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REHABILITATION OF DRAINAGE STRUCTURES AT EFD

HAS Project No. 954

Engineer's Report
Ellington Airport (EFD)

AUGUST 2023

100% Submittal

Notice

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1. General Scope of Project

1.1. History

Ellington Airport (EFD), located in Harris County, is approximately 15 miles southeast of downtown Houston, Texas. EFD is a joint operation base airport in the Houston Airport System that supports U.S. Military, NASA, and a variety of general aviation tenants. EFD has three runways: 17R-35L (9,001' x 150'), 17L-35R (4,609' x 75'), and 4-22 (8,001' x 150'), with parallel and cross taxiway systems.

1.2. Scope of Project

The project includes the design services to reconstruct two headwalls/culverts at EFD Ditch C culvert on the south side of the airfield, and a headwall on the northeast corner of the airfield. The project locations are shown below in Figure 1-1.

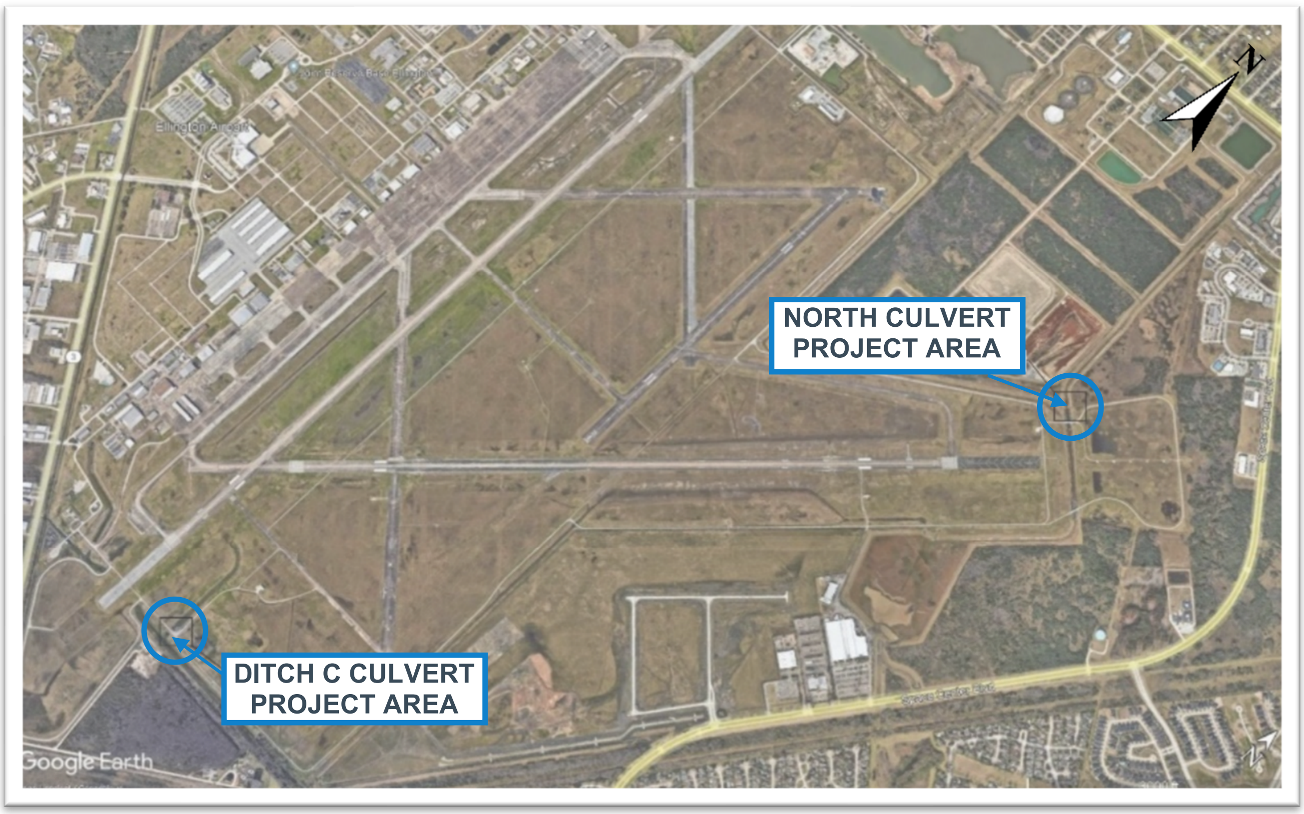

Figure 1-1: Culvert Locations

2. Consideration for Airport Operational Safety

2.1. Operational Constraints

Work shall be performed on an active airfield.

2.2. Staging Area, Haul Routes, and Project Access

Contractor access to the AOA shall be limited to gate W-21. Access to the project sites will be via the existing perimeter Vehicle Service Road (VSR), excepting larger delivery vehicles limited by turning radii, particularly delivery of the concrete box culverts. These deliveries will be coordinated with airport operations and escort provided across the Runway 35L overrun. General staging area will be outside the AOA near gate W-21. The detailed haul routes, access routes, and staging area are shown in Figure 2-1.

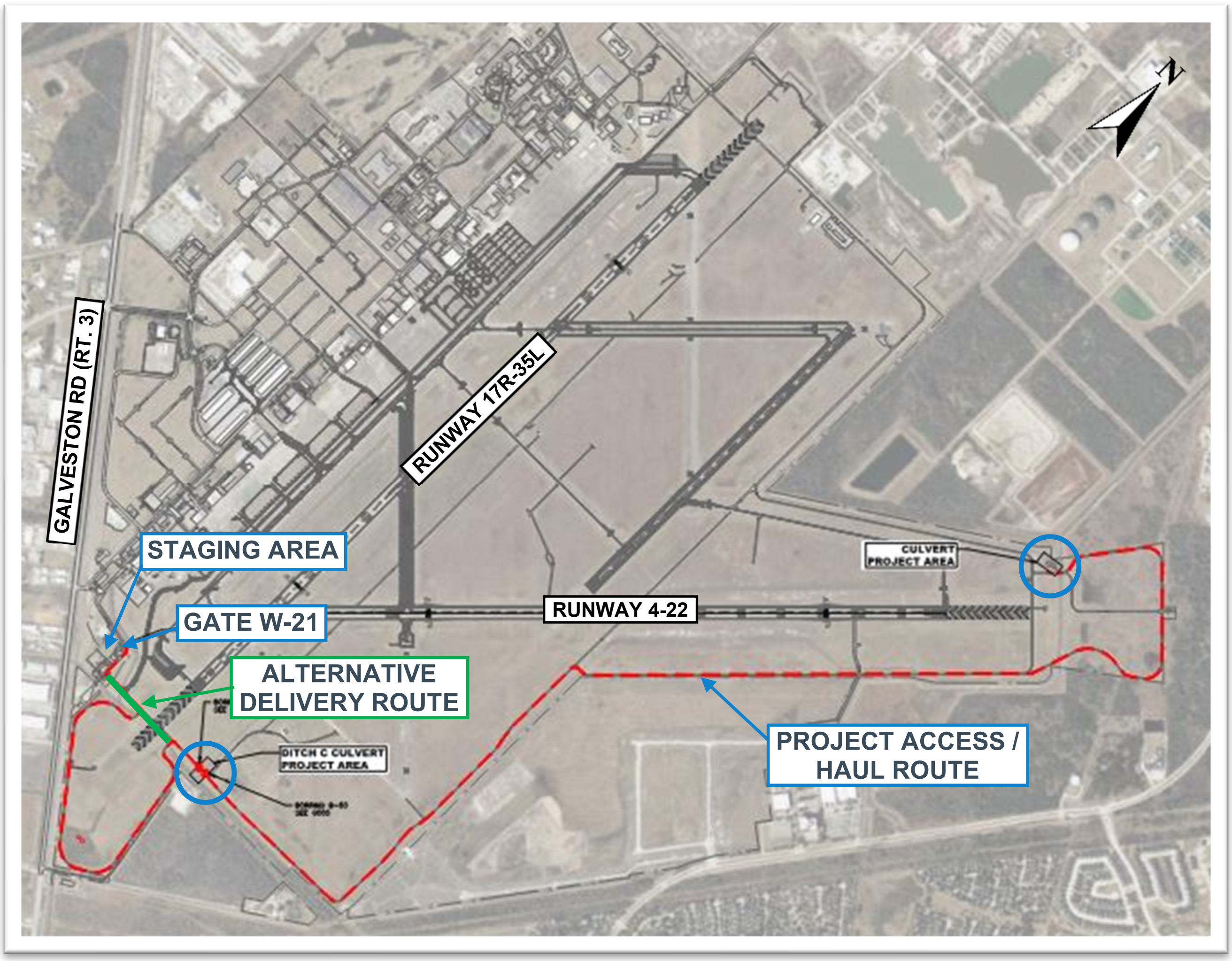

Figure 2-1: Staging Area, Haul Routes, and Project Access

2.3. Project Phasing and Sequencing

The following considerations were made when developing phasing and sequencing of the project:

- No unauthorized interruption to daytime aircraft operations will be allowed
- Maintain Vehicle Service Road (VSR) operational access.

The work within the Ditch C project site is proposed in three phases. Phase 1 includes the demolition and reconstruction of the southern half and provides a temporary VSR on the north side of the project site. Phase 2 then shifts traffic onto a temporary VSR above the reconstructed south portion to allow demolition and reconstruction of the north area. Phase 3 includes removing the temporary VSR and construction of the new permanent VSR. The phasing is shown below in Figure 2-2. Phase 4 includes the entirety of the north culvert site.

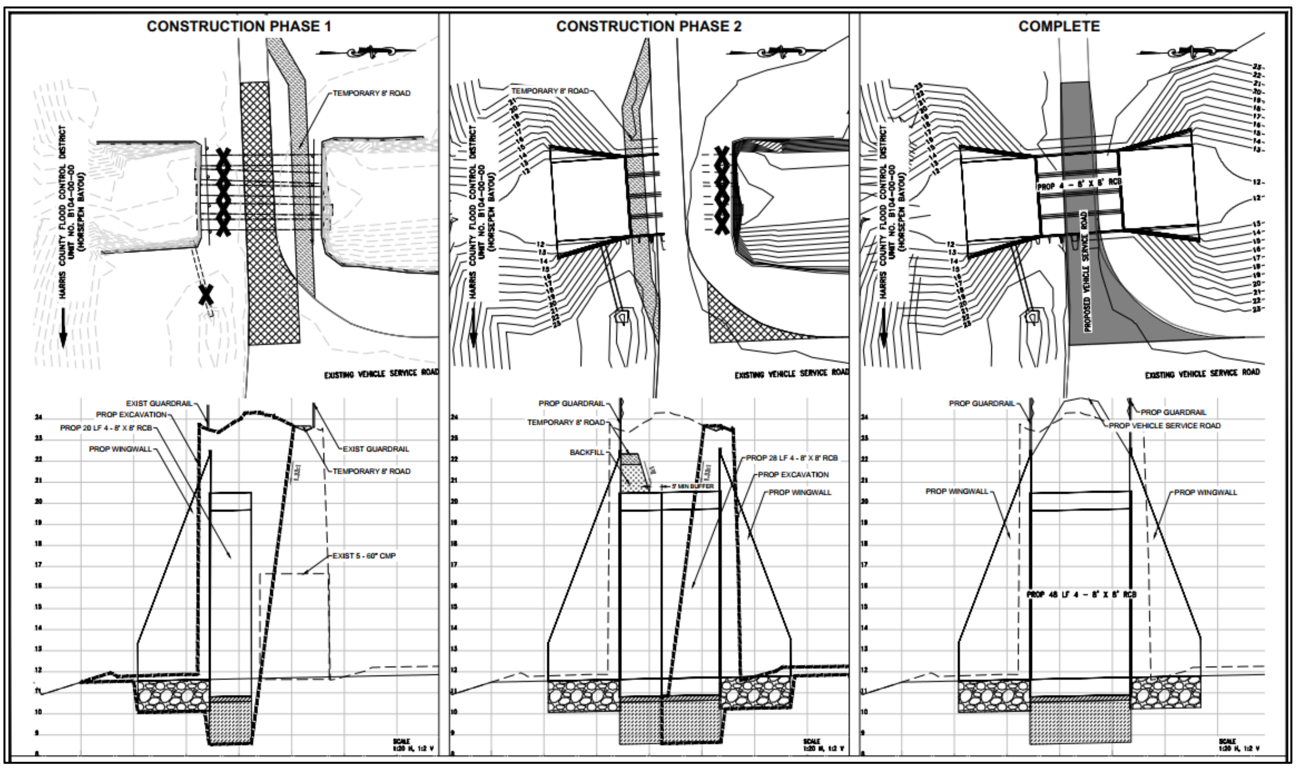

Figure 2-2: Ditch C Culvert Rehabilitation Phasing

2.4. Construction Safety and Phasing Plan

Atkins developed a Construction Safety and Phasing Plan (CSPP) in accordance with FAA AC 150/5370-2G, *Operational Safety on Airports During Construction.* The CSPP will be provided to the Contractor as part of the Contract Documents. The Contractor is required to submit a Safety Plan Compliance Document (SPCD) detailing all the elements of construction documented in the CSPP.

3. Ditch C Culvert

3.1. Introduction

The Houston Airport System (HAS) plans to reconstruct an existing culvert crossing at Ellington Field Airport. The culvert crossing is located on the south side of the Airport, near the Runway 35L end. The culverts outfall to Horsepen Bayou (HCFCD Unit No. B104-00-00). An existing Airport vehicle service road runs across the culvert crossing.

The existing culvert crossing consists of 5 – 60” CMPs with timber headwalls and has reached its end of life. EDGE Engineering, PLLC (EDGE), as part of the Atkins on-call team provided the hydrology & hydraulic drainage no-impact analysis for the proposed culvert reconstruction. The analysis evaluates the existing and proposed culvert hydraulics to ensure no peak flow impacts. The existing and proposed culverts were evaluated for the 2- (50%), 10- (10%), and 100-year (1%) Atlas 14 storm events.

3.2. Existing Conditions

The existing culvert crossing is approximately 50 LF with a 14’ wide vehicle service road running across. The culverts serve an approximately 630-acre drainage area that includes the existing apron tie-down areas, Taxiway H, Runway 17R/35L, and infield areas. This drainage area is known as catchment B104C in the Federal Emergency Management Agency’s (FEMA) effective HEC-HMS hydrologic model. The existing culvert crossing serves as the outfall for the B104C catchment into Horsepen Bayou (HCFCD Unit No. B104-00-00). Figure 3-1 presents the drainage area map.

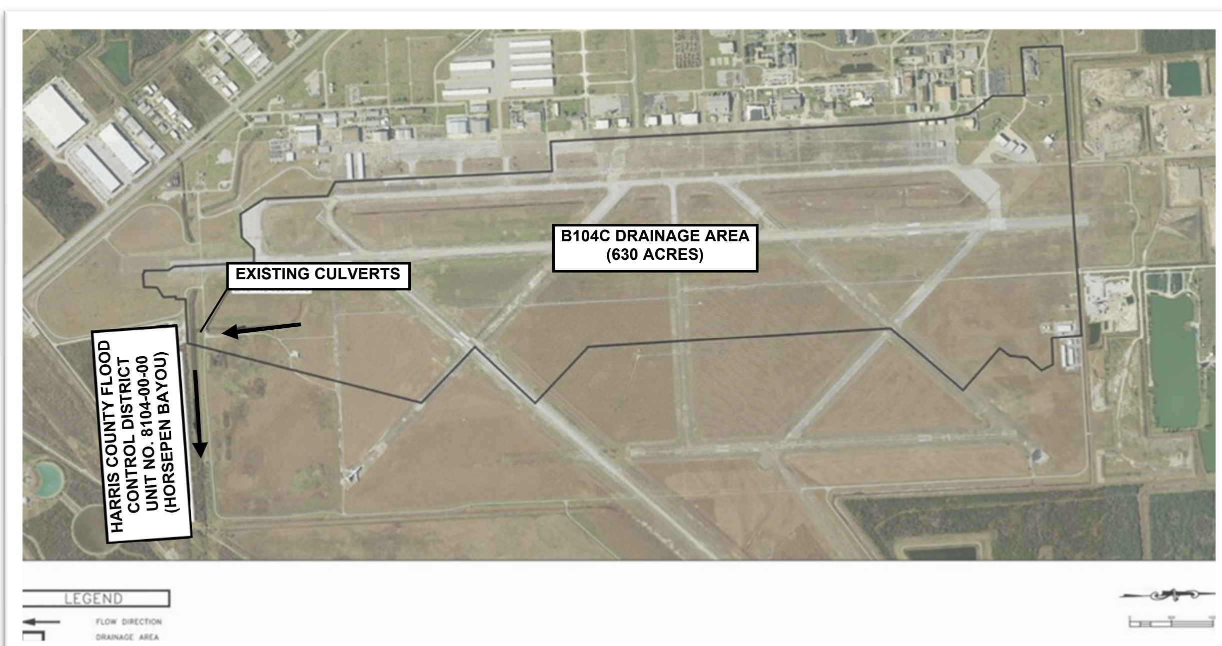

Figure 3-1: Drainage Area Map

3.3. Hydrologic and Hydraulic Methodologies

The XPSWMM version 2019.1.3 software was used to model the hydrology & hydraulics (H&H) of the existing and proposed drainage systems. The model consists of links and nodes to represent the storm sewer pipes and storm sewer structures, respectively.

The Federal Emergency Management Agency (FEMA) effective H&H models were used to calibrate different parameters in the XPSWMM model. The HEC-HMS model for Armand Bayou watershed (B100-00-00) and the HEC-RAS model for Horsepen Bayou (B104-00-00) were obtained from the Harris County Flood Control District (HCFCD) Model and Map Management System (M3).

3.3.1. Hydrologic Methodology

3.3.1.1. Rainfall

The Harris County Atlas 14 Region 3 rainfall depths for the 2-, 10-, and 100-year storm events were obtained from the County's Modeling, Assessment and Awareness Project (MAAPnext), https://www.maapnext.org/Data-Library

3.3.1.2. Land Use

In November 2022, EDGE (formerly HT&J, LLC) as part of the Atkins on-call team, provided the Taxiway L bridging documents drainage study and design. For this culvert reconstruction project, the Taxiway L developments were incorporated into the HEC-HMS model and defined as "existing" conditions. This allows the proposed culverts to be sized for future conditions whenever Taxiway L is constructed.

3.3.1.3. Peak Flows

The HEC-HMS model was updated and ran with the new Atlas 14 rainfall depths and Taxiway L developments. The resulting peak flow time series of the B104C catchment were then imported into XPSWMM as "User Inflow".

3.3.2. Hydraulic Methodology

3.3.2.1. Culvert Layout

Culvert sizes, slopes, and flowlines in the XPSWMM model were based on topographic survey.

3.3.2.2. Downstream Boundary Conditions (Tailwater)

"Free outfall" was used as the downstream boundary condition for the 2- and 10-year storm events. Typically, for small frequent storm events such as the 2- and 10-year storms, top-of-pipe is used as the starting tailwater. However, that is more applicable to urban storm sewer networks where the downstream project limit ties into another existing storm system. For this project, the culverts outfall into Horsepen Bayou, and therefore the top-of-pipe is not an accurate representation of the 2- and 10-year storm tailwater. Free outfall was chosen based on engineering judgement to represent the boundary conditions.

For the 100-year storm event, variable tailwater was used. For the variable tailwater, the time to peak (Tp) was obtained from the B1040000_0315_J Junction in the Armand Bayou (B100-00-00) HEC-HMS model. The effective 100-year water surface elevation (WSE) was obtained from cross section 31595.28 in the Horsepen Bayou (B104-00-00) HEC-RAS model, which is nearest to the culvert crossing.

Table 3-1: Variable Tailwater

HEC-RAS STA	100-YR WSE	MIN CH EL	T_p (Hrs)
31595.28	24.43	11.82	18

3.4. Design Criteria and Design Considerations

The drainage analysis and designs were done in accordance with the following criteria:

3.4.1. Federal Aviation Administration Advisory Circular 150/5320-5D, *Airport Drainage Design*

- 2-2.5 – Surface Runoff from Storms Exceeding Design Storm. Center 50% of taxiways should be free from ponding resulting from storms of a 10-year frequency and intensity determined by the geographic location.

