United States Department of Justice

Engineering Evaluation of Bollard Fence

Rio Grande, Hidalgo County, Texas

United States of America v. Fisher Sand and Gravel Company, Civil Action No. 7:19-CV-403

August 2021

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Acronyms and Abbreviations

1D	one-dimensional
2D	two-dimensional
ACI	American Concrete Institute
Arcadis	Arcadis U.S., Inc.
ASCE	American Society of Civil Engineers
ASTM	ASTM International
C	cohesion
CBR	California Bearing Ratio
cfs	cubic feet per second
Defendants	Fisher Sand and Gravel Company and related entities
DHS	U.S. Department of Homeland Security
DOJ	U.S. Department of Justice
EM	Engineer Manual
FEMA	Federal Emergency Management Agency
Fisher	Fisher Sand and Gravel Company
FOS	factor of safety
fps	feet per second
GIS	geographic information system
H:V	horizontal to vertical
HEC	USACE Hydrologic Engineering Center
HEC-RAS	HEC River Analysis System
Hydrograph	Graph of river flow or stage versus time (time series)
IBC	International Building Code
IBWC	International Boundary and Water Commission
Ka	coefficient of active earth pressure
Kp	coefficient of passive earth pressure
L&G	L&G Engineering Laboratory
lb/ft	pounds per foot
LL	liquid limit
mV	millivolts
NDT	non-destructive testing
NRCS	USDA Natural Resources Conservation Service
ohm-cm	ohm-centimeters
pcf	pounds per cubic foot

Phi or φ	internal angle of friction
PI	plasticity index
PL	plastic limit
ppm	parts per million
psf	pounds per square foot
psi	pounds per square inch
SEI	Structural Engineering Institute
Terrane	Terrane Engineering Corporation
TGR	TGR Construction, Inc.
U.S.	United States
USACE	U.S. Army Corps of Engineers
USCS	Unified Soil Classification System
USDA	U.S. Department of Agriculture
USGS	U.S. Geological Survey
USIBWC	U.S. International Boundary and Water Commission
WSEL	water surface elevation

Executive Summary

Fisher Sand and Gravel Company (Fisher) and related entities (Defendants) constructed a 3-mile-long bollard fence on the Texas bank of the Rio Grande, extending from 3.4 to 6.4 miles upstream of Anzalduas Dam near Mission, Texas. The dam is owned by the International Boundary and Water Commission (IBWC) and is operated for diversion of the United States (U.S.) share of Rio Grande floodwaters to an interior floodway on the U.S. side of the border and for regulated diversions during periods of normal flows to Mexico's main irrigation canal.

The fence, constructed in 2019 and 2020, consists of 6-inch by 6-inch square tube steel bollards raised to a height of 18 feet above ground, with 5 inches of open space between bollards. The United States filed suit to enjoin the construction of the bollard fence due to potential obstruction and deflection of river flow in violation of the 1970 Boundary Treaty between the United States and Mexico. This report documents methods, models, data, and assumptions contributing to findings of the impacts of the fence on river and floodplain hydrodynamics simulated to occur during the IBWC-designated design flood, derived based on recorded flow at Rio Grande City during Hurricane Beulah in September 1967. The results of the hydraulic model were applied to assess the fitness of use of the fence from geotechnical and structural engineering perspectives.

The most important findings of these investigations are subsequently summarized.

Hydraulic assessment: Contrary to opinions expressed in the Defendants' hydraulic model report, the model developed by Arcadis U.S., Inc. (Arcadis) shows that the bollard fence greatly alters the hydrodynamics of flow in the adjacent river and floodplains from their natural (pre-fence construction) state in the following ways:

- The fence significantly impedes movement of water between the river and the floodplain behind the fence. The reduction in floodplain conveyance capacity and storage causes flow in the river along the fence to increase by up to 27 percent from pre-project conditions. Increased river flow signifies flow deflection and increased potential for migration of the river channel and the U.S.-Mexico boundary during high-flow events. The IBWC-designated measures of deflection based on changes in maximum flow and maximum water surface elevations on the U.S. and Mexico sides of the river-floodplain system also strongly indicate deflection toward the Mexico side.
- The loss in floodplain conveyance capacity due to the fence causes reductions in freeboard along the Mission Levee to the north of the fence by up to 0.29 feet.
- Arcadis model results also show reduced conveyance from west to east within the model domain, creating a large ineffective ponding area behind the fence, head differentials between the river and the floodplain on either side of the fence, and high flow velocities (in excess of 7 feet per second) through the fence openings. High-velocity flows through the fence in combination with expected near-submergence of the bollards during design flood conditions contribute to structural loading of the bollards and to potential scouring of the base of the fence.

Geotechnical assessment: Overall conclusions relative to the fitness for use of the Fisher bollard fence are summarized as follows:

- The fence was constructed on a continuous, shallow reinforced concrete footing after clearing vegetation from the site. Site soil comprises mixtures of clay, silt, and sand. Up to about 3 feet of native material was used as fill at various locations. Where tested, the fill generally does not meet International Building Code (IBC) compaction standards.
- The foundation for the Fisher fence extends to a depth of 3 feet 2 inches below finished grade, compared to foundation depths for three other fences in Texas ranging between 10 feet and 10 feet 9 inches. Because the

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foundation was constructed at the ground surface with no burial, it is unlikely to be capable of carrying service loads during floods on the Rio Grande with expected hydrostatic and hydrodynamic loads, and impact loads from floating debris. Consequently, the foundation system is likely not fit for use under all reasonably anticipated service loads.

- The location of the fence near the riverbank and the presence of erodible soils require that the fence be protected from wind and water erosion. Without adequate protection, satisfactory performance of the fence over the long-term is questionable and may create a situation where the fence is not fit for use.
- Dispersive soil is present at various locations along the fence alignment, which, unless removed or contained, could erode and compromise fence integrity and its fitness for use.

Structural assessment: Overall conclusions relative to the structural integrity and stability of the bollard fence are summarized as follows:

- The plans prepared by TGR Construction, Inc. and dated October 30, 2019 were not signed and sealed by a licensed professional engineer in the State of Texas, and do not include design criteria, concrete notes, reinforcing and structural steel notes, foundation notes, datum, benchmarks, items requiring structural observation and inspection, and other contents considered to meet industry standards.
- The minimum lap of 24 inches for shrinkage and temperature reinforcement does not meet the 31-inch requirement for a Class B splice, unless the lap is staggered to meet the requirements of American Concrete Institute (ACI) 318-14 Building Code Requirement for Structural Concrete, Section 25.5.2.1.
- The TGR Construction, Inc. stability analysis and design calculations account for dead loads, wind pressure, and earth pressure; however, hydrostatic, hydrodynamic, and floating debris loads that may result from an unusual event such as the IBWC design flood were entirely missing.
- At some locations where the edge of the foundation was exposed due to erosion, the thickness of the footing was less than the 1 foot shown on plans. This non-conformance has an adverse impact on the external and internal stabilities of the bollard fence. Similarly, any existing erosion would have an adverse impact on the passive resistance assumed by design and for the purpose of this assessment unless effectively mitigated.
- The structural engineering assessment of the external stability of the bollard fence system included the following loading conditions that could result from an unusual event such as the IBWC design flood:
  - Maximum flow velocity during rising water levels from the river side. For this loading condition, the fence does not meet sliding and location of resultant force criteria.
  - Maximum water surface elevation on both the river and land sides in the western and eastern portions, respectively, of the bollard fence. For these loading conditions, the fence not only does not meet sliding, flotation, location of resultant force, and bearing pressure criteria, but would effectively slide, overturn, and become buoyant.
- The structural engineering assessment of the light/camera monopole external stability included the following two loading conditions that may result from the design flood:
  - Maximum flow velocity during rising water levels from the river side. For this loading condition, the monopole does not meet sliding and location of resultant criteria, and it would effectively slide and/or overturn.
  - Maximum water surface elevation during rising water levels from the land side in the western segment of the bollard fence. For this loading condition, the monopole does not meet sliding and location of resultant criteria, and it would effectively slide and/or overturn.

In summary, the bollard fence restricts movement of water between the river and floodplain during large floods and in the case of the IBWC-designated design flood, increases volume of flow in the river channel by up to 27 percent in comparison to natural (pre-project) conditions. Increased flow in the river channel indicates deflection of the river and potential violation of the 1970 U.S.-Mexico Boundary Treaty. The hydrodynamic effects of the fence include increasing flow depths and flow velocities through the bollards, which, in combination with the geotechnical and structural deficiencies described in this report, indicate that the fence is likely not fit for use under all reasonably anticipated service loads.

1 Introduction

Fisher Sand and Gravel Company (Fisher) and related entities (Defendants) constructed a bollard fence along the Texas bank of the Rio Grande between Bentsen State Park and Anzalduas Park, south of Mission, Texas. An aerial view of the site showing the fence alignment is displayed in Figure 1.1. The fence is 2.96 miles in length and extends from 3.4 to 6.4 miles upstream of Anzalduas Dam. The dam is owned and operated by the International Boundary and Water Commission (IBWC) for diversion of the United States (U.S.) share of Rio Grande floodwaters to an interior floodway on the U.S. side of the border, and for regulated diversions of non-flood flow to Mexico's main irrigation canal.

The fence was constructed in 2019 and 2020 approximately 8 to 20 feet from the Rio Grande shoreline at normal water levels. It consists of 6-inch by 6-inch square tube steel bollards oriented at 45 degrees to the river channel and spaced at approximately 13.5 inches on center to a height of 18 feet above ground. A 20-foot-wide paved road and 30-foot-tall light poles with security cameras on 6-foot-tall, 3-foot-diameter, precast concrete foundations are placed approximately every 200 feet along the land (U.S.) side of the fence. The bollards are constructed of 1/8-inch-thick galvanized steel and embedded with 5 inches of open space into a reinforced concrete T-shaped footing as shown in Figure 1.2.

The United States filed suit to enjoin the construction of the bollard fence due to potential obstruction or deflection of river flow in violation of the 1970 Boundary Treaty between the United States and Mexico. The United States Department of Justice (DOJ) McAllen Division retained Arcadis U.S., Inc. (Arcadis) to:

- Analyze the impacts of the bollard fence on river and floodplain hydrodynamics upstream of Anzalduas Dam during a design flood event identified by IBWC.
- Evaluate the design and construction of the fence's foundation system and assess its fitness for use based on its anticipated performance during the design flood event. Fitness for use as referred to in this report means that the structure can be safely used for its intended purpose.
- Calculate the structural stability of the fence as designed and constructed based on construction materials, site conditions, and wind, hydrostatic, hydrodynamic, debris impact, and soil loads to which the fence would be subjected during the design flood event.
- Review government-furnished information and photographs, drawings, plans, data, models, and model reports prepared by TGR Construction, Inc. (TGR), a subsidiary of Fisher, related to the hydraulic, geotechnical, and structural assessments.
- Conduct a site visit and field inspection to determine fence materials and construction methods, as well as fence performance and maintenance performed since construction.

Jason Vazquez and John Sparks (Arcadis) completed a site visit on April 27, 2021, accompanied by Paxton Warner (DOJ) and Tommy Fisher (Fisher). During the site visit, Mr. Fisher described the fence materials and construction methods, as well as fence performance and maintenance conducted since construction. Arcadis documented site conditions with photographs. During the site visit, non-destructive testing (NDT) was conducted by a company under contract to Arcadis to measure the thickness of the steel bollard tubes, estimate the configuration of reinforcing steel, and measure the compressive strength of the concrete footing.

Plan sheets for fence construction prepared by TGR are provided in Appendix A. Geotechnical and structural field testing results (including NDT results) are provided in Appendices B through D. A site and subsurface investigation report is provided in Appendix B, and a site-specific soils report compiled using U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) information is provided in Appendix C.

