridge (Contech, Big R Bridge, Pioneer, Excel, etc) be installed? **(Submitted: Apr 22, 2013 12:30:11 PM CDT)**
Answer

- Please elaborate on your question. The bridge to be placed is an existing iron bridge that is sitting on the project site. **(Answered: Apr 22, 2013 12:34:37 PM CDT)**

Question 4
The contractor will have to get drilling equipment, concrete trucks, hoisting equipment and other equipment on both sides of the creek. Please designate a proposed access to the bridge site from the street so we have an idea of the area we may have to repair for grading and ground cover. **(Submitted: Apr 29, 2013 4:47:43 PM CDT)**
Answer

- A map with proposed access will be released in addendum 2. **(Answered: Apr 30, 2013 3:36:32 PM CDT)**

Question 5
Will any trees have to be removed to clear the area in and around the creek for the bridge and dirt work? **(Submitted: May 1, 2013 7:39:19 AM CDT)**
Answer

- No trees are to be removed under this bid. **(Answered: May 1, 2013 9:07:53 AM CDT)**

Question 6
What type of containment is required for sandblasting the bridge? **(Submitted: May 1, 2013 8:27:42 AM CDT)**
Answer

- Can you please clarify if you are concerned about TCEQ or disturbing the vegetation? **(Answered: May 1, 2013 10:00:17 AM CDT)**

Question 7
May we obtain a copy of the soils report? **(Submitted: May 1, 2013 9:08:31 AM CDT)**
Answer

- Yes, it will be included in addendum 2. **(Answered: May 1, 2013 9:08:49 AM CDT)**

Question 8
A handrail is not shown on the drawings. Is it required? **(Submitted: May 1, 2013 9:09:42 AM CDT)**
Answer

- The handrail is not included in the scope of the bridge placement project. It will be added as part of the phase II scope. **(Answered: May 1, 2013 9:12:07 AM CDT)**

Question 9
The question about containment for the sandblasting was asked for the purpose of knowing what the contract obligations are for Collin County and the requirements by TCEQ. **(Submitted: May 2, 2013 9:54:20 AM CDT)**

Answer

- Follow TCEQ regulations, protect existing trees when sandblasting, and cleanup so that the area can be reseeded. **(Answered: May 6, 2013 4:07:19 PM CDT)**

Question 10
What is the weight of the bridge? **(Submitted: May 3, 2013 4:22:15 PM CDT)**
Answer

- The weight of the bridge is unknown. If the weight is a concern then we would recommend a site visit with a steel manufacturer to estimate the weight. Please see section 1.3 of 00200, Instructions to Bidders. **(Answered: May 3, 2013 4:25:34 PM CDT)**

Question 11
How many days are allowed in the schedule to complete the work? **(Submitted: May 6, 2013 10:18:15 AM CDT)**
Answer

- Bidder is to state how long they feel the work will take on the bid form. **(Answered: May 6, 2013 2:36:33 PM CDT)**

Question 12
For cleaning and painting the bridge. Are we using TXDOT spec Class B Blast Cleaning and Paint System II? **(Submitted: May 6, 2013 2:34:14 PM CDT)**
Answer

- Please reference the paint specification provided in addendum No. 1 **(Answered: May 7, 2013 8:23:34 AM CDT)**

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