Liberty Ditch Bank Stabilization

Preliminary Engineering Report

Prepared for:

City of Killeen, Texas

February 8, 2017

PRELIMINARY
This document is for interim review and not intended for regulatory approval, permitting, bidding, or construction purposes. This document was prepared by or under the supervision of:

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HALFF

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Acronyms and Abbreviations

WCID	Water Control & Improvement District No.1
cfs	cubic feet per second
CoK	City of Killeen
FEMA	Federal Emergency Management Agency
ft	feet/foot
GIS	Geographic Information System
HEC	Hydrologic Engineering Center (U.S. Army Corps of Engineers)
HMS	Hydrologic Modeling System (U.S. Army Corps of Engineers)
LFE	Langerman Foster Engineering Company
LiDAR	Light Detection and Ranging
NAIP	National Agriculture Imagery Program
NWP	Nationwide Permit
PCN	pre-construction notification
RAS	River Analysis System
RS	HEC-RAS River Station
TxDOT	Texas Department of Transportation
USACE	U.S. Army Corps of Engineer
WSEL	Water Surface Elevation
WWL	wastewater line/sanitary sewer line
WL	water line
1% ACE	1% annual chance exceedance

1. INTRODUCTION

This memorandum documents the preliminary engineering analysis conducted for the Liberty Ditch Bank Stabilization project in Killeen, Texas. Halff Associates, Inc. (Halff) was retained by the City of Killeen to stabilize approximately 200 linear feet of bank that is experiencing erosion due to high creek velocities and threatening an existing aerial water line and waste water line which is a critical line that serves the City of Killeen and Fort Hood. This reach of Liberty Ditch is a concrete lined channel that originally failed during the June 2015 storm event where creek flows overtopped the channel banks causing the concrete lined channel to fail. The bank failure exposed approximately 20-25 feet beyond the southern channel bank exposing an existing 30-inch wastewater line and the pier support for an existing aerial 42-inch water line. In order to protect the critical infrastructure, City crews constructed a fix by filling the bank failure and installing a concrete lined channel with mortared rock rip rap. The fix constructed by City crews has since failed during the extreme storm event on October 2015. It is critical the City of Killeen and Bell County Water Control & Improvement District No. 1 (WCID) stabilize the Liberty Ditch bank erosion to protect these threatened utilities.

1.1 Project Location

Liberty Ditch Bank Stabilization project limits is located in the City of Killeen, on the north side of the Bell County Water Control & Improvement District No. 1 (WCID) property. The project area analyzed is bounded by the BNSF railroad line to the north, the WCID fence line to the south, approximately 300 feet from the BNSF railroad culvert to the west, and approximately 350 feet downstream of the BNSF railroad crossing to the east, as shown in Appendix A Exhibit 1 – Project Location Overview.

1.2 Study Purpose and Intent

The purpose of this preliminary engineering report is to present a discussion of the analyses conducted and alternative solutions to stabilize the bank erosion of Liberty Ditch, as well as associated preliminary plans and probable cost estimates to assist the City staff in the project planning process. Halff gathered detailed ground survey data, conducted subsurface utility investigations, coordinated with a subconsultant for a geotechnical investigation, developed alternative solutions to stabilize the Liberty Ditch bank and protect existing utilities, and prepared preliminary construction plans and probable cost estimates for the alternative solutions. Liberty Ditch experiences high velocity flows from occasional flooding, which lead to undermining and failure of the concrete bank. Furthermore, these failures threaten crucial utilities (aerial and subsurface) adjacent to the area that service Fort Hood and the City of Killeen. The proposed project will evaluate the potential for moving the erosive energy of the flows in Liberty Ditch to minimize downstream impacts and further stabilize the channel banks.

2. PRELIMINARY DESIGN INVESTIGATIONS

A preliminary design investigation was conducted which included: site visits; topographic ground survey of the project area; subsurface utility investigations; and geotechnical investigations. Data obtained from these investigations was used to guide the development of preliminary solutions to further stabilize the Liberty Ditch bank erosion.

2.1 Data Collection

Existing data was collected as part of this preliminary engineering study. The various data obtained was used to provide an understanding of existing conditions in order to achieve the project's primary objectives. Table 2-1 lists the data collected along with their respective sources.

Table 2-1: Data Collected and Sources

Data	Source	Notes
Aerial imagery	NAIP	2014
Hydrologic Flow Data	FEMA FIS / Bell County	September 26, 2008
Hydraulic model	Halff	January 2017
Survey	Halff	November 2016
Subsurface Investigation	Halff	October 2016
Geotechnical Investigation	Langerman Foster	January 2017

2.2 Topographic Survey

Halff survey crews conducted a topographic survey of the project area on November 4, 2016. This survey obtained elevation information on existing topographic features, including but not limited to the railroad, channel features, aerial utilities, and natural ground. The survey bounds used for this study correspond to the project location as shown in Appendix A Exhibit 1.

2.3 Geotechnical Investigation

Halff contracted with Langerman Foster Engineering Company (LFE) to conduct a geotechnical investigation of the project site and recommendations of the proposed solutions. On November 22, 2016, LFE conducted a total of three (3) borings of varying methodologies to a depth of 15-16 feet below ground surface. The geotechnical assessment consisted of split spoon sampling in conjunction with Standard Penetration Tests. Langerman Foster Engineering provided a summary of existing soil parameters as well as recommendations for use in stabilizing the bank and for pier support design for the existing aerial pipeline. The complete geotechnical report is titled *Geotechnical Investigation – Liberty Ditch Repair*, dated January 17, 2017 and is located in Appendix B of this report.

2.4 Utility Investigation

Halff conducted a subsurface utility investigation that consisted of review of site plans, GIS utility data from the City, record drawings from Bell County WCID, and Halff subsurface utility investigations. Halff's subsurface test hole data sheets are located in Appendix C and the identified utilities are shown on the plan sheets located in Appendix E. Utilities identified include the following:

- 42-inch Aerial Water Line
- Verizon Telephone/TV/Fiber Optics Line
- 30-inch waste water force main
- Overhead Electric Line

3. HYDROLOGIC AND HYDRAULIC ANALYSIS

Halff conducted a hydrologic and hydraulic analysis for the Killeen Liberty Ditch to determine existing conditions and proposed conditions velocities and shears for the channel and banks. This was imperative as for determining where the most erosive portions of the reach occurred.

3.1 Hydrologic Discharges

Halff utilized the current effective FEMA Flood Insurance Study (FIS) peak discharges for this analysis. During the analysis, it was determined that storage existed upstream of the railroad and a limited hydrologic model was setup to simulate the storage effect of the BNSF railroad on flows for the project reach. As expected, when storage due to the BNSF railroad was considered, the Liberty Ditch discharges were lower than the FEMA FIS published flows. Leaning on the conservative side, the FEMA FIS flows were used for this study. Table 3-1 displays the range of flows utilized.

Table 3-1: Liberty Ditch FEMA FIS Flows

Recurrence Interval	Peak Flow, (cfs)
10-Year	1,370
25-Year	1,680
50-Year	1,930
100-Year	2,210
500-Year	2,750

3.2 Hydraulic Model Analysis

A hydraulic modeling analysis was conducted to simulate the depth of water, velocity, and shear stress for the project reach along Liberty Ditch. Halff utilized Hydrologic Engineering Center (HEC), HEC-RAS program (Version 5.0.3, 2016) to accomplish this task. Hydraulic methods used for this study were in accordance with the City of Killeen Drainage Criteria Manual and standard engineering practices. An appropriate QC process was followed to ensure the simulated results were as accurate as possible. The hydraulic cross section layout for this analysis is displayed in Exhibit 2. The following sections describe the procedure and assumptions made in establishing hydraulic parameters for this analysis.

3.2.1 Manning's Roughness Coefficients

Manning's n-values were assigned by visual site inspection and evaluation of recent aerial imagery. Manning's roughness coefficients for the concrete channel were set at 0.015 for Liberty Ditch. These overbank n-values ranged from 0.025-0.075. It should be noted that structures were modeled as blocked obstructions. These areas were modeled using an n-value of 0.025. Table 3-2 lists the overbank values used in this analysis.

Table 3-2: Manning's N-Values

Land Cover Designation	N-Value
Channel (Concrete)	0.015
Structure	0.025
Grass	0.045
Thick Grass	0.050
Light Woods	0.075

3.2.2 Boundary Conditions

A downstream channel slope of 2.9% was used for the normal depth boundary condition.

3.2.3 Geometry Data

Channel cross sections were cut from the 3ft x 3ft DEM and updated with topographic survey. Cross section spacing on average is approximately every 40 feet along the Liberty Ditch reach. The BNSF railroad crossing was modeled as a culvert structure utilizing a combination of LiDAR and field survey to estimate the channel flow line elevations of the existing condition. Exhibit 2 shows the cross section layout used for the hydraulic model.

Ineffective areas and blocked obstructions were set following standard practices as outlined in the *HEC-RAS Hydraulic Reference Manual*. Ineffective area transition ratios for the BNSF railroad crossing were specified as 1:1 upstream and 2:1 downstream.

Cross-section expansion/contraction coefficients were typically left at the default values of 0.1 and 0.3, unless physical conditions warranted an adjustment to these values. The expansion/contraction coefficients at hydraulic cross sections immediately upstream and downstream of the BNSF railroad crossing were modified to 0.3 and 0.5, respectively.

3.2.4 Discharge Locations

The hydraulic model incorporated peak discharges from the effective FEMA FIS as discussed above. The peak discharges used in the hydraulic model for each frequency event included the 10% (10-year), 4% (25-year), 2% (50-year), 1% (100-year), and 0.2% (500-year) annual chance events and are shown in Table 3-1 above.

The following section described the results of the existing conditions hydraulic results along with the alternative solutions developed for this analysis.

4 EXISTING AND PROPOSED IMPROVEMENT ALTERNATIVES

4.1 Existing Conditions Hydraulic Results

Halff conducted a hydraulic analysis of the existing channel to develop a baseline model to determine the channel's maximum's velocity and shear stress the concrete channel liner and overbanks are experience during high flows. A summary of results is shown in Appendix D. Our analysis indicates a velocity ranging from 7.5

ft/sec up to approximately 12 ft/sec for the 100-year storm event. The channel shear stresses ranged from 0.31 lb/sq. ft. up to 0.55 lb/sq. ft. The computed resultant water surface elevation profile is also shown in Appendix D.

4.2 Proposed Channel Improvement Alternatives

Halff developed a channel bench solution that would allow the channel flow to make the 90 degree bend as the flow exists the BNSF railroad culvert. The following section discusses two material options that include a concrete lined channel and a gabion mattress lined channel. Table 4-3 shows the comparisons of the 100-year results of the existing conditions to each of the proposed alternatives.

4.2.1 Alternative 1: Concrete Lined Benched Channel Section

In order to stabilize Liberty Ditch and to minimize future bank erosion, Halff recommends a concrete lined channel with a benched section to help convey Liberty Ditch flow through the severe bend as it exits the BNSF railroad culvert. This alternative involves constructing a new concrete channel liner. A channel benched section would be constructed around the corner and taper to a point before the next downstream bend. The concrete liner would be designed and poured with tie-downs and keyways as necessary to minimize water seepage and subsequent uplift of the concrete. The width of the benched section would vary up to 15 feet in length and would be designed so that there would be a clearance of no less than 12-inches around underground 30-inch force main. To complement this channel, a surface swale would be constructed along the southwest side of Liberty Ditch to convey any overflow water back to the channel instead of seeping down and behind the concrete liner. Additionally, bendway weirs are also being considered along the channel bottom to direct the flow around the bend.

Alternative 1 plans are shown in Appendix E in plan Sheets 1 through 7. Cross section 1+80 on Sheet 3 shows where the proposed channel benching will allow about a 12-inch clearance to the existing 30-inch force main. A concrete cap is proposed to be installed through this section to protect the force main. Table 4-1 shows our estimate of probable cost to construct Alternative 1 of approximately $304,000.

Table 4-1: Alternative 1 Estimate of Probable Construction Costs

PAY ITEM NO.	DESCRIPTION	UNITS	QUANTITY	UNIT PRICE	ITEM COST
REMOVAL					
1	REMOVE CONCRETE CHANNEL	SY	818	$ 25.00	$ 20,448.33
2	REMOVE CONCRETE RIPRAP	SY	98	$ 11.00	$ 1,083
		Subtotal REMOVAL =			**$ 21,532**
DRAINAGE					
3	EXCAVATION (CHANNEL)	CY	527	$ 20.00	$ 10,540
4	EXCAVATION (SURFACE SWALE)	CY	62	$ 8.00	$ 494
5	CONCRETE STRUCTURES, CL C, 6-IN	CY	172	$ 500.00	$ 86,000
6	RIPRAP (CONC) (CL B) (4 ")	CY	4	$ 400.00	$ 1,756
7	EMBANKMENT, ROCK DRAINAGE LAYER	CY	181	$ 70.00	$ 12,670
		Subtotal DRAINAGE =			**$ 111,461**
EROSION CONTROL					
8	FURNISHING AND PLACING TOPSOIL (4")	SY	468	$ 1.50	$ 702
9	BROADCAST SEED (PERM) (URBAN) (CLAY)	SY	468	$ 1.35	$ 632
10	BROADCAST SEED (TEMP) (WARM)	SY	234	$ 0.20	$ 47
11	BROADCAST SEED (TEMP) (COOL)	SY	234	$ 0.20	$ 47
12	SOIL RETENTION BLANKETS (CL 1) (TY A)	SY	468	$ 2.00	$ 936
13	VEGETATIVE WATERING	MG	42	$ 7.36	$ 310
14	ROCK FILTER DAM, INSTALL, TY 3 OR 4	LF	30	$ 50.00	$ 1,500
15	ROCK FILTER DAM, REMOVE	LF	30	$ 8.18	$ 245
16	TEMPORARY SEDIMENT CONTROL FENCE INSTALL	LF	106	$ 2.76	$ 293
17	TEMPORARY SEDIMENT CONTROL FENCE REMOVE	LF	106	$ 0.67	$ 71
18	STABILIZED CONSTRUCTION EXIT, INSTALL, TY 1	SY	111	$ 21.50	$ 2,389
19	STABILIZED CONSTRUCTION EXIT, REMOVE	SY	111	$ 6.30	$ 700
		Subtotal EROSION CONTROL =			**$ 7,872**
MOBILIZATION					
20	PREPARE ROW / CLEARING AND GRUBBING	STA	4	$ 5,200.00	$ 20,800
21	MOBILIZATION @ 10%	LS	1	$ 21,310.55	$ 21,311
		Subtotal MOBILIZATION =			**$ 42,111**
STRUCTURAL					
22	EXISTING PIER STRUCTURAL SUPPORT	EA	2	$ 25,000.00	$ 50,000
23	CONCRETE PIPE CAP, CL A, 12-INCHES	SY	14	$ 100.00	$ 1,441
		Subtotal STRUCTURAL =			**$ 51,441**
				SUBTOTAL =	**$ 234,416**
				CONTINGENCY @ 20% =	$ 46,883
				TOTAL CONSTRUCTION COST =	**$ 281,299**
				ENGINEERING CONSTRUCTION PHASE SERVICES @ 8% =	$ 22,504
				TOTAL =	**$ 303,803**

The unit prices shown are based on recent bids on siilar projects in the Central Texas area. The quantities shown should be considered a rough estimate based on the preliminary engineering report. Since the design professional has no control over the cost of labor, materials, or equipment, or over the contractor's method of determining prices, or over the competitive bidding or market conditions, his opinions of probable cost provided for herein are to be made on the basis of his experience and qualifications. These opinions represent his best judgment as a design professional familiar with the construction industry. However, the design professional can not and does not guarantee that proposals, bids, or construction cost will not vary from the opinions of probable cost he has prepared. If the Owner wishes greater assurance as to the construction cost, he shall employ an independent cost estimator.