Figure 1.1. Aerial imagery showing fence alignment (red line)

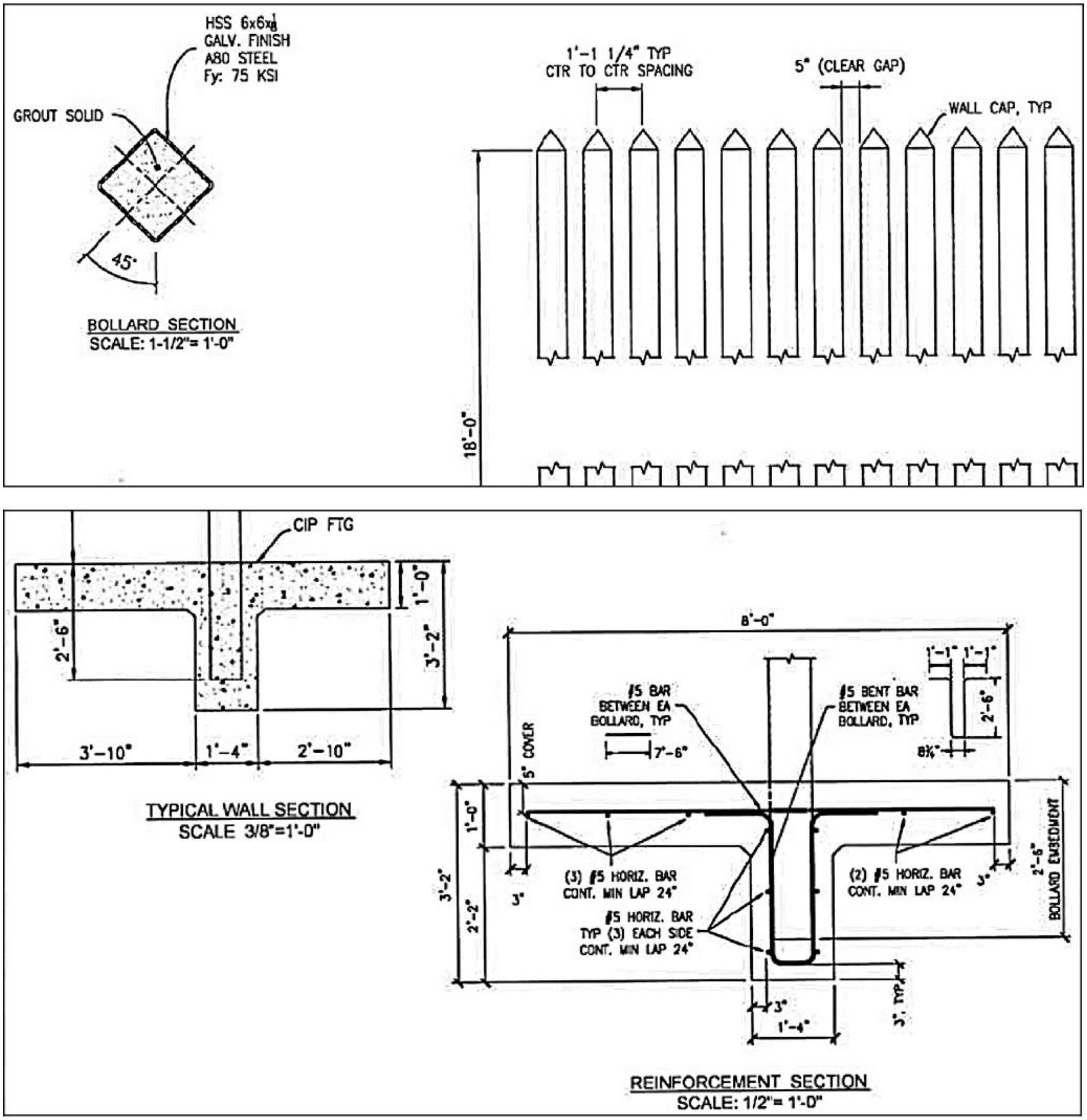

Figure 1.2. Construction details from plans by TGR, dated October 30, 2019

2 Site Conditions

Figures 2.1 through 2.3 display photographs of varying degrees of bank caving and surface erosion taken by IBWC by airboat on July 14, 2021. Figure 2.4 shows one area with well-established vegetation and relatively large fence setback with no observable erosion. The photographs were taken following cumulative precipitation totals of 4 to 10 inches measured in the McAllen/Mission area the previous week, mapped in Figure 2.5.

Figure 2.1. Bank erosion near fence, northeast side of fence

Figure 2.2. Surface erosion, southeast side of fence

Figure 2.3. Bank caving, northwest side of fence

Figure 2.4. Vegetation along eastern side of fence

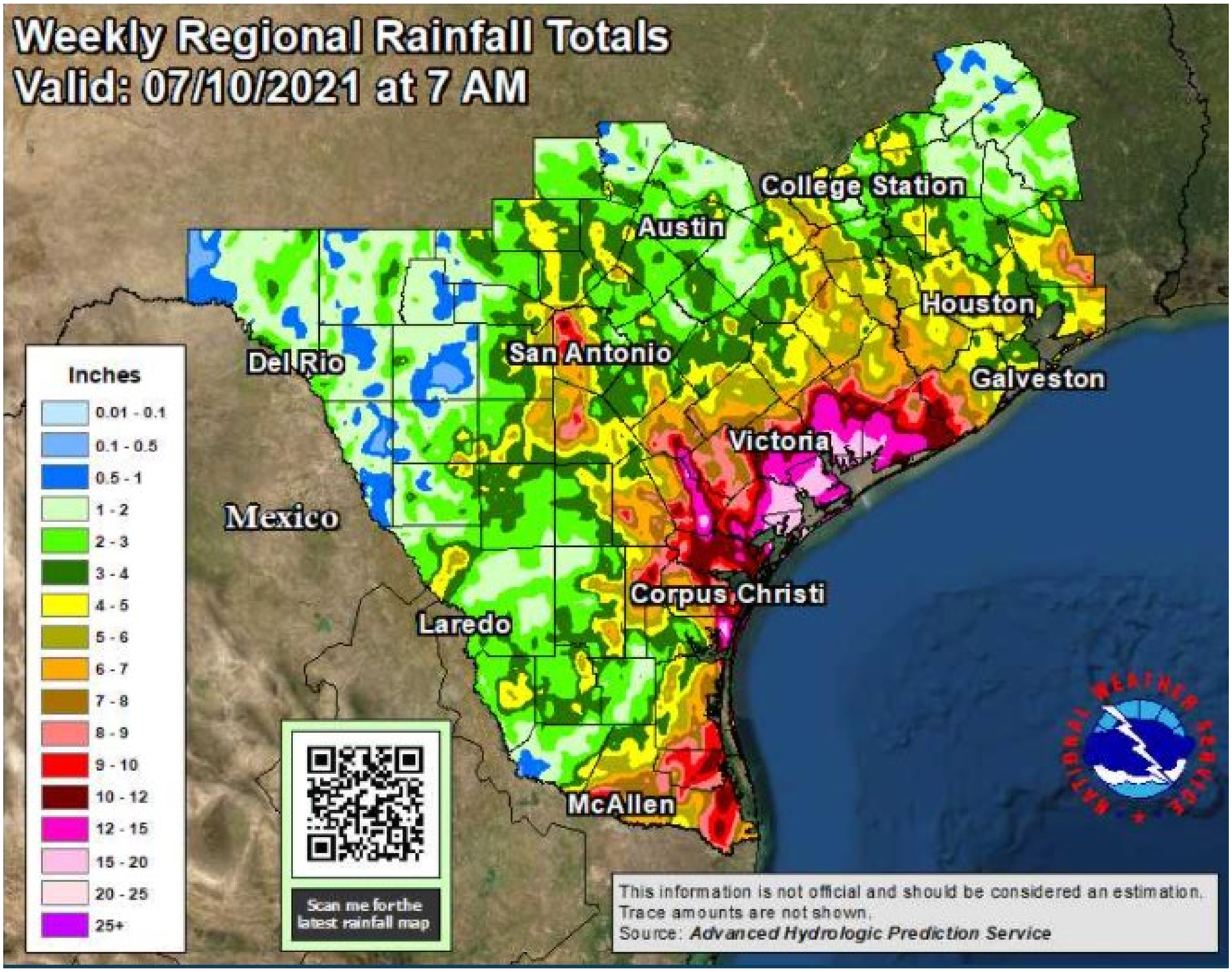

Figure 2.5. Weekly regional rainfall totals for week ending July 10, 2021 (National Oceanic and Atmospheric Administration Advanced Hydrologic Prediction Service)

3 Purpose and Scope

The U.S. Attorney's Office, Southern District of Texas, U.S. Department of Justice retained Arcadis U.S., Inc., to provide expert services for the United States of America v. Fisher Sand and Gravel Co., TGR Construction, and Neuhaus & Sons, LLC; Civil Action No. 7:19-CV-403. As part of these services, Arcadis conducted hydraulic, geotechnical, and structural engineering assessments of the impacts of the bollard fence system recently constructed on the Rio Grande near Mission, Texas.

The principal purpose of investigations authorized by DOJ is to determine whether the fence and related construction including earthworks, fill placement, and removal of vegetation from the riverbank could obstruct or deflect river flow due to its configuration and proximity to the riverbank, potentially leading to migration of the river channel and violation of the 1970 Boundary Treaty between the United States and Mexico. Additional objectives include determination of the fence's fitness for use considering the (1) anticipated performance of the fence's foundation system when flooded, and (2) structural stability of the fence under flood-induced hydrostatic, hydrodynamic, and debris impact loading, as well as wind and soil loading.

Hydraulic studies documented in this report include the following:

- Review of models, model studies, and reports prepared by TGR and assessment of their suitability for analysis of the hydraulic impacts of the bollard fence.
- Development and application of two-dimensional (2D) HEC-RAS (Hydrologic Engineering Center-River Analysis System) model(s) to determine impacts of the bollard fence on (1) flow conveyance and circulation between the river and floodplains within the model domain, (2) deflection of flow in the Rio Grande river channel by the fence, measured by volumetric change in river flow from natural (pre-project) conditions, and (3) turbulence, structural loading, and erosion potential created by the fence.

The scope of geotechnical investigations is as follows:

- Review of available site-specific geotechnical information, including materials furnished to DOJ by Fisher and published in geologic and soils reports.
- Review of various codes and standards for foundation design.
- Review of geotechnical reports and construction plans for other fence projects in Texas.
- Site reconnaissance, field testing, and laboratory testing of soil samples for (1) characterization of site soils and foundation conditions, and (2) determination of soil properties for use in geotechnical engineering and design.
- Evaluation of the fence foundation relative to long-term fence stability and soil erodibility.

The following structural evaluations are described in this report:

- Assessment of government-provided information.
- Site visit and field testing.
- Derivation of structural analysis parameters from hydraulic and geotechnical engineering assessments.
- Structural analysis of bollard fence system, including (1) external stability assessment and (2) internal (strength) stability assessment.

This report documents the findings and expert opinions associated with the hydraulic, geotechnical, and structural engineering assessments.

4 Hydraulic Assessment of Bollard Fence

Rio Grande, Hidalgo County, Texas

August 2021

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4.1 Hydraulic assessment – summary

Potential hydraulic impacts of the fence include turbulence, vorticity (rotation), and changes in circulation induced by river flow against the bollards, through the openings between the bollards, or against debris lodged in the bollards. Hydraulic impacts may be localized or cumulatively interfere with the normal filling and emptying of floodplain storage behind Anzalduas Dam during flood operations. Hydraulic forces on the fence and erosion of the riverbank at the base of the fence due to turbulence, grading, and removal of vegetation could also affect the structural stability of the fence. The United States has alleged that the combination of hydraulic, erosion, and structural problems caused or exacerbated by the fence could potentially cause the riverbank to erode or the river channel to migrate, in violation of the 1970 Boundary Treaty.

The principal findings of the hydraulic assessment are summarized as follows:

- The model developed by TGR for the Defendants does not realistically simulate the hydrodynamics or distribution of flow in the river and floodplain behind the fence, and consequently is not well-suited to analysis of (1) deflection of Rio Grande river flow, (2) potential for erosion of the riverbank and foundation of the bollard fence, or (3) hydraulic forces and moments acting on the bollard fence.
- By distorting the horizontal dimensions of the bollard fence by a factor of 12, the 2D TGR model does not preserve hydraulic similitude, i.e., accurate relationship between model and prototype. For any given river flow depth and velocity, flow and turbulence around 6-foot bollards spaced 5 feet apart are not similar to flow around 6-inch bollards with 5-inch openings. During the design flood, the TGR model simulates large (10 to 20 feet in diameter) whirls and eddies along the riverbank along the fence, and velocities up to 10 feet per second (fps) around the downstream terminus of the fence. The Arcadis model with the fence at prototype scale does not reproduce these conditions.
- The manner in which the bollards and fence openings are represented in the TGR terrain model does not reflect 30 percent blockage by debris as stipulated in IBWC guidelines. In the TGR model, flow is not actually blocked but is instead redirected around the bollards by virtual cylindrical piers placed several feet in front of or behind the bollards at different locations. With 30 percent of the bollard fence openings blocked, less than 26 percent of the total fence length – less than 0.8 of 3 miles – is open to flow through the fence. The fence as configured in the TGR model, however, appears to have 37 percent open area – more than 40 percent larger than specified by IBWC criteria. Moreover, plant and woody debris would more likely obstruct three out of 10 openings at prototype scale than reduce each 5-inch opening by 30 percent (1.5 inches) represented in distorted scale by the cylindrical piers in the TGR model.
- The Arcadis hydraulic model indicates that the fence significantly impedes movement of water between the river and the floodplain behind the fence, creating as a result a large ineffective or ponding area behind the fence. This reduction in floodplain storage increases cumulative flow in the river along the fence by up to 27 percent from pre-project conditions. Higher river flow and flow velocity indicate flow deflection and increased potential for migration of the river channel during high-flow events. The IBWC-designated measures of deflection based on changes in maximum water surface elevation (WSEL) and peak flow across the river-floodplain system also confirm flow deflection from the U.S. to the Mexico side of the river channel.
- The fence causes reductions in freeboard along the Mission Levee to the north of the fence by up to 0.29 feet.
- The Arcadis model indicates that maximum velocity of flow from the river through unblocked bollards on the western portion of the fence reaches 7.9 fps, and exceeds 5 fps from the floodplain to the river on the eastern

portion of the fence. These results contribute to structural loading of the bollards and indicate increased potential for scouring of the base of the fence in comparison to natural (pre-project) conditions.

- Because the bollard fence is mostly submerged at the peak of the flood and produces head differentials of up to 0.25 feet across the fence, hydraulic loading on individual bollards could affect structural stability.
- Since construction of the fence, large areas of vertical sloughing and caving of the bank have been observed, some of which are documented by photographs subsequently presented in this report. With average daily river stage fluctuations of 0.5 feet upstream of Anzalduas Dam and wakes generated by frequent high-speed river patrol boats, the raising and steepening of the bank and removal of natural vegetation in construction of the fence may have contributed to vertical caving and reduced bank stability. In addition, boat wakes and wind-generated waves could add to structural loading of the fence due to hydraulic head and velocity through the bollards. Calculation of effects of river level fluctuations, vessel, or wind-generated waves on foundation erosion, bank erosion, or structural stability of the fence was outside the scope of this investigation.

