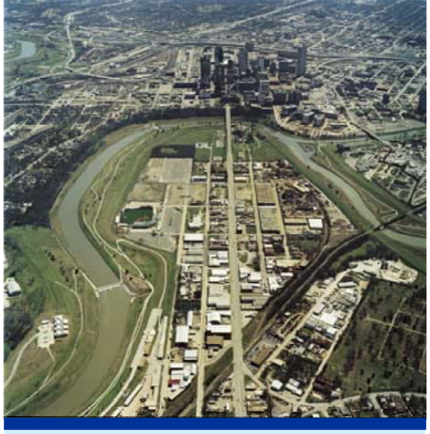
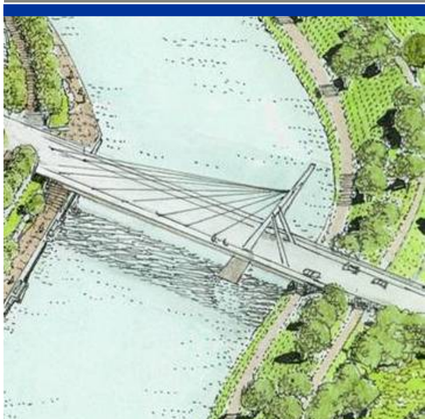


Fort Worth Central City Preliminary Design

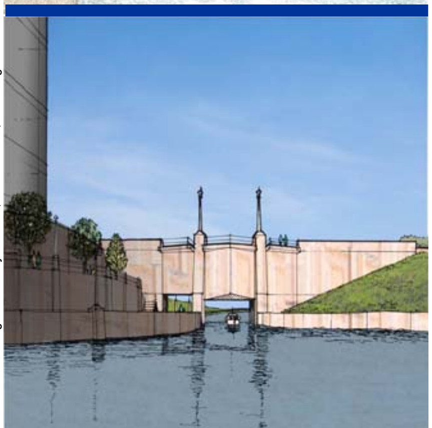
Geotechnical – Initial Geotechnical Investigation



Draft Environmental Impact Statement

Appendix B

May 2005



Images courtesy of CDM, Gideon Toal, and Bing Thom Architects

Fort Worth Central City
Preliminary Design

Geotechnical –
Initial Geotechnical Investigation

Draft Environmental
Impact Statement

Appendix B

May 2005

Executive Summary

The Fort Worth Central City Project consists of a bypass channel, levee system and associated improvements to divert flood flows around a segment of the existing Trinity River adjacent to downtown Fort Worth. An initial geotechnical investigation was performed to assess the soil characteristics for the proposed bypass channel, three isolation gates, Samuels Avenue Dam, three vehicular bridges (Main Street, Henderson Street, and White Settlement Road), and earth retaining structures**.** The investigation consisted of a review of existing geotechnical and geologic data, field drilling exploration program, and laboratory testing program.

Investigation

A total of twenty soil borings were drilled as part of the work. Nine borings were drilled along the route of the bypass channel and monitoring wells were installed adjacent to six of these borings. Two borings were drilled at the approximate location of the Samuels Avenue Dam with piezometers installed adjacent to each boring to monitor groundwater levels. One boring was drilled at each of the three isolation gates locations. A piezometer was installed adjacent to each of these three borings to monitor ground water levels. Six borings were drilled for the three bridges, (two borings for each bridge).

Standard Penetration Tests (SPT) split-spoon samples and Shelby (thin-walled) tube samples were taken at each of the borings. Field hand pocket penetrometer tests were performed on the cohesive soil samples. Bedrock was cored when auger refusal was encountered above a depth of 40 feet below ground surface (ft-bgs). Four bulk (bag) soil samples were taken from the drilling cuttings at selected locations for further laboratory analysis. Groundwater levels were measured in borings immediately after completion on the following day.

Laboratory testing on soil samples consisted of hand pocket penetrometer tests, moisture content and dry unit weight tests, Atterberg limit tests, grain-size analyses, unconfined compression tests, permeability tests, consolidated undrained (CU) triaxial compression test with pore water pressure measurements, and standard Proctor compaction tests. Unconfined compression tests were performed on rock samples.

Subsurface Conditions

The initial geotechnical investigation encountered alluvial soils overlying bedrock. The alluvial soils consisted primarily of clay and overlying generally fresh, unweathered limestone bedrock.

The majority of the clay can be described as having a medium potential for volume change, which is defined as clay with a Plasticity Index ranging from 15 to 28% and a Liquid Limit ranging from 35 to 50%. The results of permeability tests performed on the clay samples show permeability values are generally low and indicate that the soils are capable of water containment within the proposed bypass channel and levees.

Seems of sand and gravel overburden soils primarily were found to occur beneath the clay and directly over the limestone bedrock. There was no significant correlation between percent fines, sands, and gravels with depth. Sand and gravel, which may cause seepage problems, were encountered in the borings for the Samuels Avenue Dam and the Clear Fork isolation gate, and sporadically in the borings along the proposed bypass channel alignment.

Limestone with shale seams was encountered in borings above the proposed lower bypass channel bottom, indicating that some rock excavation will be necessary during construction of the bypass channel. The limestone was found to be generally fresh and unweathered, and can be classified as moderately hard.

Groundwater levels were generally found at approximately Elevation 520 feet, with one exception. Some groundwater will be encountered during excavation, however, the quantities are not expected to be significant due to low permeabilities.

Summary

Results of the initial field exploration and laboratory tests indicate that soils generally varied throughout the project area. Results of this investigation also indicate that the channel can be excavated and levees can be constructed of primarily native materials. Special considerations will be given to local conditions during the design of isolation gates, dam and retaining walls. Design parameters and factors of safety were developed for various structures and loading conditions.

The bypass channel and levees will be designed with three horizontal to one vertical (3H:1V) side slopes. This slope is expected to be stable based on existing configurations and stability analyses. Levee settlement was estimated by correlating compression index values to laboratory index test results using empirical equations. Using the correlated compression indexes, settlement of the levee was estimated to be on the order of four inches.

Seepage is not expected to be a problem for the low permeability soils that exist along the proposed bypass channel alignment. The bypass channel and levees are anticipated to contain the river water without excessive seepage or hydrostatic blow outs.

A permeable gravel layer was found just below the proposed bottom of the bypass channel in some locations. This permeable layer, if under hydrostatic pressure and overlain by a less permeable layer may require enhanced dewatering, cut-off walls or excavation in the wet during construction.

Final design parameters will be developed during the Final Design Phase based upon subsequent geotechnical investigations and analysis.

Contents

Figures

Tables

Section 1 Introduction

1.1 Project Description

The Fort Worth Central City Project consists of a bypass channel, levee system and associated improvements to divert flood flows around a segment of the existing Trinity River adjacent to downtown Fort Worth. The proposed bypass channel is approximately 8,400 feet long and approximately 300 feet wide between the top of levees. The bypass channel will be approximately 30 feet below existing grade. Figure 1-1 shows the bypass channel and other significant project components.