4.3 Alternative 2: Gabion Mattress Lined Benched Channel Section

Alternative 2 solution to the Liberty Ditch stabilization issues is to construct a channel with a concrete invert and side slopes but with gabion mattress lined benched section. This alternative would construct the same channel as proposed in Alternative 1, except with gabion mattresses (18-inch thickness) on the benched sections in lieu of concrete lining. Gabion mattresses are typically used for erosion control and embankment stability, as they are more flexible than concrete. Another advantage of gabion matresses is they would allow for water to permeate, thus reducing the hydrostatic pressure behind the wall that tends to push / uplift the concrete panels. Bendway weirs would be installed along the channel invert to direct flows around the bend. As in Alternative 1, a surface ditch would be constructed along the southwest side of the channel to convey any overflow surface water back to Liberty Ditch at a point downstream.

Alternative 2 plans are shown in Appendix E in plan Sheets 8 through 14. Cross section 1+80 on Sheet 9 shows where the proposed channel benching would be close to the existing 30-inch force main. A concrete cap is proposed to be installed through this section to protect the force main, but there would be a minimal clearance distance of protection. Table 5-2 shows our estimate of probable cost to construct Alternative 2 of approximately $328,000.

Table 5-2: Alternative 2 Estimate of Probable Construction Costs

PAY ITEM NO.	DESCRIPTION	UNITS	QUANTITY	UNIT PRICE	ITEM COST
REMOVAL					
1	REMOVE CONCRETE CHANNEL	SY	818	$ 25.00	$ 20,448.33
2	REMOVE CONCRETE RIPRAP	SY	98	$ 11.00	$ 1,083
		Subtotal REMOVAL =			**$ 21,532**
DRAINAGE					
3	EXCAVATION (CHANNEL)	CY	549	$ 20.00	$ 10,980
4	EXCAVATION (SURFACE SWALE)	CY	62	$ 8.00	$ 494
5	EMBANKMENT (FINAL) (ORD COMP) (TYA)	CY	2	$ 50.00	$ 100
6	CONCRETE STRUCTURES, CL C, 6-INCHES	CY	133	$ 500.00	$ 66,500
7	GABION MATTRESSES, 18-INCHES	CY	120	$ 325.00	$ 39,000
8	RIPRAP (CONC) (CL B) (4 ")	CY	4	$ 400.00	$ 1,756
9	EMBANKMENT, ROCK DRAINAGE LAYER	CY	126	$ 70.00	$ 8,820
		Subtotal DRAINAGE =			**$ 127,651**
EROSION CONTROL					
10	FURNISHING AND PLACING TOPSOIL (4")	SY	468	$ 1.50	$ 702
11	BROADCAST SEED (PERM) (URBAN) (CLAY)	SY	468	$ 1.35	$ 632
12	BROADCAST SEED (TEMP) (WARM)	SY	234	$ 0.20	$ 47
13	BROADCAST SEED (TEMP) (COOL)	SY	234	$ 0.20	$ 47
14	SOIL RETENTION BLANKETS (CL 1) (TY A)	SY	468	$ 2.00	$ 936
15	VEGETATIVE WATERING	MG	42	$ 7.36	$ 310
16	ROCK FILTER DAM, INSTALL, TY 3 OR 4	LF	30	$ 50.00	$ 1,500
17	ROCK FILTER DAM, REMOVE	LF	30	$ 8.18	$ 245
18	TEMPORARY SEDIMENT CONTROL FENCE INSTALL	LF	106	$ 2.76	$ 293
19	TEMPORARY SEDIMENT CONTROL FENCE REMOVE	LF	106	$ 0.67	$ 71
20	STABILIZED CONSTRUCTION EXIT, INSTALL, TY 1	SY	111	$ 21.50	$ 2,389
21	STABILIZED CONSTRUCTION EXIT, REMOVE	SY	111	$ 6.30	$ 700
		Subtotal EROSION CONTROL =			**$ 7,872**
MOBILIZATION					
22	PREPARE ROW / CLEARING AND GRUBBING	STA	4	$ 5,200.00	$ 20,800
23	MOBILIZATION @ 10%	LS	1	$ 22,986.79	$ 22,987
		Subtotal MOBILIZATION =			**$ 43,787**
STRUCTURAL					
24	EXISTING PIER STRUCTURAL SUPPORT	EA	2	$ 25,000.00	$ 50,000
25	CONCRETE PIPE CAP, CL A, 12-INCHES	SY	20	$ 100.00	$ 2,014
		Subtotal STRUCTURAL =			**$ 52,014**
				SUBTOTAL =	**$ 252,855**
				CONTINGENCY @ 20% =	$ 50,571
				TOTAL CONSTRUCTION COST =	**$ 303,426**
				ENGINEERING CONSTRUCTION PHASE SERVICES @ 8% =	$ 24,274
				TOTAL =	**$ 327,700**

The unit prices shown are based on recent bids on siilar projects in the Central Texas area. The quantities shown should be considered a rough estimate based on the preliminary engineering report. Since the design professional has no control over the cost of labor, materials, or equipment, or over the contractor's method of determining prices, or over the competitive bidding or market conditions, his opinions of probable cost provided for herein are to be made on the basis of his experience and qualifications. These opinions represent his best judgment as a design professional familiar with the construction industry. However, the design professional can not and does not guarantee that proposals, bids, or construction cost will not vary from the opinions of probable cost he has prepared. If the Owner wishes greater assurance as to the construction cost, he shall employ an independent cost estimator.

Table 4-3: 100-year ACE Hydraulic Results Comparison

XS Station	WSEL, (ft)			Velocity Total, (ft/s)			Shear Total, (lb/sqft)			WSEL Difference, (ft)		Velocity Difference, (ft/s)		Shear Difference, (lb/sqft)	
	Existing	Alt 1	Alt 2	Existing	Alt 1	Alt 2	Existing	Alt 1	Alt 2	Alt 1	Alt 2	Alt 1	Alt 2	Alt 1	Alt 2
957	785.78	784.36	784.7	9.4	11.83	11.03	0.31	0.54	4.04	-1.42	-1.08	2.43	1.63	0.23	3.73
948	786.03	784.93	785.11	6.47	7.72	7.39	0.16	0.22	1.64	-1.1	-0.92	1.25	0.92	0.06	1.48
939	784.68	784.97	785.15	8.07	6.76	6.43	0.26	0.2	1.38	0.29	0.47	-1.31	-1.64	-0.06	1.12
924	785.02	783.66	784.68	6.84	10.52	7.3	0.22	0.37	1.61	-1.36	-0.34	3.68	0.46	0.15	1.39
908	784.25	784.04	784.73	8.9	7.52	5.8	0.22	0.13	0.79	-0.21	0.48	-1.38	-3.1	-0.09	0.57
885	784.26	784.18	784.84	5.38	5.04	3.84	0.14	0.09	0.41	-0.08	0.58	-0.34	-1.54	-0.05	0.27
860	784.03	784.34	784.92	4.76	3.36	2.65	0.11	0.07	0.19	0.31	0.89	-1.4	-2.11	-0.04	0.08
804	783.73	783.91	783.86	4.53	3.77	3.76	0.15	0.12	0.12	0.18	0.13	-0.76	-0.77	-0.03	-0.03
758	784.18	784.02	784.03	2.46	2.73	2.68	0.06	0.06	0.06	-0.16	-0.15	0.27	0.22	0	0

4.4 *Nationwide 404 Permit Implications*

Nationwide Permits (NWPs) are a type of general permit designed to authorize certain activities that have minimal individual and cumulative adverse effects on the aquatic environment and generally comply with the related laws cited in 33 CFR 320.3. NWP 3 allows for the repair /rehabilitation of the Liberty Ditch as long as the repair/rehabilitation is commenced within two (2) years of the date of destruction by flood.

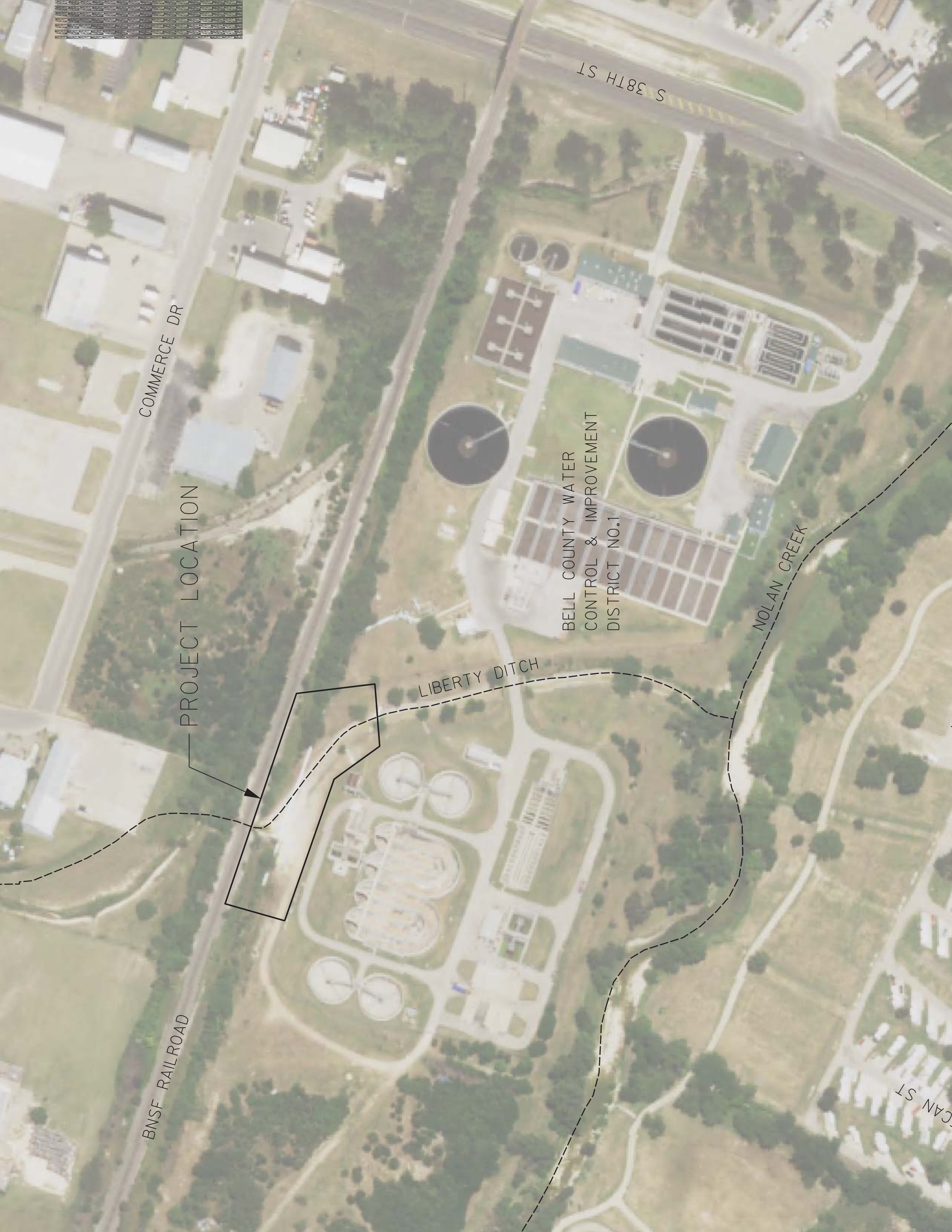
5 SUMMARY AND RECOMMENDATIONS

The primary goal of this preliminary engineering analysis is to develop alternative solutions to stabilize a bank failure along Liberty Ditch that will protect existing critical infrastructure. Halff considered two (2) alternative solutions to stabilize the bank of Liberty Ditch. Alternative 1 requires the construction of a concrete lined channel and benched section to reduce the velocities and subsequently stabilize the channel section. Alternative 2 requires construction of a concrete lined invert and side slopes and gabion mattress benched section. Each of the aforementioned alternatives require re-construction of approximately 300 linear feet of Liberty Ditch.

Halff recommends Alternative 1 because the concrete section will have a higher structural capacity and provide a longer service life compared to the gabion mattresses alternative. Alternative 1 allows approximately 12-inches of clearance to the existing 30-inch force main while Alternative 2 allows minimal clearance. In addition, the probable cost difference between Alternative 1 is approximately $24,000 less than the probable cost estimate of Alternative 2.

Halff recommends construction of Alternative 1 to successfully reduce the velocities and shear stresses of the flows in the Liberty Ditch channel. These reductions, in addition to the keyed concrete, granular filter drain, and surface swale, are expected to provide a significantly stabilized channel section over present conditions, at a total project cost of approximately $304,000. Estimates of probable construction cost for the proposed Liberty Ditch bank stabilization improvements were obtained from the October-December 2016 bid tab data for the Texas Department of Transportation. Tables 4-1 and 4-2 summarize the estimated costs of Alternatives 1 and 2, respectively.