3.4.2. Harris County Flood Control District Policy Criteria & Procedure Manual (PCPM), dated July 2019

- Section 6.7.4 – 0.5 entrance loss coefficient for culvert with headwall
- Section 8.2.1 – Design the culvert to pass the 0.2%, 1%, and 10% exceedance event flows without causing adverse impacts or erosion problems in the channel
- Section 8.2.1 – For multi-barrel culverts, accommodate low flow by setting the center barrel flowline at least one foot lower than the other barrels.
- Section 8.2.1 – Use 0.013 for Manning's n roughness coefficient
- 1:1 mitigation required for floodplain fill within the 0.2% (500-year) floodplain

3.4.3. City of Houston Infrastructure Design Manual (IDM), dated July 2020

- Section 9.2.01(B) – Use HCFCD Region 3 rainfall
- Section 9.2.01(C)(5) – Variable tailwater for 100-year storm event per TP-100

3.4.4. City of Houston Infrastructure Design Manual (IDM), dated July 2020

Design considerations included:

- Vehicle service road to remain open and operational during reconstruction
- Avoid floodplain fill impacts through grading cut/fill design
- Avoid HCFCD ROW, if possible. Work within the HCFCD ROW would require a drainage report submittal to HCFCD for their review and approval.
- TxDOT precast box culvert detail, SCP-8

- TxDOT flared wingwall detail, FW-0Pavement Section Design

3.5. Proposed Drainage Design

New 48LF of 4 – 8' x 8' RCB is proposed to replace the existing 50LF of 5 – 60" CMPs. This provides a greater opening area for improved flow conveyance while remaining within the existing channel footprint. The shorter culvert length avoids any floodplain fill impacts with a net grading cut and still allows for ample roadway shoulders. Additional drainage analysis results are discussed in Section 3.6. Three alternatives were developed to reconstruct the culverts:

3.5.1. Alternative 1 – Reconstruct to the North

Reconstruct the proposed RCB culverts entirely to the north of the existing CMPs as presented in Figure 3-2.

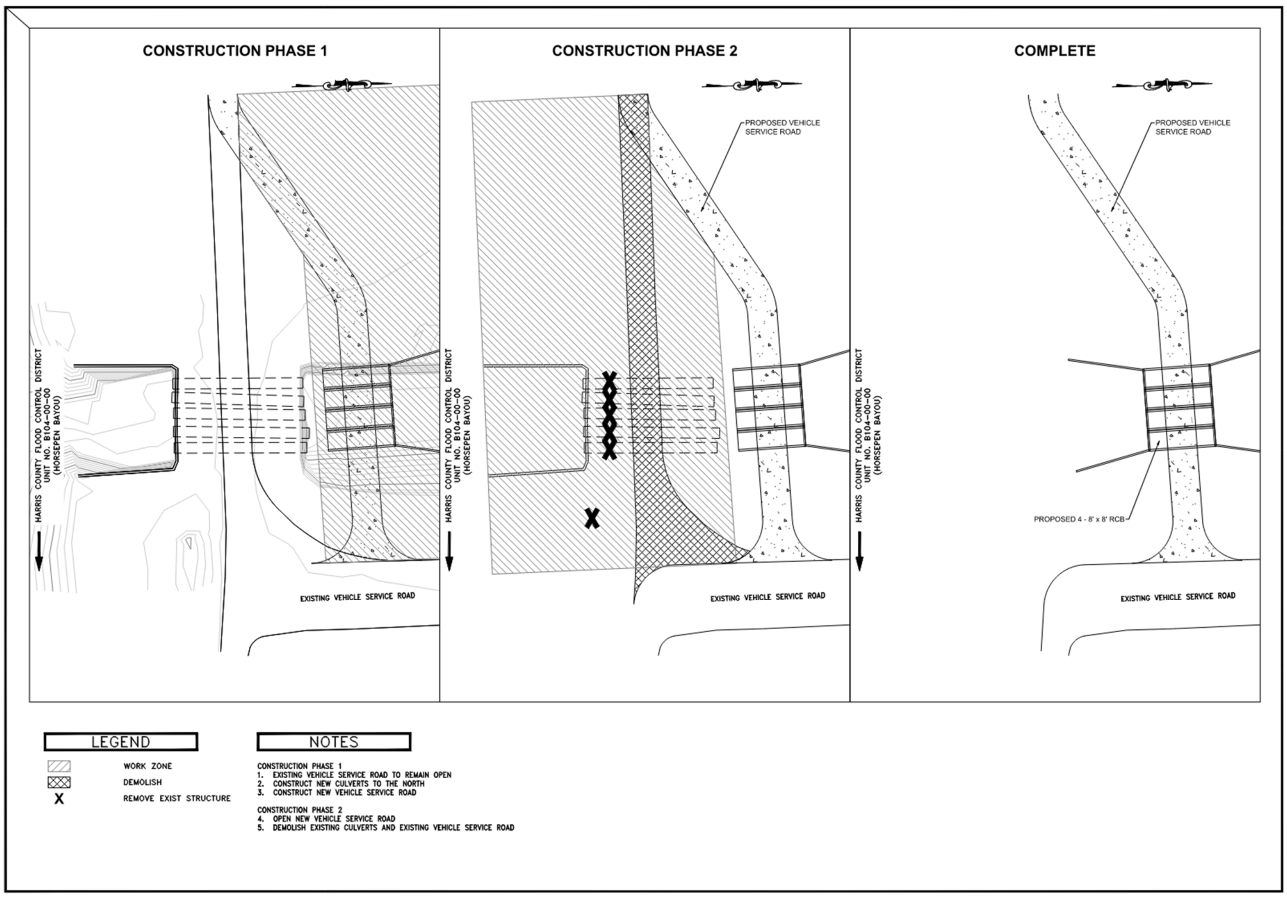

Figure 3-2: Design Alternative 1

3.5.1.1. Pros

This allows the existing vehicle service road to remain open operational until the new RCB culverts are completed. Once the new RCB culverts are constructed, the vehicle service road switches over, and the existing vehicle service road and CMPs are demolished.

This alternative also offers simpler construction phasing, since the existing and proposed structures are independent of each other. The Contractor does not have coordinate roadway/lane closures.

3.5.1.2. Cons

This alternative introduces several roadway geometry changes:

1. New alignment – the proposed roadway alignment introduces an "S" curve to tie-back into the existing vehicle service road.
2. New intersection layout – the "T" intersection gets shifted north.

3.5.2. Alternative 2 – Reconstruct in Halves

Reconstruct the proposed RCB culverts one-half at a time with a temporary 12-foot-wide roadway provided during phase 1. Alternative 2 is detailed graphically in Figure 3-3.

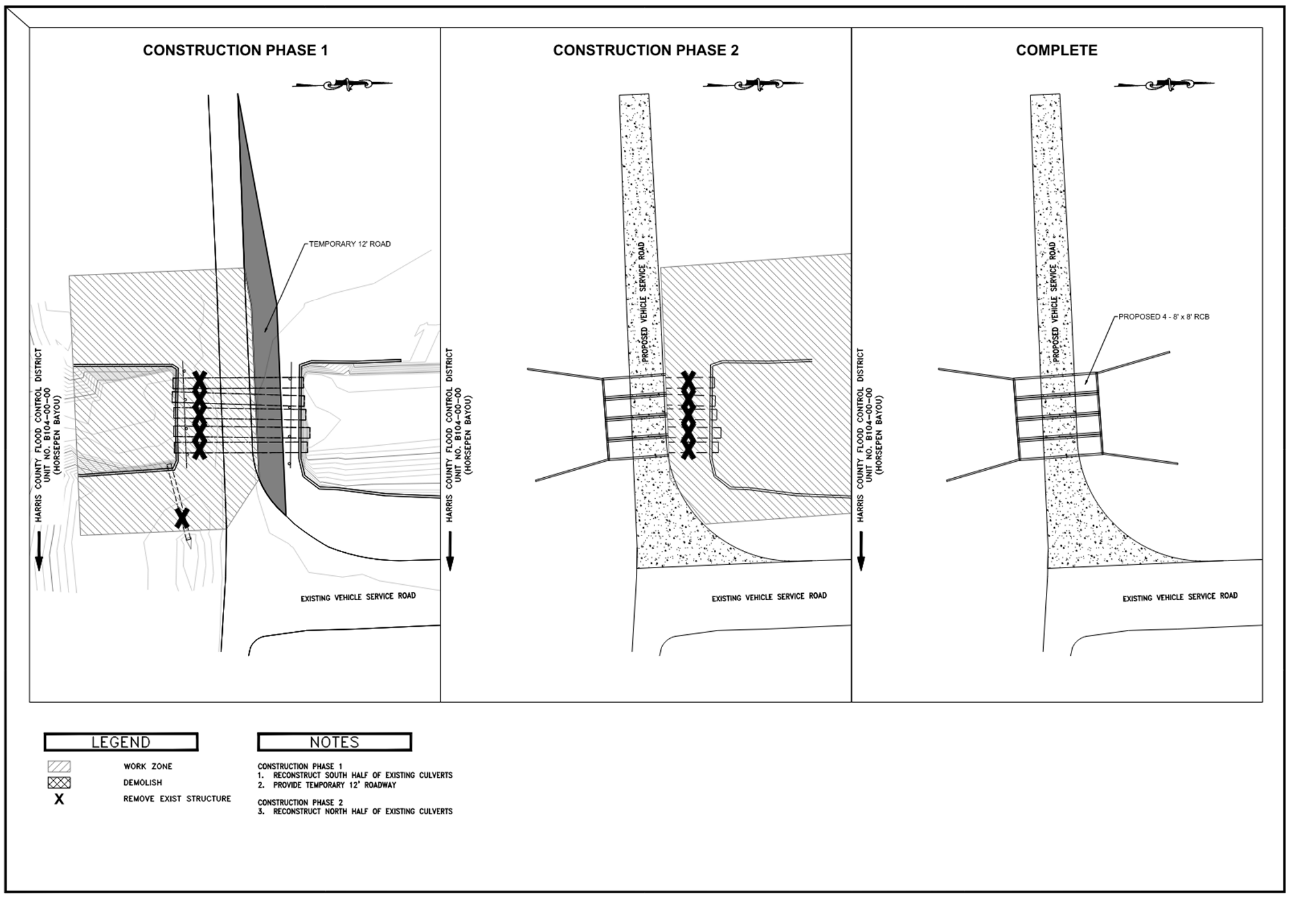

Figure 3-3: Design Alternative 2

3.5.2.1. Pros

The existing CMP culverts are approximately 50LF and offers ample room to temporarily shift the vehicle service road to allow for the demolition and reconstruction of the culverts one-half at a time.

This alternative allows the vehicle service road to remain open and operational during construction, and at completion, keeps the same roadway geometry alignment.

3.5.2.2. Cons

The alternative requires construction phasing and traffic control as each half is constructed. The vehicle service road will be temporarily shifted and narrower during construction.

3.5.3. Alternative 3 – Shutdown and Reconstruct

Shutdown the vehicle service road entirely and reconstruct the culverts as shown in Figure 3-4.

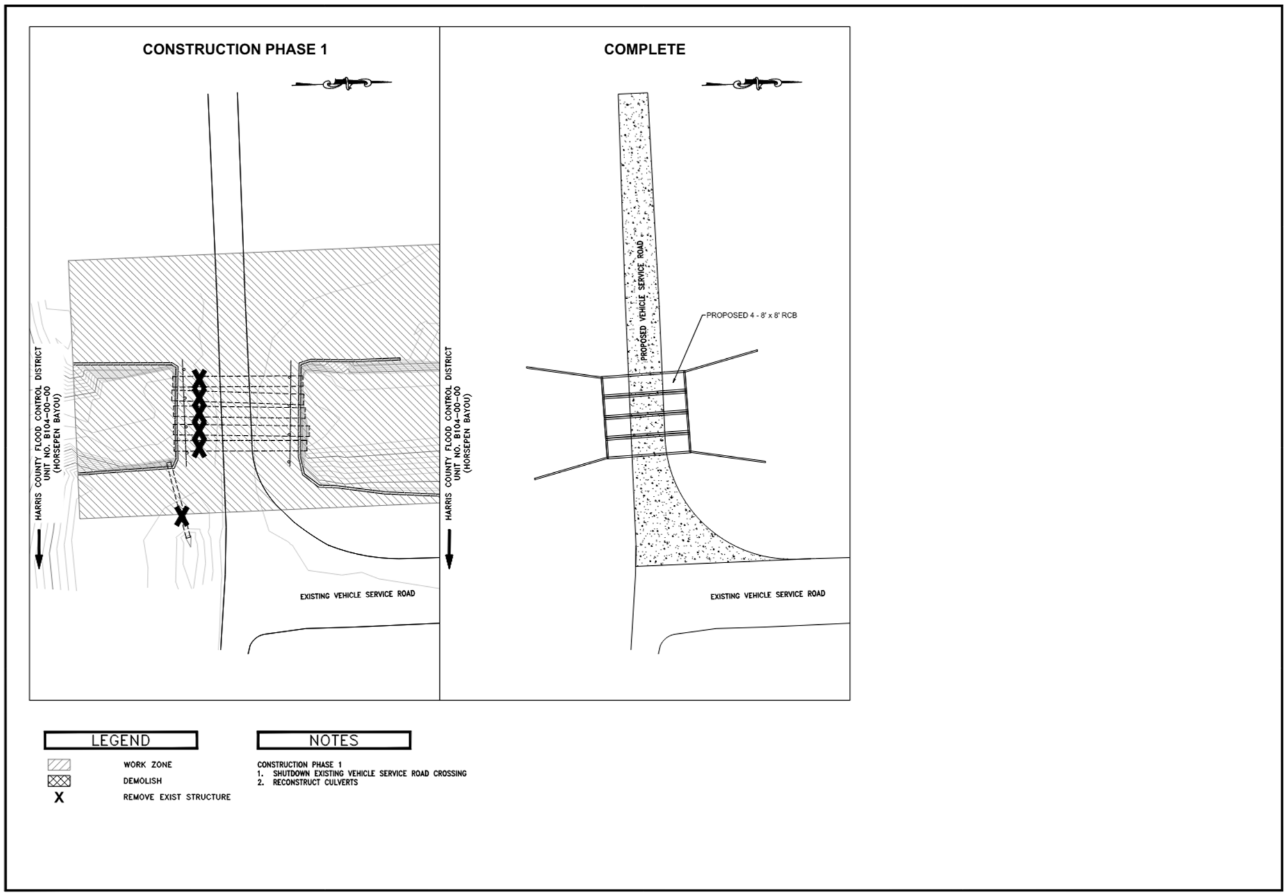

Figure 3-4: Design Alternative 3

3.5.3.1. Pros

This alternative offers the simplest construction phasing. The total closure allows the Contractor to demolish and reconstruct without any restrictions.

3.5.3.2. Cons

The closed vehicle service road limits access for several months. Operations must go around.

3.5.4. Recommended Alternative

The above three alternatives were presented to HAS and discussed during a monthly progress meeting on March 21, 2023. Alternative 2, reconstruct in halves was chosen as the preferred alternative by HAS.

3.6. Proposed Conditions Analysis

3.6.1.1. Proposed Hydrology

No developments are proposed, therefore there are no changes to the existing hydrology, including land use, impervious coverage, and time of concentration. As previously mentioned, the Taxiway L developments were defined as “existing” conditions. Any Taxiway L associated drainage impacts and detention and mitigation needs were separately studied and designed under the Taxiway L bridging documents previously from November 2022.

3.6.1.2. Proposed Hydraulics

The XPSWMM model was updated with the proposed 4 – 8’ x 8’ RCB.

3.6.1.3. Results Comparison

The existing and proposed hydrographs at the existing outfall into Horsepen Bayou are presented in **Figure 3-5** and **Figure 3-6**, respectively. The hydrographs are split into “pipe flow” and “surface flow.” In both existing and proposed conditions, a portion of the flows overtop the culvert and flows across the vehicle service road during the 100-year storm event. This is primarily due to the downstream Horsepen Bayou tailwater limitations. The flows from both pipes and surface runoff were summed timestep-by-timestep to determine the true peak flow.

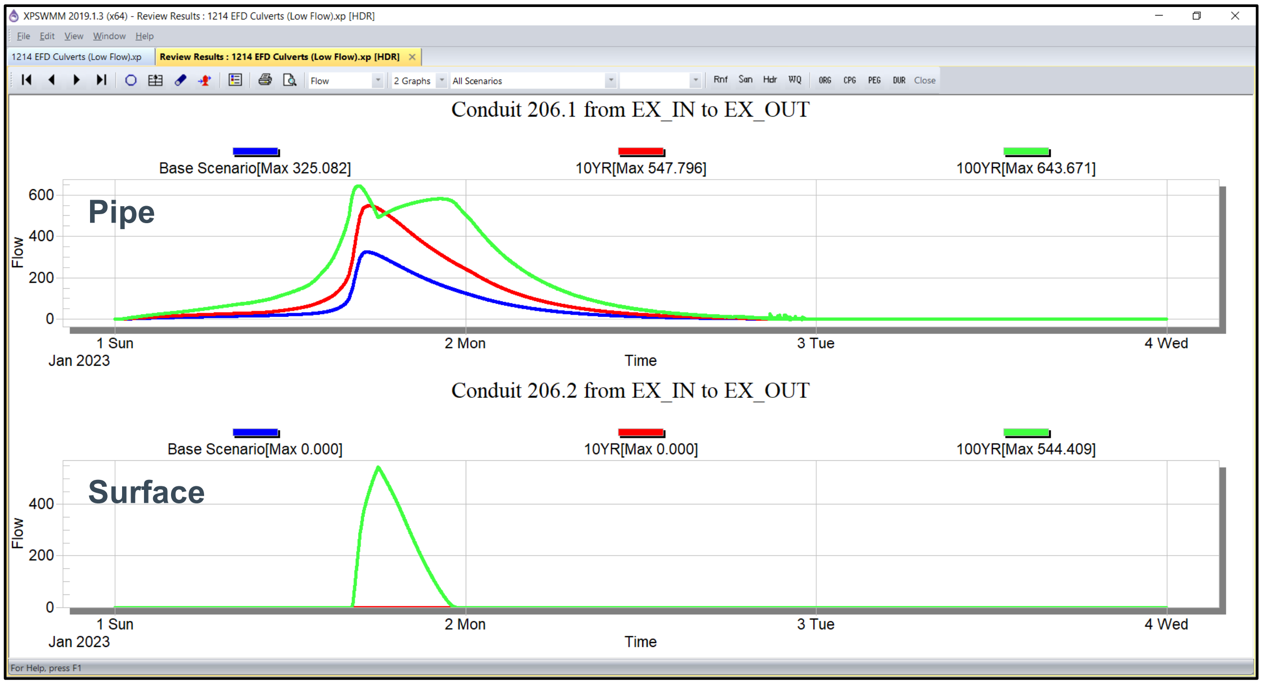

Figure 3-5: Existing Conditions Peak Flow Hydrographs

As shown in **Figure 3-5**, the estimated peak surface overflow occurs approximately 18 hours into the storm event, with a maximum flow of 544 cfs. The existing pipe peak flow, representing the available capacity, is approximately 644 cfs. The combined pipe and surface flow, shown in **Table 3-2**, is approximately 1041 cfs.