Water levels in the bypass channel will be controlled by a dam with crest gates. The dam is proposed on the West Fork of the Trinity River just east of the Samuels Avenue bridge and will be designed to maintain normal water level of approximately 525 feet above sea level in the bypass channel and interior area. Flood isolation gates will be incorporated into the levee system to protect the interior area, otherwise known as Trinity Uptown. The gates are located upstream at the confluence of the bypass channel and the Clear Fork (Clear Fork Gate), at the midpoint of the bypass channel and the West Fork confluence (Trinity Point Gate), and downstream at the confluence of the bypass channel and the West Fork (TRWD Gate).

Construction of the bypass channel, dam and isolation gates will create an approximately two-mile segment of the existing West Fork Trinity River as a controlled, quiescent watercourse. A water feature or urban lake, approximately 900 feet long, is proposed for the interior area (Trinity Uptown). The water feature will extend from the bypass channel southeast to the existing West Fork and Clear Fork confluence of the Trinity River.

Six bridges are proposed for the project, including four vehicular bridges and two pedestrian bridges. Vehicular bridges are proposed over the bypass channel at North Main Street, over the bypass channel and Fort Worth and Western Railroad (FW&W Railroad) at Henderson Street and White Settlement Road, and on the White Settlement Road extension over the urban lake. Two pedestrian bridges are also proposed, across the bypass channel downstream of Henderson Street, and across the West Fork, approximately 500 feet upstream of the existing FW&W Railroad Bridge.

The project also includes proposed modifications to University Drive, which will effectively raise the roadway approximately 10 feet from existing grade and out of the 100 year floodplain. The proposed modifications begin north of the existing bridge over the West Fork extending to Jacksboro Highway (State Highway 199).

The project could result in a loss of floodplain or valley storage due to the diversion of flood flows through the bypass channel as the bypass channel is shorter than the existing river channel. To prevent this potential loss of floodwater retention, valley storage mitigation sites are included in the preliminary design. Valley storage mitigation sites will be provided in three areas, along the West Fork of the Trinity

River upstream of the project area, in the vicinity of the Samuels Avenue Dam, and slightly downstream of the dam in proximity to Riverside Park. Construction of the bypass channel and associated valley storage sites will not increase downstream water surface elevations or downstream flows.

Major project components are identified in Figure 1-1.

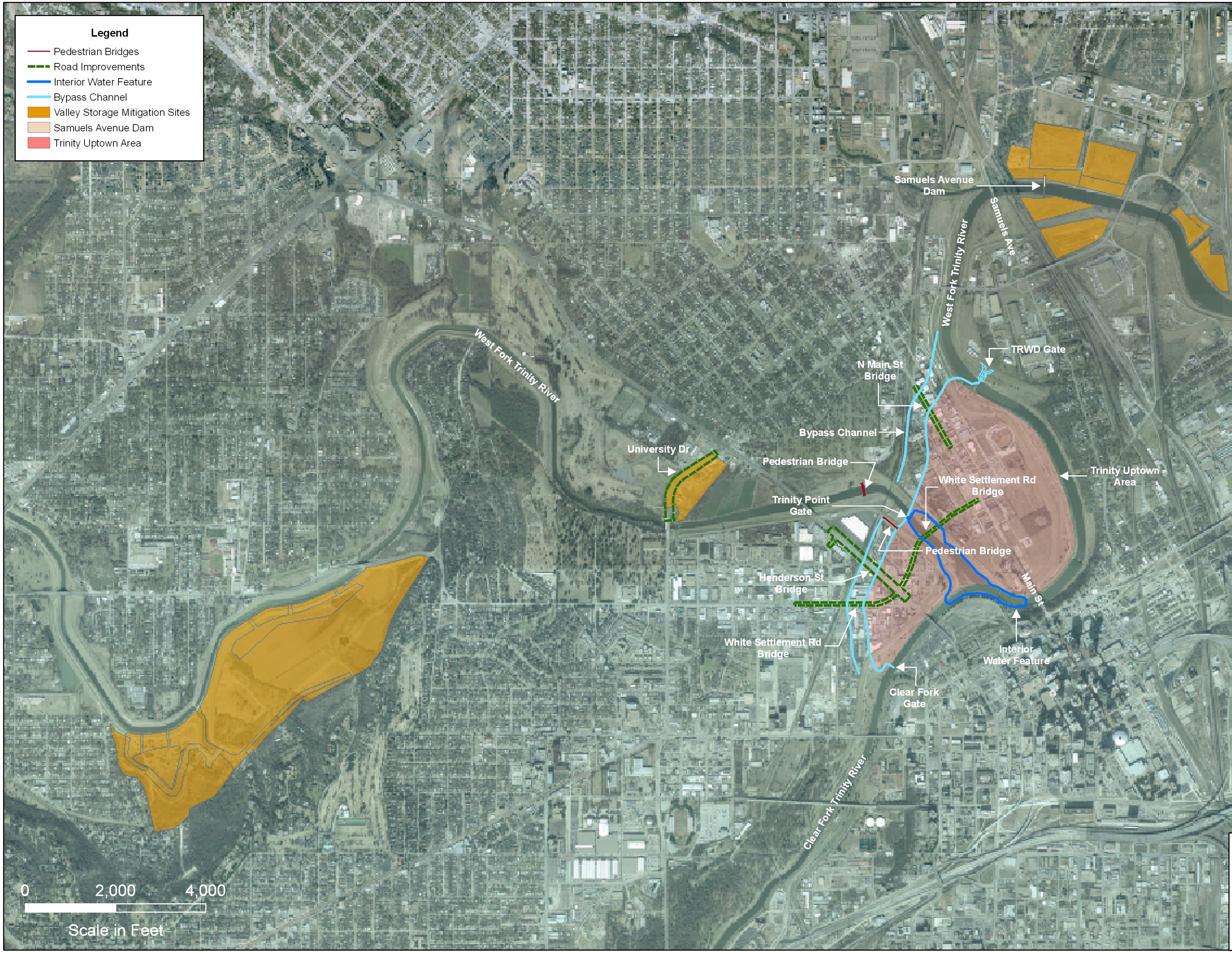
1.2 Purpose

This Appendix addresses the bypass channel, Samuels Avenue Dam, three isolation gates, three vehicular bridges, and earth retaining structures. The purpose of the submittal is to:

- Summarize results of the Initial Geotechnical Field Investigation;
- Summarize results of the Initial Geotechnical Laboratory Testing;
- Identify design considerations; and
- Define design guidelines.

1.3 Scope of Work

The geotechnical investigation is planned to be performed in phases. The initial phase of the geotechnical investigation was completed to establish feasibility and develop design concepts for planning and cost estimating. Additional geotechnical investigations will be developed and performed at a later date during detailed design when project features are further defined. This latter investigation will supplement the initial geotechnical investigation and will provide additional information on the proposed project elements and for potential new project elements.



TARRANT REGIONAL WATER DISTRICT

CDM

FORT WORTH CENTRAL CITY
PRELIMINARY DESIGN

PROJECT LOCATION

DATE: JANUARY 2005 | FIGURE No. 1-1

Section 2
Background Information

2.1 Geology and Site Conditions

During the Cretaceous Period, the seas rose and fell across Tarrant and Dallas Counties, leaving multiple layers of deposits as the sea level changed. The layers are generally thin - most are only tens of feet thick. These formations represent several deposition environments: shallow marine, deltaic, beach, and coastal.