Appendix A: EXHIBITS



S 38TH ST
COMMERCE DR
PROJECT LOCATION
BELL COUNTY WATER
CONTROL & IMPROVEMENT
DISTRICT NO.1
NOLAN CREEK
LIBERTY DITCH
BNSF RAILROAD

Proposed Benched Channel
BNSF Railroad
Liberty Ditch
WCID Road
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939
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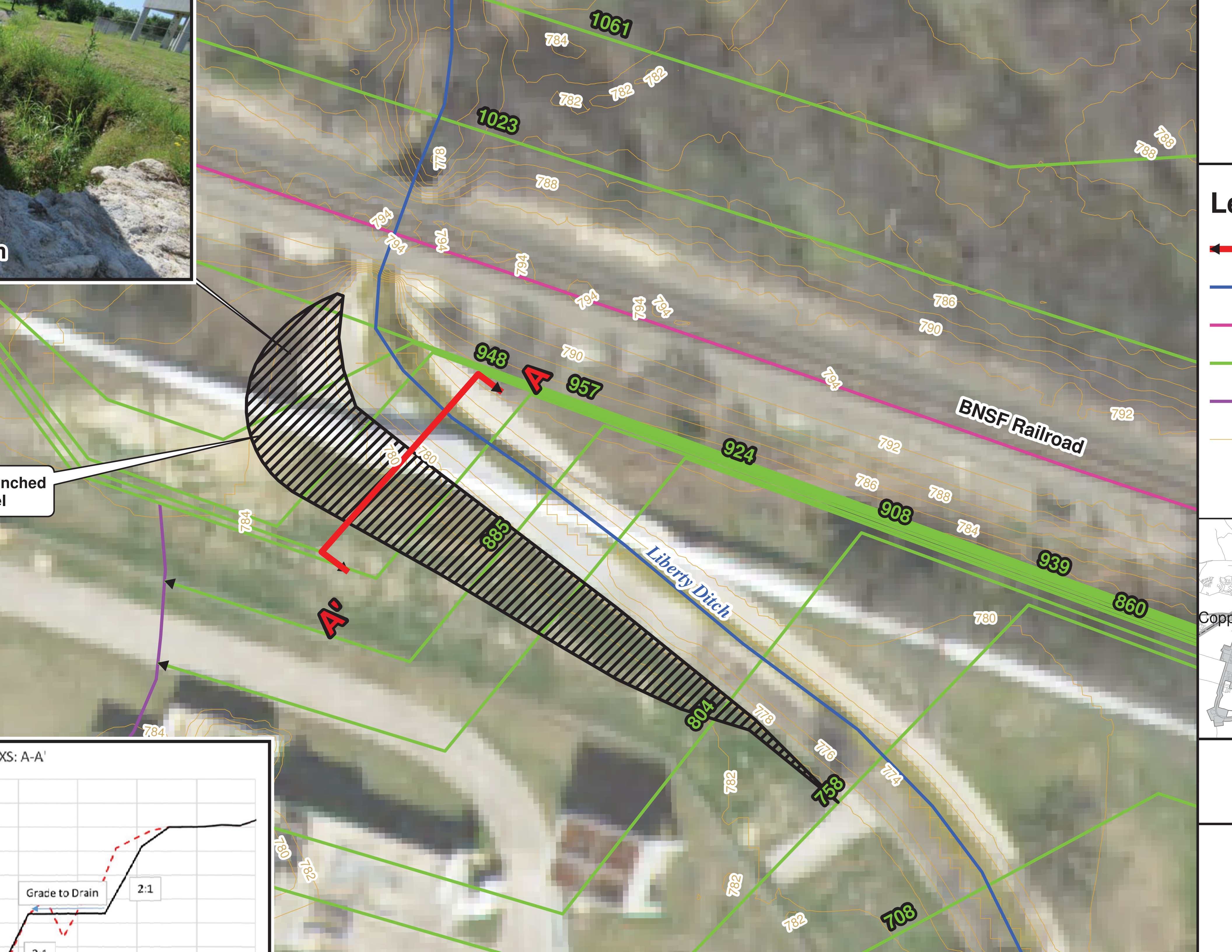

BNSF Railroad
Liberty Ditch
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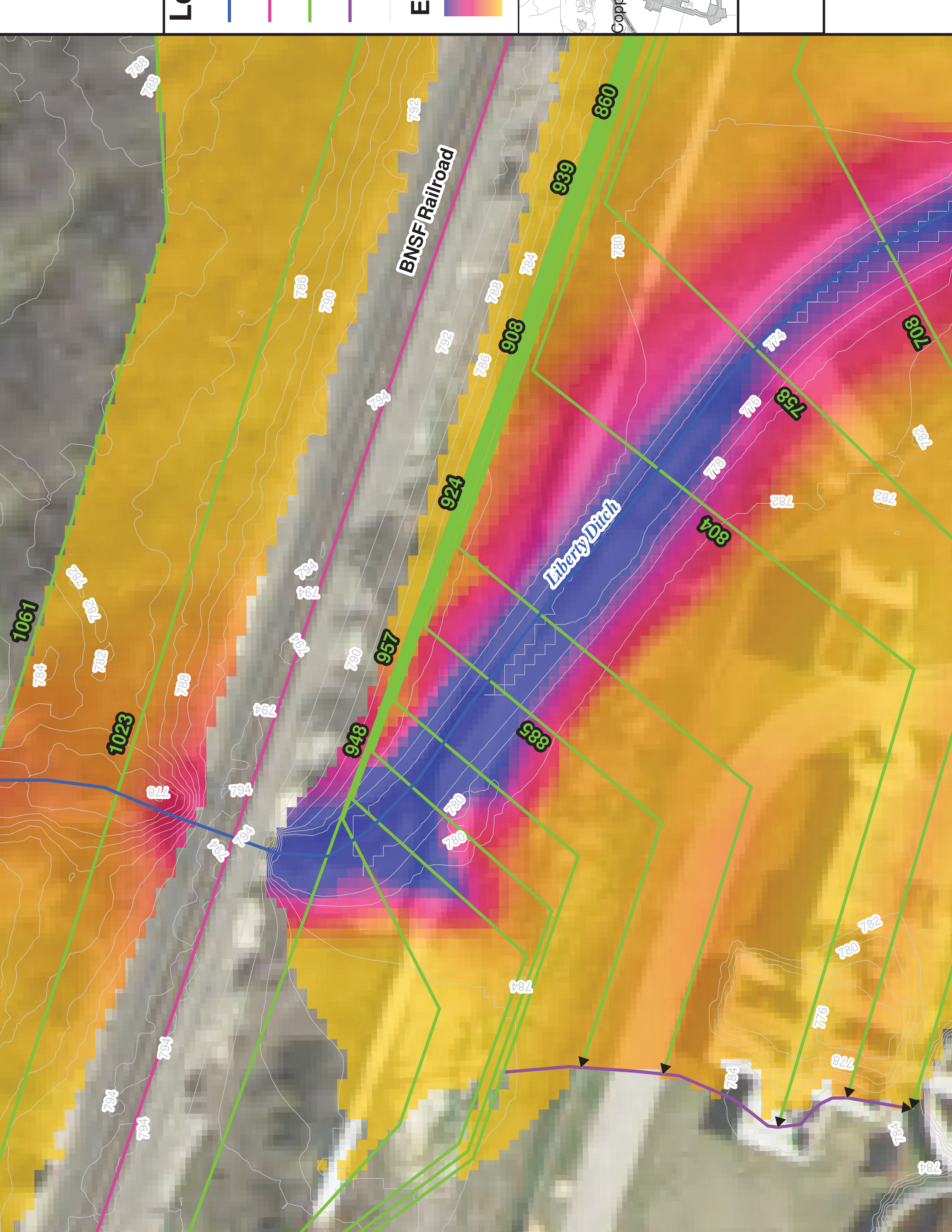

BNSF Railroad
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Appendix B:
GEOTECHNICAL INVESTIGATION

LANGERMAN FOSTER
ENGINEERING COMPANY

January 17, 2017

Halff Associates, Inc.
4030 West Braker Lane, Suite 450
Austin, TX 78759-5356

Attention: Paul Morales, PE, CFM, CPESC, Senior Project Manager
Email: pMorales@Halff.com

Reference: Geotechnical Investigation
Liberty Ditch Repair, Bell County WCD
Killeen, Texas
LFE Project No. W16-090

This letter transmits our report, which has been electronically produced. We appreciate the opportunity to provide geotechnical engineering services for you.

Once the project plans and specifications are completed, we would be pleased to review those portions that pertain to this report. We would also appreciate the chance to provide construction phase services such as materials testing as a part of the success of the project.

If you have any questions regarding our report, please call me at (254) 235-1048.

Respectfully Submitted,

LANGERMAN FOSTER ENGINEERING COMPANY, LLC
Texas Registered Engineering Firm No. F-13144

Ottis Foster

Ottis Foster, P.E.
Principal / Geotechnical Engineer

GEOTECHNICAL INVESTIGATION

Liberty Ditch Repair, Bell County WCD
Killeen, Texas

LFE Project No. W16-090

Report Prepared For:

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Austin, TX 78759-5356

Report Prepared By:

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STATE OF TEXAS
OTTIS FOSTER
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January 17, 2017

LANGERMAN FOSTER ENGINEERING COMPANY

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GEOTECHNICAL INVESTIGATION
LIBERTY DITCH REPAIR, BELL COUNTY WCD
KILLEEN, TEXAS

1.0 INTRODUCTION

Project Description: A drainage channel bank section has 'failed' beneath an aerial pipeway, and a pier support is being remediated. The project vicinity is shown on Plate 1. The scope of services is described in Langerman Foster Engineering (LFE) proposal numbers GEO16-061R2 and R3, dated respectively November 14 and December 1, 2016.

Purpose: The purpose of this geotechnical investigation has been to:

- Provide information for use by others to stabilize the south channel bank to prevent further erosion; and

- Provide foundation design and construction recommendations for use by a structural engineer to design a pier to support the existing aerial pipeway, including LPile parameters and sulfate test results.

These recommendations are based on field investigations, laboratory investigations, and engineering analysis of the investigation results.

2.0 SUBSURFACE EXPLORATION

Drilling Date: November 22, 2016.

Boring Layout: Three bores were done at the approximate locations shown on Plate 2. A registered professional land surveyor should be retained if elevation data and more precise location information is desired.

Sampling and Drilling Operations: Due to equipment access issues, a track-mounted Geoprobe was used for drilling. Push-tubes were used in cohesive soils, and a split-spoon sampler in conjunction with the Standard Penetration Tests (SPT, ASTM D 1586) in clayey and granular soils and rock. Sampling and drilling details are shown on the boring logs in the Appendix.

Borings were observed for water while drilling. Observations are noted on the borings logs and discussed subsequently. Borings were backfilled upon completion.

3.0 LABORATORY TESTS

Test Procedures: The following tests were conducted in general conformance with the standards noted in Table 3.1.

TABLE 3.1: LABORATORY TESTS	
Test Name	***Test Method***
Atterberg Limits	ASTM D 4318
-#200 Mesh Sieve	ASTM D 1140
Moisture Content	ASTM D 2216
Soil Classification	ASTM D 2487
Unconfined Compression (soil)	ASTM D 2166
Sulfate Detection	TEX 145-E, Part II

Test Results: Laboratory test results are tabulated on Plate 3 in the Appendix, and on the boring logs. Test results are also discussed subsequently.

4.0 SUBSURFACE MATERIALS

Geology: The Geologic Atlas of Texas[1] maps the site in the Walnut Clay (K_{wa}) geologic formation. The Walnut is described as consisting of clay, limestone, and shale, with fossil beds common in the lower part. Residual soils overlying the limestone vary from fat to lean clays with varying granular content.

Stratigraphy: The boring logs in the Appendix provide more detailed material descriptions. Material descriptions are general and range of depths approximate because boundaries between different strata are seldom

[1] Virgil E. Barnes, Project Director, Geologic Atlas of Texas, Waco Sheet, The University of Texas at Austin Bureau of Economic Geology, 1970.

clear and abrupt in the field. Differentiating between fill and natural material is especially difficult when the fill is similar to natural soils.

Rock was encountered at the approximate depths shown in Table 4.1. In Boring 1, rock base fill material was encountered to about 6 ½ feet, followed by severely weathered limestone (a mixture of clay, silt, sand, and broken limestone) to the top of limestone. In Boring 2, clayey fill was encountered to about 4 feet, followed by fat clay to about 6 feet, and then clayey sand to clayey gravel to the top of the limestone. In Boring 3, about 2 feet of fat clay fill was encountered over fat clay that extended to the top of rock (about 12 feet deep).

TABLE 4.1 ROCK DEPTH (ft)

Boring No.	*Weathered Limestone, Tan*	*Limestone, Gray*
1	9	10
2	10	12
3	12	15

Groundwater: Borings 1 and 3 were dry while drilling. Water was initially observed in Boring 2 at about 8 feet below ground surface (BGS), and remained at 8 feet after about 10 minutes.

It is common to encounter shallow groundwater in the Central Texas area, especially during and after periods of rainfall. The water tends to percolate down through the surficial soils until encountering a relatively impervious layer, and then either flow down gradient or become trapped. Water also tends to fill fractures and joints within the rock mass. Water can also move from the drainage channel into the surrounding soils.

These short-term readings do not constitute a groundwater study. The presence of water at certain depths does not mean the absence of water at other depths, only that it was not observed elsewhere while drilling. It could exist elsewhere on site, and it could exist within the boring depths at different elevations at different times. Excavation operations should be prepared to handle subsurface water and/or caving soils, as is discussed further in the design and construction recommendations section of this report. LFE must be contacted if groundwater is encountered during construction to evaluate whether subsurface drain systems or other improvements are needed.

Site Observations: Following are site pictures. Plates 2 and 2B also depict the site.

Photo 1. Boring 1.

Photo 2. Boring 2.

Photo 3. Boring 3.

Photo 4. General Area

Photo 5. General Area.

Photo 6. General Area.