The Arcadis hydraulic model was developed using a more recent version of the software than used in development of the TGR model. The improvements incorporated in the newer version enabled undistorted representation of the bollard fence at prototype scale in a variable 2D grid. The Arcadis modeling approach, data, assumptions, and results are described in detail in this report.

4.2 Hydraulic assessment – scope and objectives

The principal objectives of this study are as follows:

- Review of models, model studies, and reports prepared by TGR and assessment of their suitability for analysis of the effects of the bollard fence on (1) deflection of the Rio Grande and potential for migration of the river channel based on increases in flow and WSELs in excess of U.S. International Boundary and Water Commission (USIBWC) threshold limits, (2) potential for erosion of the riverbank and foundation of the bollard fence caused by the bollard fence and associated grading and removal of vegetation, and (3) simulation of flow depths and velocities on all sides of the bollards required for calculation of structural forces and moments.
- Development of fully 2D HEC-RAS model(s) for analysis of flow depth, flow velocity, and flow direction around the bollard fence, in the Rio Grande river channel adjacent to the fence, and in the floodplains on both sides of the river. The Arcadis model is designed to remedy the most serious deficiencies of the TGR model relative to these determinations.
- Application of Arcadis model simulation results for determination of (1) stages, flows, and velocities throughout the model domain during a large flood during which the river overflowed its banks, and (2) head differential and velocities acting on the bollard fence at various locations and times during the flood.
- Assessment of hydraulic impacts of the fence based on Arcadis model simulation results relative to river-floodplain circulation within the model domain, potential for flow deflection and migration of the Rio Grande river channel, erosion of the base of the fence, and forces and moments acting on the fence due to velocity against and around the open and obstructed bollards along the fence.

The scope of investigations documented in this report is as follows:

- Review of the hydrodynamic model and model report prepared for the Defendants by TGR.
- Development of an updated hydrodynamic model of the river-fence-floodplain system upstream of Anzalduas Dam; the updated model provides a more realistic representation of the bollard fence than the TGR model while using the same model domain, fence alignment, boundary conditions, pre- and post-project terrain, and roughness coefficients as the TGR model.
- Evaluation, based on the updated hydrodynamic model, of effects of the bollard fence on distribution of flow in the Rio Grande river and floodplain, flow velocities, turbulence, and WSELs along the fence during the September 19 to 23, 1967 IBWC-designated design flood.
- Development of hydraulic and hydrodynamic information required for geotechnical and structural assessment of the bollard fence.
- Quantitative and qualitative assessments of the effects of the bollard fence on circulation and river flow deflection within the model domain.

This report documents methods, data, assumptions, and findings of each of the above-listed investigations. The state of the river and floodplain prior to fence construction is referred to as the pre-project condition, and after fence construction as the post-project condition.

4.3 Review of TGR hydrodynamic model

TGR developed a hydraulic model using the HEC-RAS program, version 5.0.7. While model development is partially documented in a 2020 report (TGR 2020), important data and assumptions are not fully described, and the findings and conclusions are not fully supported by analysis results presented in the report.

4.3.1 TGR model description

The TGR HEC-RAS model domain, shown in Figure 4.1, extends approximately 9 river miles upstream of Anzalduas Dam. The model is fully two-dimensional with a variable grid covering the Rio Grande river and floodplains on the U.S. and Mexico sides of the river to the domain boundaries. Cell face lengths average 2 feet adjacent to the fence, increasing to 50 feet in the river and floodplains moving 150 feet from the fence. The model was developed using HEC-RAS version 5.0.7 (United States Army Corps of Engineers [USACE] 2019).

Pre- and post-project terrains are represented in the TGR model, and the associated simulations are labeled “Existing” and “Improved,” respectively. The pre-project terrain represents the natural ground and riverbank. The post-project terrain is characterized by a raised and steepened riverbank, creating a low levee on which the fence and a paved access road on the landward side of the fence were constructed. Both terrains are essentially identical upstream and downstream of the fence.

The post-project terrain also includes a 12:1 horizontally distorted bollard fence, constructed so that horizontal dimensions in inches map to the same number of feet, i.e., a 5-inch spacing between bollards becomes 5 feet, and 6-inch square bollards become 6-foot square bollards. The TGR model appears to interpret IBWC guidelines (IBWC undated) requiring 30 percent debris blockage as blockage of each bollard opening, although the model report does not explain how blockage was effected. Examination of the post-project model terrain shows virtual cylindrical piers approximately 4 feet in diameter centered about 5 feet in front of (river side) or behind (land side) each 5-foot fence opening, presumably to represent debris blockage although this is not confirmed in the text of the report. An enlarged view of the bollard and pier configurations represented in the TGR model is shown in Figure 4.2.

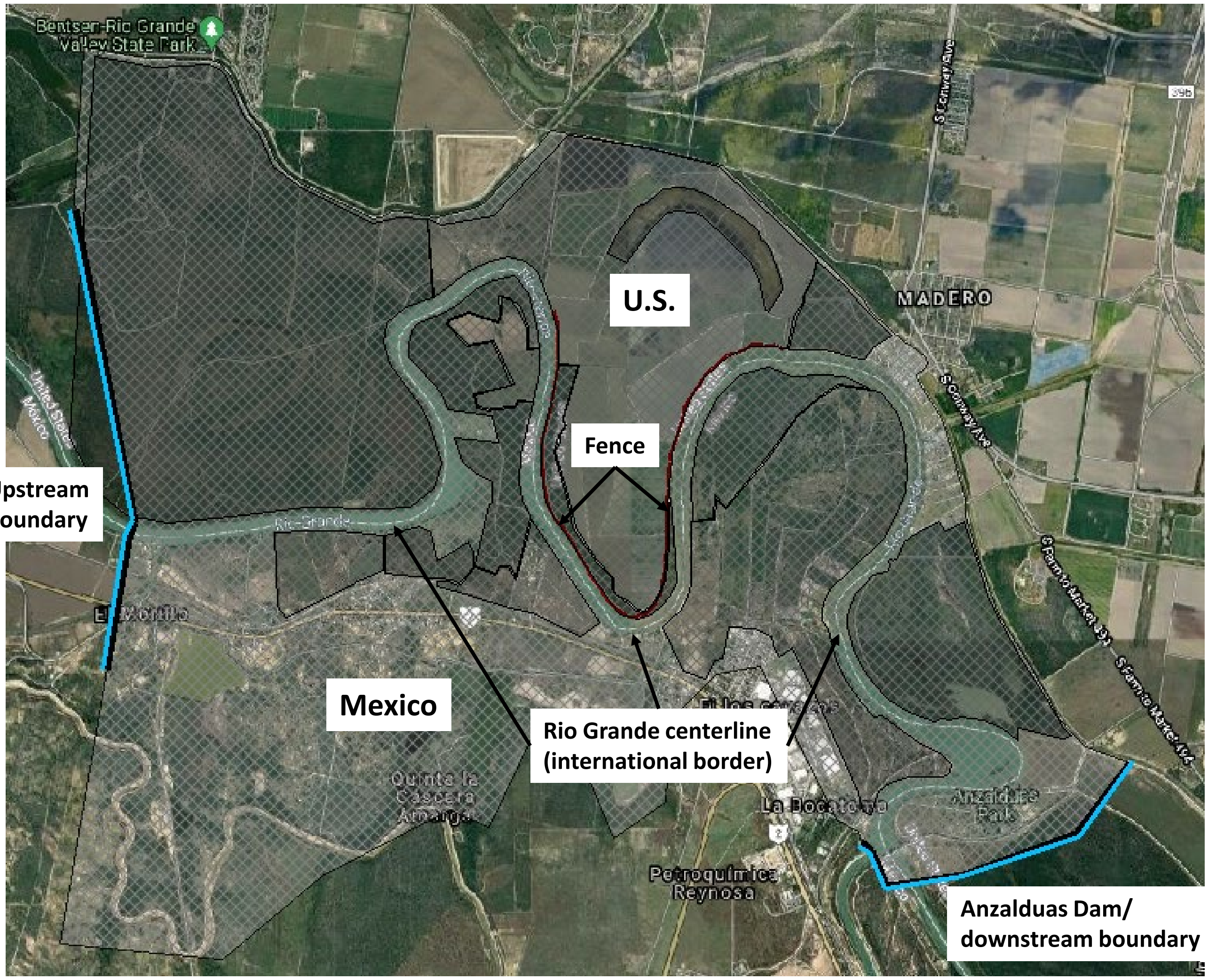

Figure 4.1. TGR HEC-RAS model domain

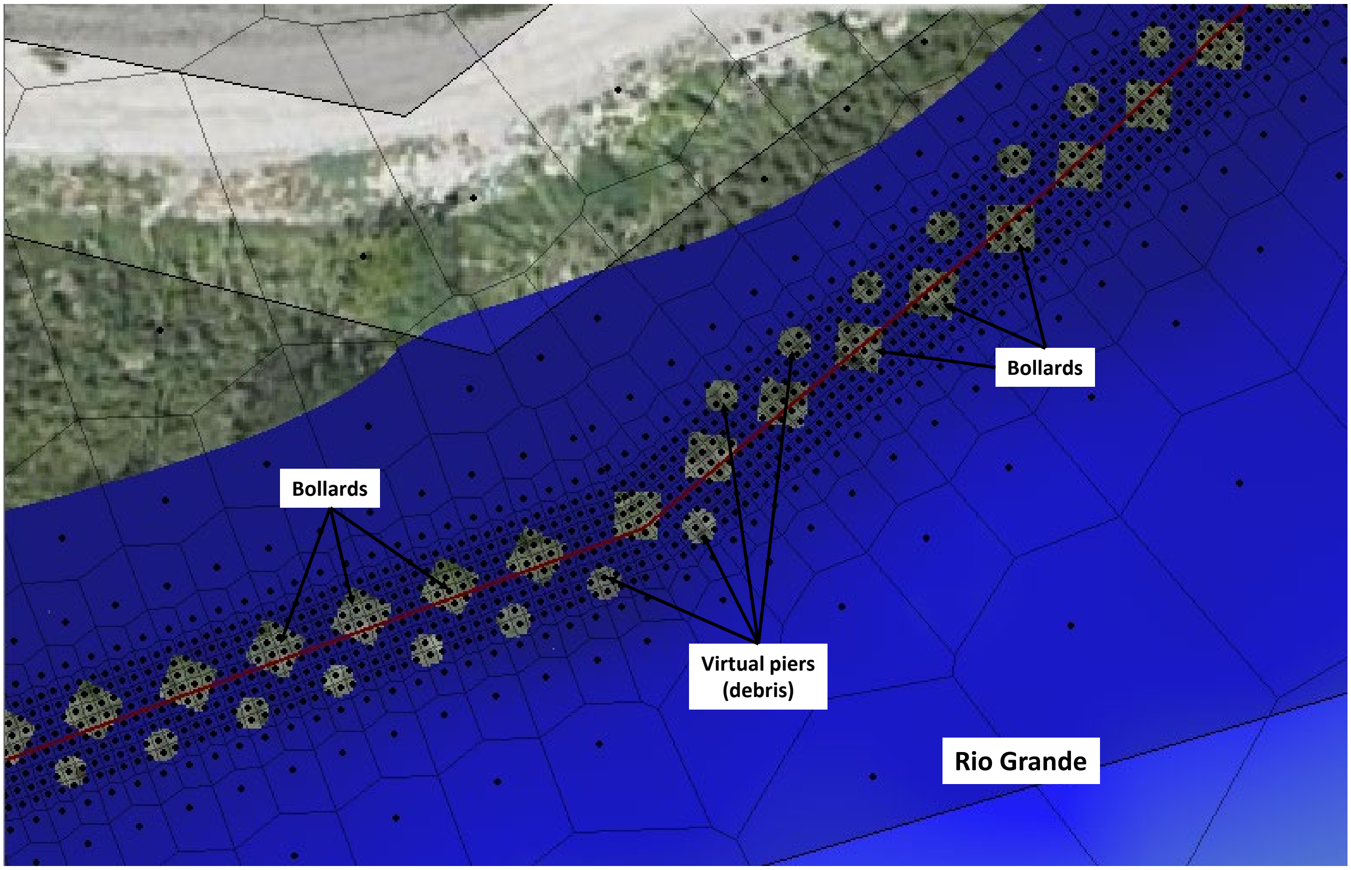

Figure 4.2. TGR bollard fence and virtual pier terrain (enlarged)

4.3.2 Boundary conditions

As prescribed by IBWC guidance, the upstream boundary condition to the TGR model consisted of the rising limb of the IBWC-designated design flood hydrograph. The recorded peak discharge of 220,000 cubic feet per second (cfs) at Rio Grande City during Hurricane Beulah in September 1967 was adjusted by IBWC to 250,000 cfs, and then reduced to the design peak inflow of 234,175 cfs at the upstream model boundary due to attenuation of the peak between Rio Grande City and Anzalduas Dam. The full design flood hydrograph is shown in Figure 4.3. The model simulation period extends from 00:00 on September 20, 1967, to 20:00 on September 23, 1967 – a total simulation time of 92 hours. The adjusted peak inflow of 234,175 cfs occurred at 12:00 on September 23, 1967.

A normal depth downstream boundary condition at Anzalduas Dam was defined in the TGR model.