After the Cretaceous Period, the area tilted slightly, causing a 1/2 degree dip due east. This slight dip across the thin deposits causes many formations to outcrop on the surface. The exposures form bands that run generally north-south. Tarrant County has fifteen formations at the ground surface.

During the Tertiary and Quaternary Period, the Trinity River carved out terraces through the Cretaceous deposits. The river deposited clays, sands, and gravel. Today, the Trinity River headwaters form in western Tarrant County and make a large "S" across Tarrant and Dallas Counties. The river causes a dendritic drainage pattern across most of western Tarrant Country.

In Tarrant County, the Paw Formation, Denton Clay, Weno Limestone, Fort Worth Limestone, and the Duck Creek Formation are geologically undivided. Along the proposed bypass channel alignment, the Fort Worth Limestone and the Duck Creek Formation are overlain by alluvium. The Fort Worth Limestone and the Duck Creek Formation are both grayish to yellow-gray or yellow-brown. Both formations are limestone and difficult to differentiate.

In summary, alluvial clay, silt, sand, and gravel overly the Fort Worth Limestone and the Duck Creek Formation along the bypass channel alignment.

2.2 References

- Dawson, W. C., McNulty, C. L., Reaser, D. F., Leaders, 1979, FieldTrip Guidebook for Stratigraphy and Structure of Selected Upper Cretaceous Rocks in Northeast Texas (Texas Academy of Science 82nd Annual Meeting, Arlington, Texas).
- Fisher, W. L., Director, 1972, "Geologic Atlas of Texas, Dallas Sheet" Bureau of Economic Geology, The University of Texas at Austin, Austin, Texas. 1972.
- Matthews, R. K., 1974, Dynamic Stratigraphy. (Prentice-Hall, Inc., Englewood Cliffs, New Jersey.)
- Winton, W. M., Adkins, W. S. 1919, The Geology of Tarrant County (University of Texas Bulletin No. 1931).

Section 3
Methodology

3.1 Overview

The initial geotechnical investigation consists of review of existing geotechnical and geologic data, a field drilling exploration program, and a laboratory testing program. A CDM geotechnical engineer and a U. S. Army Corps of Engineers (USACE) geologist observed the field drilling and logged the borings. The boring locations (plan and elevation) were surveyed by a licensed land surveyor. All field work and laboratory testing were performed in accordance with USACE requirements and in compliance with USACE Engineering Manual (Geotechnical Investigations) EM 1110-1-1804 and Engineering Manual (Laboratory Soils Testing) EM 1110-2-1906, respectively. USACE has completed the field exploration and sampling, and USACE subcontractors (Team Consultants, Inc. and Henley Johnston & Associates, Inc.) completed the laboratory testing.

The proposed bypass channel alignment and the initial boring and piezometer locations are shown on Figure 3-1.

3.2 Bypass Channel

The bypass channel was investigated drilling nine test borings (C-1 thru C-4, and C-6 thru C-10) along the proposed alignment. The borings were spaced at approximately 1,000-foot intervals and near the bypass channel centerline as access on City property permitted. Note that Boring C-5 could not be drilled because property access could not be obtained. Borings C- 1 and C-2 were drilled to depths of 57.5 and 30 feet below ground surface (ft-bgs), respectively. The remaining seven borings were drilled to a depth 40 ft-bgs. The average channel excavation depth is expected to range from 20 to 30 ft-bgs.

Soil samples were taken at approximately 5-foot depth intervals and at changes in strata. Standard Penetration Tests (SPT) split-spoon samples were typically taken in granular soils. SPT involves driving a split-spoon sampler a distance of 18 inches with a 140-pound hammer falling 30 inches. The blows required to drive the sampler each of three 6-inch increments are recorded and the sum of the blows for the last two increments is defined as the SPT blow count or referred to as the N-value. Shelby (thin-walled) tube samples were taken in the cohesive soils, on which hand pocket penetrometer tests were performed in the field to measure the unconfined compressive strength. Shelby tube samples comprised the majority of the samples that were taken. Only seven SPT samples were obtained in the bypass channel borings. Four bulk (bag) soil samples were taken from the drilling cuttings at selected locations in Borings C-2, C-4, C-6, and C-9.

Bedrock was cored when auger refusal was encountered above a depth of 40 ft-bgs. This occurred in Borings C-2, C-3, and C-4. The core was placed in boxes for storage at a nearby Tarrant Regional Water District facility. Select rock core samples were taken for laboratory unconfined compression testing.

As part of the separate environmental investigation program, monitoring wells were installed adjacent to the six (C-1, C-4, C-7, C-8, C-9, and C-10) of the nine geotechnical borings for groundwater sampling and analyses. Depths of the Monitoring wells varied from approximately 15 to 35 ft-bgs. The monitoring well for Boring C-2 was omitted, because bedrock was encountered at a depth of only seven ft-bgs. The groundwater levels in these wells also will be monitored for both environmental and geotechnical needs. The monitoring well details are summarized in the environmental report as part of this submittal.

Laboratory tests performed on cohesive soil for the bypass channel and included 35 moisture content tests and unit dry weight tests, and 37 Atterberg limits, which were performed on 35 Shelby tube samples and two SPT split-spoon samples. Tests performed on the undisturbed Shelby tube samples consisted of 12 unconfined compression tests, five permeability tests, and four consolidated undrained triaxial compression tests with pore water pressure measurements. Pocket penetrometer tests were also performed in the laboratory on all the Shelby tube samples. The laboratory pocket penetrometer results are considered to be more representative of soil strength than the field results, because the laboratory samples could be entirely inspected and representative test locations could be selected on the samples. Four standard Proctor compaction tests were performed on the bulk soil samples. Samples compacted to 95% of the maximum density at optimum moisture content from each of the four tested bulk samples were prepared for the performance of two unconfined compression tests, four permeability tests, and two consolidated undrained triaxial compression tests with pore water pressure measurements. Unconfined compression tests were performed on five rock cores.

3.3 Samuels Avenue Dam

The proposed Samuels Avenue Dam location was explored by two borings (D-1 and D-2), one on each side of the Trinity River. The borings are being drilled to refusal on bedrock and then rock core was obtained. The total depths of Borings D-1 and D-2 were 61 and 37 ft-bgs with rock encountered at 53 and 17 ft-bgs, respectively. SPT split-spoon samples were taken in granular soils and Shelby tube samples were taken in the cohesive soils. To monitor groundwater levels, piezometers with 10-foot long screens were installed in separate bore holes adjacent to the Borings D-1 and D-2 to respective depths of 30 and 23 ft-bgs.

Laboratory tests performed for the dam site include nine moisture content tests, two unit dry weight tests, one Atterberg limits (on cohesive soils), and six grain-size analyses (wet sieve on non-cohesive soils). The dam site borings primarily encountered granular soils; therefore, no unconfined compression tests, permeability tests, and consolidated undrained triaxial compression tests were performed. Unconfined compression tests were performed on two rock cores.