Subsurface Material Characteristics:

Soil Movement Potential. Clay soils in the Central Texas area tend to swell when allowed to increase in moisture content, and shrink when allowed to decrease in moisture content. The moisture fluctuations occur due to seasonal wet and dry cycles, but are also influenced after construction by site grading, drainage, landscaping, and groundwater. Some clay soils swell when the overlying load is reduced, such as in the bottom of excavations. Soil movements can occur vertically, affecting foundations, and laterally, affecting retaining walls. Actual soil movement is difficult to predict due to the many variables involved.

Table 4.2 shows the potential vertical movement (PVM, up or down) estimated by the TxDOT method[2] for the clays encountered in the borings, as well as the depth of removal and replacement required to reduce the movement potential.

TABLE 4.5: PVM AND MOVEMENT REDUCTION			
Boring No.	***Existing PVM (in)***	***Depth of R&R for PVR < 2 in***	***Depth of R&R for PVR < 1 in***
B-1	2-1/2 to 3	2	7
B-2	2-1/2 to 3	3	7
B-3	2-1/2 to 3	6	10
Note: Movement estimates do not account for random settlement in the existing fill.			

[2] Tex-124-E. Assuming moisture changing from dry to wet conditions, a 15 foot depth of potential soil movement, and using the laboratory test results.

Sulfates. Two sulfate tests were done in Boring 1. One sample was from the 0 to 2 foot depth, and one from the 8 ½ to 10 foot depth. Both results were less than 100 ppm.

Subsurface Strength Characteristics. Non-engineered fill has unknown strength characteristics. No unusually weak materials were encountered in the natural materials. The limestone offers the most strength.

Laboratory strength tests are tabulated on Plate 3, and shown on the bore logs. Field strength tests are also on the bore logs.

5.0 GEOTECHNICAL RECOMMENDATIONS

Project Information: A drainage channel bank section has 'failed' beneath an aerial pipeway, and a pier support is being remediated. Following are geotechnical parameters and recommendations for these items.

Expansive Soil: Project elements and features must be designed to account for the estimated soil movement potential. Soil movement potential estimates are provided on Table 4.1. Remove and replace existing expansive soils with stable material as shown in Table 4.1 to reduce the movement potential as needed.

Actual soil movements will depend on the subsurface moisture fluctuations over the life of the structure. Soil movements may be less than those calculated if moisture variations are minimized after construction. However, significantly larger soil movements than estimated could occur due to inadequate site grading, poor drainage, ponding of rainfall, and/or leaking utilities.

Existing Fill: Existing fill material was logged in the borings. Because fill materials can be similar to native soils, it can be difficult to distinguish between the fill and native. Therefore, the depths and characteristics of the materials shown on the boring logs are approximations.

Improperly compacted, and/or mixed fill materials are subject to random settlement, and significant total and differential settlements of elements supported in the fill can occur. ***To ensure consistent support for construction elements, existing fill that is discovered below construction***

elements must be removed and replaced with select fill, or re-worked and re-compacted if deemed acceptable by the Geotechnical Engineer.

Be aware that the borings drilled for this investigation were exclusively for geotechnical purposes. Geotechnical reasons for removing the fill were not apparent to LFE, but could be revealed during construction.

LFE does not provide environmental services such as may be warranted for such fill. Other environmental-related investigations by an environmental professional may be warranted.

Pier Support: Piers should penetrate at least 3 feet into gray limestone, and terminate on a hard layer. Gray limestone was encountered between 10 and 15 feet in the borings. It may be shallower or deeper in the field. 3,000 psf allowable side shear may be used beginning at the top of the gray limestone, and 20,000 psf in allowable bearing capacity.

Steel reinforcement is required, and must meet code and other project requirements. LFE advises at least ½ % steel reinforcement.

Groundwater was encountered in Boring 2 at about 8 feet, but not in the other bores. Groundwater and/or caving soils may be present during installation of the drilled shafts. Temporary steel casing will be necessary for such conditions. We recommend that the contractor verify the drilling and groundwater conditions prior to starting drilled shaft installation.

Use the L-Pile parameters provided in Table 5.1 to design lateral load resistance. The values shown are subject to interpretation, and LFE should be contacted if the structural engineer believes the values generate overly (or insufficiently) conservative designs.

TABLE 5.1: LATERAL LOAD DESIGN PARAMETERS									
Depth (feet)	***LPile Soil Type***	***Soil Type***	***Cohesion (psf)***	***φ (deg)***	***ε50 (Soil) or k_{rm} (Rock)***	***Unit Wt [B] (pcf)***	***RQD (%)***	***Initial Modulus of Rock Mass (psi)***	***Uniaxial Comp. Strength (psi)***
0 to 3	Soft Clay	Clay	100 [A]	---	0.02	115	---	---	---
Varies	Clay[C]	Soil of varying clay and granular content	1,000	--	0.008	115	--	--	--
Varies	Weak Rock	WLS[D] and LS[E]	---	---	0.0005	130	20	15,000	1,500

[A]Minor cohesion, unit weight, and ε50 as listed above may be used to account for the effect of the overburden soils.

[B]Unit weights listed above represent total unit weights. In conditions where soils are below the water table, submerged unit weights should be used. Submerged unit weight is obtained by subtracting the unit weight of water from the unit weight listed above. Unit weight of water is 62.4 pcf.

[C]includes severely weathered limestone. [D]Weathered limestone. [E]Limestone.

Channel Bank Remediation: Boring 1 is nearest the channel bank that is to be remediated. Based on the available information, it appears existing materials in the problem area should be removed vertically and laterally as needed to reveal firm, natural ground. Additional excavation should be conducted as needed to provide suitable area for installing appropriate embankment materials. The existing material may be acceptable for re-use based on Boring 1, but should be stockpiled for confirmation by the geotechnical engineer with respect to final plans and specifications.

Others are responsible for designing appropriate erosion control measures to prevent undermining, washing out, or other undesirable results from drainage events. LFE understands that numerous products are available and common in such environments, including:

- Concrete or rock rip rap
- Gabions
- Reinforced concrete facing
- Concrete retaining wall
- Roller-compacted concrete

If the existing fill is deemed suitable for re-use behind the erosion-control elements, it must be well-mixed to achieve a consistent material and installed in controlled lifts. Larger rock complicates compaction efforts, and in general rock size should be less than about 3 inches unless appropriately accounted for during construction. In general, fat clay should not be used as fill, and is recommended for removal unless the design team is aware of and accounts for the negative effects that can result from fill that includes fat clay. Cement can be added to many types of fill material to increase strength and erosion resistance.

Assuming the slope is composed of properly compacted materials such as were encountered in Boring 1 and that water does not saturate the slope[3], a 2:1 slope less than 10 feet high provides at least a 1.5 safety factor. However, the face requires erosion protection as discussed previously. For steeper angles, the slope face must resist lateral loads such as are provided in Table 6.1 in the following Section 6. The lateral design loads may be reduced proportionally with the slope angle, since the lateral load decreases with decreasing wall angle.

Miscellaneous Geotechnical Parameters

Existing Fill: Existing fill materials were logged in the borings. Because fill materials can be similar to native soils, it can be difficult to distinguish between the fill and native. Therefore, the depths and characteristics of the materials shown on the boring logs are approximations. Improperly compacted, and/or mixed fill materials are subject to random settlement, and significant total and differential settlements of elements supported in the fill can occur. ***To ensure consistent support for construction elements, existing fill that is discovered below construction elements must be removed and replaced with select fill, or re-worked and re-compacted if deemed acceptable by the Geotechnical Engineer.***

Be aware that the borings drilled for this investigation were exclusively for geotechnical purposes. LFE does not provide environmental services such as may be warranted for such fill. Other environmental-related investigations by an environmental professional may be warranted.

Seismic: For structural designs based upon the 2012 IBC, the following criteria will apply. The Site Class is C. The Mapped Spectral Response Acceleration at short periods (SS) is about 0.10g, and the Mapped Spectral Response Acceleration at a 1 second period (S1) is about 0.04g. Site Coefficients are as follows: F_a= 1.2 and F_v= 1.7.

[3] This assumes a relatively conservative 16° internal friction and 100 psf cohesion.

Utility Connections
In Expansive Soil
Situations: Utilities resting on or within expansive soils are subject to soil movements. Utility connections should account for such movement potential, such as by using flexible connections.

6.0 RETAINING WALL RECOMMENDATIONS

Project Information: Figure 6.1 and Table 6.1 provide information for retaining structures less than about 10 feet in height. Since the site is on a creek, the design engineer must address erosion issues. Include hydrostatic pressures in the lateral load calculations unless water such as might exist immediately after a flood event can be dissipated prior to imposing its lateral load. In our opinion, rapid draw-down conditions would exist immediately after a flood event, and hydrostatic pressures would be difficult to dissipate prior to imposing its lateral load. Table 6.1 assumes undrained conditions.

Constructing near creek beds may expose softer/weaker soils that were not revealed by the borings and which may not adequately support the foundation for a retaining wall unless firmed up for foundation construction. If such soils are encountered during construction, they must be removed and replaced with select fill, or may be appropriate for re-working to appropriate compaction and moisture content. Another alternative is to use clean crushed stone placed at the base of the excavation to create a working surface. Such conditions should be analyzed on a case by case basis.

If the option of using clean crushed stone is selected to improve the subgrade support for the wall foundation, we expect that a layer of about 6 to 12 inches in thickness will be needed, but field conditions will dictate the thickness.

The crushed stone must be clean, and should generally range in size from 3 to 6 inches. Compaction specifications do not apply; however, the rock should be placed in such a manner that will stabilize the bottom of the excavation. This type of clean rock is normally used to stabilize construction entrances, and should be readily available.

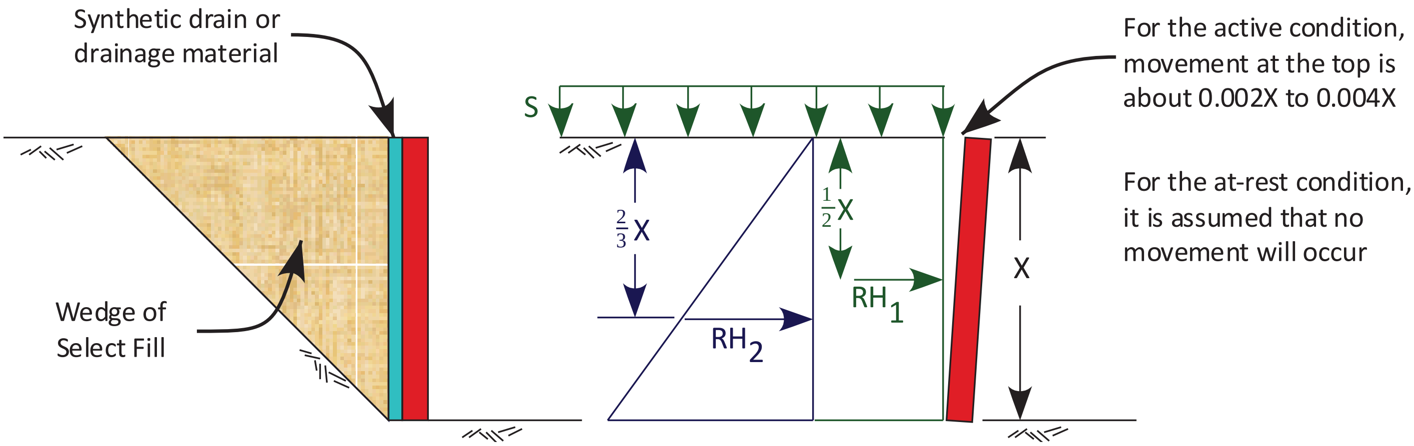


Figure 6.1

TABLE 6.1: EARTH PRESSURE PARAMETERS BELOW WATER				
Earth Pressure	**Coefficient**	**Equivalent Fluid Pressure (pcf)**	**Surcharge Pressure, P_1 (psf)**	**Earth Pressure, P_2 (psf)**
At-Rest (K_0)	0.50	91	(0.50)S	91X
Active (K_A)	0.33	81	(0.33)S	81X

- Values assume a 1H:1V wedge of select fill behind the wall with a unit weight of 120 pcf.
- Values include hydrostatic pressure. Values may be reduced accordingly for a drained condition.
- Earth pressure parameters do not include a factor of safety
- Drainage material: ASTM C-33, Size 67 gravel aggregate, uniformly compacted
- Base sliding resistance: 500 psf (or a 0.3 coefficient of friction)
- Footing bearing pressure: 2,000 psf
- Resultant Horizontal Forces per linear foot:
  - R_{H1}= (P_1)(X), where R_{H1} is acting at ½X from the top of the wall
  - R_{H2}= (0.5)(P_2)(X), where R_{H2} is acting at ⅔X from the top of the wall

7.0 GEOTECHNICAL CONSTRUCTION RECOMMENDATIONS

Site Preparation: For general site preparation, remove at least the existing 6 inches plus any organics, debris, utilities, underground structures, non-engineered fill or other unsuitable material. Additionally remove the existing fat clays to the depth required to achieve movement reduction goals. The stripping depth must consider field observations with attention given to old drainage areas, uneven topography, and wet soils.

Proof-roll to detect soft spots or pumping subgrade areas for remediation using a heavy pneumatic tired roller, loaded dump truck, or similar piece of equipment weighing at least 25 tons. Subgrade soils must also be compacted as required by the following Subgrade Preparation subsection.

Use special care when removing subsurface structures to make sure that disturbed ground is appropriately compacted to prevent future unwanted settlement.

Subgrade Preparation: Scarify and re-compact the exposed subgrade soils to at least 95 percent of ASTM D698 (or TEX-113-E) maximum dry density at 0 to +3% of the optimum moisture content, based on a 6-inch compacted lift thickness. Limestone does not need compacting and should not be excessively disturbed.

Subgrade Improvement: The onsite soils may be unstable during construction, and subgrade pumping may occur. Where needed, crushed stone can be placed to create firm working surfaces. Field conditions will dictate the needed thickness.

The crushed stone must be clean, and should generally range in size from 3 to 5 inches. Compaction specifications do not apply; however, the rock should be placed in such a manner that will stabilize the soils. This type of clean stone is normally used to stabilize construction entrances, and should be readily available. However, this stone may interfere with utility installations and excavations in general. If more than about a foot of crushed stone is used, a filter fabric should be installed to prevent fines from migrating out of the overlying soils into the crushed stone.