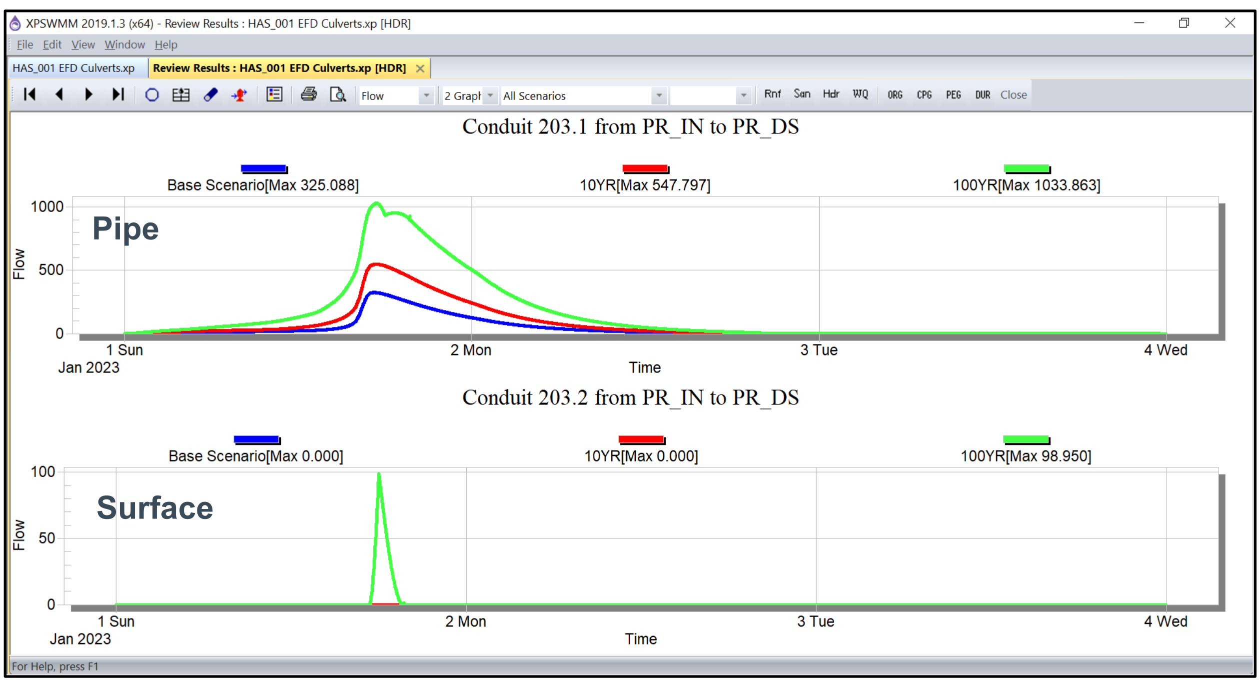

Figure 3-6: Proposed Conditions Peak Flow Hydrographs

As shown in **Figure 3-6**, the proposed condition 4 – 8' x 8' RCBs has greater capacity and passes approximately 795 cfs of flow. The surface overflow is reduced significantly to 99 cfs. The combined pipe and surface flow, as shown in **Table 3-2**, remains the same at approximately 1041 cfs.

Table 3-2 compares the existing and proposed condition peak flows. As shown, the 2-, 10- and 100-year proposed condition peak flows do not exceed existing conditions. The overtopping surface flow is reduced by approximately 445 cfs, from 544 cfs down to 99 cfs, with the proposed 4 – 8' x 8' RCBs.

Table 3-2 - Peak Flow Comparison

	2-Year (cfs)	10-Year (cfs)	100-Year (cfs)
Existing (Maximum Allowable Outflow)	325.08	547.80	1040.65
Proposed (Maximum Outflow Provided)	325.09	547.80	1040.40
Difference (Proposed – Existing)	0.01	0.00	(0.25)

The effective 100-year base flood elevation (BFE) of Horsepen Bayou at the culvert location is elevation 24.43. Existing ground elevations in the area are lower than the BFE. This means existing ground in the area becomes inundated during a 100-year storm event as Horsepen Bayou "backs up." This will continue to occur even with the proposed culvert reconstruction as the flooding is due to the backwater effects of Horsepen Bayou, and not due to any conveyance capacity of the proposed culverts. However, a comparison of the 100-year water surface elevation (WSE) shows that the proposed 4 – 8' x 8' RCBs reduces the WSE by approximately 6-inches from WSE 25.30 to WSE 24.76. **Table 3-3** and **Figure 3-7** below present the existing and proposed conditions WSE.

Table 3-3 - Existing and Proposed Water Surface Elevations

	2-Year (cfs)	10-Year (cfs)	100-Year (cfs)	BFE
Existing WSE	15.13	16.38	25.30	24.43
Proposed WSE	15.24	16.25	24.76	
Difference	0.11	(0.13)	(0.54)	

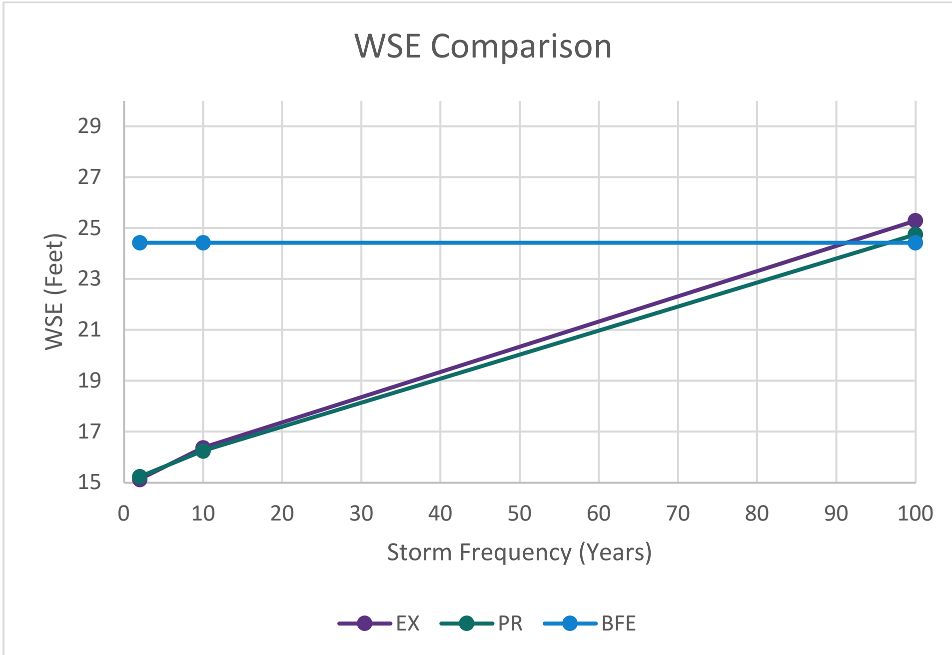

Figure 3-7: WSE Comparison

3.7. Conclusions and Recommendations

Based on the hydrologic and hydraulic analysis of the existing and proposed conditions, we conclude the proposed culvert reconstruction will cause no adverse impact to the receiving stream for storm events up to and including the 1% chance exceedance Atlas 14 event.

4. Northeast Culvert

4.1. Introduction

The Houston Airport System (HAS) plans to reconstruct a failed apron and headwall at the Northeast corner of Ellington Field Airport.

4.2. Existing Conditions

The existing Headwall appears to have been constructed for a larger Corrugated Metal Pipe than the existing pipe with the differences in size grouted to eliminate voids. The grout has completely failed and has caused erosion behind the headwall.

4.3. Proposed Design

The proposed design will consist of (+/-) 16 LF of 48” Type I corrugated metal pipe (CMP) to tie into the existing CMP. A new concrete headwall with flared concrete wingwalls will be installed. The existing concrete channel lining will be removed and reconstructed. The wash out area behind the existing structure will be backfilled and regraded for proper support and drainage.

5. Environmental

5.1. Environmental Considerations

All work items for this project are on existing airport property. It is anticipated a Categorical Exclusion (CATEX) will be filed by Houston Airport Systems to satisfy the NEPA component for the project construction.

6. Construction Specifications

FAA technical specifications will be utilized for this project where applicable. The anticipated applicable Division 01-16 technical specifications are given in Table 6-1.

Table 6-1: Anticipated Technical Specifications

SPECIFICATION NUMBER	DESCRIPTION
C-102	Temporary Air and Water Pollution, Soil Erosion, and Siltation Control
C-105	Mobilization
P-101	Preparation/Removal of Existing Pavements
P-151	Clearing and Grubbing
P-152	Excavation, Subgrade, and Embankment
P-155	Lime-Treated Subgrade
P-219	Recycled Concrete Aggregate Base Course
D-701	Pipe for Storm Drains and Culverts
D-751	Manholes, Catch Basins, Inlets and Inspection Holes
D-752	Concrete Culverts, Headwalls, and Miscellaneous Drainage Structures
T-901	Seeding
T-904	Sodding
T-908	Cement Stabilized Sand

Additional specifications are required for specific aspects and/or materials for this project. City of Houston, TxDOT and Harris County Flood Control District standard specifications and details were utilized where appropriate and are listed below in Table 6-2.

Table 6-2: Additional Technical Specifications

SPECIFICATION NUMBER	DESCRIPTION
City of Houston Specification	
01555	Traffic Control and Regulation
01578	Control of Ground and Surface Water
02260	Trench Safety System for Trench Excavations
02714	Flexible Base Course for Temporary Driveways
02741	Type D Hot Mix Asphalt Concrete Surfacing
TxDOT Specifications	
169	Soil Retention Blankets
432	Riprap
540	Metal Beam Guard Fence
Harris County Flood Control District (HCFCD) Specifications	
02376	Concrete Channel Lining and Concrete Interceptor Structures

7. Project Schedule

7.1. Overall Project Schedule

Milestone	Approximate Time Period
Design:	
Design Begin	December 2022
Design Complete	August 2023
Bidding:	
Advertisement for Bids	August 2023
Bid Opening	September 2023
Construction:	
Construction Contract Execution	TBD 2023
Notice to Proceed	TBD 2023
Begin Construction	30 days after NTP
Complete Construction	120 days after Begin Construction
Closeout documentation	30 days after Construction Complete

Appendices

Appendix A. Geotechnical Reports

GEOTECHNICAL INVESTIGATION

HOUSTON AIRPORT SYSTEM
DITCH C CULVERT REPLACEMENT
AT ELLINGTON AIRPORT (EFD)
HAS PROJECT NO. 707
HOUSTON, TEXAS

Reported to

Atkins North America, Inc.
Houston, Texas

by

Aviles Engineering Corporation
5790 Windfern
Houston, Texas 77041
713-895-7645

REPORT NO. G155-22 (Final)

June 2023

ENGINEERING CORP.

June 20, 2023

Mr. John Verburg, P.E.
Atkins North America, Inc.
200 Westlake Park Boulevard, Suite 1100
Houston, Texas 77079

Reference: **Geotechnical Investigation**
Houston Airport System
Ditch C Culvert Replacement at Ellington Airport (EFD)
HAS Project No. 707
Houston, Texas
AEC Report No. G155-22 (Final)

Dear Mr. Verburg,

Aviles Engineering Corporation (AEC) is pleased to present this report of the results of our geotechnical investigation for the above referenced project. This investigation was authorized by you on January 18, 2023, via Task Order No. 8 of Subcontract No. 1011417. Project terms and conditions were in accordance with the Master Subcontract Agreement between Atkins North America, Inc. (Atkins) and AEC, dated June 24, 2019. The project scope of services is in accordance with AEC Proposal No. G2022-10-08, dated October 24, 2022.

AEC appreciates the opportunity to be of service to you. Please call us if you have any questions or comments concerning this report or when we can be of further assistance.

Respectfully submitted,
Aviles Engineering Corporation
(TBPELS Firm Registration No. F-42)

STATE OF TEXAS
WILBER L. WANG
99226
LICENSED
PROFESSIONAL ENGINEER

Wilber L. Wang, P.E. 06/20/2023
Senior Engineer

Reports Submitted: 1 Atkins North America, Inc. (electronic)

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GEOTECHNICAL INVESTIGATION

HOUSTON AIRPORT SYSTEM
DITCH C CULVERT REPLACEMENT
AT ELLINGTON AIRPORT (EFD)
HAS PROJECT NO. 707
HOUSTON, TEXAS

1.0 INTRODUCTION

1.1 Project Description

The report submitted herein presents the results of AEC's geotechnical investigation for the Houston Airport System's (HAS) proposed Ditch C Culvert Replacement at Ellington Airport (IATA Airport Code: EFD) in Houston, Texas (Houston/Harris County Key Map No.: 617C). A vicinity map of the project location is presented on Plate A-1, in Appendix A.

According to 65 percent submittal drawings (dated April 4, 2023) provided by Atkins North America, Inc. (Atkins), the project consists of reconstruction of an existing culvert along a vehicle access road crossing Ditch C at the south end of the airport. The new culvert will consist of four eight foot high by eight foot wide reinforced concrete boxes (RCB) in parallel, with new headwalls and wingwalls on each side of the boxes. The flowline of the culverts will be approximately 12 to 13 feet below the vehicle service road. The vehicle access road crossing above the culvert will also be reconstructed.

1.2 Purpose and Scope

The purpose of this geotechnical investigation is to evaluate the subsurface soil and groundwater conditions at the site and develop geotechnical engineering recommendations for design and construction of the access road and culvert. The scope of this geotechnical investigation is summarized below:

1. Drilling and sampling two soil borings ranging from a depth of 20 to 40 feet below existing grade.
2. Performing soil laboratory testing on selected soil samples to determine the index and strength properties of the subgrade soils.
3. Engineering analyses and recommendations for subgrade preparation of the proposed access road.
4. Engineering analyses and recommendations for culvert installation, including loadings on pipes, lateral earth pressure soil parameters for headwalls, and backfill requirements.
5. Construction recommendations and groundwater control guidelines for the proposed improvements.

2.0 SUBSURFACE EXPLORATION

Subsurface conditions at the site were investigated by drilling two soil borings (Borings B-82 and B-83) to a depth ranging between 20 and 40 feet below existing grade. Borings B-1 through B-81 were drilled for the EFD Taxiway L and 30 percent Bridging projects, presented in AEC Report G103-21. The boring locations were marked in the field by Landtech, Inc. and were surveyed as they were marked. Boring survey data (in State Plane Grid Coordinates, Texas South Central Zone 4204, US Survey Feet) is presented on the representative boring logs and is also summarized on Table 1. The boring locations are presented on the Boring Location Plan on Plate A-2, in Appendix A.

Table 1. Summary of Boring Survey Data

Boring No.	Northing (Grid, ft)	Easting (Grid, ft)	Boring Surface Elevation (ft)
B-82	13782784.01	3188203.77	24.52
B-83	13782787.66	3188268.08	24.17

Soil Borings: Prior to drilling, existing pavement and base at the boring locations were cut with a core barrel. The field drilling was performed using a truck-mounted drilling rig. Boring B-82 was initially advanced using dry auger method, but completed using wet rotary method once groundwater was encountered. Boring B-83 was advanced using dry auger method alone. Undisturbed samples of cohesive soils and some granular (i.e., clayey sand) soils were obtained from the borings by pushing 3-inch diameter thin-wall, seamless steel Shelby tube samplers in accordance with ASTM D 1587. Granular soils were sampled with a 2-inch split-barrel sampler in accordance with ASTM D 1586. Standard Penetration Test resistance (N) values for these samples were recorded as "Blows per Foot" and are shown on the boring logs. Strength of the cohesive soils was estimated in the field using a hand penetrometer. The undisturbed samples of cohesive soils were extruded mechanically from the core barrels in the field and wrapped in aluminum foil; all samples were sealed in plastic bags to reduce moisture loss and disturbance. The samples were then placed in core boxes and transported to the AEC laboratory for testing and further study. Groundwater readings were obtained during drilling and upon completion of drilling. The boreholes were then grouted with cement-bentonite upon completion of drilling, and existing pavement was patched with high strength non-shrink grout.

3.0 LABORATORY TESTING PROGRAM

Soil laboratory testing was performed by AEC personnel. Samples from the borings were examined and classified in the laboratory by a technician under supervision of a geotechnical engineer. Laboratory tests were performed on selected soil samples to evaluate the engineering properties of the foundation soils in accordance with applicable ASTM Standards. Atterberg limits, moisture contents, percent passing a No. 200 sieve, sieve analysis, and dry unit weight tests were performed on selected samples to establish the index properties and confirm field classification of the subsurface soils. Strength properties of cohesive soils were determined by means of torvane (TV), unconfined compression (UC), and unconsolidated-undrained (UU) triaxial tests performed on undisturbed samples. The laboratory test results are presented on the representative boring logs (see Plates A-3 and A-4, in Appendix A). A key to the boring logs, classification of soils for engineering purposes, terms used on boring logs, and reference ASTM Standards for laboratory testing are presented on Plates A-5 through A-8, in Appendix A. Sieve analysis results are presented on Plate A-9, in Appendix A.

4.0 SITE CONDITIONS

A summary of existing pavement sections encountered in AEC's borings is presented on Table 2. Photographs of pavement core sections are presented on Plate 1, in the Illustrations.

Table 2. Summary of Existing Pavement Thickness

Boring No.	Core Location	Pavement Section
B-82	Access Road	1" asphalt, 9" cement stabilized base, 7" cement stabilized shell
B-83	Access Road	1.25" asphalt, 9.5" cement stabilized base, 3.5" cement stabilized shell

4.1 Subsurface Conditions

Details of the soils encountered during drilling are presented in the boring logs on Plates A-3 through A-11, in Appendix A. Soil strata encountered in our borings are summarized below.

Boring	Depth (ft)	Description of Stratum
B-82	0 - 1.4	Pavement and base: see Table 2 in Section 4.0 of this report.
	1.4 - 6	Stiff to very stiff, Fat Clay with Sand (CH), with slickensides
	6 - 12	Soft to very stiff, Fat Clay (CH)
	12 - 14	Clayey Sand (SC), with abundant silty sand seams
	14 - 40	Loose to dense, Silty Sand (SM)

Boring	Depth (ft)	Description of Stratum
B-83	0 - 1.2	Pavement and base: see Table 2 in Section 4.0 of this report.
	1.2 - 6	Fill: stiff to very stiff, Lean Clay with Sand (CL), with fat clay and calcareous nodules
	6 - 8	Fill: firm to stiff, Fat Clay with Sand (CH), with ferrous nodules
	8 - 12	Firm to very stiff, Lean Clay (CL), with abundant silty sand seams
	12 - 16	Loose, Clayey Sand (SC), with silty sand seams
	16 - 20	Medium dense to dense, Silty Sand (SM)

AEC notes that thick (8 feet) fill layers were encountered in Boring B-83. It is likely that the fill layers encountered in this boring is backfill for the existing culvert crossing.

Subsurface Soil Properties: The cohesive soils encountered in the borings (including fill, but excluding clayey sand) have high to very high plasticity (see "Degree of Plasticity of Cohesive Soils" on Plate A-6, in Appendix A), with Liquid Limits (LL) ranging from 41 to 73 and Plasticity Indices (PI) ranging from 25 to 54. The cohesive soils encountered are classified as "CL" and "CH" type soils and the granular soils are classified as "SC" and "SM" type soils in accordance with ASTM D 2487.

Groundwater Conditions: Groundwater levels encountered in the borings during drilling are summarized in Table 3.

Table 3. Summary of Boring Groundwater Depths

Boring No.	Date Drilled	Boring Depth (ft)	Groundwater Depth (ft)
B-82	3/1/2022	40	15 (Drilling) 12.7 (15 min.)
B-83	3/1/2022	20	15 (Drilling) 12.7 (Complete)

The information in this report summarizes conditions found on the date the borings were drilled. However, it should be noted that our groundwater observations are short term; groundwater depths and subsurface soil moisture contents will vary with environmental variations such as frequency and magnitude of rainfall and the time of year when construction is in progress.

4.2 Hazardous Materials

No signs of visual staining or odors were encountered during field drilling or during processing of the soil samples in the laboratory.

4.3 Subsurface Variations

It should be emphasized that: (i) at any given time, groundwater depths can vary from location to location, and (ii) at any given location, groundwater depths can change with time. Groundwater depths will vary with seasonal rainfall and other climatic/environmental events. Subsurface conditions may vary away from and in between the boring locations.

Clay soils in the Greater Houston area typically have secondary features such as slickensides, calcareous/ferrous nodules, and contain sand/silt seams/lenses/layers/pockets. It should be noted that the information in the boring logs is based on 3-inch diameter soil samples which were obtained continuously at intervals of 2 feet from the ground surface to a depth of 20 feet, and then at 5 foot intervals thereafter. A detailed description of the soil secondary features may not have been obtained due to the small sample size and sampling interval between the samples. Therefore, while a boring log shows some soil secondary features, it should not be assumed that the features are absent where not indicated on the boring logs.