3.4 Isolation Gates

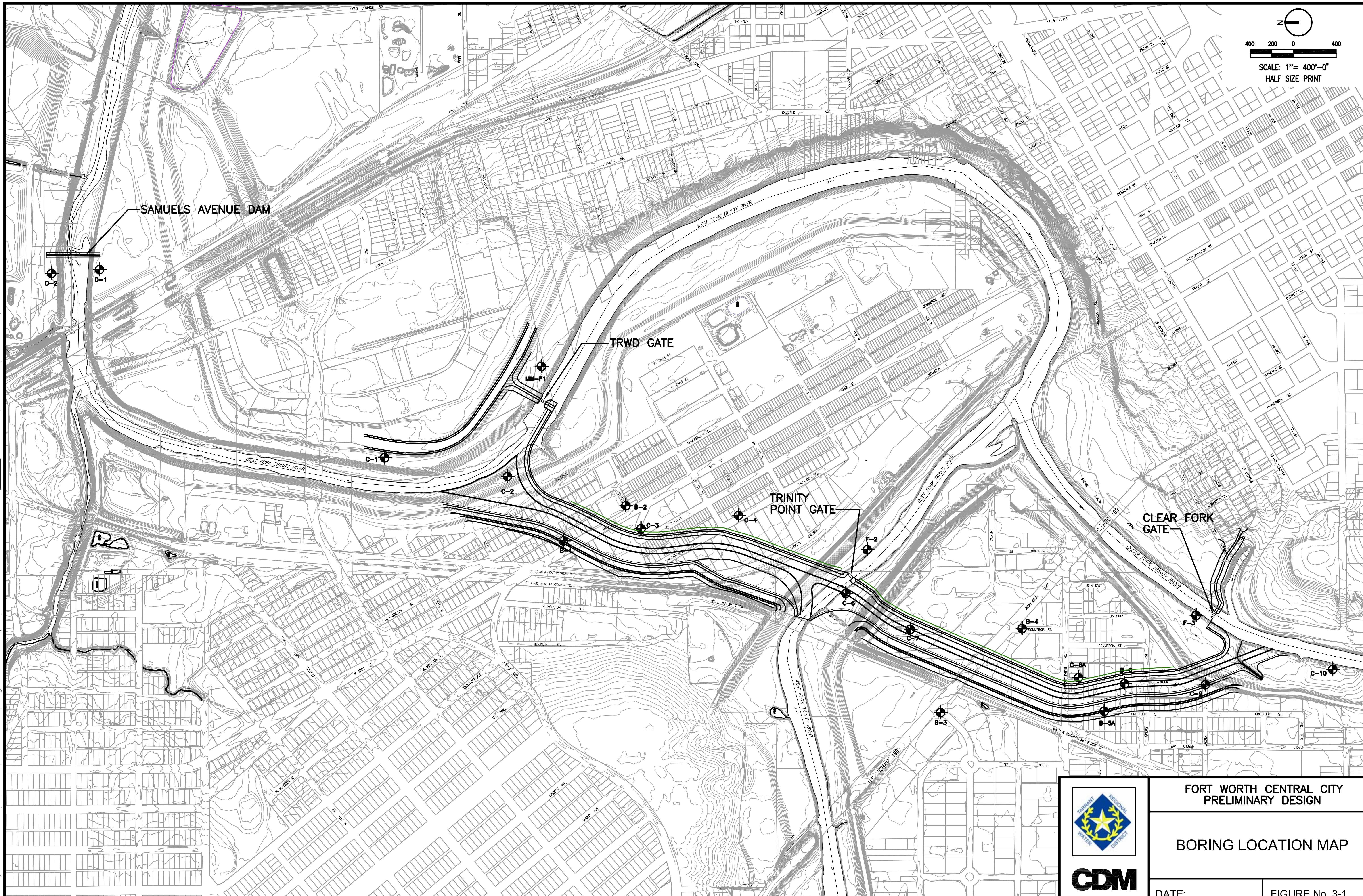
The three isolation gates were explored by three soil test borings (F-1, F-2, and F-3), one drilled at each isolation gate near the river. Boring F-1 was drilled to a depth of 49 ft-bgs, and Borings F-2 and F-3 were terminated at a depth of 40 ft-bgs. No rock core was taken. The SPT split-barrel and Shelby tube sampling was performed in a similar manner to the borings for the bypass channel. To monitor groundwater levels, piezometers were installed in separate bore holes adjacent to the three isolation gate borings. The piezometers for Borings F-1, F-2, and F-3 were installed with 10-foot long screens to respective depths of 29, 16, and 18 ft-bgs.

Laboratory tests performed for the isolation gates include 13 moisture content tests, seven unit dry weight tests, nine Atterberg limits (on cohesive soils), and five grain-size analyses (wet sieve on non-cohesive soils). Tests performed on undisturbed Shelby tube samples consisted of three unconfined compression tests and laboratory pocket penetrometer tests.

3.5 Bridges

The three vehicular bridge sites were explored by six soil test borings (B-1 thru B-6), one drilled at each of the proposed bridge abutments. The SPT split-spoon and Shelby tube sampling was performed in a similar manner as the borings for the bypass channel. Borings B-1, B-2, and B-3 had to be prematurely terminated at respective depths of 12, 13, and 20 ft-bgs due to potential environmental hazards for the drilling crew that were encountered during drilling. Later, two environmental monitoring wells were installed at the locations of Borings B-1 and B-2 by a hazardous waste trained and prepared drilling crew. These wells were installed to a depth of approximately 15 ft-bgs. Borings B-4, B-5, and B-6 were drilled to depths of 48.8, 50.3, and 50.5 ft-bgs, respectively. Bedrock was cored in these three latter borings and select soil and rock samples were taken for laboratory testing.

Laboratory tests performed on soil samples from Borings B-4, B-5, and B-6 consist of 15 moisture content tests, eight unit dry weight tests, and six Atterberg limits (on cohesive soils). Tests performed on undisturbed Shelby tube samples consisted of six unconfined compression tests and laboratory pocket penetrometer tests. Unconfined compression tests were performed on two rock cores.



FORT WORTH CENTRAL CITY
PRELIMINARY DESIGN

BORING LOCATION MAP

CDM

DATE:

FIGURE No. 3-1

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Section 4
Summary of Results

4.1 Overview

The boring and piezometer construction logs along with profiles and laboratory test result summaries are also presented in Attachment A. Team Consultants' laboratory test result submittals are provided in Attachment B. Laboratory testing results are summarized in Attachment C. The soil classifications are based on visual inspection, field pocket penetrometer tests, in-situ SPT blow counts (N-values), and the geotechnical laboratory test results. Figures illustrating and correlating the data are presented in Attachment D. Results of slope stability analyses for the channel and levees are summarized in Attachment E. Results of the settlement analyses for levees and retaining walls are presented in Attachment F.

4.2 Groundwater

Groundwater levels were measured in borings immediately after completion, and the next day when it was possible to leave the bore holes open overnight. The groundwater levels were generally found at approximately Elevation 520 feet. The only exception was in Boring C-1, where the groundwater was measured at approximately Elevation 500 feet. The groundwater level in this boring may not have stabilized when the measurement was taken.