Lime can also be used to create a working platform. Since this is a temporary measure to facilitate construction for the contractor's convenience, the contractor shall determine the lime percentage and procedure as the contractor may deem appropriate. However, lime can affect vegetation, and the Design Team must be made aware of plans for using lime to assess the effect on vegetation or other project aspects.

Select Fill: Fill should meet the requirements of 2014 TxDOT Item 247, Type A, Grade 3 or better. If another local source of select fill is desired, the following specification may be used as a guide:

Maximum Aggregate:	3 inches
Percent Retained on #4 Sieve:	25 - 50
Percent Retained on #40 Sieve:	50 - 75
Plasticity Index:	5 - 15
Non-Organic	

Other locally available non-expansive fill may be acceptable, but should be evaluated by LFE on a case-by-case basis. Compact the select fill material to at least 95 percent of ASTM D698 maximum dry density at ±3% of the optimum moisture content. Specify a 6-inch maximum compacted lift thickness, with each lift tested for compliance prior to adding subsequent lifts. LFE or a similarly qualified testing laboratory must observe, monitor, and test the fill placement and compaction on a full-time basis.

Grading: Water is often detrimental to the performance of constructed items. Site grading should prevent water from ponding around constructed items, and good drainage should be obvious to the casual observer.

Foundations: Foundation construction recommendations are listed below.

1. Follow ACI Manual of Concrete Practice, Item 336 for piers. LFE strongly recommends that the minimum slump requirements specified therein be followed, in part to more easily obtain uniform distribution of the concrete around the reinforcement and against the pier sides than is obtained with lower slumps.
2. Observe a 24-inch minimum pier shaft diameter to allow for cleaning, minimum construction tolerances, and conventional concrete mix designs. Smaller diameters may be used at the discretion of the structural engineer.

3. LFE or other qualified testing laboratory must observe the foundation construction to determine that the proper bearing material has been reached in accordance with the recommendations given herein. Bearing depth variations should be expected.
4. Remove water from foundation excavations prior to concrete placement. Prolonged exposure or inundation of the bearing surface with water may result in changes in bearing strength and compressibility characteristics. If delays occur, deepen and clean the drilled shaft excavations to provide a fresh bearing surface.
5. Place concrete promptly after excavations are completed, cleaned, and observed. Place pier concrete before the end of the work day.
6. Design the reinforcement steel cage placed in the shaft to meet at least the following two requirements: (1) the structural requirements for the imposed loads; and (2) stability requirements during the placement of concrete. Other structural or code requirements may also apply.
7. Groundwater or caving soils may require temporary steel casing per ACI 336.1 and ACI 336.3R. Place pier concrete at a minimum 6-inch slump when temporary steel casing is used. Some pier holes may not require temporary casing, and we advise that the bid schedule include installation of temporary casing as a separate unit-price bid item.

Construction Materials Testing: Foundation construction monitoring and materials testing (CMT) is strongly recommended to help the construction conform to the plans and specifications and to document how the construction conforms to the plans and specifications. This should be considered an essential part of achieving acceptable foundation performance.

8.0 DESIGN REVIEW AND LIMITATIONS

Design Review: The recommendations contained in this report were based on preliminary site plans and design information provided by the Client. Our recommendations may not be applicable if changes have been made to the original information that formed the basis for this report, and we must be retained to make such a determination if such changes have been made. We also must be given the opportunity to review construction documents to affirm that our recommendations have been interpreted correctly. We cannot be responsible for misinterpretations if not given the opportunity to review aspects of the project that are based on the contents of this report. Such a review is considered an additional service.

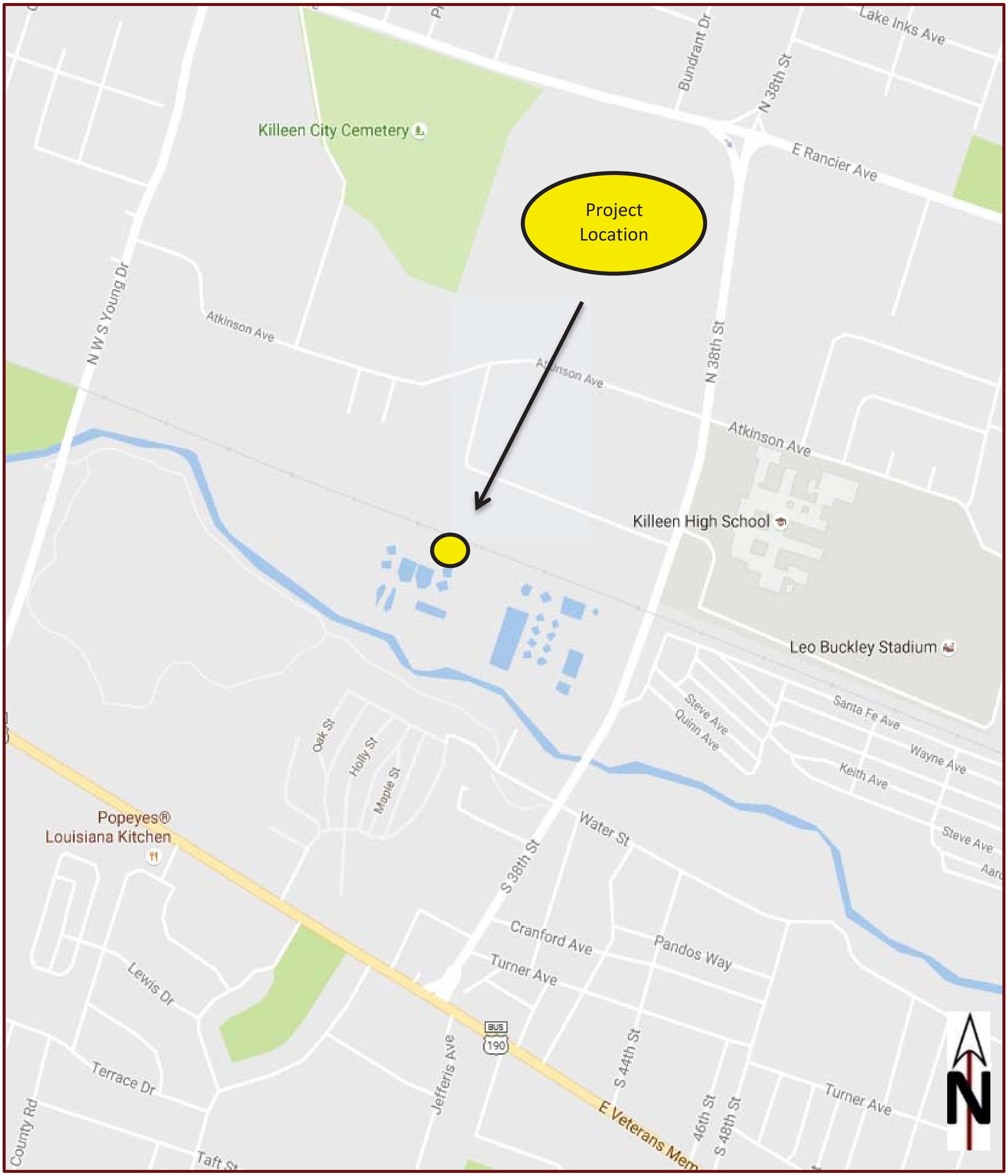
Limitations: This report has been prepared for the exclusive use of our client and their designated project design team. Preparation of the report has been performed using that degree of care and skill ordinarily exercised under similar conditions by reputable geotechnical engineers in the same locality. No warranties, express or implied, are intended or made.

As stated in the attachment titled 'Important Information About Your Geotechnical Engineering Report," the subsurface conditions are interpreted from samples taken only at the boring locations. During construction, variations will be encountered, and will require interpretation by LFE to verify the adequacy of the geotechnical recommendations. Other limitations and considerations are discussed in the attachment and are a part of this report.

LFE does not provide environmental services, and this investigation did not include environmental testing or evaluation. LFE does not know whether environmental services may be appropriate or required for this project. An environmental professional should be retained to evaluate whether such services are appropriate and/or necessary, and to provide such services when so deemed.

9.0 APPENDIX

Site Location Map – Plate 1
Boring Location Sketch – Plate 2
Laboratory Test Results – Plate 3
Explanation of Boring Log Symbols and Terms
Boring Logs
Important Information About Your Geotechnical Engineering Report



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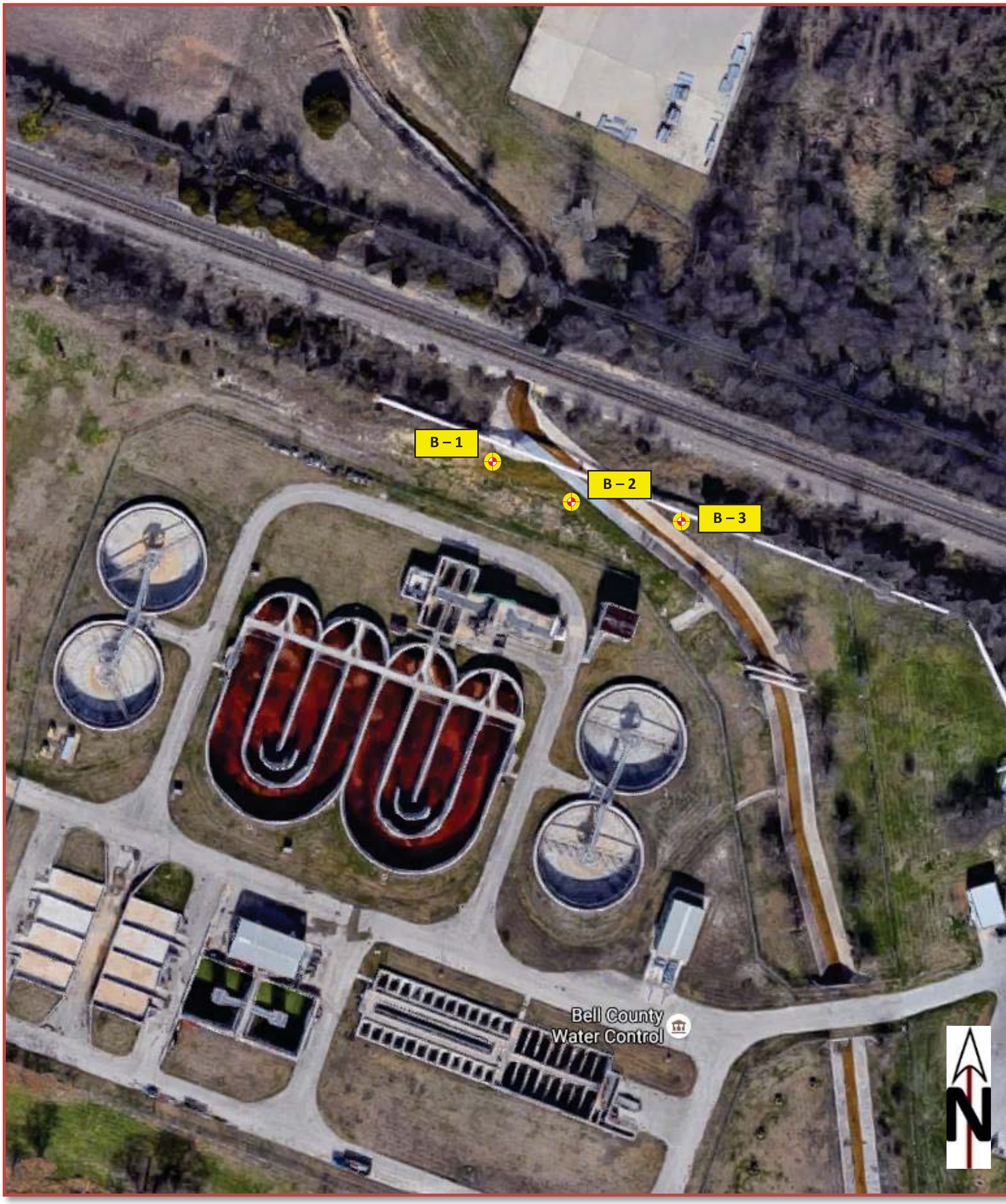
SITE LOCATION MAP

LIBERTY DITCH REPAIR, BELL COUNTY WCD
KILLEEN, TEXAS
LFE PROJECT NO. W16-090

PLATE

1



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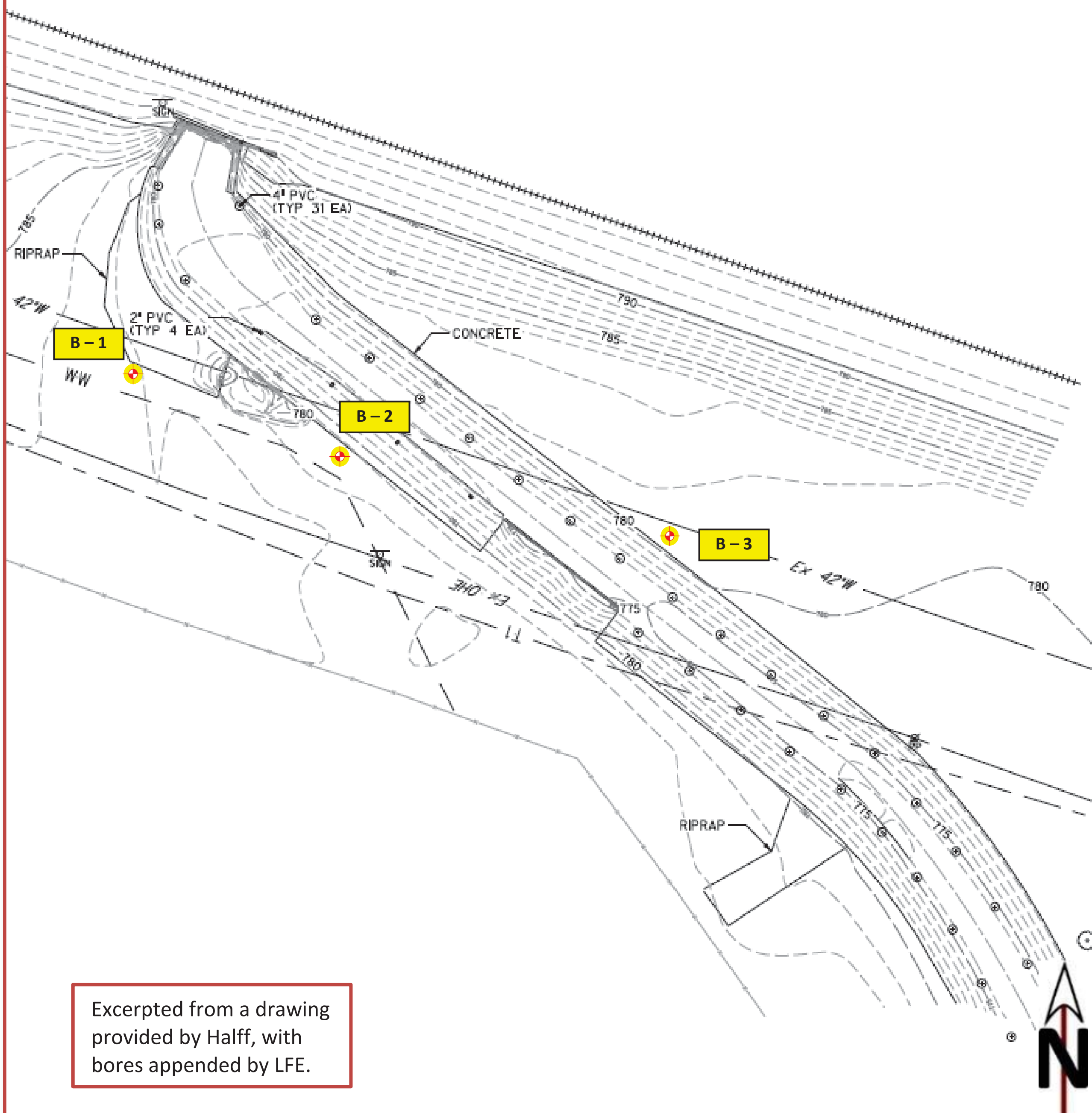
BORING LOCATION MAP

LIBERTY DITCH REPAIR, BELL COUNTY WCD
KILLEEN, TEXAS
LFE PROJECT NO. W16-090

PLATE

2



Excerpted from a drawing provided by Halff, with bores appended by LFE.