4.4 Geologic Faults

AEC previously performed a preliminary geologic fault study investigation for the Taxiway L alignment (see AEC Report G103-21). AEC's previous fault study indicates that there are multiple faults in the vicinity of EFD, which is repeated here for convenience (in relation to Taxiway L). The closest fault to the Taxiway L alignment, the Liberty Fault is an east-west oriented fault that terminates approximately 0.18 miles west of the Taxiway L alignment. The next closest fault, according to the 1984 fault map, is an un-named northeast-southwest oriented fault that is located approximately 0.24 miles southwest of the Taxiway L alignment.

A map in the 1975 publication, "Active Faults in Southeastern Harris County, Texas", Geo I, pages 149 – 154, by Clanton, U.S. and Amsbury, D.L., indicates multiple faults in or near the project area. The map indicates at least two or more faults crossing the southern portion of the Taxiway L alignment. According to the article, these faults were associated with structural damages seen in nearby buildings, streets, and runways.

If there are no historical Phase I Fault Investigation studies readily available of the Taxiway L alignment, AEC recommends a Phase I Fault Investigation for the project be performed since there are variations in fault locations on different maps and the article by Clanton, U.S. and Amsbury, D.L indicates multiple faults near and or crossing the southern half of the Taxiway L alignment.

Limitations: The desktop fault study provided in this report is limited to a review of available literature, aerial photographs, and maps. Distances are scaled from maps. Faults may exist in, cross, or adjoin the Project Alignment which were not identified in this report due to the following reasons: limitations of the scope of work and cost, no field observations were conducted; lack of documentation in the literature; and faults may have not been visible on the aerial photographs due to clarity of the aerial photographs, the presence of vegetation and environmental features, and modification of the land surface by human activities. Faults may also be present below ground but do not currently have surface expressions. Identification of these faults is beyond the scope of work for this study.

5.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

According to 65 percent submittal drawings (dated April 4, 2023) provided by Atkins, the project consists of reconstruction of an existing culvert along a vehicle access road crossing Ditch C at the south end of the airport. The new culvert will consist of four eight foot high by eight foot wide RCB in parallel, with new headwalls and wingwalls on each side of the boxes. The flowline of the culverts will be approximately 12 to 13 feet below the vehicle service road. The vehicle access road crossing above the culvert will also be reconstructed.

Design Standards: In accordance with Section 2.16.12 of the HAS 2015 Design Criteria Manual, pavement design for all aircraft rated pavements shall be based on Federal Aviation Administration (FAA) methodology and requirements in Advisory Circular (AC) 150/5320-6G (or latest edition).

Construction Standards: AEC has referenced construction standards from FAA AC 150/5370-10H (or latest edition) "Standard Specifications for Construction of Airports", where applicable.

5.1 Ditch C Culvert Reconstruction

5.1.1 Geotechnical Parameters for Culvert Design

Recommended geotechnical parameters to be used for design of the proposed Ditch C culvert replacement is presented on Plate B-1, in Appendix B. The design values are based on the results of field and laboratory test data on individual boring logs as well as AEC's experience with local soil conditions. It should be noted that because of the variable nature of soil stratigraphy, soil types and properties along the project alignment or at locations away from a particular boring may vary substantially.

5.1.2 Culvert Design

According to Atkins' drawings, the new culvert crossing will consist of four, 8 foot by 8 foot RCB in parallel. One RCB will have a invert elevation at +11.7 feet Mean Sea Level (MSL), matching the existing ditch flowline, while the remaining three RCB will have an invert elevation of 12.7 feet MSL. The bottom of the RCBs will bear at an elevation ranging from 10.8 to 11.8 feet MSL. The Ditch C flowline is at an elevation of approximately +12 feet MSL. Based on Borings B-82 and B-83, the proposed culvert RCBs will bear directly on top of a loose/soft clayey/silty sand (SC/SM) strata. Placing the RCB culverts on top of the loose/soft sand strata could result in excessive settlement and differential settlement of the culvert crossing, which could impact the channel flowline and also result in distress to the access road pavement on top of the crossing, such as cracking and/or differential movement.

In order to mitigate the settlement impact on the performance of the culvert and access road, **AEC recommends that a minimum of 24 inches of existing loose/soft sand that is present beneath the culvert invert depth be over-excavated and replaced with a 24 inch thick flexible base layer that is wrapped in a woven geo-textile fabric.**

Subgrade Preparation: After the existing culvert boxes and headwalls have been demolished, AEC recommends that the exposed subgrade where the new culvert boxes will be installed be over-excavated to a depth of 24 inches (i.e., excavate to an elevation of approximately +9 to +8.5 feet MSL). After excavation to grade, AEC recommends that a competent soil technician inspect the exposed subgrade to determine if there are any unsuitable soils or other deleterious materials. Excavate and dispose of unsuitable soils and other deleterious materials which will not consolidate; the excavation depth should be increased when inspection indicates the presence of soft soils, organics, or deleterious materials to greater depths After the exposed subgrade has been inspected, a minimum 24 inch thick layer of flexible base should be placed. The entirety of the base layer should be wrapped with a woven geotextile filter fabric. The flexible base should be a Type D (crushed stone or recycled crushed concrete), Grade 1-2 flexible base, in accordance with the 2014 Texas Department of Transportation (TxDOT) Standard Specifications for Construction and Maintenance of highways, Streets, and Bridges. The base shall be placed in 12 inch thick loose lifts and compacted to 100 percent of its maximum dry density determined by TxDOT Test Method Tex-113-E, at a moisture content within 2 percent of optimum.

Based on Table 3 in Section 4.1 of this report, groundwater may be present within the excavation zone at the bottom of the culvert. See Section 5.1.4 of this report ("Soil and Groundwater" for discussion of excavation shoring and the need for groundwater control.

Allowable Bearing Capacity: Considering the 24 inch thick flexible base layer (see above), AEC recommends the culvert bottom be designed considering a net allowable bearing capacity of 1,500 psf for sustained loads and 2,250 psf for total loads. A factor of safety (FS) of 3.0 has been applied to the net allowable bearing capacity for sustained loads and a FS of 2.0 has been applied to the net allowable bearing capacity for total loads. Whichever net allowable bearing capacity results in a larger RCB footprint should be used for culvert design.

The net footing pressure may be determined by:

1. Summing the weight of the load applied to the foundation, the weight of the foundation, and the weight of soil backfill placed above the foundation.
2. Subtracting the weight of soil excavated from the foundation.
3. Dividing the result of items 1 and 2 by the base area of the foundation.

Uplift Resistance: The proposed culvert should be designed to resist hydrostatic uplift. For uplift design of the culvert, AEC recommends that the water level be assumed to be at the ground surface or 100-year flood elevation, whichever is more critical. If the dead weights of the structures are inadequate to resist uplift forces, toe extensions of the base slabs may be constructed so that the effective weight of the soil above the extended slabs can be utilized to resist the uplift forces. The unit buoyant weight of concrete can be taken as 90 pcf. The minimum recommended FS against uplift should be 1.1 for concrete weight, 1.5 for soil weight, and 3.0 for soil friction. Design soil parameters are included on Plate B-1, in Appendix B. Recommended design criteria for uplift resistance are shown on Plate C-1, in Appendix C.

Traffic Loads: The HL-93 design live load required by the AASHTO Load and Resistance Factor Design (LRFD) Specifications includes two types of vehicular design loads and consists of a combination of: (i) the Design Truck or the Design Tandem, and (ii) the Design Lane Load. HL-93 loading is representative of the worst case between these two loading scenarios. The Design Truck used in the AASHTO LRFD Specifications has the same configuration as the HS-20 Design Truck in the AASHTO Standard Specifications. The Design Tandem load configuration consists of a pair of 25-kip axles spaced 4 feet apart. The transverse spacing of wheels is 6 feet. Load factors, dynamic load allowance, and other factors are then applied to these loads in the proper load combination(s) based on the AASHTO LRFD Specifications. The HS-20 design truck 32,000-pound design axle and the Design Tandem 25,000-pound design axle are carried on dual wheels. The loads from both the Design Truck and the Design Tandem are assumed to be distributed transversely within a 10 feet wide design lane. A rectangular tire contact area (typically 10 inches by 20 inches) is used in the design. AASHTO wheel loads and wheel spacings for both Design Truck and Design Tandem, as well as AASHTO wheel load surface contact area are shown on Plate B-2, in Appendix B. Design Lane load is considered as a 640 pound/foot load uniformly distributed in the longitudinal direction across a 10-foot-wide lane at all depths of earth cover over the top of the conduit, up to a depth of 8 feet. This converts to an additional lane load intensity of 64 psf applied to the top of the conduit for any depth of burial less than 8 feet. Details of application and calculation of design vehicular live load can be found in Section 3.6.1.2. of the AASHTO Bridge Design Specifications.

The average pressure intensity caused by a wheel load is calculated from Equation (1).

$$w = P\,(1.0 + IM)/A \qquad \text{...........Equation (1)}$$

$$IM = 33\,(1.0 - 0.125H)/100 \qquad \text{...........Equation (2)}$$

where:
- w = vertical pressure on the top of the conduit resulting from wheel load (psf).
- P = total live wheel load applied at the surface (lb).
- A = spread wheel load area at the outside top of the conduit (ft^2).
- IM = dynamic load allowance (also known as Impact Factor).
- H = height of earth over the top of the conduit (ft).

The critical wheel load and spread dimensions for the height of earth cover, H, over the outside top of the conduit are presented on Plate B-3, in Appendix B. The spread live load area, A, equal to "spread a" times "spread b", is also shown on Plate B-3, in Appendix B.

The total live load acting on top of the culvert, W_L, can be calculated from Equation (3):

$$W_L = (w + L_L)\,S_L\,L / L_e \qquad \text{...........Equation (3)}$$

$$L_e = L + 1.75\,(3/4\,R_0) \qquad \text{...........Equation (4)}$$

where:
- W_L = live load on the top of the culvert (lb/ft).
- w = vertical pressure on the top of the culvert resulting from wheel load (psf).
- L_L = lane load intensity (psf):
  $L_L = 64$ psf for $0 \leq H < 8$,

L_L = 0 psf for H ≥ 8.

L = dimension of A parallel to the longitudinal axis of culvert (ft):
L = "spread a" for vehicles traveling perpendicular to the culvert,
L = "spread b" for vehicles traveling parallel to the culvert.

S_L = outside horizontal span of culvert, D_o, or spread wheel load area, A, transverse to the longitudinal axis of culvert, whichever is less (ft).

L_e = effective supporting length of culvert (ft), as shown on Plate B-3, in Appendix B.

R_0 = outside vertical rise of culvert (ft).

5.1.3 Headwalls

Lateral Earth Pressures: The magnitudes of the lateral earth pressures on headwalls will depend on the type and density of the backfill behind the walls, surcharge on the backfill, and hydrostatic pressure, if any. If the backfill is over-compacted or if highly plastic clays are placed behind the walls, the lateral earth pressure could exceed the vertical pressure. Instead, AEC recommends that select clay fill be used as backfill material behind the headwalls. Select clay backfill requirements are presented in Section 5.1.5 of this report.

Lateral pressure resulting from construction equipment, pavement and traffic, or other surcharges on the top of the walls should be considered by adding the equivalent uniformly distributed surcharge to the design lateral pressure. AEC also recommends that at least 250 psf surcharge be considered for design of the headwalls, although the determination of the loading surcharge should ultimately be performed by the culvert designer. Hydrostatic pressure, if any, should also be considered.

Lateral earth pressures acting on the headwalls will depend on whether the top of the headwalls will be allowed to deflect. If the headwalls are allowed to deflect to a minor degree, then the headwalls can be designed based on active earth pressures. If the headwalls are not allowed to deflect (i.e., considered fully restrained), then the headwalls should be designed based on at-rest earth pressures. The determination of which lateral earth pressure condition to be used for headwall design will be up to the culvert designer.

The active or at-rest earth pressures at depth z acting on a headwall can be determined by Equations (5) and (6), respectively. The design soil parameters for headwall lateral earth pressure design are presented on Plate B-1, in Appendix B. AEC recommends that headwall design first consider short-term soil conditions and then consider long-term soil conditions. Whichever soil condition results in a more conservative headwall design should then be used, regardless of the actual service life of the walls.

$$p_a = (q_s + \gamma h_1 + \gamma' h_2)K_a - 2c\sqrt{K_a} + \gamma_w h_2 \qquad \text{...........Equation (5)}$$

where: p_a = active earth pressure (psf).
q_s = uniform surcharge pressure (psf).
γ, γ' = wet unit weight and buoyant unit weight of soil (pcf).
h_1 = depth from ground surface to groundwater table (ft).
h_2 = z-h_1, depth from groundwater table to the point under consideration (ft).
z = depth below ground surface for the point under consideration (ft).
K_a = coefficient of active earth pressure.
c = cohesion of clayey soils (psf); c can be omitted conservatively for long-term soil conditions.
γ_w = unit weight of water, 62.4 pcf.

$$p_0 = (q_s+\gamma\ h_1+\gamma'\ h_2)K_0 + \gamma_w h_2 \qquad \text{...........Equation (6)}$$

where, p_0 = at-rest earth pressure (psf).
q_s = uniform surcharge pressure (psf).
γ, γ' = wet and buoyant unit weights of soil (pcf0
h_1 = depth from ground surface to groundwater table (ft).
h_2 = z-h_1, depth from groundwater table to point under consideration (ft).
Z = depth below ground surface (ft).
K_0 = coefficient of at-rest earth pressure.
γ_w = unit weight of water, 62.4 pcf.

Wall Sliding Resistance: The sliding resistance of the headwall foundations can be determined by the summation of friction resistance between the foundation and the underlying soil, the adhesion resistance between the foundation and the underlying soil, and passive earth pressure resistance in front of the wall (if any); while horizontal driving forces are determined by active or at-rest earth pressure caused by backfill materials behind the walls, as well as traffic or construction surcharge. Headwall sliding resistance can be determined using Equation (7). Foundation design should consider both short-term and long-term conditions. Whichever soil condition results in a more conservative foundation design should be used. Passive pressure resistance can conservatively be omitted from design. If passive earth pressure resistance is considered in the design, a FS of 2.0 should be applied to the passive pressure resistance component. Passive earth pressure resistance can be determined using Equation (8). Design soil parameters for wall foundation sliding resistance are presented on Plate B-1, in Appendix B.

$$\Sigma F_r = \Sigma V \text{ x } \tan(\delta) + B_f \text{ x } C_\alpha + P_p \qquad \text{...........Equation (7)}$$

where: ΣF_r = sum of horizontal resistance forces (plf).
ΣV = sum of vertical forces (plf).
δ = angle of friction between soil and footing; can be taken as 2/3 ϕ.
ϕ = angle of internal friction.
B_f = width of footing (ft).

C_α = soil adhesion (psf), can be taken as 0.6 times the cohesion of the layer.
P_p = passive pressure resistance (psf), see Equation (8).

$$p_p = \gamma z K_p + 2c(K_p)^{1/2} \qquad \text{...........Equation (8)}$$

where:
p_p = passive earth pressure (psf).
γ = wet unit weight of soil (pcf).
z = depth below ground surface for the point under consideration (ft).
K_p = coefficient of passive earth pressure.
c = cohesion of clayey soils (psf).

5.1.4 Excavation Stability

Cohesive soils in the Greater Houston area contain many secondary features which affect excavation stability, including sand seams and slickensides. Slickensides are shiny weak failure planes which are commonly present in fat clays; such clays often fail along these weak planes when they are not laterally supported, such as in an open excavation. The Contractor should not assume that slickensides and sand seams/layers/pockets are absent where not indicated on the logs.

The Contractor should be responsible for designing, constructing, and maintaining safe excavations. The excavations should be performed in a manner so as to not cause any distress to existing structures.

Excavations may be shored, sheeted and braced, or laid back to a stable slope for the safety of workers, the general public, and adjacent structures, except for excavations which are less than 5 feet deep and verified by a competent person to have no cave-in potential. The excavation should be in accordance with OSHA Safety and Health Regulations, 29 CFR, Part 1926.

Critical Height is defined as the height a slope will stand unsupported for a short time; in cohesive soils, it is used to estimate the maximum depth of open-cuts at given side slopes. Critical Height may be calculated based on the soil cohesion. Values for various slopes and cohesion are shown on Plate C-2, in Appendix C. Cautions listed below should be exercised in use of Critical Height applications:

1. AEC conservatively recommends a FS of 2.0 be applied to the determination of critical height; as a result, no more than 50 percent of the Critical Height computed should be used for vertical slopes. Unsupported vertical slopes are not recommended where granular soils or soils that will slough when not laterally supported are encountered within the excavation depth.

2. If the soil at the surface is dry to the point where tension cracks occur, any water in the crack will increase the lateral pressure considerably. In addition, if tension cracks occur, no cohesion should be assumed for the soils within the depth of the crack. The depth of the first waler should not exceed the depth of the potential tension crack. Struts should be installed before lateral displacement occurs.

3. Shoring should be provided for excavations where limited space precludes adequate side slopes, e.g., where granular soils will not stand on stable slopes and/or for deep open cuts.

4. All excavation and shoring should be designed and constructed by qualified professionals in accordance with OSHA requirements.

The maximum (steepest) allowable slopes for OSHA Soil Types for excavations less than 20 feet are presented on Plate C-3, in Appendix C.

If limited space is available for the required open excavation side slopes, the space required for the slope can be reduced by using a combination of bracing and open-cut as illustrated on Plate C-4, in Appendix C. Guidelines for bracing and calculating bracing stress are presented below.

Computation of Bracing Pressures: The following method can be used for calculating earth pressure against bracing for open-cuts. Lateral pressure resulting from construction equipment, traffic loads, or other surcharge should be considered by adding the equivalent uniformly distributed surcharge to the design lateral pressure. Hydrostatic pressure, if any, should also be considered. The active earth pressure at depth z can be determined by Equation (5) in Section 5.1.3 of this report. The design soil parameters for excavation bracing design are presented on Plate B-1, in Appendix B. AEC recommends that excavation bracing design first consider short-term soil conditions and then consider long-term soil conditions. Whichever soil condition results in a more conservative excavation bracing design should then be used, regardless of the actual time the shoring will remain in place during construction.

Pressure distribution for the practical design of struts in open-cuts for clays and sands are illustrated on Plates C-5 through C-7, in Appendix C. Struts in mixed soil (i.e., sand and clay) conditions should be based on whichever soil condition (either sand or clay) results in a more conservative shoring design.

Bottom Stability: In open-cuts, it is necessary to consider the possibility of the bottom failing by heaving, due to the removal of the weight of excavated soil. Heaving typically occurs in soft plastic clays when the excavation depth is sufficiently deep enough to cause the surrounding soil to displace vertically due to bearing capacity failure of the soil beneath the excavation bottom, with a corresponding upward movement of the soils in the bottom of the excavation. In fat and lean clays, heave normally does not occur unless the ratio of Critical Height (see Plate C-2, in Appendix C) to Depth of Cut approaches one. In very sandy and silty lean clays and granular soils, heave can occur if an artificially large head of water is created due to installation of impervious sheeting while bracing the cut. This can be mitigated if groundwater is lowered below the excavation by dewatering the area. Guidelines for evaluating bottom stability in clay soils are presented on Plate C-8, in Appendix C.

Soil and Groundwater: AEC anticipates that open-cut excavations for the proposed culvert will generally encounter soft to very stiff fat/lean clay (CH/CL) in the top 12 feet of the excavation, extending into loose clayey/silty sand (SC/SM) at the bottom of the excavation.

Based on the groundwater levels presented on Table 3 in Section 4.1 of this report, there may groundwater seepage towards the bottom of the excavation. There is a possibility to control the seepage into the excavation using an open drainage method (i.e., sump and pump), although this is not guaranteed. If the seepage inflow into the excavation exceeds what can be handled by open drainage method, then predrainage (i.e., wellpoints in well-graded sands or eductors/ejectors for poorly-graded sands and silts) groundwater control method may be necessary. If predrainage groundwater control is used, the groundwater level should be lowered to at least 3 feet below the bottom of the excavation. AEC notes that groundwater depths can vary from location to location and at any given location, and the groundwater depths could be higher than anticipated during construction, depending on time of year and amount of rainfall. If required, groundwater control recommendations are presented in Section 6.2 of this report.