4.2.1 Piezometers

Geotechnical piezometers MWD-1, MWD-2, MWF-1, MWF-2, and MWF-3 were installed adjacent to the corresponding dam borings (D-1 and D-2) and isolation gate borings (F-1, F-2, and F-3). Environmental monitoring wells (MWB-1, MWB-2, MWC-1, MWC-4, MWC-7, MWC-8, MWC-9, and MWC-10) were installed adjacent to corresponding geotechnical borings drilled for or along the bypass channel alignment. Groundwater levels will be measured and will continue to be measured in these piezometers and monitoring wells as part of the on-going geotechnical and environmental investigations (see the environmental section of this submittal).

4.3 Soils

4.3.1 Clay Overburden Soils

4.3.1.1 Soil Classification

This geotechnical investigation encountered alluvial soils overlying limestone. The alluvial soils were found to be mostly "CL" clay in accordance with the Unified Soil Classification System (see Plasticity Chart on Figure D-1 in Attachment D). The results of the Atterberg limits are summarized on Table C-1 in Attachment C. The majority of the clay can be described as having a medium potential for volume change, which is defined as a clay with a Plasticity Index ranging from 15 to 28% and a Liquid Limit ranging from 35 to 50% (Bowles 1996, Table 7-1 as referenced in Section 5.1, Guidance Documents).

4.3.1.2 Clay Consistency vs. Depth

The consistency (or strength) of the clay generally decreases with depth, which is probably the result of groundwater level fluctuation and desiccation of the upper subsoils. In accordance with Visual Identification of Soil Samples in Appendix F-3 of the USACE Manual EM 1110-1-1906, the surficial clay (upper 10 feet) was found to be firm to hard in consistency with laboratory pocket penetrometer values ranging from 1.0 to 4.5 tons per square foot (tsf). From a depth from 10 to 20 ft-bgs, the clay was typically medium to very firm with the laboratory pocket penetrometer values in the range from 0.5 to 4.0 tsf. From a depth from 20 to 30 ft-bgs, the clay was generally medium to firm with penetrometer values ranging from about 0.75 to 1.75 tsf. From a depth from 30 to 40 ft-bgs, the clays were defined as soft to medium firm with penetrometer values ranging from 0.25 to 1.0 tsf. Soft clay at or near the bypass channel bottom may affect the stability of the bypass channel slopes. See the Penetrometer vs. Depth relationship on Figure D-2 in Attachment D. The laboratory pocket penetrometer values are summarized along with field SPT blow counts on Table C-2 in Attachment C. The field and laboratory pocket penetrometer values are shown on the boring logs in Attachment A. Note that average measured unconfined compression strength is about 20 percent less than the average measured laboratory pocket penetrometer strength (See Pocket Penetrometer vs. Unconfined Compression Tests on Figure D-3 in Attachment D). The unconfined compression tests are tabulated on Table C-3 in Attachment C.

4.3.1.3 Clay Properties vs. Depth

Other clay property relationships exist with depth below the ground surface. The plasticity indexes vary considerably from approximately 5 to 30 %, but show a slight decreasing trend (but no clear correlation) with depth. The liquid limits have a wide scatter from about 20 to 50 %, but also show a decreasing trend with depth. The plastic limits and natural moisture contents exhibit no trend with depth. The plastic limits range from approximately 10 to 20 % and the natural moisture contents typically vary from about 10 to 30 %. Near the ground surface, the plastic limit and natural moisture content values are relatively close to each other, but at depth, the moisture contents become greater than the plastic limits (see Moisture, LL, and PL vs. Depth on Figures D-4 in Attachment D). This is an indication that the upper soils are over-consolidated, which is probably the result of desiccation.

In three of the four standard Proctor compaction tests performed (discussed in the next section.), optimum moisture contents were approximately equal to the plastic limits. Having the optimum moisture contents very close to the plastic limits is significant, because the clay appears to have natural moisture contents greater than (at least for the deeper clay) the plastic limits and therefore, the optimum moisture contents, thus affecting the compaction. The implication is that drying or stabilization (with lime or other similar materials) of wet clay may be necessary to meet the compaction requirements.

4.3.1.4 Compaction Tests

The results of the four standard Proctor compaction tests (summarized on Table C-4 in Attachment C) had optimum moisture contents ranging from 10.2 to 16.6 percent and maximum unit dry weights ranging from 111.7 to 124.1 pounds per cubic foot. Measured permeabilities for the four compacted samples were low, ranging from 5.81 x 10^{-9} to 7.37 x 10^{-10} cm/sec. The results of two unconfined compression tests performed on bag samples from Borings C-2 and C-4 were respectively 1.2 and 0.72 tsf.

4.3.1.5 Permeability Tests

The result of five permeability tests performed on undisturbed clay samples ranged from 1.26 x 10^{-7} to 6.43 x 10^{-9} cm/sec. These permeability values are low and indicate that the site soils are capable of water containment within the channel and levees. The permeability of the site soils (undisturbed and compacted samples) tend to increase with decreasing Plasticity Index values, but show no strong correlation (see Plasticity Index vs. Permeability on Figure D-5 in Attachment D). The permeability results are summarized on Table C-4 in Attachment C.

4.3.1.6 Triaxial Compression Tests

Five consolidated undrained triaxial compression tests with pore water pressure measurements were performed. The triaxial test results performed on three undisturbed (Shelby tube samples) soil samples from Borings C-3, C-7 and C-9 are friction angle/cohesive values of 25°/230 psf, 27°/251 psf, and 29°/504 psf, respectively. The test results performed on two remolded (from bag samples) soil samples from Borings C-6 and C-9 are 23°/286 psf, and 30°/223 psf, respectively. The triaxial test results are summarized on Table C-5 in Attachment C.

4.3.2 Granular Overburden Soils

Sand and gravel overburden soils primarily occur directly over the limestone bedrock. There was no significant correlation between percent fines, sands, gravels with depth. The grain size analyses results are summarized on Table C-4 in Attachment C.

The gravels overlying the bedrock appeared to be both alluvial and resulting from the in-situ weathering of the bedrock. Gravels were encountered in Borings F-3 and C-10 at approximately the elevation (515 feet) proposed for the upstream bottom of the bypass channel. Gravels were found in Boring C-3 and C-4 just below Elevation 520 feet above the proposed bypass channel bottom. Sands and gravels were the primary overburden soils encountered in Borings D-1 and D-2, drilled for the Samuels Avenue Dam.

4.3.3 Bedrock

Limestone with shale seams was encountered in Borings C-2, C-3, and C-4 above the proposed bypass channel bottom. Rock excavation will be necessary. The limestone is generally fresh and unweathered with a Rock Quality Designation (RQD) of 100 percent. RQD is obtained by summing the total length of core recovered, but counting only pieces of hard, sound core, which are four inches in length or longer, and taking the total core length as a percentage of the total length cored. Results of all the unconfined compression tests performed on rock core samples ranged from 121.1 to 377 tsf. For rock core samples obtained above the proposed channel bottom in Borings C-2, C-3, and C-4, the unconfined compression tests results were 177.6, 164.7, 144.2, and 237 tsf. This rock can be classified as moderately hard (Hunt 1984, Table 5.20 as referenced in Section 5.1, Guidance Documents). Rock within a strength range from approximately 100 to 250 tsf is defined to have moderate hardness.