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BORING LOCATION MAP

LIBERTY DITCH REPAIR, BELL COUNTY WCD
KILLEEN, TEXAS
LFE PROJECT NO. W16-090

PLATE

2B

Boring No.	Sample Depth (ft.)	Liquid Limit	Plastic Limit	Plasticity Index	Percent Passing No. 200 Sieve	Moisture Content (%)	Unit Dry Weight (pcf)	Unconfined Compressive Strength (tsf)	Strain at Failure (%)
B-1	0.0 - 2.0				38	7			
B-1	2.0 - 4.0	28	16	12	24	7			
B-1	4.0 - 6.0				42	10			
B-1	6.5 - 8.0	27	15	12	91	15			
B-1	8.5 - 9.6					9			
B-1	10.5 - 10.9					10			
B-1	13.5 - 13.6					9			
B-1	15.0 - 15.3					11			
B-2	0.0 - 2.0	40	17	23	55	17			
B-2	2.0 - 4.0				83	23			
B-2	4.0 - 6.0	56	18	38	64	17			
B-2	6.0 - 8.0				46	19	106.2	1.1	12.3
B-2	8.0 - 10.0	38	16	22	38	22			
B-2	10.0 - 10.7					14			
B-2	12.0 - 12.8					23			
B-2	13.5 - 13.9					22			
B-2	15.0 - 15.2					24			
B-3	0.0 - 2.0					14			
B-3	2.0 - 4.0	74	21	53	67	12	114.9	6.2	7.8
B-3	4.0 - 6.0				85	26			
B-3	6.0 - 8.0	62	19	43	90	25	101.0	2.0	6.3
B-3	8.0 - 10.0				78	23			
B-3	10.0 - 12.0	51	16	35	65	18			
B-3	13.0 - 13.7					12			
B-3	15.0 - 15.8					14			

LANGERMAN FOSTER
ENGINEERING COMPANY

Summary of Laboratory Results

Project: Liberty Ditch Repair

Project Number: W16-090

Langerman Foster Engineering Company
Waco and Harker Heights (Killeen), Texas
Ph: 254-235-1048 www.LFECTX.com

KEY TO SYMBOLS

CLIENT Halff Associates, Inc.
PROJECT NAME Liberty Ditch Repair
PROJECT NUMBER W16-090
PROJECT LOCATION Killeen, Texas

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

- CH: USCS High Plasticity Clay
- CHS: USCS SANDY FAT CLAY
- CLG: USCS Low Plasticity Gravelly Clay
- FILL: Fill (made ground)
- LIMESTONE: Limestone
- SC: USCS Clayey Sand
- SEVERELY WEATHERED LIMESTONE: bolder than SP-SM
- WEATHERED LIMESTONE: bolder than SP-SM

SAMPLER SYMBOLS

- Bag Sample
- Split Spoon
- Shelby Tube

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
-200 - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION

Water Level at Time Drilling, or as Shown
Water Level at End of Drilling, or as Shown
Water Level After 24 Hours, or as Shown

Langerman Foster Engineering Company
Waco and Harker Heights (Killeen), Texas
Ph: 254-235-1048 www.LFECTX.com

BORING NO. B-1

CLIENT Halff Associates, Inc. **PROJECT NAME** Liberty Ditch Repair

PROJECT NUMBER W16-090 **PROJECT LOCATION** Killeen, Texas

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION Approximate Surface Elevation feet	SAMPLE TYPE	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)	STRAIN AT FAILURE (%)
0		FILL - CRUSHED LIMESTONE; tan, with clay, with varying granular content throughout	ST			2.5				38	7			
			ST			NT	28	16	12	24	7			
5			ST			NT				42	10			
			A											
		SEVERELY WEATHERED LIMESTONE; tan, (a mixture of clay, silt, sand, and broken limestone)	SS		6-20-50 (70)		27	15	12	91	15			
			A											
		WEATHERED LIMESTONE; tan, broken, with clayey seams and layers	SS		7-17-50/1"						9			
10		LIMESTONE; gray, broken, with clayey seams and layers	A											
			SS		50/4"						10			
			A											
			SS		50/5"									
			A											
			SS		50/1"						9			
			A											
15			SS		50/3"						11			

Completion Depth: 15.3 ft.
Date Started: 11/22/16
Completed: 11/22/16
Logged by: I. Lovett

Remarks: Boring was drilled without drilling fluid. Groundwater was not observed in the boring.

Langerman Foster Engineering Company
Waco and Harker Heights (Killeen), Texas
Ph: 254-235-1048 www.LFECTX.com

BORING NO. B-2

CLIENT Halff Associates, Inc.

PROJECT NAME Liberty Ditch Repair

PROJECT NUMBER W16-090

PROJECT LOCATION Killeen, Texas

Approximate Surface Elevation feet

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)	STRAIN AT FAILURE (%)
0	FILL - SANDY CLAY to CLAY with SAND; tan and gray, with gravel	ST			2.5	40	17	23	55	17			
		ST			2.5				83	23			
5	FAT SANDY CLAY; brown to gray, with gravel --- with a 3" gravelly sand layer at 5 feet	ST			2.0	56	18	38	64	17			
	CLAYEY SAND; brown to gray, with gravel	ST			1.5				46	19	106	1.1	12.3
10	LEAN CLAYEY GRAVEL; brown	ST			3.5	38	16	22	38	22			
		SS		17-50/2"						14			
	WEATHERED LIMESTONE; tan, broken, with clayey seams and layers	A											
	LIMESTONE; gray, broken, with clayey seams and layers	SS		11-50/3"						23			
		A											
		SS		50/4"						22			
		A											
15		SS		50/2"						24			

Completion Depth:	15.2 ft.	Remarks:	Boring was drilled without drilling fluid. Groundwater was initially measured about 8 feet below ground surface (BGS). After about 10 minutes, groundwater remained about 8 feet BGS.
Date Started:	11/22/16		
Completed:	11/22/16		
Logged by:	I. Lovett		

Langerman Foster Engineering Company
Waco and Harker Heights (Killeen), Texas
Ph: 254-235-1048 www.LFECTX.com

BORING NO. B-3

CLIENT Halff Associates, Inc. **PROJECT NAME** Liberty Ditch Repair

PROJECT NUMBER W16-090 **PROJECT LOCATION** Killeen, Texas

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION Approximate Surface Elevation feet	SAMPLE TYPE	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)	STRAIN AT FAILURE (%)
0		FILL - FAT CLAY; brown to gray, with gravel	ST			3.0					14			
		FAT CLAY; gray to light gray and tan, with varying granular content	ST			4.5+	74	21	53	67	12	115	6.2	7.8
5			ST			1.5				85	26			
			ST			2.0	62	19	43	90	25	101	2.0	6.3
10			ST			2.5				78	23			
			ST			2.0	51	16	35	65	18			
		WEATHERED LIMESTONE; tan, broken, with clayey seams and layers	A											
			SS		49-50/2"						12			
15			A											
		--- gray below 15 feet	SS		30-50/3"						14			

Completion Depth: 15.8 ft.
Date Started: 11/22/16
Completed: 11/22/16
Logged by: I. Lovett

Remarks: Boring was drilled without drilling fluid. Groundwater was not observed in the boring.

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

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

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

Appendix C:
SUBSURFACE UTILITY INVESTIGATION

SUE TEST HOLE DATA SHEET

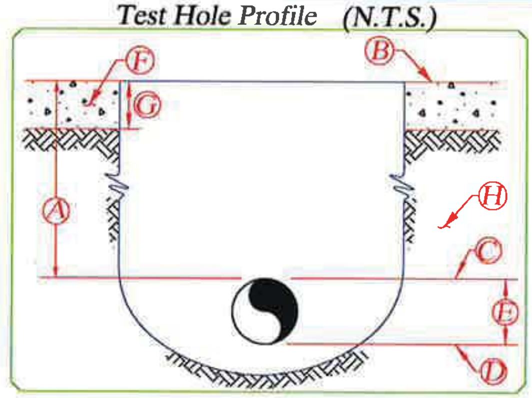
Halff Project # 32033
CSJ # N/A
City/ County Name: KILLEEN/BELL COUNTY
Location: LIBERTY DITCH
Limits: BETWEEN BNSF RAILROAD AND BELL COUNTY WC&ID #1 WW PLANT
Test Hole # 1
Utility: FORCE MAIN
Owner: CITY OF KILLEEN
Date: 10/24/16
Weather: CLOUDY/WARM

HALFF
TBPE #F-312
9500 Amberglen Blvd.
Building F, Suite 125
Austin, Texas 78729
Tel: 512.777.4600
Fax: 512.252.8141

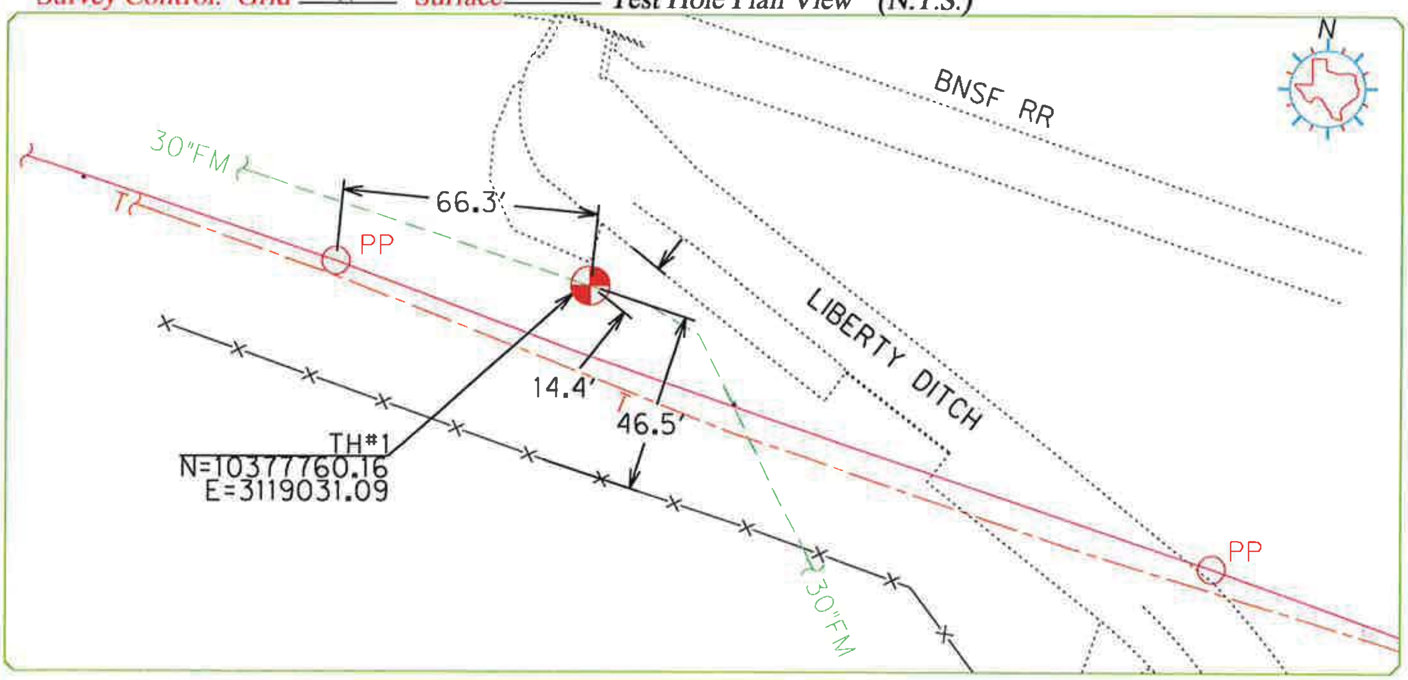
Test Hole Data

Approx. Sta. N/A Offset: N/A
Offset pulled from: N/A
(A) Measured Depth of Hole: 4.50'
(B) Surface Elevation: 782.80'
(C) Elev. @ Top of Utility: 778.30'
(D) Elev. @ Bottom of Utility: 775.80' (CALC.)
(E) Size of Utility: 30" (PER RECORDS)
Utility Material: DUCTILE IRON
Utility Condition: GOOD
(F) Thickness of Pvmt.: N/A
(G) Type of Pvmt., Base, etc: N/A
(H) Description of soil: CLAY/ROCK
Unit of Measure: English X Metric ____
Survey Control: Grid X Surface ____

Test Hole Profile (N.T.S.)



Test Hole Plan View (N.T.S.)