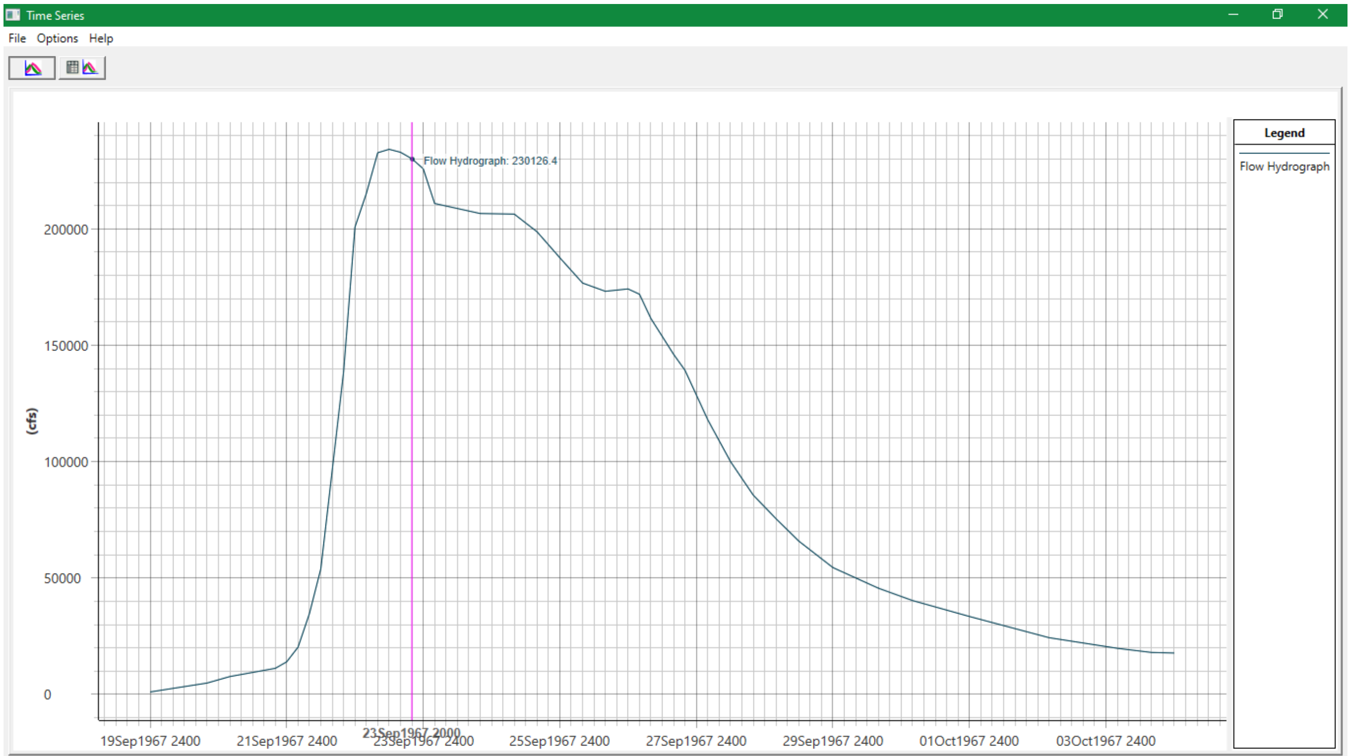

Figure 4.3. TGR HEC-RAS model discharge hydrograph upstream boundary condition

4.3.3 TGR model assessment

The TGR model is fully two-dimensional for both the pre- and post-project simulations, exceeding IBWC guidelines for one-dimensional (1D) or 1D/2D modeling of existing conditions. The post-project model further exceeds IBWC criteria for 2D analysis by superimposing the bollard fence (albeit at a 12:1 distorted horizontal scale) on the post-project terrain to represent the shape of the bollards more accurately than the typical gate shape simulated as weir flow in HEC-RAS version 5.0.7. Notwithstanding the added detail, the TGR report makes very limited use of 2D model features for display and analysis of simulation results, or for assessing the hydraulic impacts of the fence on river and floodplain hydrodynamics.

4.3.3.1 Model capabilities for analysis of hydraulic impacts of bollard fence

By distorting the horizontal dimensions of the bollard fence by a factor of 12, the 2D TGR model does not preserve hydraulic similitude. For any given river flow depth and velocity, flow and turbulence around 6-foot bollards spaced 5 feet apart are not similar to flow around 6-inch bollards with 5-inch openings. As shown in Figure 4.4, the TGR model simulates large (10 to 20 feet) whirls and eddies along the riverbank and fence, which ordinarily would not be expected to occur with river flow velocities between 2 and 4 fps. In addition, velocities of 8 to 10 fps are shown in Figure 4.5 at the downstream terminus of the fence where velocities would be expected to fall as the reservoir fills. Large-scale turbulence and high velocities simulated by the TGR model do not appear to accurately characterize river and floodplain hydrodynamics with flow velocities averaging less than 4 fps as simulated by the subsequently described Arcadis HEC-RAS model. The TGR report does not disclose high velocities and turbulence simulated by its model and does not propose or discuss mitigation measures.

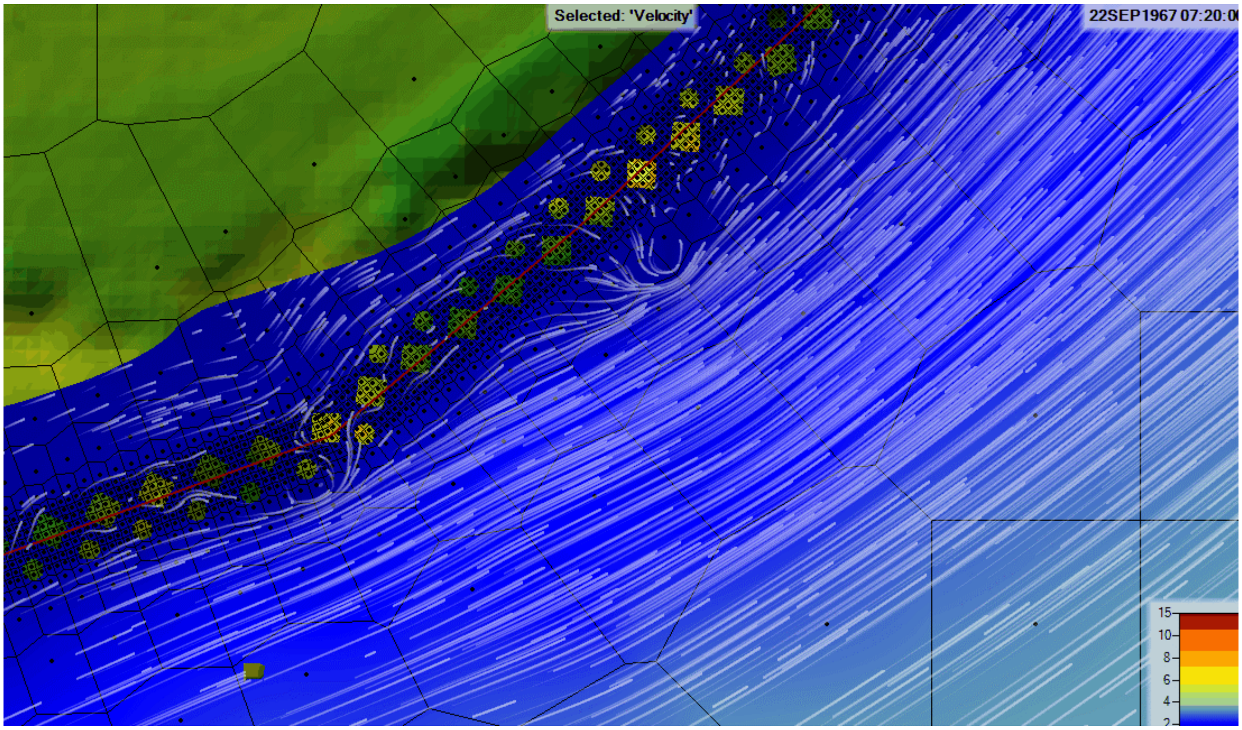

Figure 4.4. TGR HEC-RAS model-simulated whirls and eddies

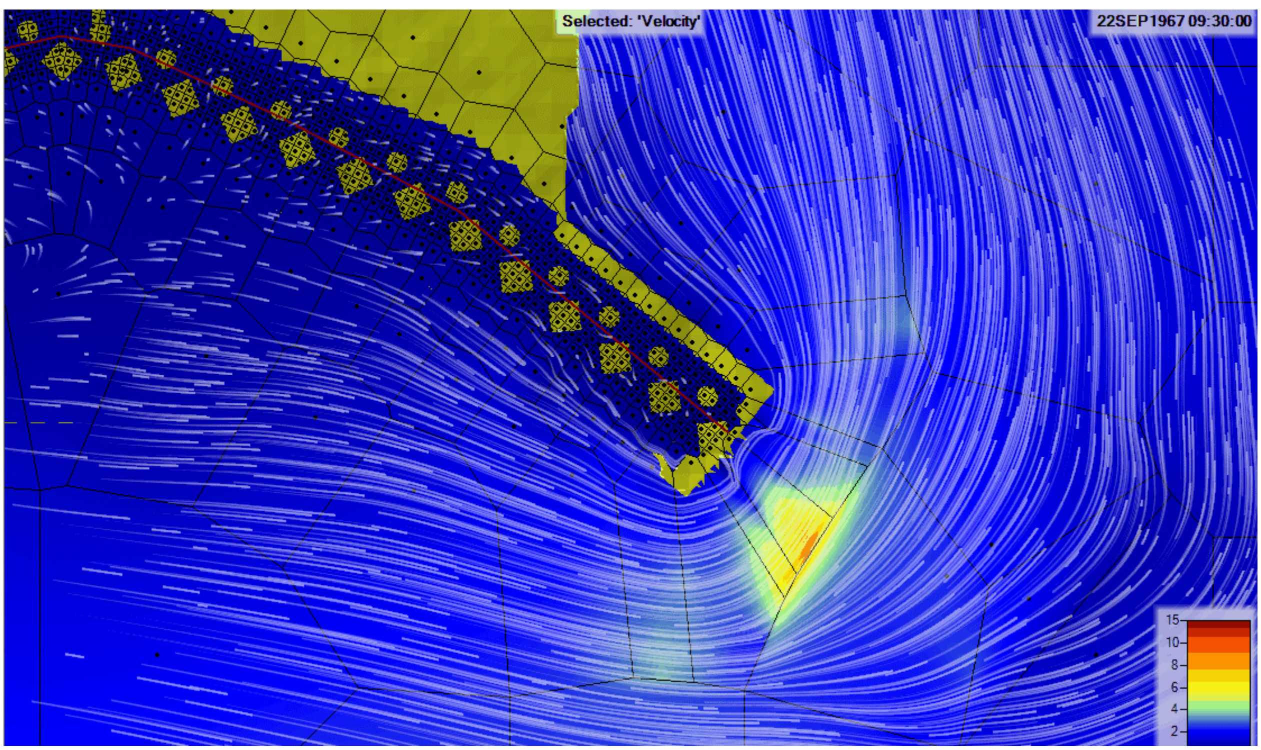

Figure 4.5. TGR HEC-RAS model-simulated velocity at downstream fence terminus

The manner in which the bollards and fence openings are represented in the TGR model does not reflect 30 percent blockage by debris as stipulated in IBWC guidelines. In the TGR model, flow is not actually blocked but is instead redirected around the bollards by virtual cylindrical piers placed several feet in front of or behind the bollards at various locations. In reality, with 30 percent of the bollard fence openings blocked, less than 26 percent of the total fence length – less than 0.8 of 3 miles – is open to flow exchange between the river and floodplain behind the fence. The fence as configured in the TGR model, in contrast, appears to have 37 percent open area – more than 40 percent larger than specified by IBWC criteria, with the smallest opening of 5 feet. In addition, at prototype scale, plant and woody debris would likely obstruct multiple 5-inch openings, as opposed to a 1.5-inch (30 percent) obstruction of each opening. The 4-foot-diameter virtual piers in the TGR model not only do not obstruct flow through the fence but act as guidewalls that deflect flow and create the whirls and eddies shown in Figure 4.4.

4.3.3.2 TGR evaluation and interpretation of model results

Notwithstanding the added detail, the TGR report makes very limited use of 2D model features for display and analysis of simulation results for assessing the overall hydraulic impacts of the fence on river and floodplain hydrodynamics. The report does not present a quantitative comparison of WSELs within the model domain and consequently draws no conclusions on the overall impacts of the fence.

Data provided to support the TGR assessment are displayed in the table shown in Figure 4.6. The data consist of peak flows in the left and right overbanks delineated by six cross sections spaced within the model domain. Only two of the TGR cross sections intersect the fence, however. Unfortunately, flow in the river channel for pre- and post-project conditions (required by IBWC guidelines) is not included in the table, and consequently no quantitative information is provided on channelization effects of the raised and steepened riverbank, flow restrictions due to the bollards and debris blockages, and hydrodynamic losses caused by the fence. As subsequently described in this report, the Arcadis HEC-RAS model shows significant (up to 30 percent) increases in flow in the river channel at some locations along the fence.

4.3.3.3 Assessment of TGR model

The documentation of the model and evaluation of model output provided in the TGR report are incomplete and insufficient to support the author’s subsequent conclusions on the hydraulic impacts of the bollard fence. Specifically, there are no quantitative data presented in the report to support two of its most important conclusions, briefly described as follows:

- “The bollards … do not significantly impede the movement of water as the reservoir fills and draws down” (TGR 2020). A 2D analysis would have instead revealed that the floodplain behind the fence fills significantly more slowly than would naturally occur without the fence as the reservoir rises. The analysis would have also shown that the reservoir does not rise as a level pool or uniformly from east to west (as described in the TGR report), but rather from the west and the east initially as the center portion behind the fence fills more slowly. A more accurate interpretation of the TGR model results is that movement of water between the Rio Grande river channel and floodplain behind the fence is significantly impeded on the rising side of the design flood (the period to the left of the vertical dashed line in Figure 4.3). Neither the TGR nor the Arcadis model was extended to simulate the recession side of the flood, and consequently no data are available for assessment of the impacts of the fence on flow conditions as the reservoir draws down.