If the excavation extends below groundwater and the soils at or near the bottom of the excavation are mainly sands or silts, the bottom can fail by blow-out (boiling) when a sufficient hydraulic head exists. The potential for boiling or in-flow of granular soils increases where the groundwater is pressurized. To reduce the potential for boiling of excavations terminating in granular soils below pressurized groundwater, the groundwater table should be lowered at least 3 feet below the excavation.

Secondary Features and Fill Soils: Ferrous and calcareous nodules, slickensides, as well as sand partings/seams were encountered in the borings. These secondary structures may become sources of localized instability when they are exposed during excavation, especially when they become saturated. AEC notes that soils with secondary structures tend to slough or cave-in when not laterally confined, such as in excavations. The Contractor should be aware of the potential for cave-in of the soils. Low plasticity soils (silts and clayey silts) will lose strength and may behave like granular soils when saturated.

Stockpile and Equipment Surcharge: To avoid surcharging the excavation walls, stockpile of excavated materials immediately adjacent to the excavation face should be prohibited. AEC recommends stockpiled materials be placed at least 6 feet away from the edge of an excavation face, and no higher than 3 feet. Construction equipment working near the excavation may also induce excessive surcharge loads; AEC recommends appropriate shoring or shield system be provided considering these impacts in addition to the lateral earth and hydrostatic pressures.

5.1.5 Select Clay Backfill

AEC recommends that select clay fill be used as backfill around the culvert RCBs and behind culvert headwalls.

'Select' Clay Fill: It is AEC's experience that 'select' fill material imported from sand and clay pits in the Greater Houston area is generally non-homogenous (i.e., composed of a mixture of sands, silts, and clays, instead of a homogenous sandy clay material) and of poor quality, and either contains too much sand or has large clay clods with high expansive potential. Use of this non-homogenous soil can result in poor long-term performance of structures and pavements placed on top of the fill.

Precautions: Prior to construction, the Contractor should determine if they can obtain qualified select clay fill meeting the below select clay fill criteria. The closest sand and clay pit to the project site may not be able to deliver fill material that meets the requirements below. The Contractor should also be aware that testing of select clay fill (see below) typically takes a minimum of 1.5 days to complete and they should accommodate testing in their fill placement in their project schedule. In addition, imported fill that is delivered to the project site may vary from day to day; material delivered to the site may pass one day but fail the next.

Select Clay Fill Requirements: Select clay fill (whether imported from offsite or excavated onsite) should consist of *uniform*, non-active inorganic lean clays with a PI between 10 and 20 percent, and more than 50 percent passing a No. 200 sieve. Any clay soil intended for use as select clay fill (whether imported from offsite or excavated onsite) shall not have clay clods with PI greater than 20, clay clods greater than 2 inches in diameter, or contain sands/silts with PI less than 10. Sand and clay mixtures/blends are unacceptable for use as select clay fill. Sand/silt with clay clods is unacceptable for use as select clay fill. Mixing sand into clay or mixing clay into sand/silt is also unacceptable for use as select clay fill. **The testing lab shall *reject* any imported material delivered to the project site that does not meet the PI, sieve, and clay clod requirements above, without exceptions.**

<u>Lifts and Compaction:</u> All material intended for use as select clay fill should be tested prior to use to confirm that it meets select clay fill criteria. The fill should be placed in loose lifts not exceeding 8 inches in thickness. Backfill within 3 feet of walls or columns should be placed in loose lifts no more than 4-inches thick and compacted using hand tampers, or small self-propelled compactors.

Select clay fill should be compacted to a minimum of 95 percent of the ASTM D 698 (Standard Proctor) maximum dry unit weight at a moisture content ranging between optimum and 3 percent above optimum.

<u>Testing:</u> **If select clay fill will be used, at least one Atterberg Limits and one percent passing a No. 200 sieve test shall be performed for each 10,000 square feet (sf) of placed fill, every second lift (with a minimum of one set of tests per), to determine whether it meets select clay fill requirements.** Prior to placement of pavement or concrete, the moisture contents of the top 2 lifts of compacted select clay fill shall be re-tested (if there is an extended period between fill placement and concrete placement) to determine if the in-place moisture content of the lifts have been maintained at the required moisture requirements.

5.2 Access Road Pavement Subgrade

AEC understands that the existing access road crossing Ditch C will need to be reconstructed, since the culvert replacement will be performed using open cut method. Atkins' drawings indicate that the new vehicle service road will be an asphalt roadway, consisting of a 1.5 inch thick Hot Mix Asphalt (HMA) surface over an 8 inch thick crushed concrete base course, and an 8 inch thick stabilized subgrade.

AEC anticipates that the roadway subgrade will partially be located on top of existing clay soils (away from the culverts) and also be located on top of select clay backfill that will be placed on top of the new culvert RCBs. In both cases, AEC recommends that the roadway subgrade be stabilized with hydrated lime.

<u>Lime Stabilized Subgrade:</u> The natural subgrade soils beneath the pavement that were encountered in Borings B-82 and B-83 generally consist of fat/lean clay (CH/CL) with high to very high plasticity (see "Degree of Plasticity of Cohesive Soils" on Plate A-6, in Appendix A). According to Section 2.16.12 of the 2015 HAS Design Criteria Manual, all subgrades should be lime/fly ash-treated or cement/fly ash-treated. However, based on the cohesive soils encountered in the borings, it is AEC's opinion that using fly ash or cement will not be effective for subgrade stabilization along the access road alignment. Instead, **AEC recommends lime stabilized**

subgrade be used alone for the construction of the access road. Based on the subsurface soil conditions, AEC recommends that a minimum of 8 inches of subgrade soils beneath the proposed pavement (whether in-situ clay or select clay backfill above the culvert RCBs) be stabilized with a minimum of 8 percent lime by dry soil weight. AEC's lime series tests (see AEC Report G103-21 Revision 1) indicate that an 8 percent application rate (by dry soil weight) will be necessary; however, the actual percentage of lime should be determined by lime-series or pH method by the CMT laboratory prior to construction.

5.2.2 Subgrade Preparation

AEC assumes that the access road pavement will be constructed at or near existing grade. Subgrade preparation should extend to 2 feet beyond the paved area perimeters. Existing pavement and base should first be demolished. Removal of existing pavement shall be performed in accordance with Item P-101 of the FAA AC 150/5370-10G Airport Construction Standards. After stripping, the subgrade should be cut to grade to accommodate the pavement section. After cutting to grade, AEC recommends that a competent soil technician inspect the exposed subgrade to determine if there are any unsuitable soils or other deleterious materials. Excavate and dispose of unsuitable soils and other deleterious materials which will not consolidate; the excavation depth should be increased when inspection indicates the presence of organics and deleterious materials to greater depths. The exposed soils should be proof-rolled in accordance with Item 216 of the 2014 TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges to identify and remove any weak, compressible, or other unsuitable materials. Excavation and subgrade preparation shall be performed in accordance with Item P-152 of the FAA AC 150/5370-10G Airport Construction Standards.

Scarify the top 8 inches of the exposed subgrade and stabilize with a minimum of 8 percent hydrated lime (by dry weight). The stabilized soils should be compacted to 95 percent of their ASTM D 1557 (Modified Proctor) dry density at a moisture content ranging from optimum to 3 percent above optimum. Lime stabilization shall be performed in accordance with Item P-155 of the FAA AC 150/5370-10G Airport Construction Standards.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Site Preparation and Grading

To mitigate site problems that may develop following prolonged periods of rainfall, it is essential to have adequate drainage to maintain a relatively dry and firm surface prior to starting any work at the site. Adequate drainage should be maintained throughout the construction period. Methods for controlling surface runoff and ponding include proper site grading, berm construction around exposed areas, and installation of sump pits with pumps.

6.2 Groundwater Control

The need for groundwater control will depend on the depth of excavation relative to the groundwater depth at the time of construction. If there is heavy rain prior to or during construction, the groundwater table may be higher than indicated in this report; higher seepage is also likely and may require a more extensive groundwater control program. In addition, groundwater may be pressurized in certain areas of the alignment, requiring further evaluation and consideration of the excess hydrostatic pressures. Groundwater control should be in general accordance with Section 01578 of the latest edition of the City of Houston Standard General Requirement (COHSGR).

The Contractor should be responsible for selecting, designing, constructing, maintaining, and monitoring a groundwater control system and adapt his operations to ensure the stability of the excavations. Groundwater information presented in Section 4.1 of this report and elsewhere, along with consideration for potential environmental and site variation between the time of our field exploration and construction, should be incorporated in evaluating groundwater depths. The following recommendations are intended to guide the Contractor during design and construction of the dewatering system.

Groundwater control methods typically can be classified into three categories: (i) open pumping, where water is allowed to flow into an excavation and is collected in ditches or sumps and pumped away; (ii) predrainage, where the water table is lowered before excavation using wellpoints, ejector/eductor systems, deep wells, etc.; and (iii) cut off or exclusion, where the groundwater is prevented from entering the excavation by an impermeable barrier, such as by sheet piling, grouting, deep soil mixing, ground freezing, slurry shields, etc.

Cohesive Soils: Groundwater control in cohesive soils can typically be performed using open pumping methods. Seepage rates are lower than in granular soils and groundwater is usually collected in sumps and/or channeled by gravity flow to storm sewers. If cohesive soils contain significant secondary features, seepage rates will be higher. This may require larger sumps and drainage channels, or if significant granular layers are interbedded within the cohesive soils, methods used for granular soils may be required. Where it is present, pressurized groundwater will also yield higher seepage rates.

Granular Soils: Groundwater control in granular soils will typically require predrainage methods or cutoff/exclusion methods. For excavations that are less than 15 feet deep that will occur within saturated sands, a predrainage method such as wellpoints can be considered. For excavations that are greater than 15 feet deep, other predrainage methods that can be considered include multiple staged wellpoints, ejectors/eductors (primarily for use when silty soils are present), or deep wells with submersible pumps. Generally, with predrainage methods, the groundwater depth should be lowered at least 3 feet below the excavation bottom to be able to work on a firm surface when water-bearing granular soils are encountered.

If predrainage methods cannot be used, then a cutoff/exclusion method such as interlocking water-tight sheet piles, drilled shaft/secant pile wall (with grout between the shafts/piles), or jet grouting of the granular strata may be necessary.

Extended Dewatering: Extended and/or excessive dewatering can result in settlement of existing structures in the vicinity of the dewatering operations; the Contractor should take the necessary precautions to minimize the effect on existing structures in the vicinity of the dewatering operation. We recommend that the Contractor verify the groundwater depths and seepage rates prior to and during construction and retain the services of a dewatering expert (if necessary) to assist them in identifying, implementing, and monitoring the most suitable and cost-effective method of controlling groundwater.

Bottom Heave or Boiling: For excavation in cohesive soils, the possibility of bottom heave must be considered due to the removal of the weight of excavated soil. In lean and fat clays, heave normally does not occur unless the ratio of Critical Height (see Plate C-2, in Appendix C) to Depth of Cut approaches one. In silty clays, heave does not typically occur unless an artificially large head of water is created using impervious sheeting in bracing the cut. If the excavation extends below groundwater and the soils at or near the bottom of the excavation are mainly sands or silts, the bottom can fail by blow-out (boiling) when a sufficient hydraulic head exists. The potential for boiling or in-flow of granular soils increases where the groundwater is pressurized. To reduce the potential for boiling of excavations terminating in granular soils below pressurized groundwater, the groundwater table should be lowered at least 3 feet below the excavation.

Perched Water: Although it may be present at a shallower depth than the normal groundwater level, perched water should still be considered a form of groundwater. If perched water is encountered during the construction phase, the groundwater control methods mentioned above would still be the same. Depending on the size of the perched reservoir and recharge rates, the contractor should not assume that perched water can be completely dewatered during a normal construction period.

6.3 Construction Monitoring

Site preparation (including clearing and proof-rolling) and earthwork operations should be monitored by qualified geotechnical professionals to check for compliance with project documents and changed conditions, if encountered. AEC should be allowed to review the design and construction plans and specifications prior to release to check that the geotechnical recommendations and design criteria presented herein are properly interpreted.

7.0 GENERAL

The information contained in this report summarizes conditions found on the dates the borings were drilled. The attached boring logs are true representations of the soils encountered at the specific boring locations on the dates of drilling. Reasonable variations from the subsurface information presented in this report should be anticipated. AEC should be notified immediately if conditions encountered during construction are significantly different from those presented in this report.

8.0 LIMITATIONS

The investigation was performed using the standard level of care and diligence normally practiced by recognized geotechnical engineering firms in this area, presently performing similar services under similar circumstances. The report has been prepared exclusively for the project and location described in this report and is intended to be used in its entirety. If pertinent project details change or otherwise differ from those described herein, AEC should be notified immediately and retained to evaluate the effect of the changes on the recommendations presented in this report and revise the recommendations if necessary. The recommendations presented in this report should not be used for other structures located along the alignment or similar structures located elsewhere, without additional evaluation and/or investigation.

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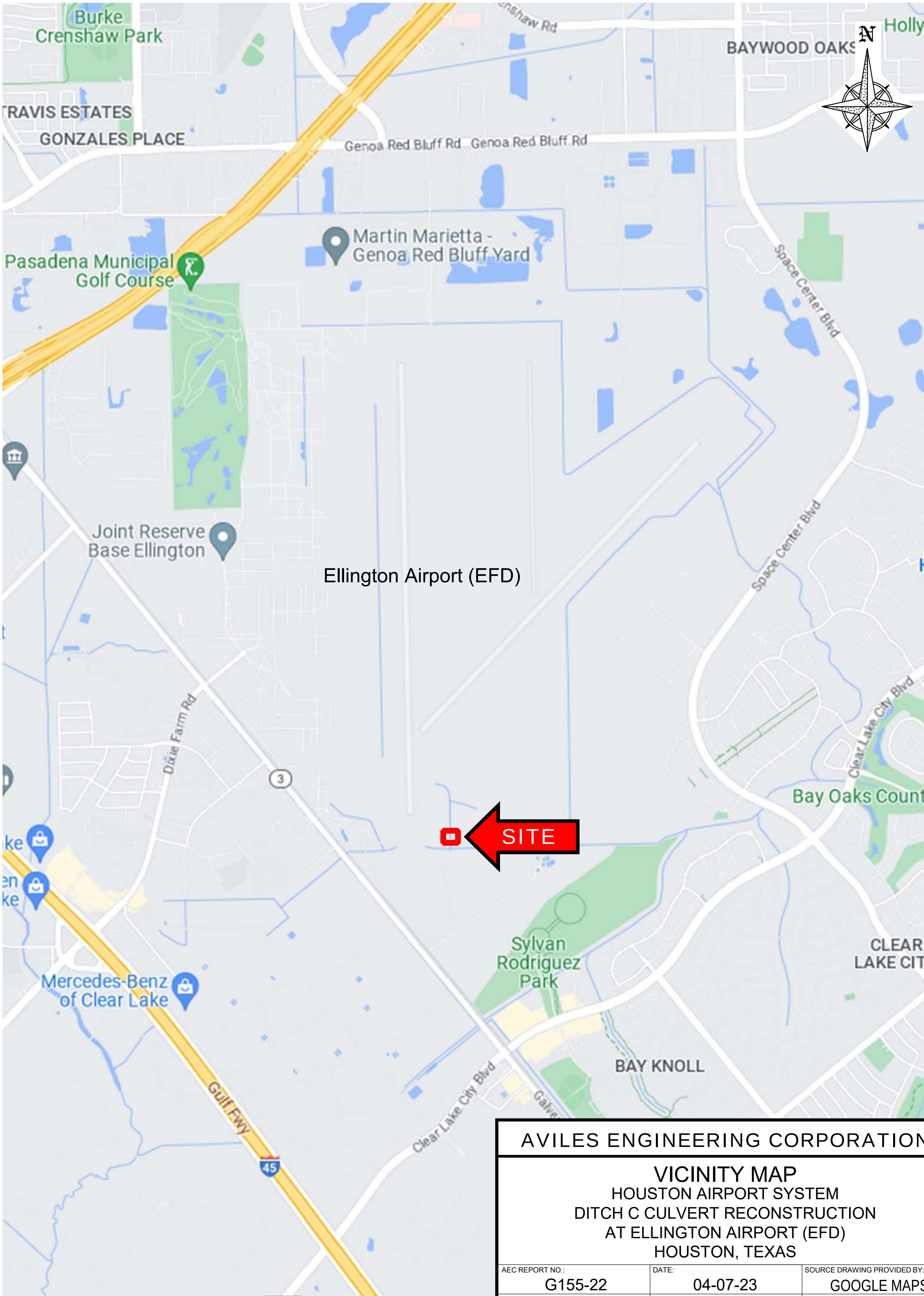
APPENDIX A

Plate A-1	Vicinity Map
Plate A-2	Boring Location Plan
Plates A-3 and A-4	Boring Logs
Plate A-5	Key to Symbols
Plate A-6	Classification of Soils for Engineering Purposes
Plate A-7	Terms Used on Boring Logs
Plate A-8	ASTM & TXDOT Designation for Soil Laboratory Tests
Plate A-9	Grain Size Analysis Results



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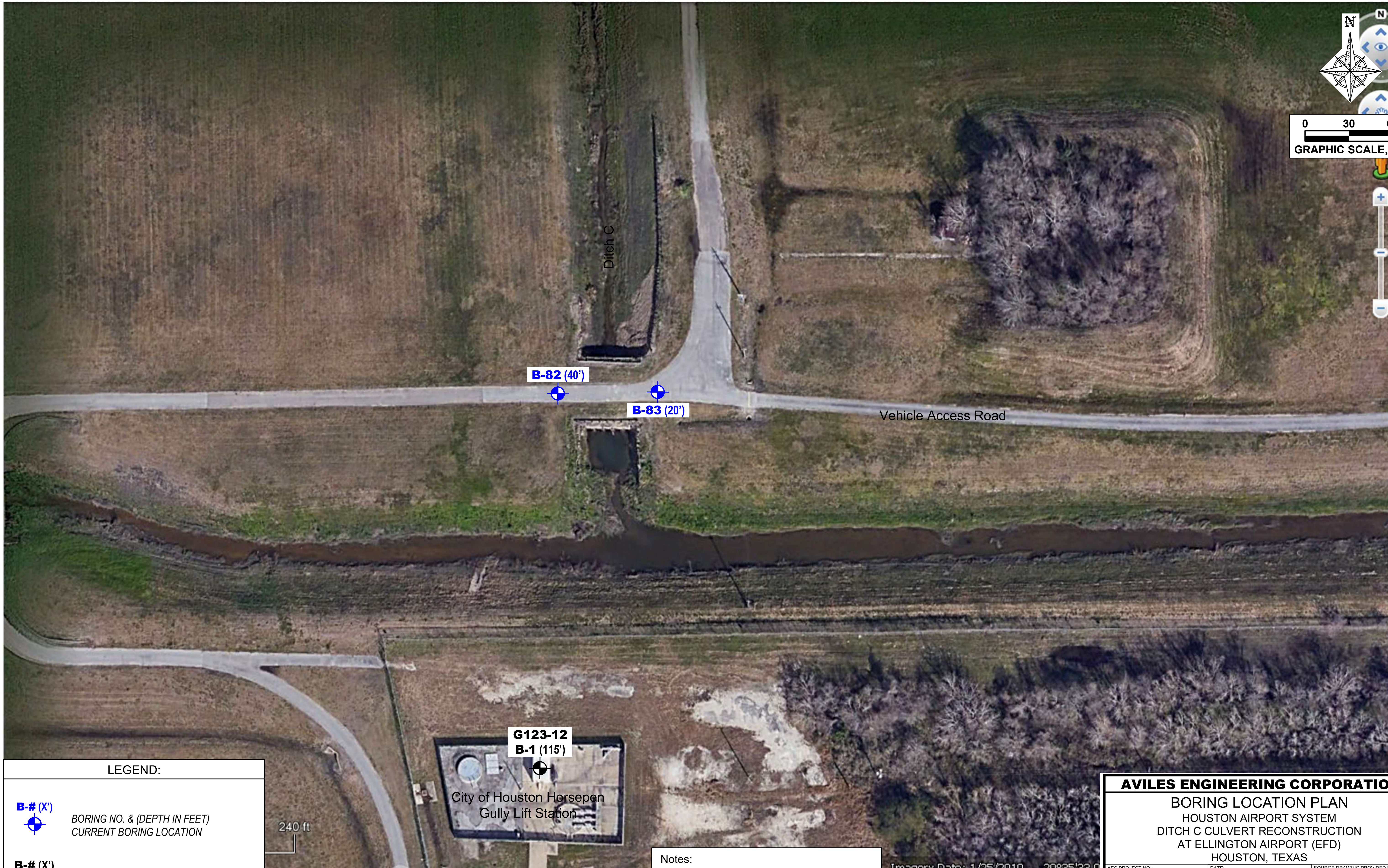
VICINITY MAP

HOUSTON AIRPORT SYSTEM
DITCH C CULVERT RECONSTRUCTION
AT ELLINGTON AIRPORT (EFD)
HOUSTON, TEXAS

AEC REPORT NO.:	DATE:	SOURCE DRAWING PROVIDED BY:
G155-22	04-07-23	GOOGLE MAPS
APPROX. SCALE:	DRAFTED BY:	PLATE NO.:
NTS	WLW	PLATE A-1



LEGEND:

B-# (X')
BORING NO. & (DEPTH IN FEET)
CURRENT BORING LOCATION

B-# (X')
EXISTING BORING NO. (DEPTH IN FEET)
AEC Report G123-12, JUNE 2012

Notes:
1) Site plan is 'best fit' to aerial photo.
2) Boring locations have been surveyed.