TIE	MEASUREMENTS	DISTANCE
1	TEST HOLE TO POWER POLE	66.3'
2	TEST HOLE TO FENCE	46.5'
3	TEST HOLE TO EDGE OF CONCRETE	14.4'

Field Notes/Legend:

- TEST HOLE
- PP POWER POLE
- EDGE OF CONCRETE
- FENCE

Certification: I hereby certify that the information shown hereon is correct to the best of my knowledge and belief.

STATE OF TEXAS
RYAN D. LEWIS
95767
LICENSED
PROFESSIONAL ENGINEER

11/9/16

SUE TEST HOLE DATA SHEET

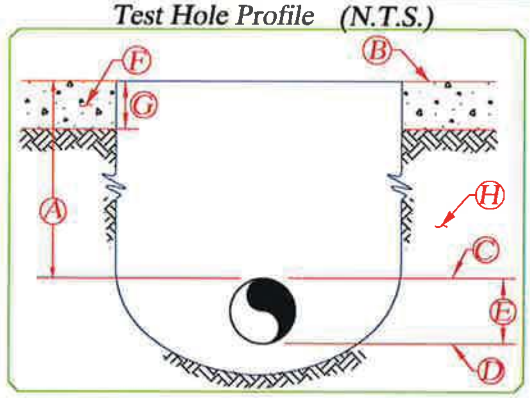
Halff Project # 32033
CSJ # N/A
City/ County Name: KILLEEN/BELL COUNTY
Location: LIBERTY DITCH
Limits: BETWEEN BNSF RAILROAD AND BELL COUNTY WC&ID #1 WW PLANT
Test Hole # 2
Utility: TELEPHONE
Owner: FRONTIER COMMUNICATIONS (VERIZON)
Date: 10/24/16
Weather: CLOUDY/WARM

HALFF
TBPE #F-312
9500 Amberglen Blvd.
Building F, Suite 125
Austin, Texas 78729
Tel: 512.777.4600
Fax: 512.252.8141

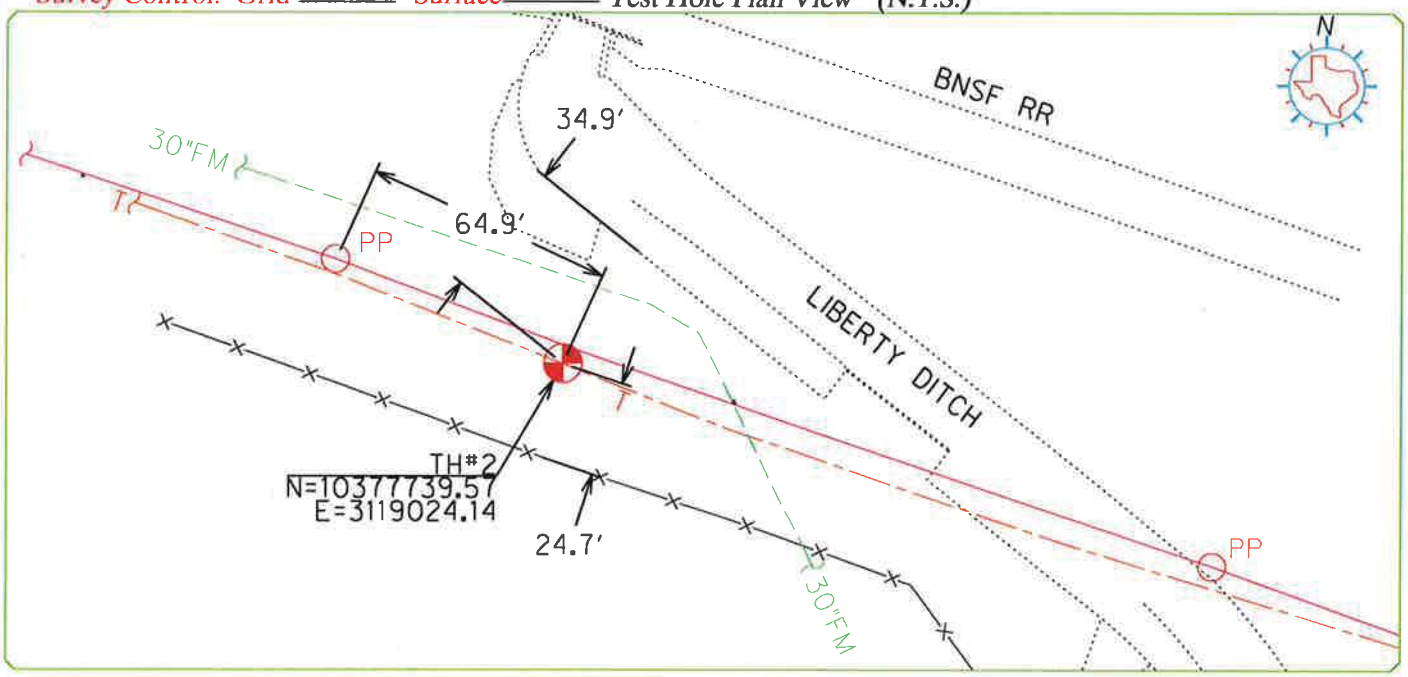
Test Hole Data

Approx. Sta. N/A Offset: N/A
Offset pulled from: N/A
(A) Measured Depth of Hole: 2.25'
(B) Surface Elevation: 784.10'
(C) Elev. @ Top of Utility: 781.85'
(D) Elev. @ Bottom of Utility: 781.81' (CALC.)
(E) Size of Utility: 1/2"
Utility Material: COPPER
Utility Condition: GOOD
(F) Thickness of Pvmt.: N/A
(G) Type of Pvmt., Base, etc: N/A
(H) Description of soil: ROCK/CLAY
Unit of Measure: English X Metric
Survey Control: Grid X Surface

Test Hole Profile (N.T.S.)



Test Hole Plan View (N.T.S.)



TIE	MEASUREMENTS	DISTANCE
1	TEST HOLE TO FENCE	24.7'
2	TEST HOLE TO POWER POLE	64.9'
3	TEST HOLE TO EDGE OF CONCRETE	34.9'

Certification: I hereby certify that the information shown hereon is correct to the best of my knowledge and belief.

STATE OF TEXAS
RYAN D. LEWIS
95767
LICENSED
PROFESSIONAL ENGINEER

11/9/16

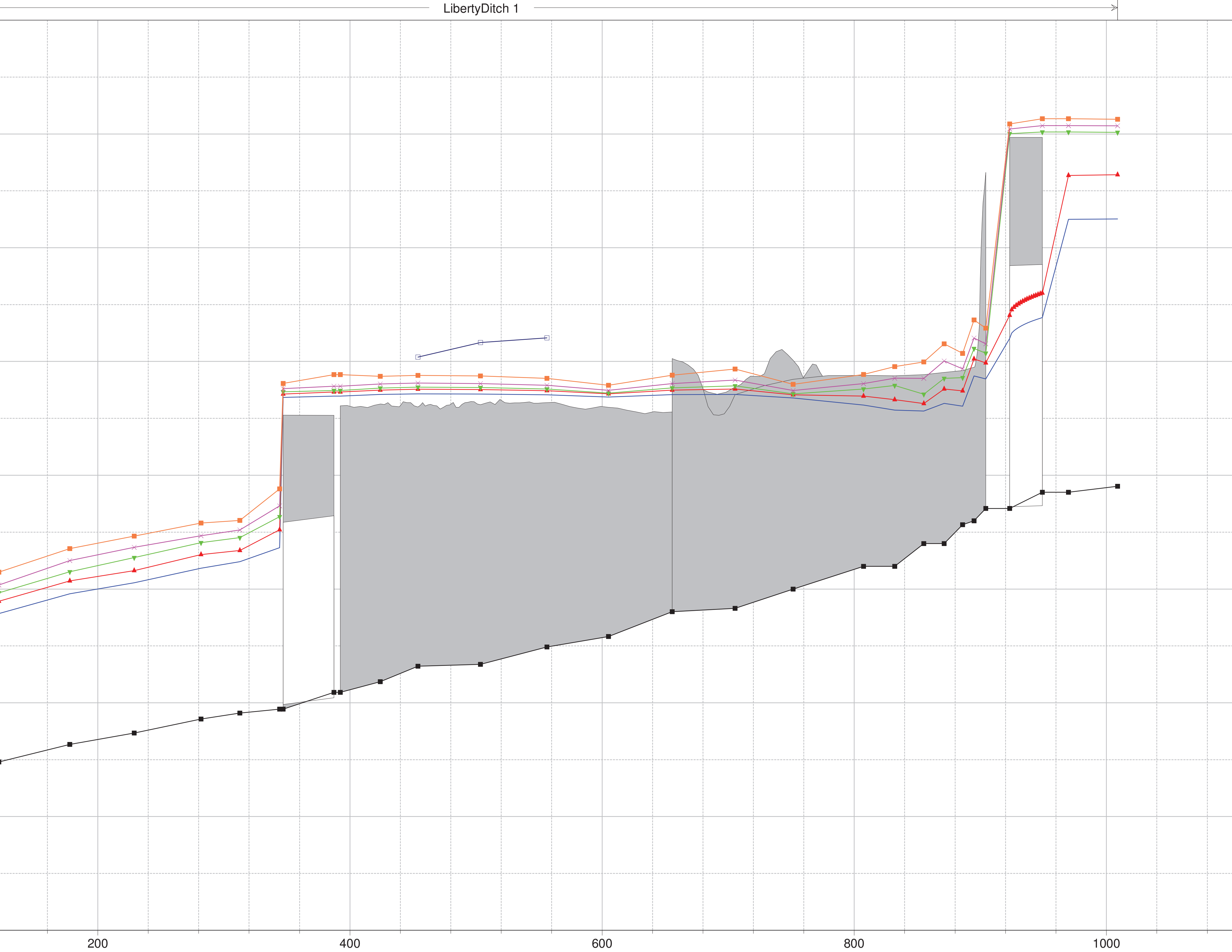
Field Notes/Legend:

- TEST HOLE
- PP POWER POLE
- EDGE OF CONCRETE
- FENCE

Appendix D:
HYDRAULIC ANALYSIS SUMMARY OF RESULTS

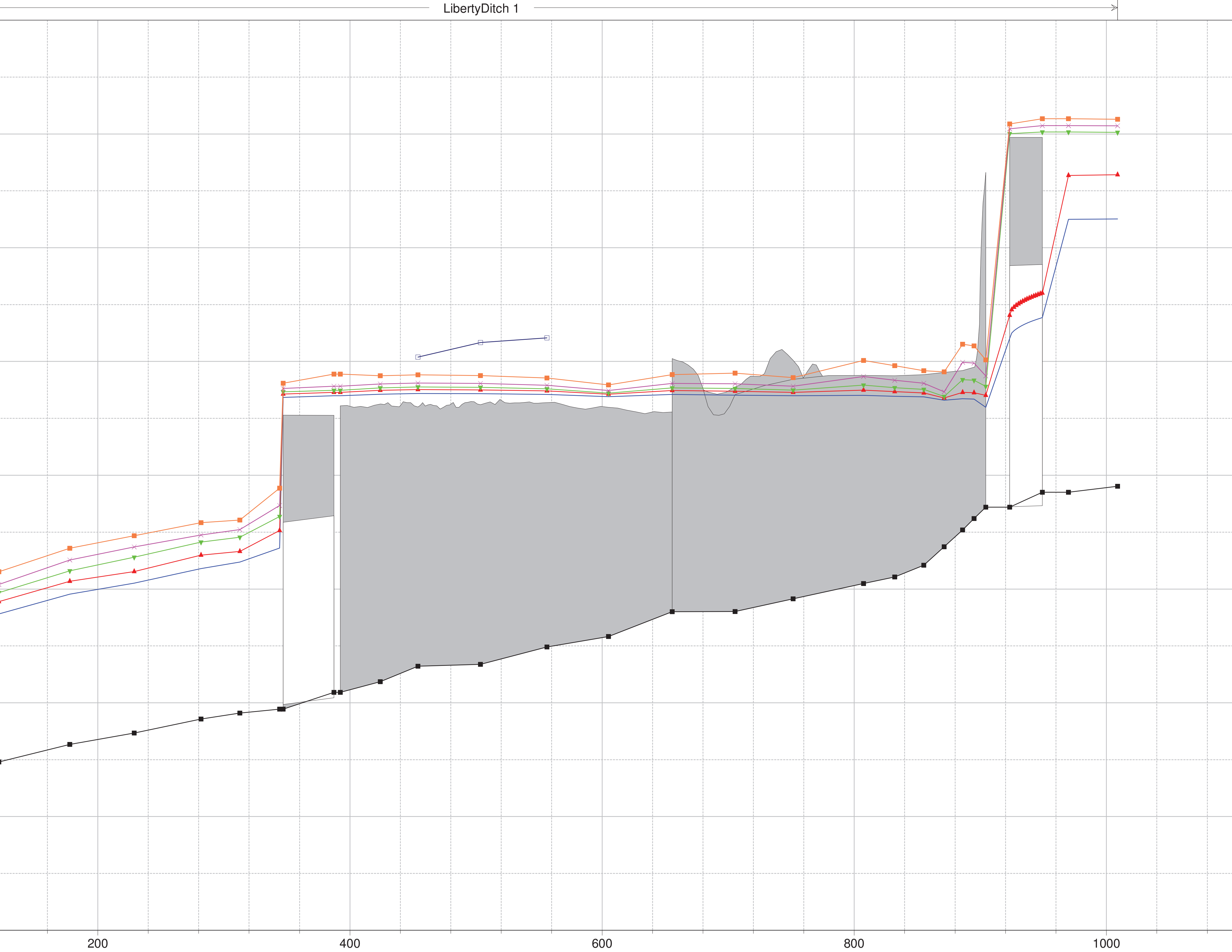
Existing Conditions Hydraulic Results

Reach	River Sta	Profile	Q Total, (cfs)	Min Ch El, (ft)	W.S. Elev, (ft)	Crit W.S., (ft)	E.G. Elev, (ft)	E.G. Slope, (ft/ft)	Vel Chnl, (ft/s)	Flow Area, (sq ft)	Top Width, (ft)	Froude # Chl
	1061	100-yr	2210	779.52	795.36	786.31	795.4	0.000016	2.14	4147.06	907.78	0.1
	1023	100-yr	2210	779.25	795.37	785.61	795.39	0.000007	1.51	5700.35	1426.17	0.07
	986	Culvert										
	957	100-yr	2210	778.54	785.78	784.38	787.15	0.000857	9.4	235.09	524.03	0.67
	956	Lat Struct										
	948	100-yr	2193.5	778	786.03	784.06	786.88	0.000536	7.5	339.11	602.89	0.53
	939	100-yr	2192.82	777.83	784.68	784.68	786.75	0.001486	11.98	271.59	581.58	0.87
	924	100-yr	2192.58	777	785.02	783.93	786.36	0.001201	9.64	320.57	610.68	0.66
	908	100-yr	2192.39	777	784.25	784.25	786.27	0.001523	11.6	246.28	571.98	0.87
	907	Lat Struct										
	885	100-yr	2174.17	776	784.26	784.26	785.76	0.001117	10.54	404.35	571.38	0.75
	860	100-yr	2144.08	776	784.03	784.03	785.22	0.000926	9.66	450.78	572.12	0.69
	804	100-yr	2130.63	775	783.73	782.91	784.91	0.000806	9.52	470.61	528.77	0.65
	758	100-yr	2129.68	774.15	784.18	782.3	784.68	0.000335	6.87	867.49	567	0.43
	708	100-yr	2090.91	774	784.03	781.87	784.65	0.000353	7.12	660.63	263.38	0.44
1	707	Lat Struct										
	658	100-yr	2058.94	772.91	783.73	781	784.6	0.000465	7.69	389.7	143.83	0.49
	609	100-yr	2034.77	772.45	783.96	780.49	784.48	0.000266	5.98	497.55	154.95	0.38
	556	100-yr	1995.62	771.69	784.03	779.86	784.43	0.000191	5.27	602.14	177.52	0.32
	507	100-yr	1917.64	771.61	784.05	779.08	784.41	0.000145	4.96	585.65	170.56	0.29
	477	100-yr	1879.06	770.93	784.01	778.39	784.4	0.00014	5.28	622.67	194.93	0.29
	445	100-yr	1857.53	770.46	783.91	778.77	784.37	0.000147	5.76	702.92	314.84	0.3
	422	Culvert										
	397	100-yr	1857.53	769.72	778.65	778.65	782.74	0.001815	16.21	114.59	37.21	1
	365	100-yr	1857.53	769.55	777.6	777.6	779.6	0.001787	11.46	192.14	79.28	0.9
	335	100-yr	1857.53	769.29	777.35	777.35	779.41	0.001715	11.68	202.89	88.76	0.89
	282	100-yr	1857.53	768.67	776.83	776.83	778.75	0.001559	11.29	217.45	107.37	0.85
	230	100-yr	1857.53	768.17	776.25	776.25	778.25	0.001675	11.49	204.56	95.03	0.88
	174	100-yr	1857.53	767.4	775.18	775.18	777.38	0.002071	11.92	164.92	60.01	0.97
	119	100-yr	1857.53	766.77	774.43	774.43	776.92	0.002089	12.69	153.25	39.2	0.98
	53	100-yr	1857.53	764.85	772.56	772.56	774.9	0.002363	12.28	151.28	33.22	1.01


LibertyDitch 1
200
400
600
800
1000

Alternative 1 Hydraulic Results

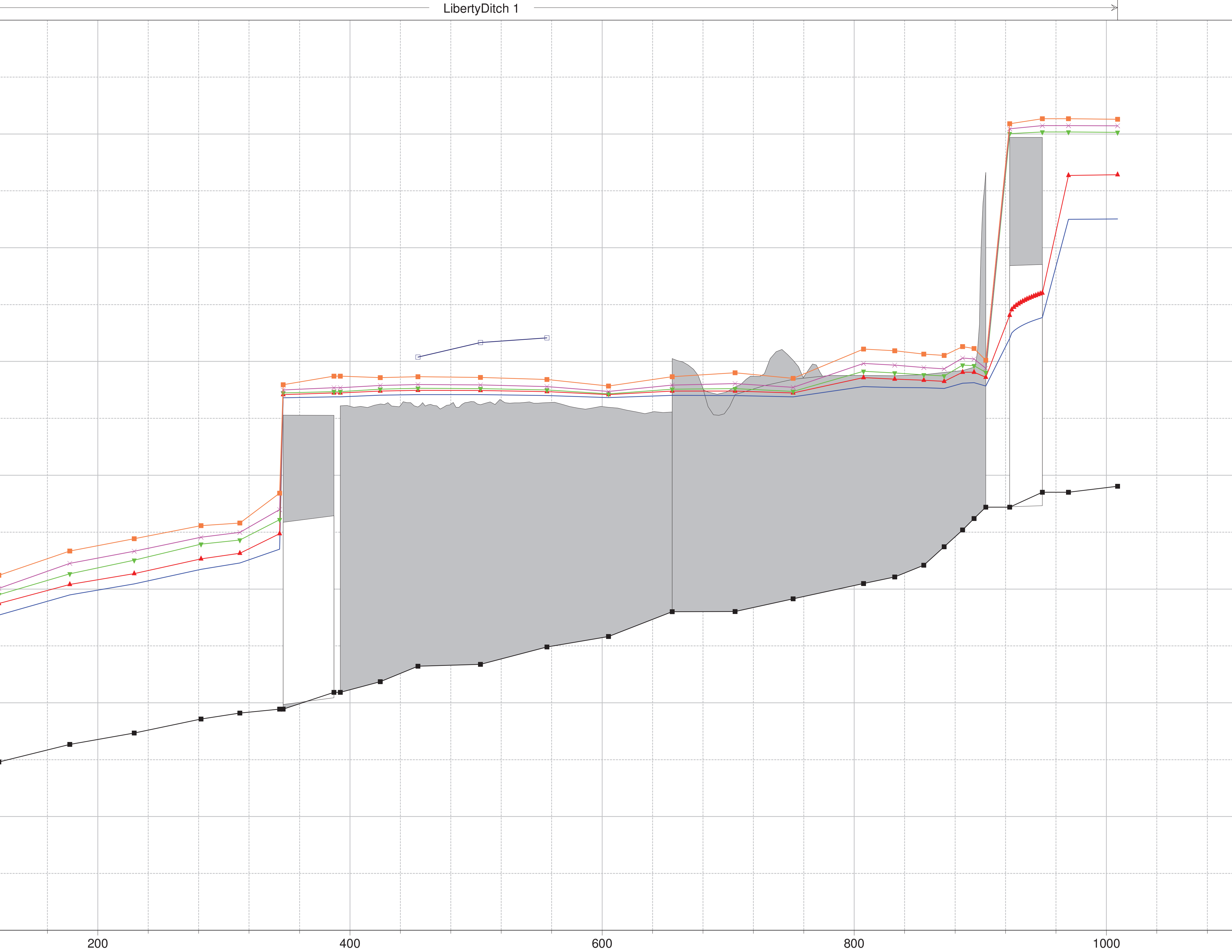
Reach	River Sta	Profile	Q Total, (cfs)	Min Ch El, (ft)	W.S. Elev, (ft)	Crit W.S., (ft)	E.G. Elev, (ft)	E.G. Slope, (ft/ft)	Vel Chnl, (ft/s)	Flow Area, (sq ft)	Top Width, (ft)	Froude # Chl
1	1061	100-yr	2210	779.52	795.36	786.31	795.4	0.000016	2.14	4147.06	907.78	0.1
	1023	100-yr	2210	779.25	795.37	785.61	795.39	0.000007	1.51	5700.35	1426.17	0.07
	986	Culvert										
	957	100-yr	2210	778.6	784.36	784.32	786.71	0.001981	12.34	186.78	463.74	0.98
	956	Lat Struct										
	948	100-yr	2209.66	778.1	784.93	783.8	786.11	0.000905	8.81	286.07	559.73	0.67
	939	100-yr	2209.49	777.6	784.97	783.7	786.05	0.001015	8.57	327.06	603.2	0.63
	924	100-yr	2209.46	776.86	783.66	783.64	785.74	0.002354	11.62	210.01	463.57	0.93
	908	100-yr	2209.46	776.05	784.04	782.79	785.32	0.000884	9.17	293.86	548.5	0.67
	907	Lat Struct										
	885	100-yr	2195.72	775.53	784.18	782.5	785.15	0.000628	8.23	436.05	566.22	0.57
	860	100-yr	2159.56	775.24	784.34	782.75	784.96	0.000429	7.1	642.04	593.79	0.48
	804	100-yr	2136.21	774.57	783.91	782.4	784.84	0.000574	8.54	567.3	539.04	0.55
	758	100-yr	2135.7	774.01	784.02	782.28	784.69	0.000374	7.87	782.6	567	0.46
	708	100-yr	2102.8	774	784.03	781.9	784.66	0.000356	7.16	661.05	263.42	0.44
	707	Lat Struct										
	658	100-yr	2070.66	772.91	783.73	781.02	784.61	0.000471	7.74	389.27	143.82	0.49
	609	100-yr	2046.3	772.45	783.96	780.5	784.49	0.000268	6.01	497.92	154.97	0.38
	556	100-yr	2006.74	771.69	784.03	779.87	784.43	0.000193	5.3	602.74	177.56	0.32
	507	100-yr	1928.21	771.61	784.06	779.09	784.41	0.000146	4.99	586.15	170.59	0.29
	477	100-yr	1889.37	770.93	784.01	778.4	784.41	0.000141	5.31	623.17	194.95	0.29
	445	100-yr	1867.71	770.46	783.92	778.79	784.38	0.000148	5.79	703.61	314.86	0.3
	422	Culvert										
	397	100-yr	1867.71	769.72	778.67	778.67	782.78	0.00182	16.26	114.86	37.25	1
	365	100-yr	1867.71	769.55	777.61	777.61	779.62	0.001782	11.48	193.61	79.42	0.9
	335	100-yr	1867.71	769.29	777.37	777.37	779.43	0.001698	11.66	205.5	89.53	0.89
	282	100-yr	1867.71	768.67	776.85	776.85	778.77	0.001552	11.29	219.71	107.82	0.85
	230	100-yr	1867.71	768.17	776.28	776.28	778.27	0.001659	11.48	207.35	96.51	0.88
	174	100-yr	1867.71	767.4	775.2	775.2	777.4	0.00206	11.93	166.28	60.49	0.96
	119	100-yr	1867.71	766.77	774.45	774.45	776.95	0.002085	12.71	154.01	39.39	0.98
	53	100-yr	1867.71	764.85	772.57	772.57	774.93	0.002364	12.31	151.77	33.29	1.01



LibertyDitch 1
200
400
600
800
1000

Alternative 2 Hydraulic Results

Reach	River Sta	Profile	Q Total, (cfs)	Min Ch El, (ft)	W.S. Elev, (ft)	Crit W.S., (ft)	E.G. Elev, (ft)	E.G. Slope, (ft/ft)	Vel Chnl, (ft/s)	Flow Area, (sq ft)	Top Width, (ft)	Froude # Chl
	1061	100-yr	2210	779.52	795.36	786.31	795.4	0.000016	2.14	4147.06	907.78	0.1
	1023	100-yr	2210	779.25	795.37	785.61	795.39	0.000007	1.51	5700.35	1426.17	0.07
	986	Culvert										
	957	100-yr	2210	778.6	784.7	784.33	786.64	0.013723	11.31	200.31	506.79	0.87
	956	Lat Struct										
	948	100-yr	2208.76	778.1	785.11	783.84	786.07	0.006578	8.1	299.04	563.16	0.61
	939	100-yr	2208.41	777.6	785.15	783.96	785.9	0.006593	7.41	343.61	604.94	0.54
	924	100-yr	2208.11	776.86	784.68	783.91	785.71	0.009442	8.58	302.63	608.19	0.62
	908	100-yr	2207.9	776.05	784.73	782.83	785.46	0.004302	7.26	380.83	617.98	0.5
	907	Lat Struct										
	885	100-yr	2163.46	775.53	784.84	782.52	785.21	0.00229	5.59	563.96	588.07	0.37
	860	100-yr	2087.85	775.24	784.92	783.39	785.06	0.001003	3.8	789.18	596.41	0.24
	804	100-yr	2044.59	774.57	783.86	782.47	784.78	0.000615	8.5	543.55	537.13	0.56
	758	100-yr	2044.1	774.01	784.03	781.95	784.57	0.000369	7.04	761.55	567	0.44
	708	100-yr	2013.07	774	783.96	781.76	784.55	0.000341	6.96	646.7	261.24	0.43
1	707	Lat Struct										
	658	100-yr	1983.46	772.91	783.69	780.86	784.51	0.000442	7.47	384.11	143.6	0.48
	609	100-yr	1961.29	772.45	783.9	780.38	784.39	0.000254	5.82	489.58	154.41	0.37
	556	100-yr	1926.12	771.69	783.97	779.72	784.35	0.000184	5.14	591.69	176.87	0.32
	507	100-yr	1855.42	771.61	783.99	778.98	784.33	0.000139	4.84	577.19	170.06	0.28
	477	100-yr	1820.55	770.93	783.95	778.28	784.32	0.000135	5.16	613.25	194.5	0.28
	445	100-yr	1801.12	770.46	783.85	778.6	784.29	0.000143	5.66	683.53	314.26	0.3
	422	Culvert										
	397	100-yr	1801.12	769.72	778.49	778.49	782.49	0.001826	16.04	112.28	36.9	1
	365	100-yr	1801.12	769.55	777.49	777.49	779.48	0.00182	11.4	183.57	78.48	0.91
	335	100-yr	1801.12	769.29	777.28	777.28	779.28	0.001689	11.5	197.11	87.61	0.88
	282	100-yr	1801.12	768.67	776.66	776.66	778.63	0.001653	11.38	199.71	97.35	0.88
	230	100-yr	1801.12	768.17	776.13	776.13	778.13	0.001715	11.45	193.45	92.29	0.89
	174	100-yr	1801.12	767.4	775.03	775.03	777.24	0.002174	11.95	156.2	52.71	0.98
	119	100-yr	1801.12	766.77	774.31	774.31	776.77	0.002124	12.6	148.67	38.04	0.99
	53	100-yr	1801.12	764.85	772.47	772.47	774.76	0.00236	12.15	148.26	32.82	1.01


LibertyDitch 1
200
400
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800
1000

Appendix E:
PRELIMINARY PLAN SHEETS