Section 5
Design Criteria and Requirements

5.1 Guidance Documents

Geotechnical design for the project will be performed in accordance with the following public agency requirements and guidance documents:

- City of Fort Worth;
- Texas Department of Transportation;
- United States Army Corps of Engineer (USACE) requirements including;
  - EM 1110-1-1804, Geotechnical Investigations;
  - EM 1110-1-1906, Soil Sampling;
  - EM 1110-2-1901, Seepage Analysis and Control for Dams;
  - EM 1110-2-1902, Slope Stability;
  - EM 1110-1-1904, Settlement Analysis;
  - EM 1110-1-1905, Bearing Capacity of Soils;
  - EM 1110-2-1906, Laboratory Soil Testing;
  - EM 1110-2-1913, Design of Construction of Levees;
  - EM 1110-2-2300, Earth & Rock-Filled Dams General Design & Construction Considerations;
  - EM 1110-2-2502, Retaining Walls and Floodwalls; and
  - EM 1110-2-2906, Design of Pile Foundations.
- Other Technical References including;
  - Foundation Analysis and Design by Joseph E. Bowles, The McGraw-Hill Companies, Inc., 1996;
  - Geotechnical Engineering Investigation Manual by Roy E. Hunt, The McGraw-Hill Company, 1984; and
  - Geotechnical Manual, Texas Department of Transportation, October 2000.

5.2 Design Requirements

5.2.1 General Parameters

Geotechnical design addresses the proposed bypass channel, three isolation gates, Samuels Avenue Dam, earth retaining structures, and three vehicular bridges (Main Street, Henderson Street, and White Settlement Road). Based on the results of the initial geotechnical investigation field exploration and laboratory tests, special considerations will be given to the design of these project components as follows:

- Plastic clays in the Fort Worth area are known to be susceptible to shallow slope failures caused by drying and wetting. Selecting clays with low plasticity indexes for surficial slope soils would mitigate shallow slope failures.
- Soft clays and loose sandy soils found at or near the bottom of the proposed bypass channel excavation may cause slope stability problems, which need to be further evaluated.
- Availability and suitability of borrow material for levees needs to be further evaluated. High natural moisture contents are likely to increase compaction costs and required additional evaluation.
- Shallow, fresh limestone bedrock was encountered along portions of the proposed bypass channel alignment. This rock will increase excavation costs. Blasting and other methods for rock removal need to be evaluated.
- A permeable gravel layer was found just below the proposed bottom of the bypass channel in some locations. This permeable layer, if under hydrostatic pressure and overlain by a less permeable layer, could cause hydraulic blow-out problems during channel excavation. Enhanced dewatering, cut-off walls, and excavation in the wet are possible solutions.
- A permeable gravel layer was found beneath the proposed location of the Clear Fork isolation gate. This permeable layer is likely to contribute to high seepage beneath the isolation gate. Constructing the gate below the permeable layer on bedrock or installing a cut-off wall may be necessary.
- Due to the permeable sands and gravel overlying the bedrock, the Samuels Avenue Dam will need to be constructed directly on the bedrock. Testing results indicate the bedrock in this area has a highly uneven surface, based on results of the two dam borings (D-1 and D-2), which encountered bedrock at depths of 53 and 17 ft-bgs, respectively.

General design parameters for preliminary design, project feasibility determination, and cost estimating are derived from the results of this initial geotechnical investigation. Refinements of design parameters will be made when additional geotechnical investigations are completed.

5.2.2 Functional and Technical Requirements

Geotechnical analyses and design will be performed in accordance with USACE requirements and computer software presented in Table 5-1.

Table 5-1
Guidance and Software

Structure	Analysis	USACE Engineering Manual	Software		
			Vender	Version	Program
Bypass Channel & Levees	Seepage Analysis	EM 1110-2-1901 EM 1110-2-1913	Geoslope	5.1	SEEP/W
	Slope Stability	EM 1110-2-1902 EM 1110-2-1913	Geoslope	5.1	SLOPE/W
			Interactive	5.0	XSTABL
	Settlement	EM 1110-1-1904			SAF-1
Concrete Dam	Seepage Control	EM 1110-2-1901	Geoslope	5.1	SEEP/W
	Bearing Capacity	EM 1110-1-1905			
	Settlement	EM 1110-1-1904			SAF-1
Isolation Gates	Seepage Control	EM 1110-2-1901	Geoslope	5.1	SEEP/W
	Bearing Capacity	EM 1110-1-1905			
	Settlement	EM 1110-1-1904			SAF-1
Retaining Walls	Settlement	EM 1110-1-1904			SAF-1
	Bearing Capacity	EM 1110-1-1905			
	Lateral Loads	Flexible Walls, EM 1110-2-2504			
		Rigid Walls, EM 1110-2-2502			
Bridge Foundations	Vertical Loads	EM 1110-2-2906			
	Lateral Loads	EM 1110-2-2906			COM624
	Construction Control	EM 1110-2-2906			GRL WEAP

Section 6
Design and Analyses

6.1 Bypass Channel and Levees

6.1.1 Global Slope Stability Analyses

The bypass channel and levees will be designed with 3 horizontal to 1 vertical (3H:1V) side slopes. This slope is expected to be stable, because the existing Trinity River Channel with 3H:1V slopes has performed well since its construction. In addition, stability analyses were performed, including:

- Developed subsurface design profile based upon geotechnical field investigation, laboratory test results, and channel and levee design configuration, and selected design parameter for each layer;
- Modeled channel, levee, and retaining wall design cross-sections, phreatic surface for end of construction condition, long-term (steady-state seepage) condition, rapid draw down condition, and long-term under earthquake loading conditions ; and
- Performed global stability analyses of channel, levee, and retaining walls using the XSTABL slope stability computer program.

The typical design cross-sections of the channel, levee, and retaining walls with the subsurface profile are shown in Figure D-6 in Attachment D.

6.1.1.1 Subsurface Design Properties

Based upon subsurface conditions presented in Attachments of this Appendix, a design subsurface profile for the channel, levee, and retaining wall stability analyses was developed. Borings B-5 and C-8 were selected as representative for the typical cross-section. A summary of subsurface design properties is presented in Table 6-1.

Table 6-1
Summary of Subsurface Design Properties

Layer No.	Materials	Unit Weight (pcf)	End of Construction Condition	Long Term and Rapid Drawdown Conditions	Basis for Parameter Selection
			Friction Angle/Cohesion (psf)	Friction Angle/Cohesion (psf)	
1	Compacted Levee	130	0°/1,000	27°/250	Lab Test Results, Boring Logs, and EM 1110-2-2502
2	Soil Layer 1 – Clay (CL)	130	0°/2,000	27°/250	Lab Test Results, Boring Logs, and EM 1110-2-2502
3	Soil Layer 2 – Clay (CL)	130	0°/1,000	27°/100	Lab Test Results, Boring Logs, and EM 1110-2-2502
4	Soil Layer 3 – Clayey Sand (SC)	125	0°/500	28°/0	Lab Test Results, Boring Logs, and EM 1110-2-2502
5	Structural Retaining Wall	150	0°/5,000	0°/5,000	Literature Search