- "There is no significant deflection due to the improved inlet conditions at all of the openings in the bollard fence" (TGR 2020). Data provided in the TGR report, displayed in Figure 4.6, are not sufficient to assess flow deflection because hydrodynamic conditions including river flows, flow velocities, and river stages are not presented along the full length of the fence for pre- and post-project conditions. In addition, peak flow is a one-dimensional quantity applicable to the river channel but not to two-dimensional floodplains where flow is not unidirectional. The TGR model is a 2D model, but the TGR report presents no information on flow velocities, flow direction, or river-floodplain circulation for more complete assessment of flow deflection.

Rio Grande Deflections Run Sep 2020 - 30% Blockage
Flows in CFS

Section	Original Peak	Improved Peak	Delta
15002 Left	64100	64236	0.2%
15002 Right	169747	169212	-0.3%
	233846.914	233447.8282	
21782 Left	205289	203948	-0.7%
21782 Right	27077	28397	4.9%
	232365.996	232345.4433	
24260 Left	152372	150484	-1.2%
24260 Right	80086.93	81967.8984	2.3%
	232458.633	232452.164	
32283 Left	45554	45074	-1.1%
32283 Right	186999	187470	0.3%
	232553.859	232544.2539	
32774 Left	49982	49128	-1.7%
32774 Right	177046	178029	0.6%
	227027.961	227156.6992	
41992 Left	113865	112721	-1.0%
41992 Right	119474	120601	0.9%
	233338.968	233322.1015	

Figure 4.6. Rio Grande flow deflection summary (from TGR 2020 report)

Overall, the TGR model is not suitable for simulation of 2D flows around and through the bollard fence, partially obstructed by debris. The reasons for this conclusion are that (1) the fence is not represented with adequate resolution in the 2D terrain due to the 12:1 horizontal scale distortion represented in the TGR model, and (2) there is a lack of debris obstruction represented by the virtual piers, which function effectively as guidewalls rather than obstructions. The interpretation of the model results in the TGR report does not accurately characterize the hydrodynamic impacts of the fence including flow deflection, erosion potential, and loading on the bollards due to hydraulic head, flow velocity, and direction of flow.

4.4 Arcadis hydrodynamic model development

Arcadis developed a refined hydrodynamic model of the bollard fence system and the Rio Grande river and floodplains upstream of Anzalduas Dam using the most recent release of HEC-RAS, version 6.0.0 (USACE 2021). The refined model was designed to take advantage of the additional capabilities of the latest release over version 5.0.7 applied in the TGR modeling. The refined model was intended to remedy the most serious deficiencies of the TGR model relative to modeling objectives, specifically simulation of hydrodynamics of flow around and through the bollard fence and assessment of flow deflection potentially caused by the fence. Some of the version improvements utilized in creation of the Arcadis model include:

- HEC-RAS Mapper editing tools and raster calculator
- Placement of breaklines within 2D flow areas to align computational mesh with geometric features, in this case the bollard fence and fence obstructions
- Weir profile capacity for 500 station-elevation points per breakline segment
- More accurate and physically realistic simulation of flow around bollards using connections and weir profiles to represent bollards and openings exactly for application of 2D equation solver
- Greater parallelization of the 2D code, making 2D model simulations 20 to 50 percent faster than previous versions

These improvements enabled a computational mesh to be developed that represents the fence geometry at prototype scale and without distortion to better preserve hydraulic similitude than the TGR model. The Arcadis model simulates hydraulic properties of the fence with bollards oriented at 45 degrees to the river centerline, with 30 percent of the fence openings blocked as prescribed by IBWC guidelines. An important distinction between the TGR and Arcadis models is that debris blockage in the Arcadis terrain model is symmetrical, i.e., applies equally regardless of flow direction. In contrast, the superposition of virtual piers in front of or behind the fence at different locations in the TGR model to represent obstructions would need to be relocated to have equal effect on flow moving into or out of the floodplain as the river rises and falls.

The computational speed improvements were critical to successful implementation of the more detailed fence geometry in the Arcadis model. Run times for simulation of a 92-hour flood hydrograph ranged from 48 to 70 hours, depending on central processing unit (CPU) speed, number of cores, and whether pre- or post-project geometry was simulated.

4.4.1 Modeling approach

The Arcadis model is a modified version of the TGR model that shares the following information with the TGR model:

- Run controls – pre- and post-processing, simulation period, computational time step and tolerances, output time step
- Model domain
- Upstream boundary condition – flow hydrograph
- Downstream boundary condition – normal depth
- Pre-project terrain, breaklines, and 2D grid
- Manning's n regions and roughness coefficients
- Fence alignment

The major changes made to the TGR model to create the Arcadis model are as follows:

- Post-fence construction breaklines and 2D grid
- Undistorted bollard fence terrain with 30 percent obstruction

In summary, the principal differences between the Arcadis and TGR models is the representation of the bollard fence and debris obstructions. The Arcadis model incorporates significant refinements to the post-project fence geometry in comparison to the TGR model, which required development of new tools for breakline and terrain development.

4.4.2 Model geometry

This section describes the development of the terrain models, geospatial layers, and breaklines comprising the digital elevation model (DEM) used in the Arcadis HEC-RAS model for generation of the computational mesh and 2D simulation of the post-project condition. As previously noted, a new terrain model and breaklines for the (30 percent blocked) bollard fence at prototype scale were created and superimposed on the TGR post-project topography. Figures 4.7 and 4.8 compare fence terrains utilized in the TGR and Arcadis models, respectively. The breaklines and grid cells shown in both figures were generated by the Arcadis model.

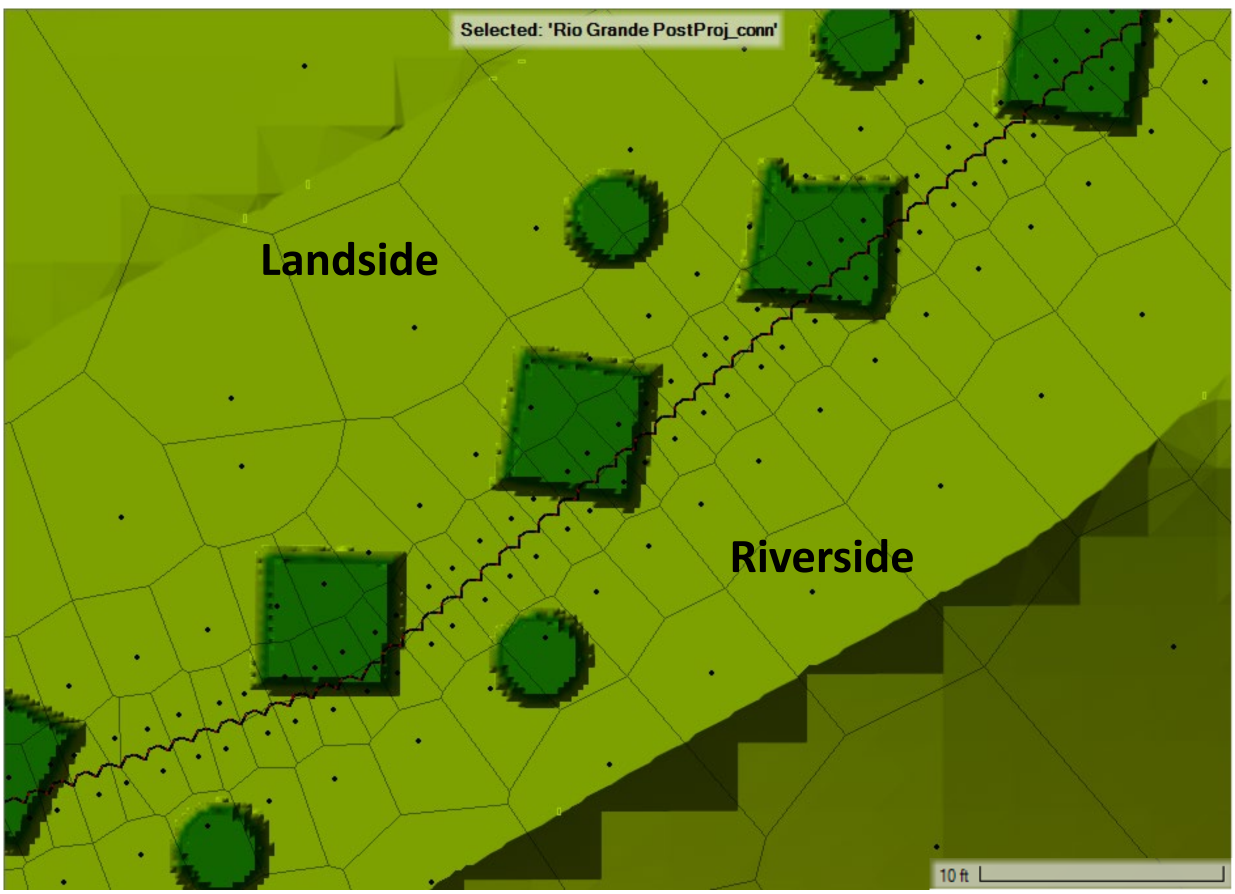

Figure 4.7. Sample TGR model fence terrain with blockage by piers

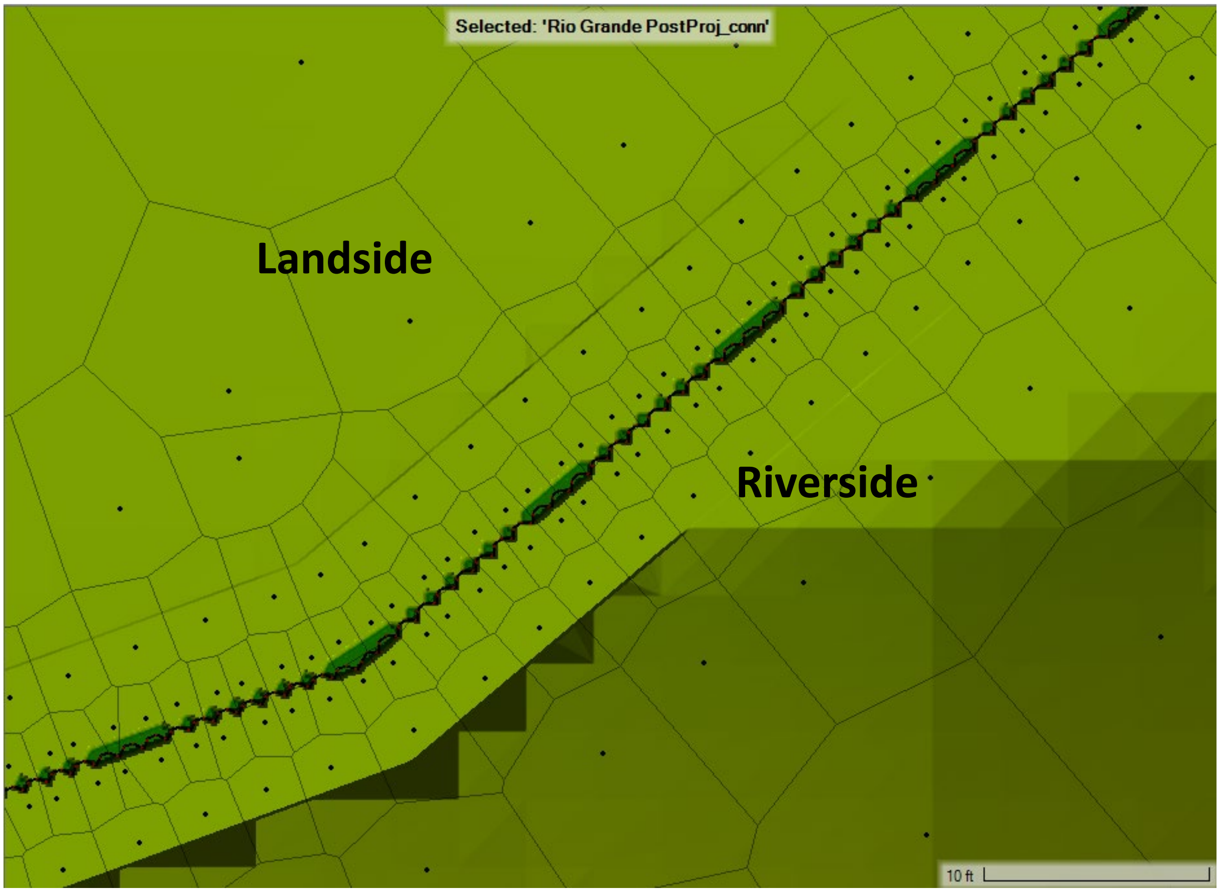

Figure 4.8. Sample Arcadis model fence terrain with blockage

The bollard fence in the Arcadis model consisted of 6-inch bollards turned 45 degrees to the river flow, with 5-inch open spaces between the turned bollards, resulting in a 13.48-inch center-to-center distance. With three of every 10 gaps blocked as prescribed by IBWC guidelines, slightly less than 26 percent of its total length is therefore open in the Arcadis model geometry to flow through the fence, whether into or out of the floodplain behind the fence.