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BORING LOCATION PLAN
HOUSTON AIRPORT SYSTEM
DITCH C CULVERT RECONSTRUCTION
AT ELLINGTON AIRPORT (EFD)
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DRAWING PROVIDED BY:
G155-22	04-07-2023	GOOGLE
APPROX. SCALE:	DRAFTED BY:	PLATE NO.:
1" = 60'	WLW	PLATE A-2

PROJECT: **EFD Culvert Repairs**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-82**

DATE **3/1/2022** TYPE **4" Wet Rotary** LOCATION **See Boring Plan**

DESCRIPTION

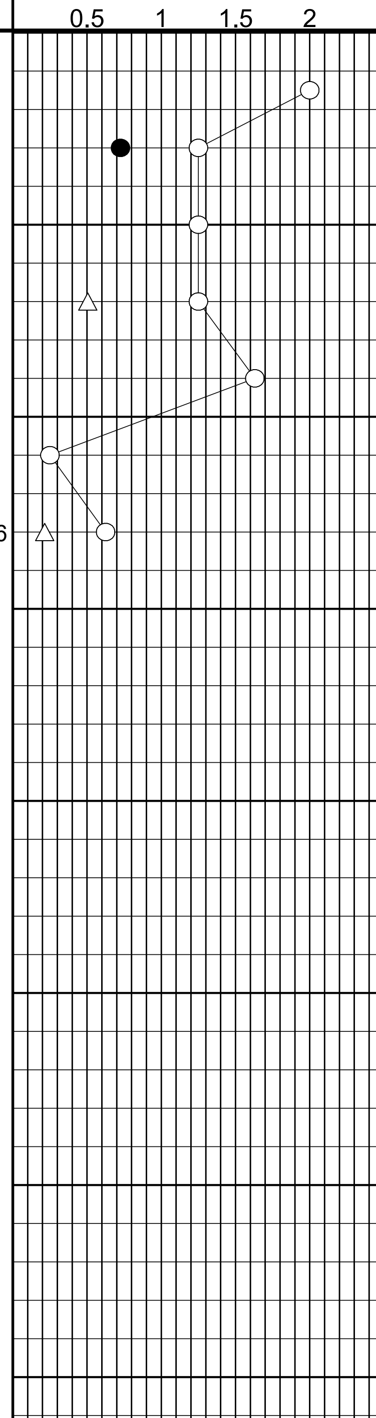
GRID Coordinates (US Survey ft):
Texas State Plane Zone:
Easting: 3188203.773213
Northing: 13782784.010817
Elevation: 24.52

Depth in feet: 0, 5, 10, 15, 20, 25, 30, 35

DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Pavement: 1" asphalt, 9" cement stabilized base, 7" cement stabilized shell							
Stiff to very stiff, dark gray Fat Clay with Sand (CH)		18					
		30	93.4	80.0	73	19	54
-gray and brown, with ferrous and calcareous nodules 4'-6'		31					
Soft to very stiff, gray and tan Fat Clay (CH) -with ferrous nodules 6'-10'		28	93.1				
-with calcareous nodules 8'-10'		33		88.4	71	20	51
-gray, with abundant silty sand seams 10'-12'		33					
Gray and tan Clayey Sand (SC), with abundant silty sand seams		24	107.6				
Loose to dense, tan Silty Sand (SM)				26.6			
	14	24					
-with calcareous nodules 16'-18' and fat clay pockets 16'-20'	6	26					
-with sandstone nodules 18'-20'	20	25					
	46	24					
-brown 28'-40'				28.6			
	30	23					
	18	30					

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **16** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **15** FEET WHILE DRILLING
WATER LEVEL AT **12.7** FEET AFTER **1/4 HR**
DRILLED BY **V&S** DRAFTED BY **WLW** LOGGED BY **DN**

ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **EFD Culvert Repairs** BORING **B-82**

DATE **3/1/2022** TYPE **4" Wet Rotary** LOCATION **See Boring Plan**

DEPTH IN FEET	SYMBOL	SAMPLE INTERVAL	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	SHEAR STRENGTH, TSF (△ Confined Compression; ● Unconfined Compression; ○ Pocket Penetrometer; □ Torvane; 0.5 1 1.5 2)	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
			Silty Sand (SM) (cont.) -with sandstone nodules 38'-40'	24	26						
40			Termination Depth = 40 feet								
45											
50											
55											
60											
65											
70											

BORING DRILLED TO **16** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **15** FEET WHILE DRILLING
WATER LEVEL AT **12.7** FEET AFTER **1/4 HR**
DRILLED BY **V&S** DRAFTED BY **WLW** LOGGED BY **DN**

PROJECT NO. **G155-22** **PLATE A-3**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **EFD Culvert Repairs** BORING **B-83**

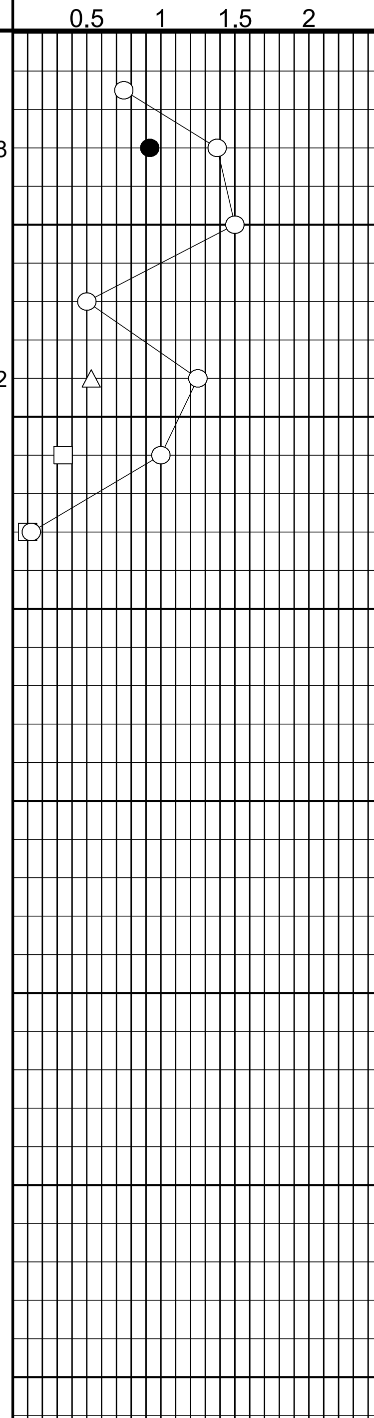
DATE **3/1/2022** TYPE **4" Wet Rotary** LOCATION **See Boring Plan**

GRID Coordinates (US Survey ft):
Texas State Plane Zone:
Easting: 3188268.082470
Northing: 13782787.657500
Elevation: 24.17

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 1.25" asphalt, 9.5" cement stabilized base, 3.5" cement stabilized shell							
	Fill: stiff to very stiff, dark gray Lean Clay with Sand (CL), with fat clay and calcareous nodules		20		75.1	41	16	25
			24	102.8				
5	-with ferrous nodules 4'-6'		21					
	Fill: firm to stiff, gray Fat Clay with Sand (CH), with ferrous nodules		31		84.1	51	17	34
	Firm to very stiff, tan and gray Lean Clay (CL), with abundant silty sand seams		20	107.2				
10			26					
	Loose, gray and tan Clayey Sand (SC), with silty sand seams		28		36.8	27	19	8
15	-tan 14'-16'	8	27					
	Medium dense to dense, tan Silty Sand (SM)	30	20					
		15	23		27.6			
20	Termination Depth = 20 feet							
25								
30								
35								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **15** FEET WHILE DRILLING
WATER LEVEL AT **12.7** FEET AFTER **COMPLETE**
DRILLED BY **V&S** DRAFTED BY **WLW** LOGGED BY **DN**

PROJECT NO. G155-22 **PLATE A-4**

KEY TO SYMBOLS

Symbol	Description
	Strata symbols
	Paving
	High plasticity clay
	Clayey sand
	Silty sand
	Fill
	Low plasticity clay
	Misc. Symbols
	Water table depth during drilling
	Subsequent water table depth
	Pocket Penetrometer
	Unconfined Compression
	Confined Compression
	Torvane
	Soil Samplers
	Auger
	Undisturbed thin wall Shelby tube
	Standard penetration test

AMLES ENGINEERING CORP.

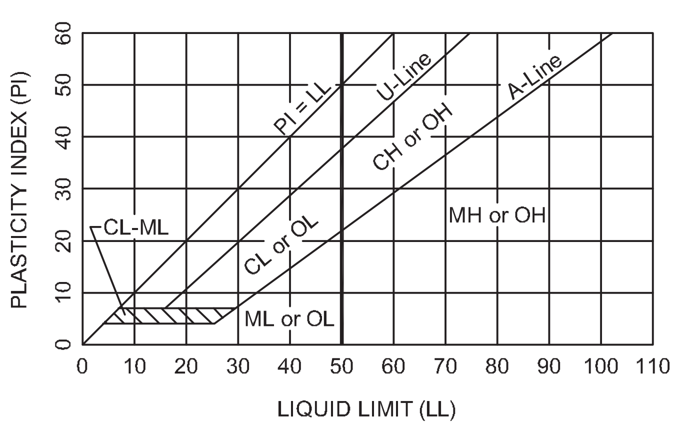
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

ASTM Designation D-2487

MAJOR DIVISIONS				GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (Less than 50% of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)		GW	Well-graded gravel, well-graded gravel with sand
				GP	Poorly-graded gravel, poorly-graded gravel with sand
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	GM	Silty gravel, silty gravel with sand
			Limits plot above "A" line & hatched zone on plasticity chart	GC	Clayey gravel, clayey gravel with sand
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 sieve)		SW	Well-graded sand, well-graded sand with gravel
				SP	Poorly-graded sand, poorly-graded sand with gravel
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	SM	Silty sand, silty sand with gravel
			Limits plot above "A" line & hatched zone on plasticity chart	SC	Clayey sand, clayey sand with gravel
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)		SILTS AND CLAYS (Liquid Limit Less Than 50%)		ML	Silt, silt with sand, silt with gravel, sandy silt, gravelly silt
				CL	Lean clay, lean clay with sand, lean clay with gravel, sandy lean clay, gravelly lean clay
				OL	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt
		SILTS AND CLAYS (Liquid Limit 50% or More)		MH	Elastic silt, elastic silt with sand, sandy elastic silt, gravelly elastic silt
				CH	Fat clay, fat clay with sand, fat clay with gravel, sandy fat clay, gravelly fat clay
				OH	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt

NOTE: Coarse soils between 5% and 12% passing the No. 200 sieve and fine-grained soils with limits plotting in the hatched zone of the plasticity chart are to have dual symbols.

PLASTICITY CHART



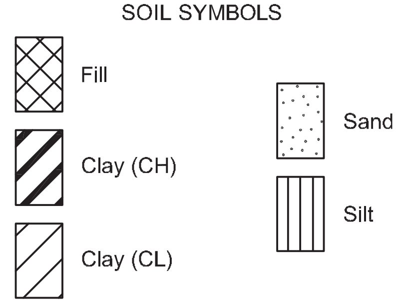
Equation of A-Line: Horizontal at PI=4 to LL=25.5, then PI=0.73(LL-20)

Equation of U-Line: Vertical at LL=16 to PI=7, then PI=0.9(LL-8)

DEGREE OF PLASTICITY OF COHESIVE SOILS

Degree of Plasticity	Plasticity Index
None	0 - 4
Slight	5 - 10
Medium	11 - 20
High	21 - 40
Very High	>40

SOIL SYMBOLS



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TERMS USED ON BORING LOGS

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

6" 3" 3/4" #4 #10 #40 #200

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

152 76.2 19.1 4.76 2.00 0.420 0.074 0.002

SOIL GRAIN SIZE IN MILLIMETERS

STRENGTH OF COHESIVE SOILS

Consistency	Undrained Shear Strength, Kips per Sq. ft.	SPT Blowcount
Very Soft	less than 0.25	< 2 bpf
Soft	0.25 to 0.50	2-4 bpf
Firm	0.50 to 1.00	4-8 bpf
Stiff	1.00 to 2.00	8-16 bpf
Very Stiff	2.00 to 4.00	16-32 bpf
Hard	greater than 4.00	>32 bpf

RELATIVE DENSITY OF COHESIONLESS SOILS FROM STANDARD PENETRATION TEST

Very Loose .... <4 bpf
Loose .... 5-10 bpf
Medium Dense .... 11-30 bpf
Dense .... 31-50 bpf
Very Dense .... >50 bpf

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows per Foot	Description
25	25 blows driving sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows driving sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows driving sampler 3 inches, during initial 6-inches seating interval.

NOTE: To avoid change to sampling tools, driving is limited to 50 blows during or after seating interval.

DRY STRENGTH ASTM D2488

None	Dry specimen crumbles into powder with mere pressure of handling
Low	Dry specimen crumbles into powder with some finger pressure
Medium	Dry specimen breaks into pieces or crumbles with considerable pressure
High	Dry specimen cannot be broken with finger pressure, it can be broken between thumb and hard surface
Very High	Dry specimen cannot be broken between thumb and hard surface

MOISTURE CONDITION ASTM D2488

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy. The degree of slickensidedness depends upon the spacing of slickensides and the easiness of breaking along these planes.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Friable	Crumbly, can be easily crushed with light pressure.
Blocky	Clays that have a block-like or polyhedral structure.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil types.
Interlayered	Soil sample composed of alternating layers of different soil types.
Intermixed	Soil sample composed of pockets of different soil types and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of calcium material.

ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

SOIL TEST	ASTM TEST DESIGNATION	TXDOT TEST DESIGNATION
Unified Soil Classification System	D 2487	Tex-142-E
Moisture Content	D 2216	Tex-103-E
Specific Gravity	D 854	Tex-108-E
Sieve Analysis	D 6913	Tex-110-E (Part 1)
Hydrometer Analysis	D 7928	Tex-110-E (Part 2)
Minus No. 200 Sieve	D 1140	Tex-111-E
Liquid Limit	D 4318	Tex-104-E
Plastic Limit	D 4318	Tex-105-E
Standard Proctor Compaction	D 698	Tex-114-E
Modified Proctor Compaction	D 1557	Tex-113-E
California Bearing Ratio	D 1883	-
Swell	D 4546	-
Consolidation	D 2435	-
Unconfined Compression	D 2166	-
Unconsolidated-Undrained Triaxial	D 2850	Tex-118-E
Consolidated-Undrained Triaxial	D 4767	Tex-131-E
Permeability (constant head)	D 5084	-
Pinhole	D 4647	-
Crumb	D 6572	-
Double Hydrometer	D 4221	-
pH of Soil	D 4972	Tex-128-E
Soil Suction	D 5298	-
Soil Sulfate	C 1580	Tex-145-E
Organics	D 2974	Tex-148-E

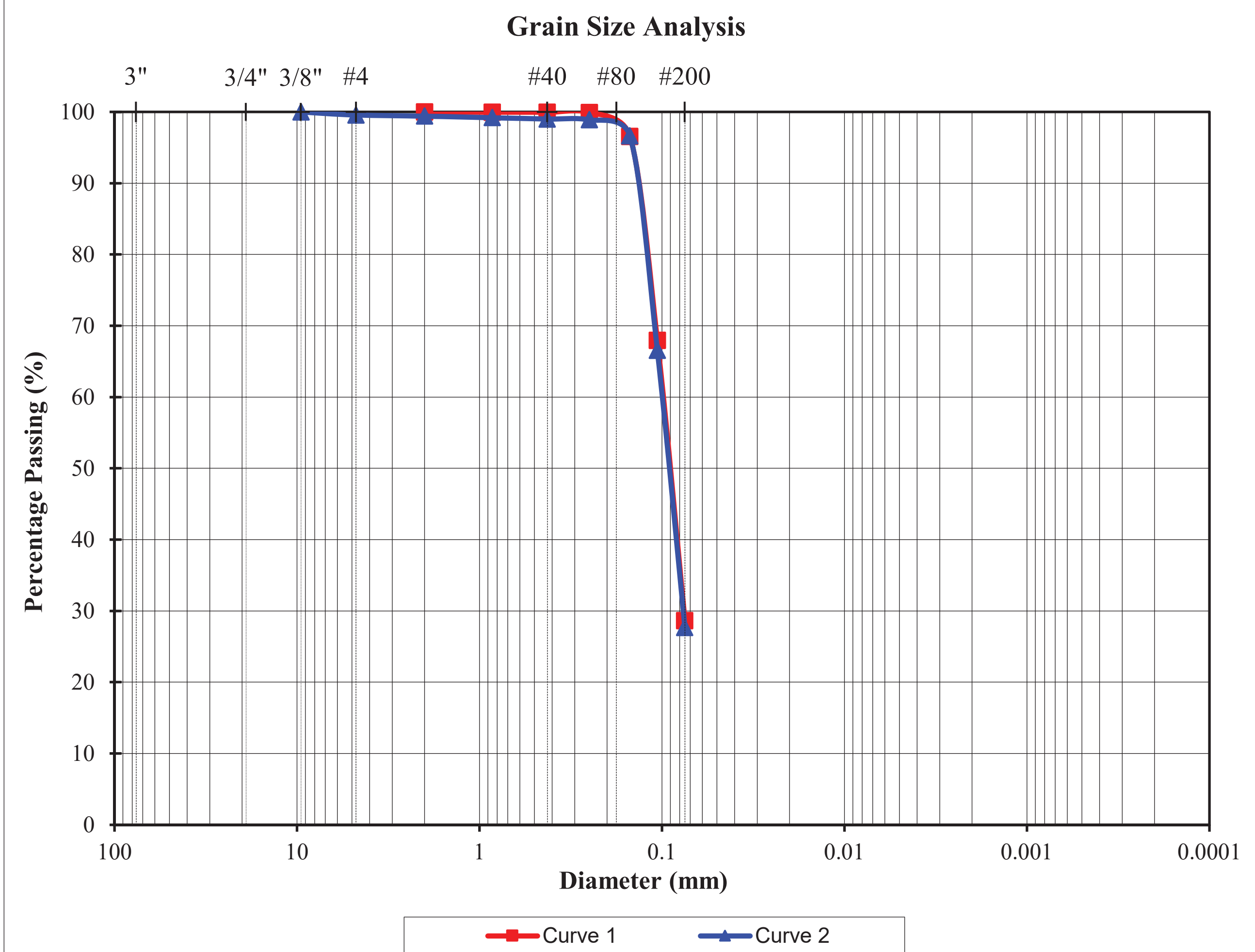
AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE (ASTM D 6913) & HYDROMETER (ASTM D 7928)

Project: EFD Ditch C Culvert Reconstruction **Job No.:** G155-22
Location of Project: Houston, Texas

	Gravel	Sand		Silt	Clay
		Coarse to Medium	Fine		



Curve	Boring	Depth (ft)	Soil Description	D50 (mm)	Cu	Cc
1	B-82	28-30	Silty Sand (SM)	0.09	N/A	N/A
2	B-83	18-20	Silty Sand (SM)	0.09	N/A	N/A

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APPENDIX B

AVILES ENGINEERING CORP.