- **Compacted Levee 1** – It was assumed that compacted levee embankment layer 1 consists of approximately 12.5 feet of silty or sandy CLAY. Based upon laboratory test results, and previous experience, a unit weight of 130 pcf and a cohesion of 1,000 psf for end of construction condition and a friction angle of 27 degrees and cohesion of 250 psf for long term and rapid draw down condition were assumed for soil layer 1.
- **Foundation Soil Layer 2** – It was assumed that foundation soil layer 2 consists of approximately 20.0 feet of silty or sandy CLAY. Based upon laboratory test results, laboratory and field pocket penetrometer reading, and previous experience, a unit weight of 130 pcf and a cohesion of 2,000 psf for end of construction condition and a friction angle of 27 degrees and cohesion of 250 psf for long term and rapid draw down condition were assumed for soil layer 2.
- **Foundation Soil Layer 3** – It was assumed that foundation soil layer 3 consists of approximately 22.5 feet of silty of sandy CLAY or lean CLAY material. Based upon laboratory test results, laboratory and field pocket penetrometer reading, and previous experience, a unit weight of 130 pcf and a cohesion of 1,000 psf for end of construction condition and a friction angle of 27 degrees and cohesion of

100 psf for long term and rapid draw down condition were assumed for soil layer 3.

- **Foundation Soil Layer 4** – It was assumed that foundation soil layer 4 consists of approximately 5.2 feet of clayey SAND. Based upon laboratory test results, laboratory and field pocket penetrometer reading, field SPT blow counts, and previous experience, a unit weight of 125 pcf and a cohesion of 500 psf for end of construction condition and a friction angle of 28 degrees for long term and rapid draw down condition were assumed for soil layer 4.
- **Structural Retaining Wall 5** – Based upon literature search a unit weight of 150 pcf, cohesion of 5,000 psf for end of construction condition, long term, and rapid draw down condition were assumed for the concrete retaining wall. The purpose of these values was only for use in the global stability analyses, which is discussed later.

6.1.1.2 Phreatic Surface

During drilling from April to July 2004, water levels were measured in the soil borings along proposed channel, levee, retaining walls, bridges, isolation gates, and dam. The average measured water level was estimated to be approximately Elevation 520 feet.

Three phreatic surface models were developed. The first model was used for end of construction with the water level in the soil outside the channel at Elevation 522 feet, and inside the channel at the ground surface elevation. The second model was used for the long term condition (steady-state seepage) with the water level inside and outside the channel at a normal pool elevation of 525 feet. For the third model and the rapid draw down condition, the water level was assumed within the soil outside the channel at Elevation 522 feet, and inside the channel at the ground surface elevation.

6.1.1.3 Seismic

Earthquake loading was analyzed using a horizontal acceleration of 0.01 g for the long term (steady-state seepage) condition of the channel and levee slopes. Based on the National Seismic Hazard Maps of 1996, the peak horizontal acceleration (%g) with 10 % probability of exceedance in 50 years is 0.01 g for the project area.

6.1.2 Global Stability Analyses

Analysis for overall stability was performed with XSTABL, slope stability software, version 5.203. This computer program uses the inputted slope geometry, soil properties and groundwater conditions to calculate safety factors against overall mass slope failures. The minimum acceptable safety factors against overall slope failure is 1.3 for end of construction conditions, 1.4 for long-term conditions, and 1.0 to 1.2 for rapid draw down condition (based upon Engineer Manual EM 1110-2-1913, Design and Construction of Levees). Analyses for circular failure surfaces through the foundation soils were performed using the Modified Bishop Method in a manner consistent with the USACE's Engineer Manual EM 1110-2-1902, Slope Stability.

The three tiers of retaining walls proposed on the east side of the bypass channel were considered in the slope stability analyses. Soft and loose soil conditions at and near the base of a 30-foot-deep channel excavation were considered. The computed safety factors for slope stability are summarized in Table 6-2 and the XSTABL output files are presented in Attachment E.

Table 6-2
Summary of Safety Factors

Location	Condition	Factor of Safety	Minimum Safety Factor per EM 1110-2-1913
Left Channel & Levee Typical Cross Section	End of Construction	1.4	1.3
	Long Term	1.5	1.4
	Rapid Draw Down	1.5	1.0 to 1.2
	Long Term with Earthquake Loading	1.5	NA
Right Channel & Levee Typical Cross Section	End of Construction	2.0	1.3
	Long Term	2.2	1.4
	Rapid Draw Down	2.0	1.0 to 1.2
	Long Term with Earthquake Loading	2.1	NA

6.1.2.1 Settlement

Settlement is estimated for planning purposes so that levees can be over built to accommodate the settlement and meet the final top of levee design elevations. Levee settlement was estimated by correlating compression index values to laboratory index test results using empirical equations. Using the correlated compression indexes, settlement of the levee was estimated to be approximately four inches. Results of the settlement estimates are presented in Attachment F. These settlement estimates are considered approximate. More precise settlements will be estimated using the results of consolidation tests that will be performed as part of additional geotechnical investigation.

6.1.2.2 Seepage

Internal and under seepage for the bypass channel and levees will be evaluated using permeability coefficient parameters determined from laboratory permeability tests, and correlation from grain-size analyses. Seepage is not expected to be a problem for the low permeability soils that exist along the proposed bypass channel alignment. The bypass channel and levees will contain the river water without excessive seepage or hydrostatic blow outs. Blow outs occur when permeable soils under hydrostatic pressure are overlain by less permeable soils, which are not of sufficient weight to resist the hydrostatic pressure.

6.2 Dam and Isolation Gates

The Samuels Avenue Dam and three isolation gates (TRWD, Trinity Point, and Clear Fork Gates) will be evaluated for seepage and foundation support as required. Internal seepage analyses through the structural elements of the roller-compacted dam and isolation gates are not required as for the earthen levees. Seepage under the Samuels Avenue Dam and the Clear Fork isolation gate need to be evaluated due to an underlying gravel layer. Cut-off walls or over-excavation to bedrock and backfilling may be necessary to control seepage. Seepage, bearing capacity, settlement, over-turning, and sliding analyses will be addressed.

Pile or drilled-shaft foundations and primarily battered piles or drilled shafts extending to bedrock through the overlying clay to resist lateral loads will probably be required for the Trinity Point and TRWD isolation gates. Pile and seepage analyses will be required, but spread foundation bearing capacity and settlement analyses would not be necessary for these structures. Pile or drilled shaft foundations are also considered for support of the storm water pump station adjacent to the TRWD Gate.

6.3 Vehicular Bridges

Abutment and pier foundations for the proposed Main Street, Henderson Street and White Settlement Road Bridges will be designed and analyzed for vertical and lateral loads. Deep foundation support is expected to be required based on the available subsurface information. If bedrock is shallow, drilled shafts are likely to be the most economical foundation type. Drilled shafts are commons used in Fort Worth for deep foundation support. If bedrock is deep, pile foundations may prove to be the more economical foundation type.

Pile or drilled shaft capacities will be computed using shear strength parameters derived from laboratory unconfined compression tests (soil and rock) and field SPT blow counts. The Texas Department of Transportation Geotechnical Manual will be used for design guidelines.