4.4.2.1 Breakline and computational mesh generation

Breaklines are used in HEC-RAS to force alignment of computational cell faces along two sides of a line or series of lines, in this case the bollard fence with 30 percent of the openings blocked and with bollards turned 45 degrees to the direction of flow in the river. Arcadis created a tool in ArcMap (version 10.7.1) to extract breakline points from the delineated fence shapefile. These breakline points reflected the alignment of the fence based on the approximate centerline provided in the TGR files. The tool was designed to split the fence shapefile into small segments and further subdivide those segments to account for the width of the bollards. The lengths of the small segments were calculated based on the distance between each bollard and width of each bollard. Software limitations resulted in a small deviation between the ArcGIS distance and the actual segment length, though not enough of a difference to materially alter the computational mesh. In the first step of breakline generation, several points were created at upstream, downstream, land side, and river side points shown in Figure 4.9.

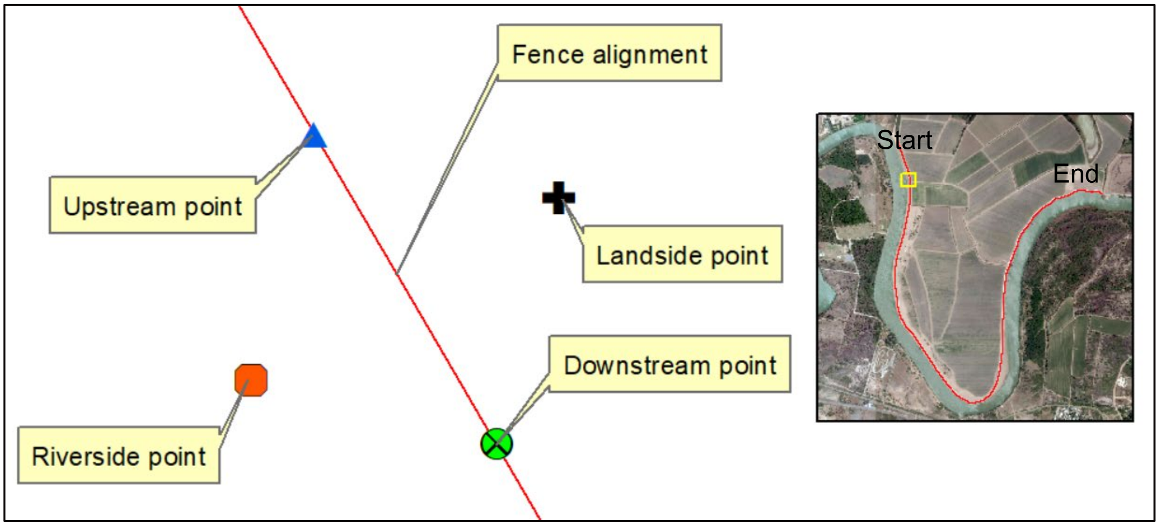

Figure 4.9. Sample of points created using the ArcMap tool

These points were generated using the tool for the entire length of the fence, proceeding counterclockwise from the northwest (upstream) end of the fence to the northeast (downstream) end of the fence. To create the breaklines, the upstream, river side, and downstream points were ordered in sequence from the upstream end of the alignment to the downstream end. An example segment of the sawtooth-pattern breakline exported to HEC-RAS is shown in Figure 4.10.

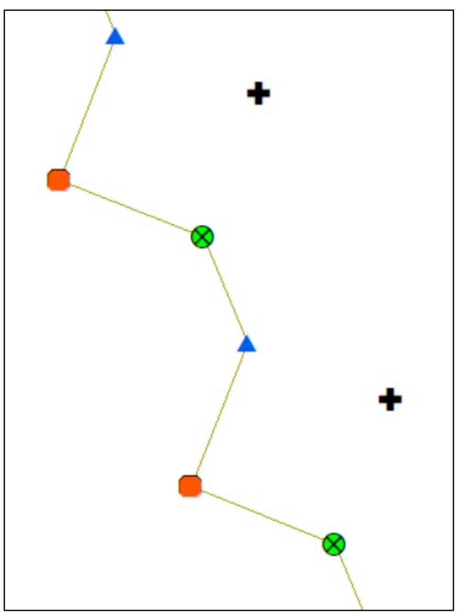

Figure 4.10. Breaklines created from upstream, downstream, and river side points

Due to the HEC-RAS limitation of 500 points used to define a breakline profile, the points generated by the tool in ArcGIS were separated into 124 500-point breakline connections, stationed as shown in Figure 4.11. A typical segment profile is shown in Figure 4.12.

Figure 4.11. Bollard fence breakline connections

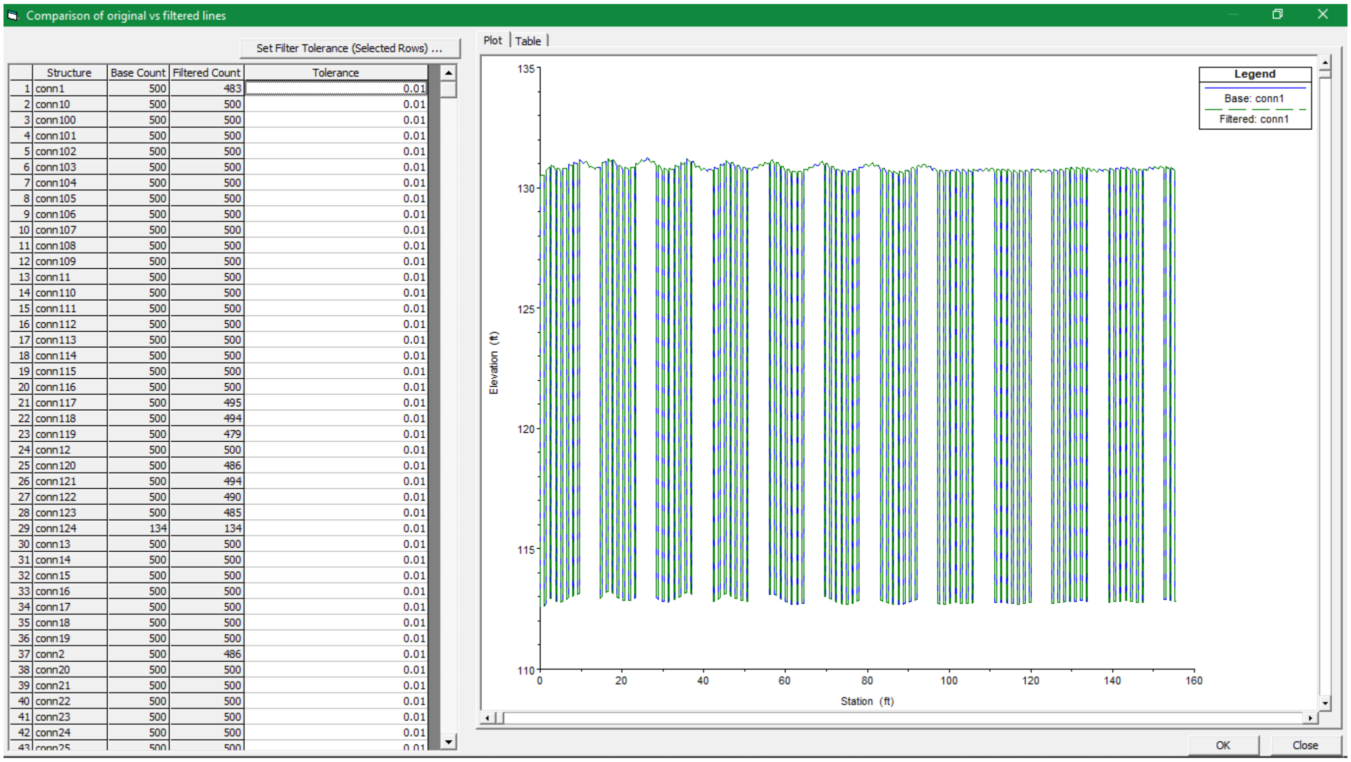

Figure 4.12. Typical bollard fence breakline profile with 30 percent obstruction

4.4.2.2 Computational mesh

The polygon boundary for the 2D area comprising the entire domain of both TGR and Arcadis models is shown in Figure 4.1. For the post-project model, HEC-RAS Mapper was used to force generation of the grid to the fence breakline previously described, with overall grid spacing of 50 feet reduced to approximately 2 feet to force cell face alignment with the breakline without exceeding the maximum number of eight cell faces. A small number of manual refinements were required, but in general the 2-foot spacing worked well. The total number of cells in the post-project mesh is approximately 146,000. Without the fence breakline, the pre-project mesh contains about 94,000 primarily rectangular cells with 50-foot average face length.

A portion of the pre-project mesh is shown in Figure 4.13, and the post-project grid is shown in Figure 4.14.