G155-22 Ditch C Culvert Reconstruction
Lateral Earth Pressure Parameters for Culvert Headwalls

Table 1. Lateral Earth Pressure Parameters for Headwall Backfills

Soil Type	γ (pcf)	γ' (pcf)	Short-Term						Long-Term					
			C (psf)	C_α (psf)	φ (deg)	K_a	K_0	K_p	C' (psf)	C'_α (psf)	φ' (deg)	K_a	K_0	K_p
Select Clay Fill	120	58	1,600	0	0	1.00	1.00	1.00	140	84	24	0.42	0.59	2.37

Table 2. Lateral Earth Pressure Parameters for Culvert Headwalls (Based on Borings B-82 and B-83)

Elevation (ft)	Soil Type	γ (pcf)	γ' (pcf)	Short-Term						Long-Term					
				C (psf)	C_α (psf)	φ (deg)	K_a	K_0	K_p	C' (psf)	C'_α (psf)	φ' (deg)	K_a	K_0	K_p
25 to 19	Stiff to very stiff CL/CH	121	59	1400	840	0	1.00	1.00	1.00	125	75	16	0.57	0.72	1.76
19 to 17	Firm to stiff CH	119	57	800	480	0	1.00	1.00	1.00	75	45	16	0.57	0.72	1.76
17 to 13	Soft to stiff CL/CH	120	58	500	300	0	1.00	1.00	1.00	50	30	16	0.57	0.72	1.76
13 to 10	Loose SC	133	71	0	0	26	0.39	0.56	2.56	0	0	26	0.39	0.56	2.56

Notes: (1) γ = unit weight for soil above water level, γ' = buoyant unit weight for soil below water level.

(2) C = Soil ultimate cohesion for short-term condition, Cα = Soil adhesion for short-term condition, φ = Soil friction angle for short-term condition.

(3) C' = Soil ultimate cohesion for long-term condition, C'α = Soil adhesion for long-term condition, φ' = Soil friction for long-term condition.

(4) K_a = Coefficient of active earth pressure, K_0 = Coefficient of at-rest earth pressure, K_p = Coefficient of passive earth pressure, for level backfill.

(5) CH = Fat Clay, CL = Lean Clay, SC = Clayey Sand.

(6) AEC recommends the use of interface friction angle, δ = 2/3 φ for short-term condition and δ' = 2/3 φ' for long-term condition.

(7) AEC recommends the use of FS = 2.0 for passive earth pressure if it is to be used in the design.

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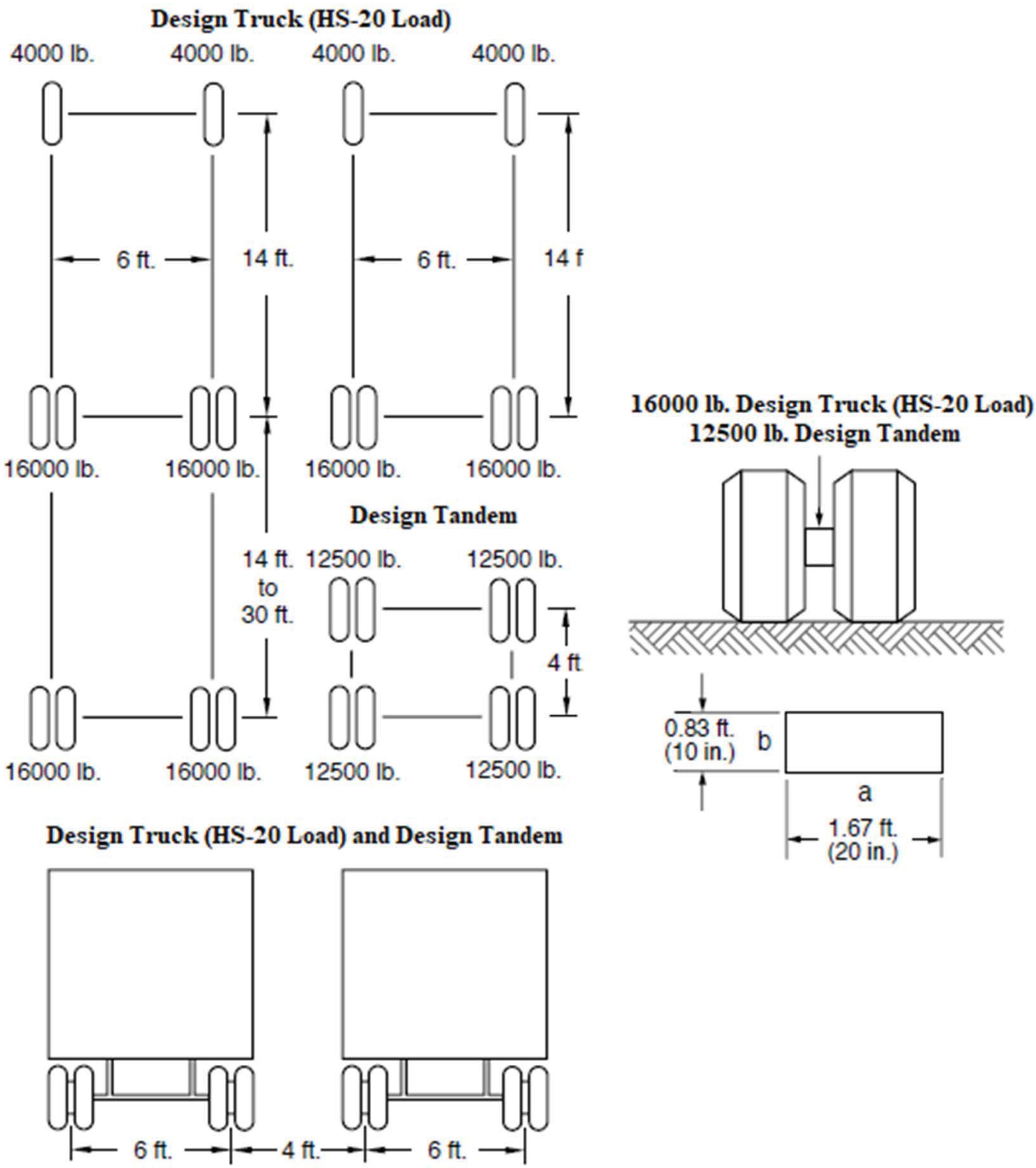
AASHTO LRFD DESIGN TRUCK AND DESIGN TANDEM LOADING



Source: American Concrete Pipe Association, 2001, Design Data 1, "Highway Live Loads on Concrete Pipe".

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LRFD Critical Wheel Loads and Spread Dimensions at the Top of Pipe for:

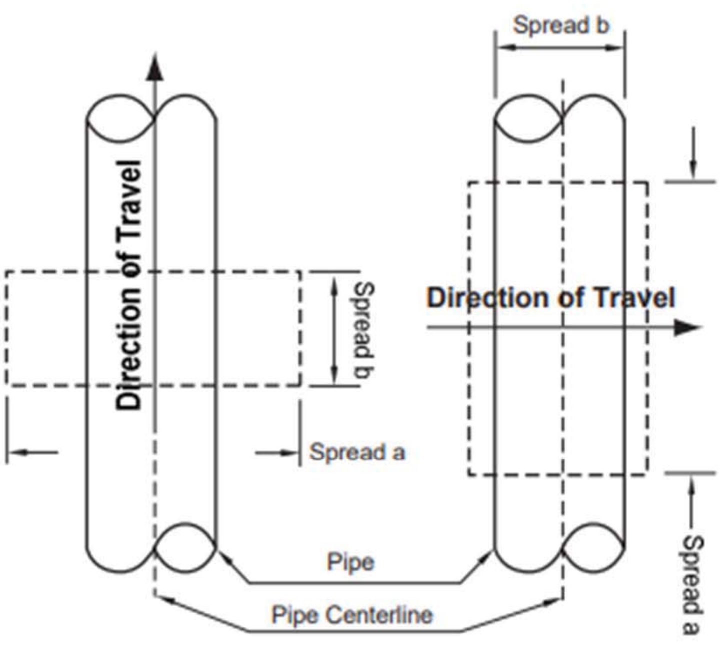
Select Granular Soil Fill

H, ft	P, lbs	Spread a, ft	Spread b, ft	Illustration
H < 2.03	16,000	a + 1.15H	b + 1.15H	4.13
2.03 ≤ H < 2.76	32,000	a + 4 + 1.15H	b + 4 + 1.15H	4.14
2.76 ≤ H	50,000	a + 4 + 1.15H	b + 4 + 1.15H	4.15

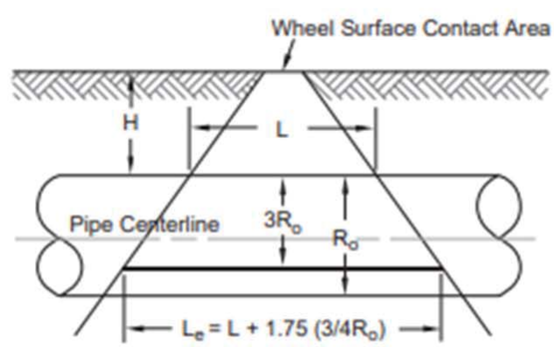
Other Soils

H, ft	P, lbs	Spread a, ft	Spread b, ft	Illustration
H < 2.33	16,000	a + 1.00H	b + 1.00H	4.13
2.33 ≤ H < 3.17	32,000	a + 4 + 1.00H	b + 4 + 1.00H	4.14
3.17 ≤ H	50,000	a + 4 + 1.00H	b + 4 + 1.00H	4.15

Spread Load Area Dimensions vs Direction of Truck



Effective Supporting Length of Pipe



Source: American Concrete Pipe Association, 2001, Design Data 1, "Highway Live Loads on Concrete Pipe".

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APPENDIX C

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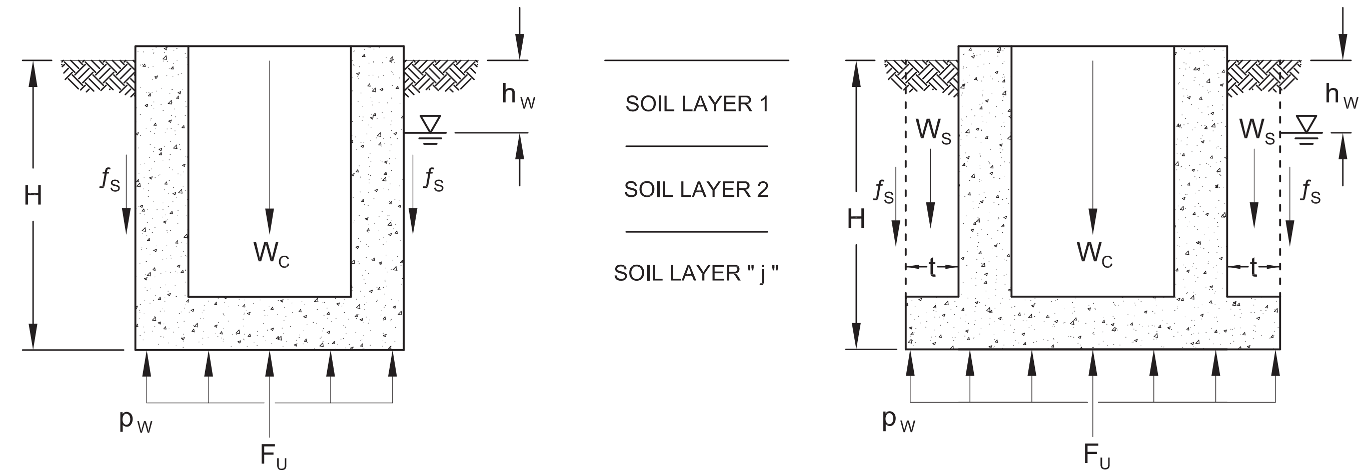
BUOYANT UPLIFT RESISTANCE FOR BURIED STRUCTURES

(a) WALL / SOIL FRICTION PLUS STRUCTURAL WEIGHT

(b) SOIL WEIGHT ABOVE BASE EXTENSION



(a)	(b)
cohesive soils: $f_{S_j} = \alpha\, c_j \leqq 3{,}000$ psf	cohesive soils: $f_{S_j} = c_j \leqq 3{,}000$ psf
cohesionless soils: $f_{S_j} = 0.75\, K_S\, \sigma_{V_j} \tan\delta_j$	cohesionless soils: $f_{S_j} = 0.75\, K_S\, \sigma_{V_j} \tan\Phi_j$
$Q_S = P_S \sum f_{S_j} h_j$	$Q_S = P_S \sum f_{S_j} h_j$
$\dfrac{W_C}{S_{f_a}} + \dfrac{Q_S}{S_{f_b}} \geqq F_U$	$\dfrac{W_C}{S_{f_a}} + \dfrac{Q_S}{S_{f_b}} + \dfrac{W_S}{S_{f_c}} \geqq F_U$

Where:

- A_B = area of base, sq. ft.
- H = buried height of structure, ft.
- h_W = depth to water table, ft.
- p_W = $\gamma_W (H - h_W)$, unit hydrostatic uplift, psf.
- γ_W = 62.4 pcf, unit weight of water
- F_U = $p_W A_B$, hydrostatic uplift force, lbs.
- f_{S_j} = unit frictional resistance of soil layer " j ", psf.
- c_j = undrained cohesion of soil layer " j ", psf.
- α = 0.55, cohesion factor between soil and structure wall
- σ_{V_j} = effective overburden pressure at midpoint of soil layer " j ", psf.
- δ_j = 0.75 Φ_j, friction angle between soil layer " j " and concrete wall, degrees
- Φ_j = internal angle of friction of soil layer " j ", degrees
- K_S = 0.4, coefficient of lateral pressure
- h_j = thickness of soil layer " j ", ft.
- j = 1, 2, ...
- P_S = perimeter of structure base, ft.
- Q_S = ultimate skin friction, lbs.
- W_C = weight of structure, lbs.
- W_S = weight of backfill above base extension, lbs.
- S_{f_a} = 1.1, factor of safety for dead weight of structure
- S_{f_b} = 3.0, factor of safety for soil / structure friction
- S_{f_c} = 1.5, factor of safety for soil weight above base extension
- t = width of base extension, ft.

NOTE: neglect f_S in upper 5 feet for expansive clay with a plasticity index > 20.

Reference:

1) *American Concrete Pipe Association, (1996), Manhole Floatation*

2) *O'Neill, M.W., and Reese, L.C., (1999), "Drilled Shafts: Construction Procedures and Design Methods", FHWA-IF-99-025*

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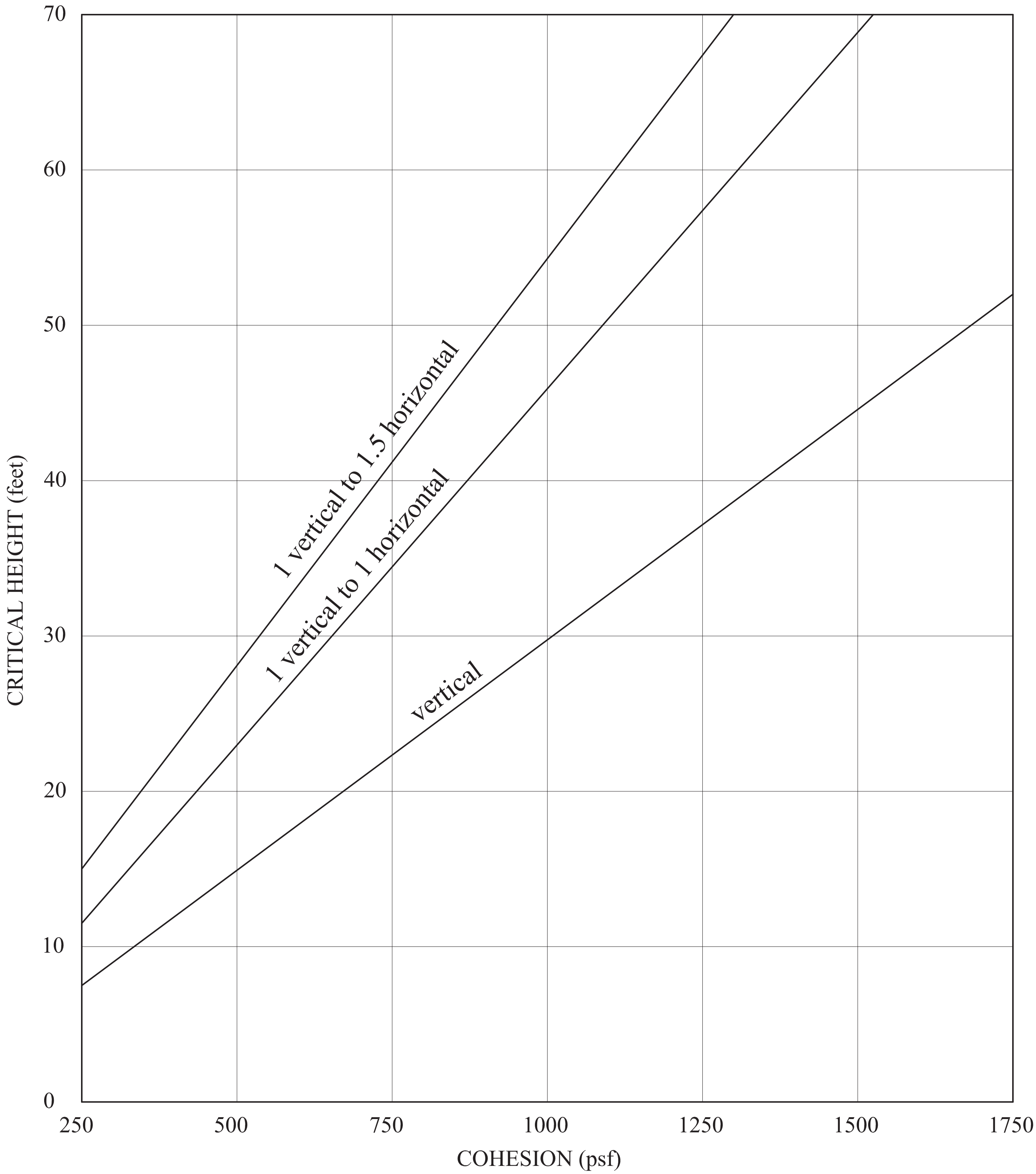
Critical Heights of Cut Slopes in Nonfissured Clays



Note: The charts are calculated based on NAVFAC DM7.1, Page 7.1-319, assuming the critical circles are toe circles, and wet unit weight of soils = 125pcf.