Borings B-1 and B-2 for the proposed North Main Street Bridge and Boring B-3 for the proposed Henderson Street Bridge had to be prematurely terminated due to potential environmental hazards. Using appropriate environmental health and safety methods and procedures, these borings will have to be re-drilled into the underlying bedrock along with additional borings to provide sufficient information for foundation bridge design.

6.4 Earth Retention Structures

Cantilever T- type reinforced concrete retaining walls were selected for earth retention along the proposed bypass channel. Design, including the use of the USACE computer program CTWALL, is discussed separately in the Atttachment C, which includes design and analyses for lateral loads, overturning, sliding, and bearing capacity.

The geotechnical parameters to be used for the design of the reinforced concrete retaining walls are described in the following sections.

6.4.1 Lower Level Walls

Unit weight moist: 130 pcf
Unit weight dry: 100 pcf
Short-Term Shear Strength: cohesion = 1,000 psf, φ = 0°
Long-Term Shear Strength: cohesion = 100 psf, φ = 27°

Allowable Bearing Capacity = 2,000 psf (for foundations bearing on in-situ clay)

6.4.2 Mid-Level Walls

Unit weight moist: 130 pcf
Unit weight dry: 100 pcf
Short-Term Shear Strength: cohesion = 1,000 psf, φ = 0°
Long-Term Shear Strength: cohesion = 100 psf, φ = 27°

Allowable Bearing Capacity = 2,000 psf (for foundations bearing on in-situ clay)

6.4.3 Upper Level Walls

In-situ Soils
Unit weight moist: 130 pcf
Unit weight dry: 100 pcf
Short-Term Shear Strength: cohesion = 1,000 psf, φ = 0°
Long-Term Shear Strength: cohesion = 250 psf, φ = 27°

Allowable Bearing Capacity = 2,000 psf (for foundations bearing on in-situ clay)
Allowable Bearing Capacity = 2,000 psf (for foundations bearing which may be founded on compacted levee fill)

For purposes of this analysis, settlement was estimated and deep-seated slope failures were evaluated. Settlement was estimated so that the retaining walls can be built to accommodate the settlement and meet their required design elevations. Retaining wall settlement was estimated in a similar manner as levee settlement, by correlating compression index values to laboratory index test results using empirical equations.

Using the correlated compression indexes, settlement of 15-foot high retaining walls was estimated to be approximately nine inches. The results of the settlement estimates are presented in Attachment F. These settlement estimates are approximate. More precise settlements will be estimated using the results of consolidation tests that will be performed as part of additional geotechnical investigation.

Deep-seated slope failures below and around the retaining walls were evaluated as part of the slope stability analyses performed on the bypass channel slope. Such slope failure modes were found to have safety factors exceeding 1.5, and therefore, the retaining walls are considered to be stable from a global stability aspect. The computed safety factors for slope stability including deep-seated failures around the retaining walls are summarized in the XSTABL output files are presented in Attachment E**.**

Over-compacting the backfill should be avoided so that excessive lateral forces are not applied to the structure, although a reasonable degree of compaction is necessary to provide adequate shear strength and reduce settlement. The appropriate degree of compaction will be specified in the construction documents.

Section 7
Potential Impacts on Existing Bridges

7.1 Overview

The proposed Samuels Avenue Dam will raise the normal pool level in the existing Trinity River to an approximate elevation of 525 MSL and raise the groundwater table adjacent to the channel upstream of the Dam. This increase in normal pool level could potentially impact the foundation support for some upstream bridges. It should be noted that this rise in pool elevation is below the inundation levels of the 100 yr and SPF flood levels and partially within the seasonal groundwater variation.

There are eight existing bridges that potentially could be impacted. Depending upon the location and design of the foundations, the increase in normal pool level could create a buoyant effect on the foundation soils, thus reducing the foundation support capacity.

Foundations and foundation support soils that are above the new normal pool level and those already below the river level and buoyant will not be impacted. In addition, foundations founded directly on bedrock or founded indirectly on bedrock, such as piles or drilled shafts extending to or into bedrock, will not be significantly impacted by the higher water level.

An extensive effort was conducted to obtain as-built drawings of the existing upstream bridges in an attempt to determine the likelihood of an impact from the proposed increase in normal pool levels. Local and State agencies along with various railroad companies were contacted in an effort to obtain information on existing bridges. However, in many cases only limited information was made available, or in other cases no information was furnished. Based on the information provided and visual observation of the existing bridges, the following determinations are presented.

7.2 Main Street Bridge

There is no impact expected on the Main Street Bridge. This is an older bridge and foundation data was not located for this bridge. However, this bridge has massive loads and its foundations are likely to be supported directly on bedrock or on deep foundations extending to or into bedrock. In addition, the Main Street Bridge is located a considerable distance upstream of the Samuels Avenue Dam and within the backwater of Nutt Dam. The water level will only be raised approximately 5 feet in this area.

7.3 Northside Drive Bridge

No impact is anticipated for the Northside Drive Bridge. Based on as-built drawings, this bridge is supported on drilled shafts penetrating into bedrock.

7.4 Railroad Bridges

There are three railroad bridges located downstream of the Samuels Avenue Bridge and just upstream of the Samuels Dam that could potentially be impacted by the increase in normal pool level associated with the proposed project. Based on the information gathered and contacts the three bridges are:

- Union Pacific Railroad Bridge – former Rock Island Bridge;

- Burlington Northern Santa Fe Railway Bridge – former Fort Worth & Denver Bridge; and

- Burlington Northern Santa Fe Railway Bridge – former Santa Fe (Gulf Colorado & Santa Fe) Bridge.

Additionally, there are two other bridges in the project area where the normal pool level will be increased. One bridge, the Fort Worth and Western Railroad, is located on the West Fork just upstream of the confluence with the bypass channel, and similarly to Main Street will have a relatively small increase in water level due to Nutt Dam. Plans are not available for this bridge. The second bridge, Union Pacific Railroad is a smaller bridge located over the Marine Creek tributary.

The ownership of these railroads has changed over the years and full plans may not be available. During the initial contact only a limited amount of information was obtained on some of the structures, however, much of the information was not sufficient to make any judgments on potential impacts.

Due to the age and lack of information on these bridge structures it is not possible to do any further review at this time. In addition, the criteria used by the individual railroads for the initial design, and the current criteria to supplement base standards (i.e. AREMA and ARBBA) are not known. Typically, the design and evaluation of structures or impacts on facilities are conducted and subject to the criteria and standards for the specific railroad and application.

Therefore, the individual railroad companies will be contacted and advised early during the design phase about the increased normal water levels. The railroad companies will be responsible making their determinations and evaluating the potential impact of higher water levels on their bridges.

7.5 Samuels Avenue Bridge

The Samuels Avenue Bridge is also an older structure with limited information which could potentially be impacted by the increase in normal pool level associated with the proposed project. Six foundation piers are presently located in the river. These foundations should not be significantly impacted by the increase in normal pool level, because the foundations and/or foundation soils are already submerged. The two southernmost pier foundations and both the north and south abutment foundations may be potentially impacted, because their foundation condition and support are unknown. The City of Fort Worth will be contacted early during the design phase about the increase in normal water levels. The City will be responsible for making their determinations and evaluating the impact of higher pool level on the Samuels Avenue Bridge.

Attachments

Attachments A – F to be provided in hard-copy format.