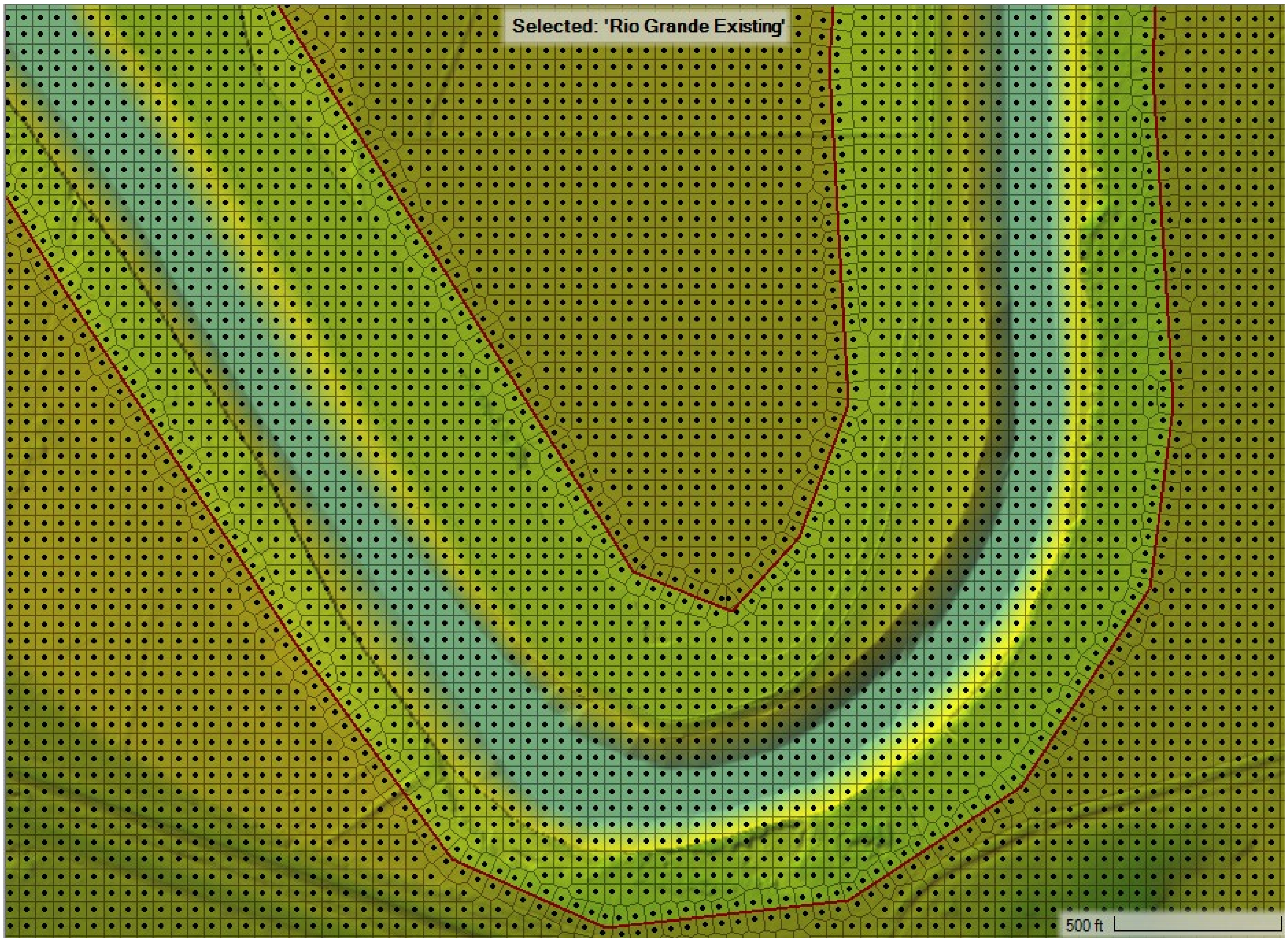

Figure 4.13. Arcadis 2D area pre-project partial computational mesh

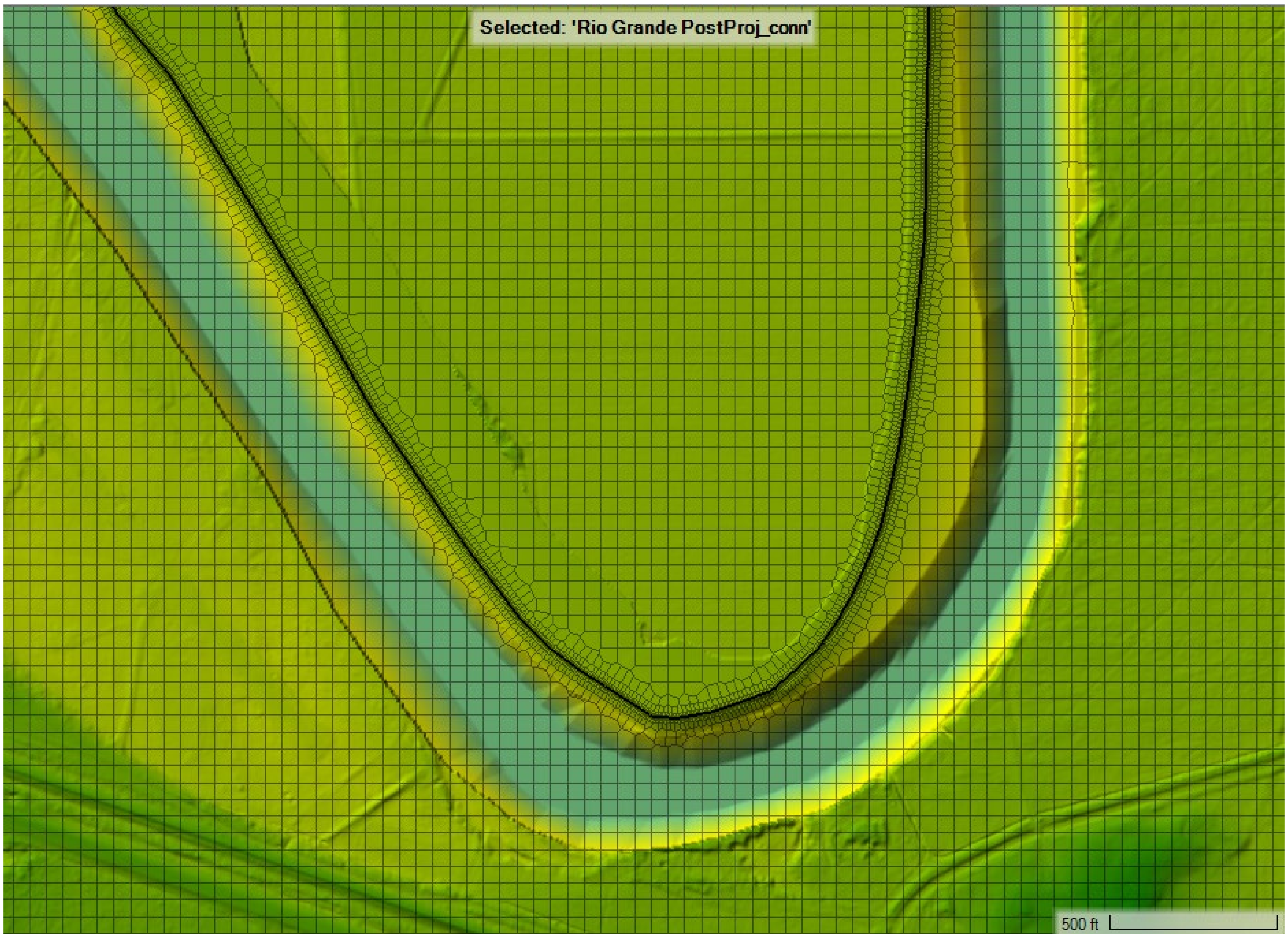

Figure 4.14. Arcadis 2D area post-project partial computational mesh

4.4.3 Model scenarios

The Arcadis HEC-RAS model incorporates the terrain model and computational grid developed by TGR for simulation of the pre-project condition, designated the 'Rio Grande Existing' plan in the model. The post-project simulation (designated 'RioGrande PostProj_conn') utilizes the TGR post-project base terrain with the prototype-scale bollard fence overlay, 124 connections representing the fence profile with 30 percent of openings blocked (three out of every ten), and computational mesh developed by Arcadis as previously described.

The upstream boundary condition for both pre- and post-project scenarios represents model domain inflow on the rising limb through the peak of the design flood, i.e. from September 20 to September 23, 1967 – a simulation period of 92 hours marked as shown in Figure 4.3.

4.5 Model results

Model results presented and discussed in this report are used to assess the impacts of the bollard fence on river and floodplain hydrodynamics, and subsequently to provide data needed for geotechnical and structural assessments. Hydraulic, geotechnical, and structural assessments provided in this report are based on the following Arcadis hydraulic model results:

- Reduction in flow conveyance and circulation between the river and floodplain behind the fence in comparison to pre-project conditions (slower filling of floodplain behind the fence, higher WSELs, and differential head across the fence).
- Deflection of flow in the Rio Grande river channel due to the fence (change in river flow volume with and without the fence).
- Deflection of total river and floodplain flow toward either the U.S. or Mexico sides of the river channel, measured by changes in maximum WSELs and maximum flow with and without the fence in accordance with IBWC-designated criteria.
- Reduction in Mission Levee Phases I and II freeboard with and without the fence.
- Turbulence, structural loading, and erosion potential created by the fence (flow direction and velocity through and around the fence).

Hydraulic model results presented in this report were generated by the Arcadis model simulation of the rising limb through the peak of the design flood (September 20 at 00:00 to September 23 at 20:00), as shown in Figure 4.3. As previously described, the fence geometry is represented in the Arcadis model at prototype scale with 30 percent debris obstruction as specified by IBWC criteria.

4.5.1 Circulation and flow exchange

Pre- and post-project flow depths are primarily within the riverbanks until the early morning hours of September 22. Without the fence, by 08:00, the floodplain behind the fence line begins to fill, as shown in Figure 4.15. With the fence in place, however, the floodplain behind the fence fills more slowly, as shown in Figure 4.16. The contrast indicates that the fence significantly restricts circulation between the river and floodplain in comparison to pre-project conditions.

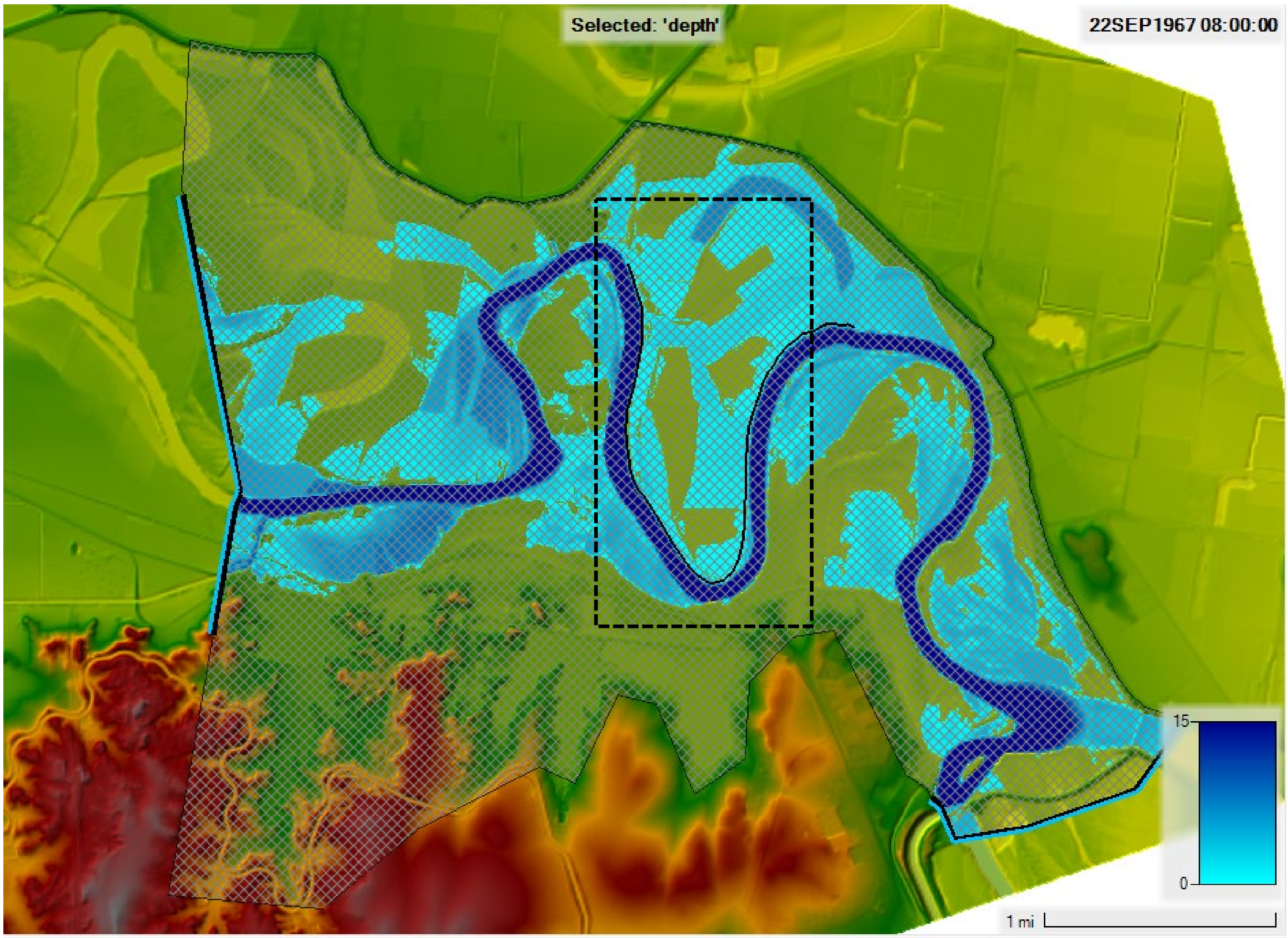

Figure 4.15. Pre-project depth of inundation (September 22, 1967, 08:00)

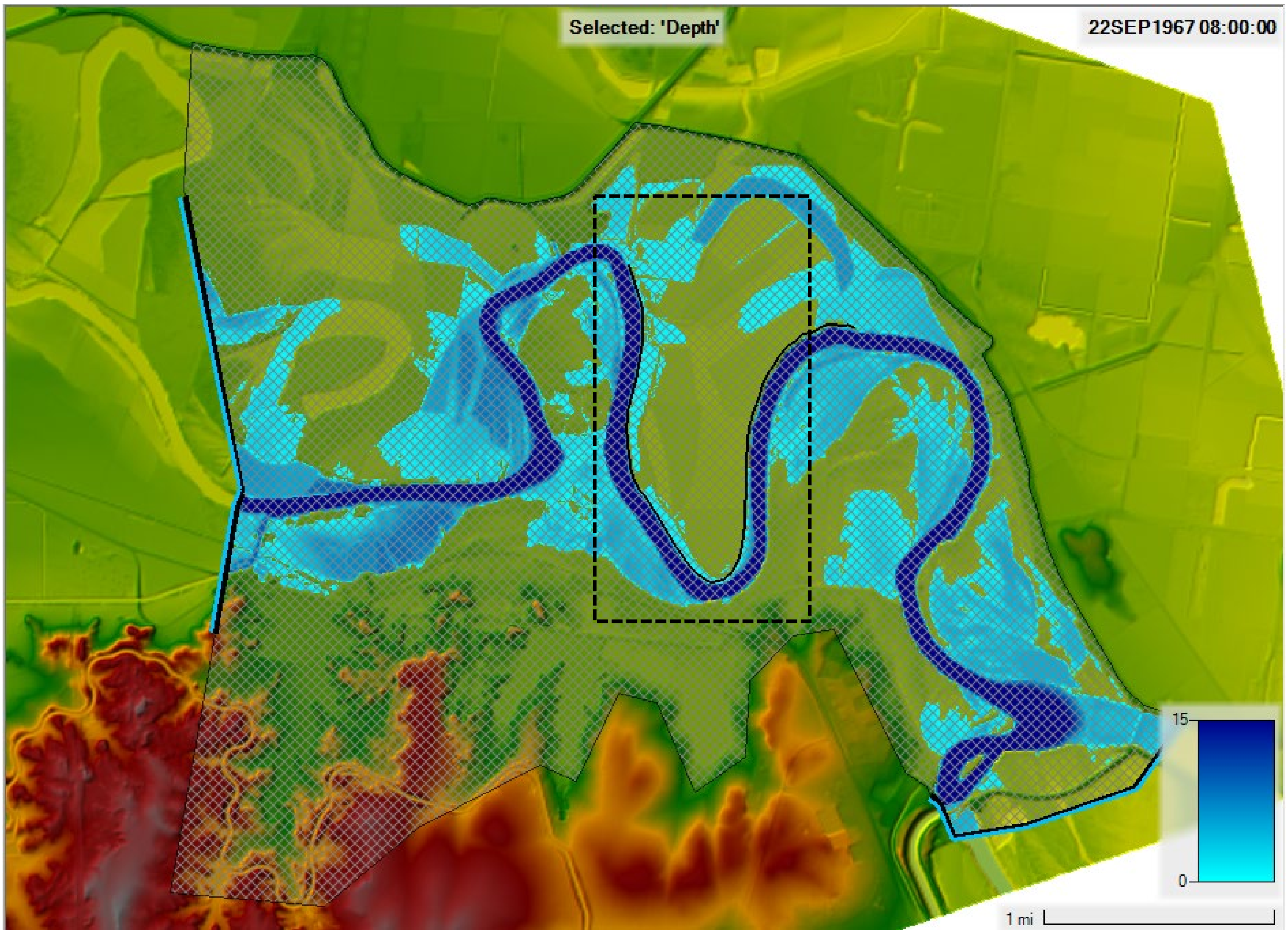

Figure 4.16. Post-project depth of inundation (September 22, 1967, 08:00)

Another indicator of altered circulation caused by the fence is differential WSEL between the land and river sides of the fence. As shown in Figure 4.17, the maximum WSEL is higher in the river than in the floodplain behind the fence on the western portion by about 0.2 feet on average. To the east of the fence midpoint (the southern tip of the peninsula bounded by the fence, shown in Figure 4.17), however, WSEL behind the fence is approximately 0.25 feet on average higher than in the river. The net effect is that movement of water into the floodplain from the west and out of the floodplain to the east is clearly impeded by the fence. By comparison, Figure 4.18 shows peak WSELs on land and river sides of the fence line to be nearly identical and lower overall than the post-project condition. Together these results indicate that the fence significantly reduces flow conveyance from west to east, creating a large ineffective or ponding area behind the fence as a result.