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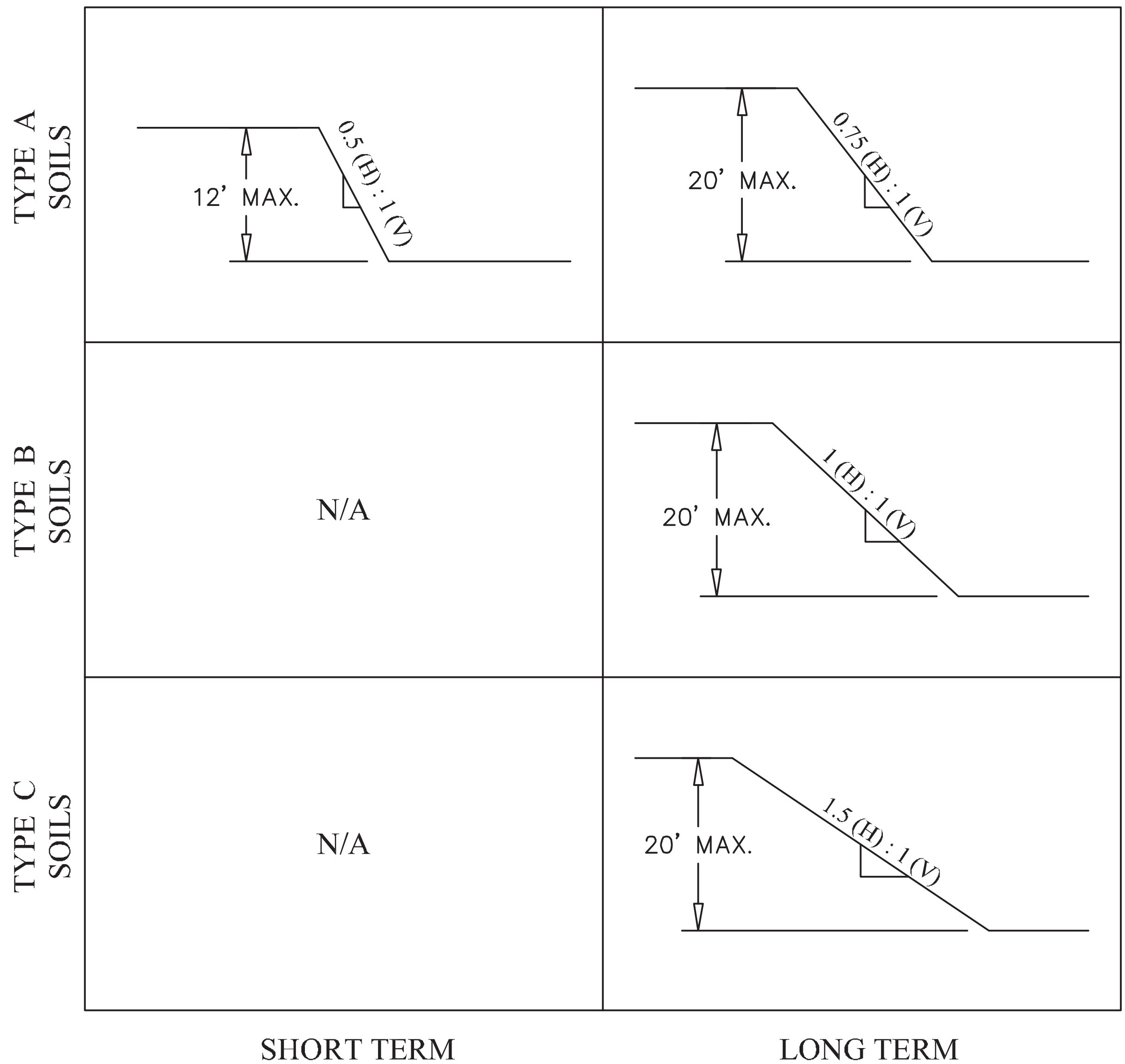
MAXIMUM ALLOWABLE SLOPES



NOTES:

(1) For Type A soils, a short term maximum allowable slope of 0.5 (H) : 1 (V) is allowed in excavations that are 12 feet or less in depth; short term (24 hours or less) maximum allowable slopes for excavations greater than 12 feet in depth shall be 0.75 (H) : 1 (V).

(2) Maximum depth for above slopes is 20 feet. For slopes deeper than 20 feet, trench protection should be designed by the Contractor's professional engineer.

(3) When surcharge loads from stored material or equipment, operating equipment, or traffic are present, a competent person shall determine the degree to which the actual slope must be reduced below the maximum allowable slope.

Reference: OSHA, Safety and Health Regulations for Construction, 1926 Subpart P.

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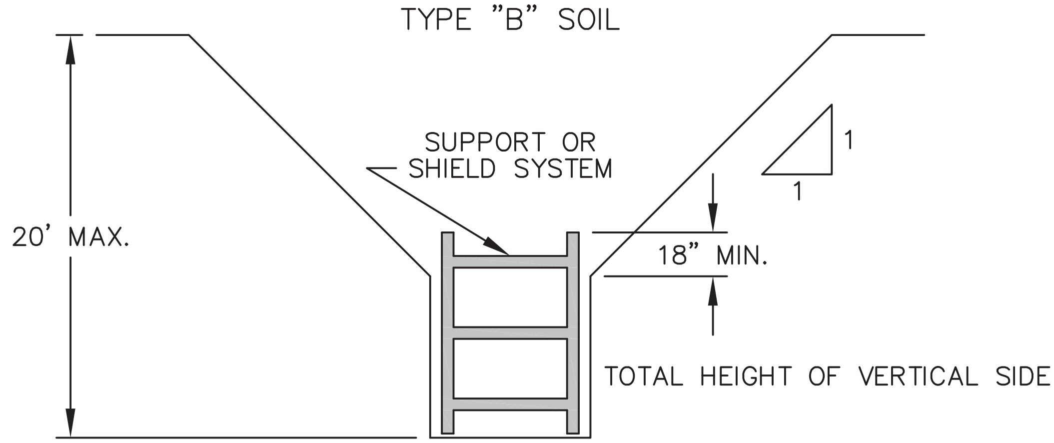
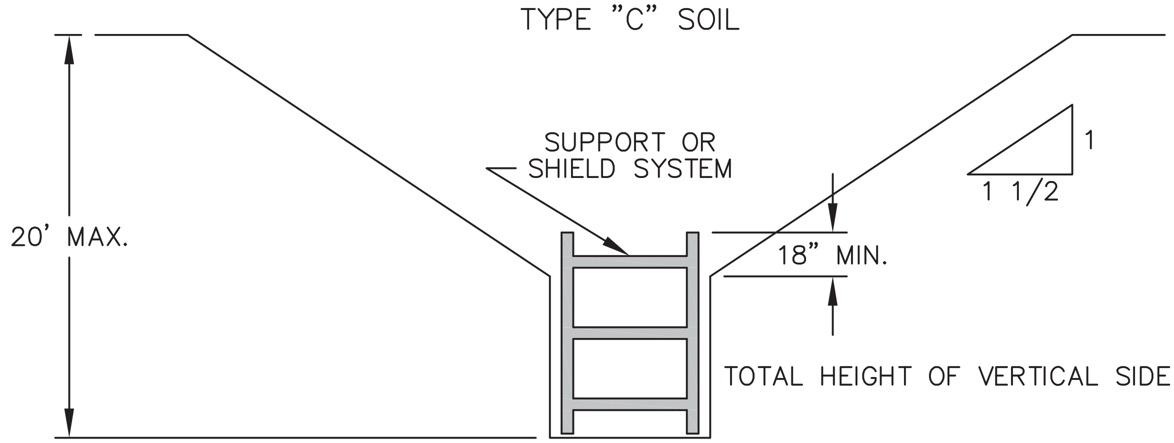
A COMBINATION OF BRACING AND OPEN CUTS

TYPE "B" SOIL

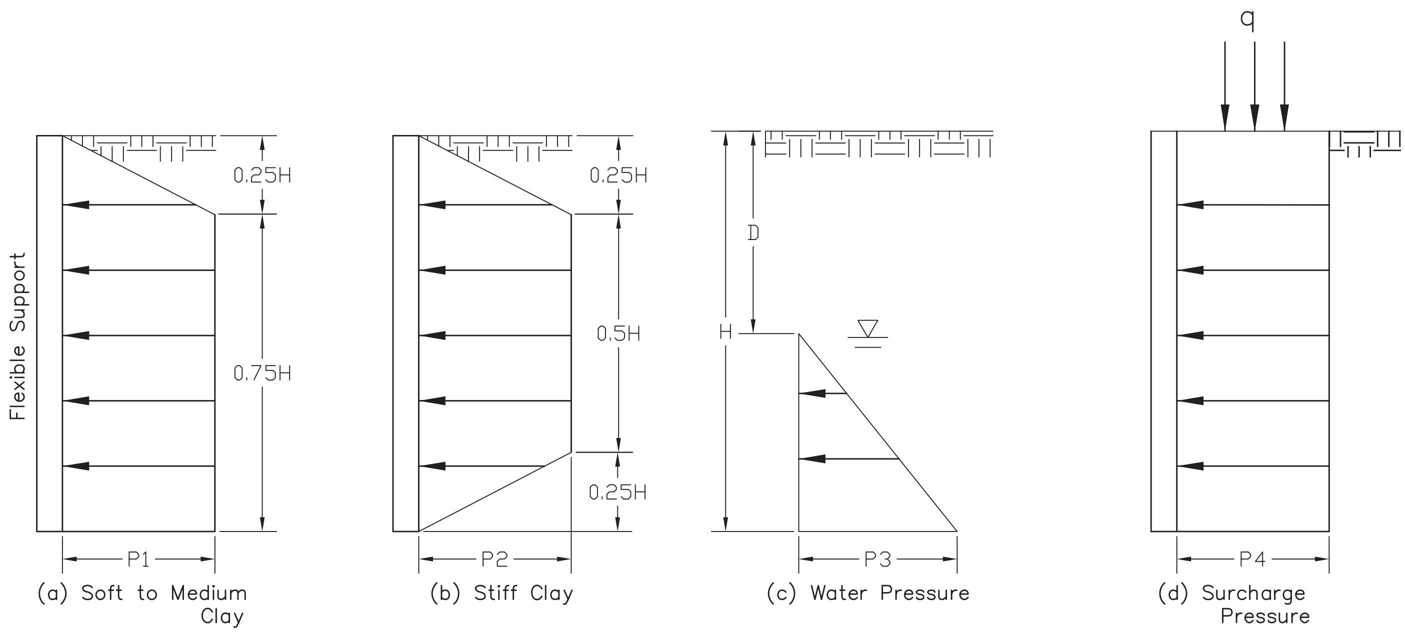


TYPE "C" SOIL



Reference: OSHA, Safety and Health Regulations for Construction, 1926 Subpart P.

LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN COHESIVE SOIL - LONG TERM CONDITIONS



Empirical Pressure Distributions

Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = γH-4C, psf
P2 = Lateral earth pressure = 0.4γH, psf
P3 = Water pressure = γ_w (H-D), psf
P4 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
C = Drained shear strength or cohesion, psf
K_a = Coefficient of active earth pressure

Notes:
1. All pressures are additive.
2. No safety factors are included.
3. For use only during long term construction.
4. If γH/C < 4, use section (b),
If 4 < γH/C < 6, use larger of section (a) or (b),
If γH/C > 6, use section (a).

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

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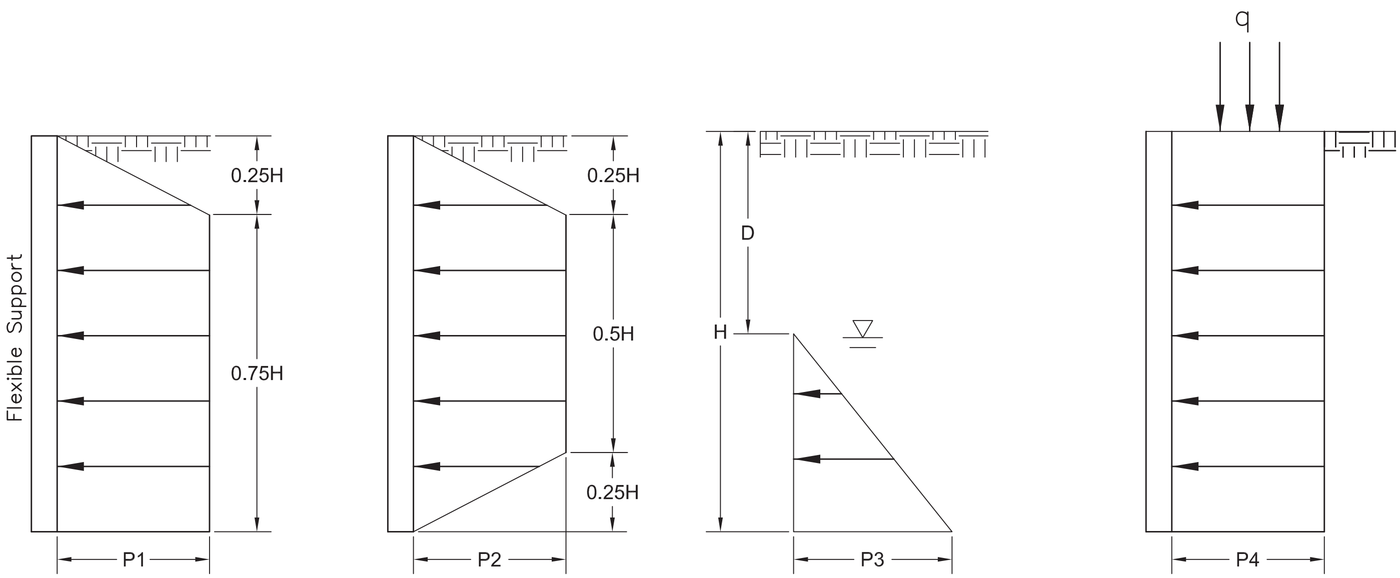
LATERAL PRESSURE DIAGRAMS

FOR OPEN CUTS IN COHESIVE SOIL - SHORT TERM CONDITIONS



(a) Soft to Medium Clay (b) Stiff Clay (c) Water Pressure (d) Surcharge Pressure

Empirical Pressure Distributions

Where:

H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $\gamma H-4S_u$, psf
P2 = Lateral earth pressure = $0.2\gamma H$, psf
P3 = Water pressure = γ_w (H-D), psf
P4 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
S_u = Undrained shear strength = $q_u/2$, psf
q_u = Unconfined compressive strength, psf
K_a = Coefficient of active earth pressure

Notes:

1. All pressures are additive.
2. No safety factors are included.
3. For use only during short term construction.
4. If $\gamma H/S_u < 4$, use section (b),
   If $4 < \gamma H/S_u < 6$, use larger of section (a) or (b),
   If $\gamma H/S_u > 6$, use section (a).

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

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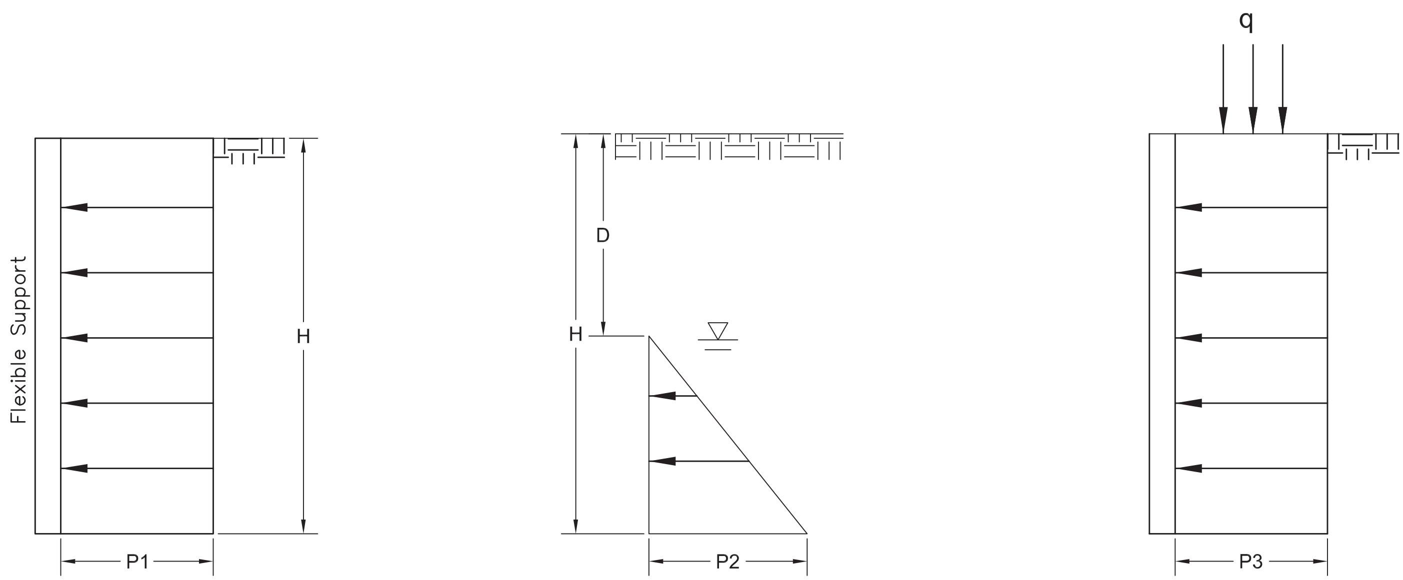
LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN SAND



(a) Sand   (b) Water Pressure   (c) Surcharge Pressure

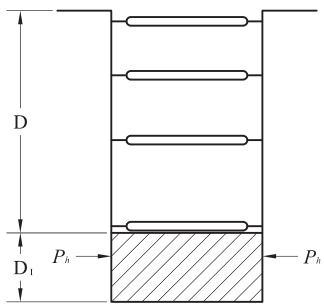
Empirical Pressure Distributions

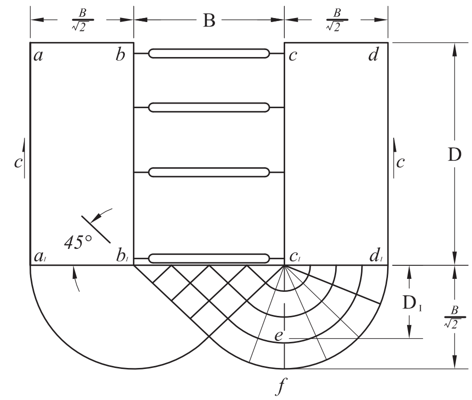
Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $0.65 * \gamma H K_a$, psf
P2 = Water pressure = γ_w (H-D), psf
P3 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
K_a = Coefficient of active earth pressure = (1-sinφ)/(1+sinφ)
φ = Drained friction angle

Notes:
1. All pressures are additive.
2. No safety factors are included.

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

BOTTOM STABILITY FOR BRACED EXCAVATION IN CLAY





Factor of Safety against bottom of heave,

$$F.S = \frac{NcC}{(\gamma D+q)}$$

where, Nc = Coefficient depending on the dimension of the excavation (see Figure at the bottom)
C = Undrained shear strength of soil in zone immediately around the bottom of the excavation,
γ = Unit weight of soil,
D = Depth of excavation,
q = Surface surcharge.

If F.S < 1.5, sheeting should be extended further down to achieve stability

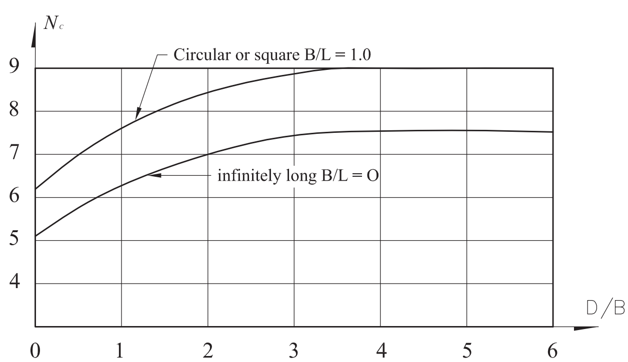
$$\textit{Depth of Buried Length, } (D_1) = \frac{1.5(\gamma D+q)-NcC}{(C/B)-0.5\gamma} \; ; D_1 \geqslant 5 \textit{ ft.}$$

Pressure on buried length, P_h:

For $D_1 < 0.47B$; $P_h = 1.5\, D_1(\gamma D - 1.4\, CD/B - 3.14C)$

For $D_1 > 0.47B$; $P_h = 0.7\, (\gamma DB - 1.4\, CD - 3.14CB)$

where; B = width of excavation



N_c rectangular = $(0.84 + 0.16B/L)N_c$ square

Reference: Bjerrum, L. and Eide, O., Stability of Strutted Excavations in Clay, Geotechnique, 6, 32-47 (1956).

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ILLUSTRATIONS

Photo 01 - Boring B-82 Pavement Core

Photo 02 - Boring B-83 Pavement Core