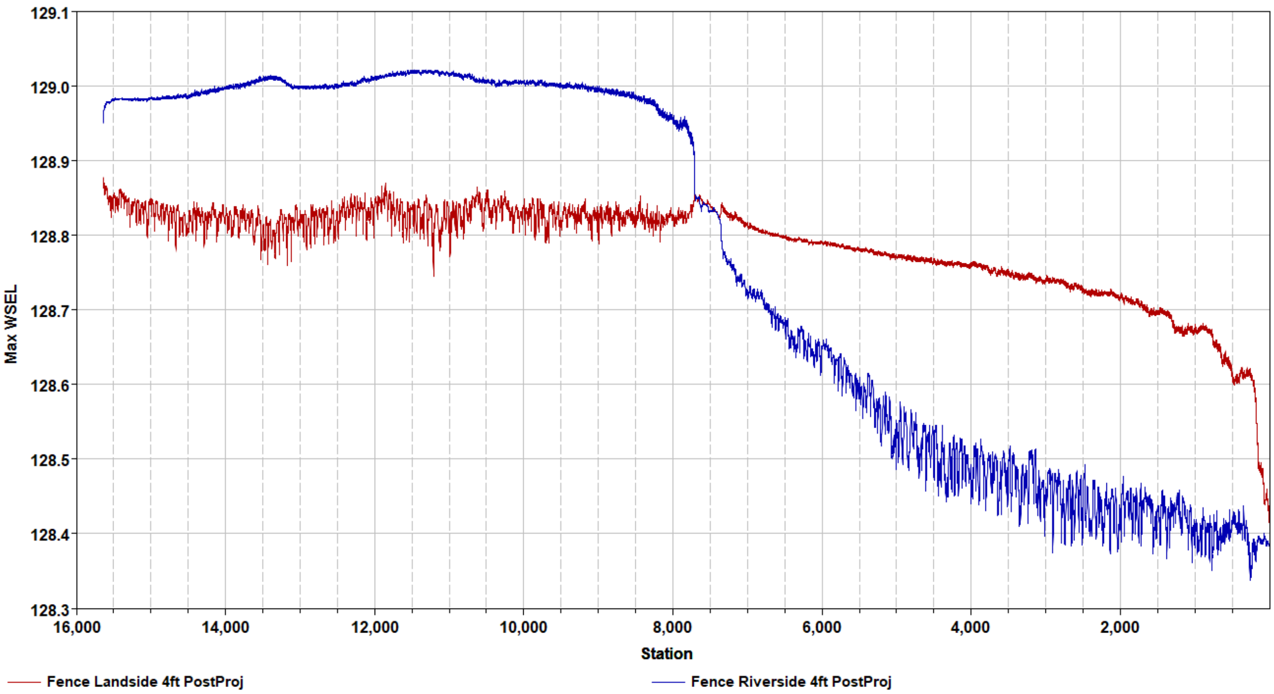

Figure 4.17. Post-project peak water surface elevation profiles on land (red line) and river (blue line) sides of fence

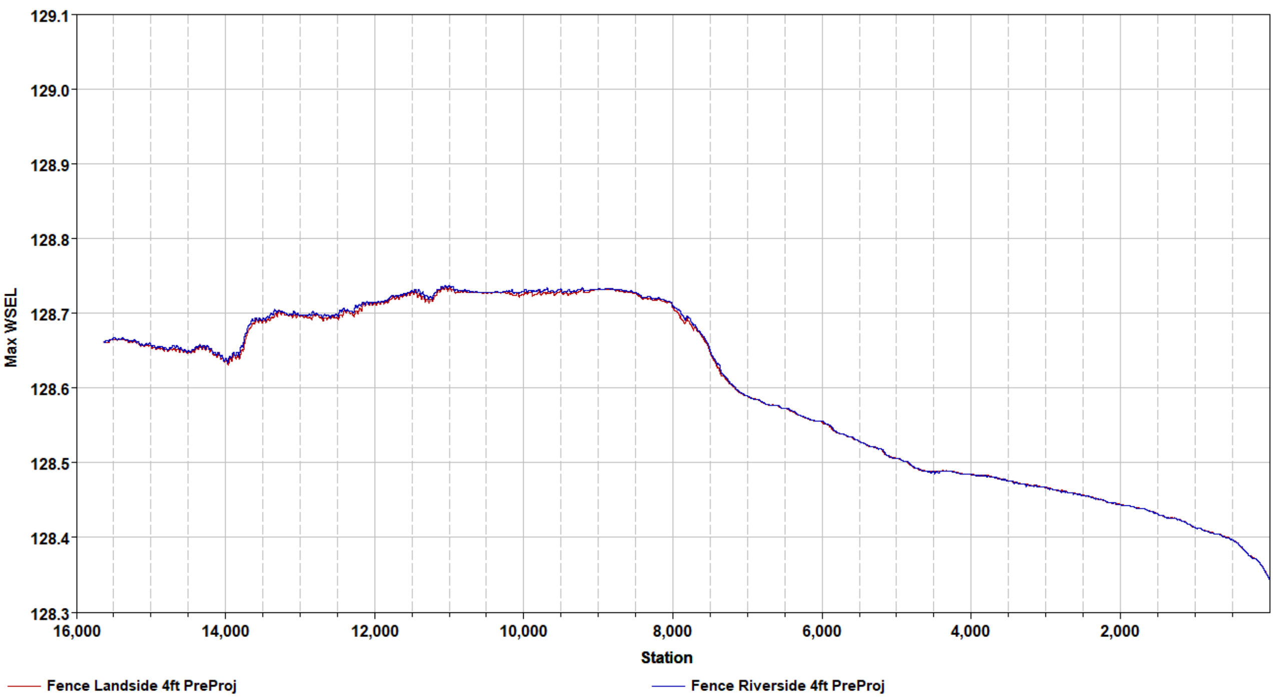

Figure 4.18. Pre-project peak water surface elevation profiles on land (red line) and river (blue line) sides of fence line

4.5.2 Flow deflection

Restrictions in river and floodplain water exchange caused by the fence have altered the balance of flow conveyed over the U.S. and Mexico sides of the river, constituting flow deflections for purposes of this report. Cumulative 92-hour flow volume at seven river cross sections, stationed from upstream to downstream as shown in Figure 4.19, were calculated from Arcadis model results for pre- and post-project conditions. With one exception, model results displayed in Figures 4.20 through 4.26 show that flow in the river increases from 11 to 27 percent above pre-project river flow.

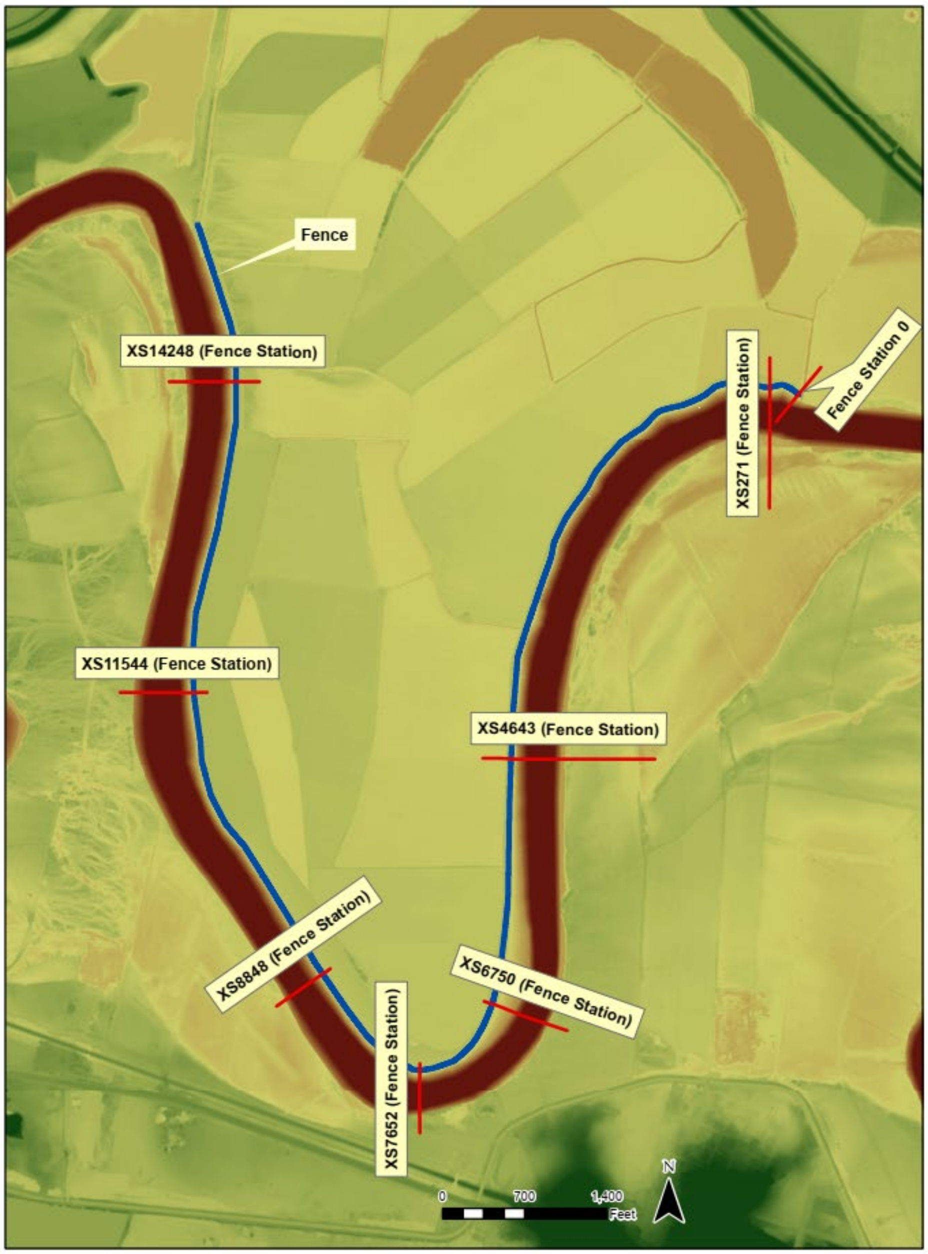

Figure 4.19. River cross-section stationing along bollard fence

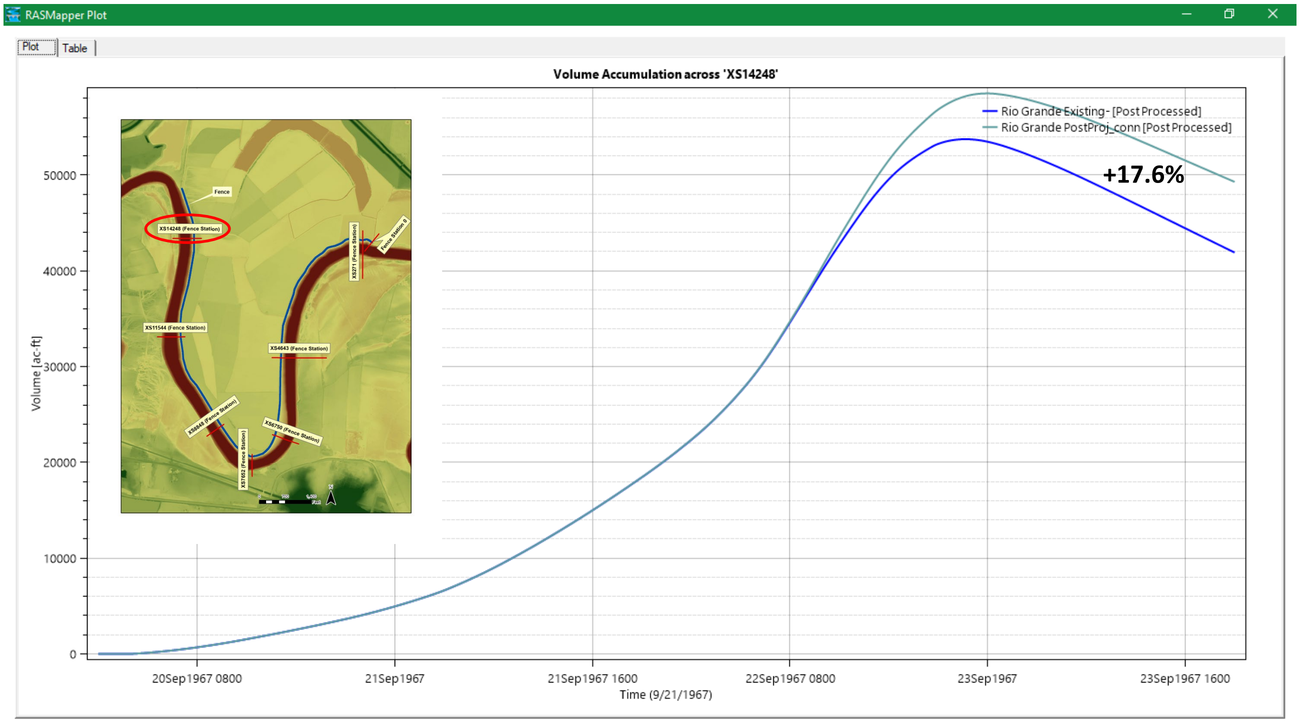

Figure 4.20. Rio Grande river channel cumulative flow at station 14248 (light blue – post-project, blue – pre-project)

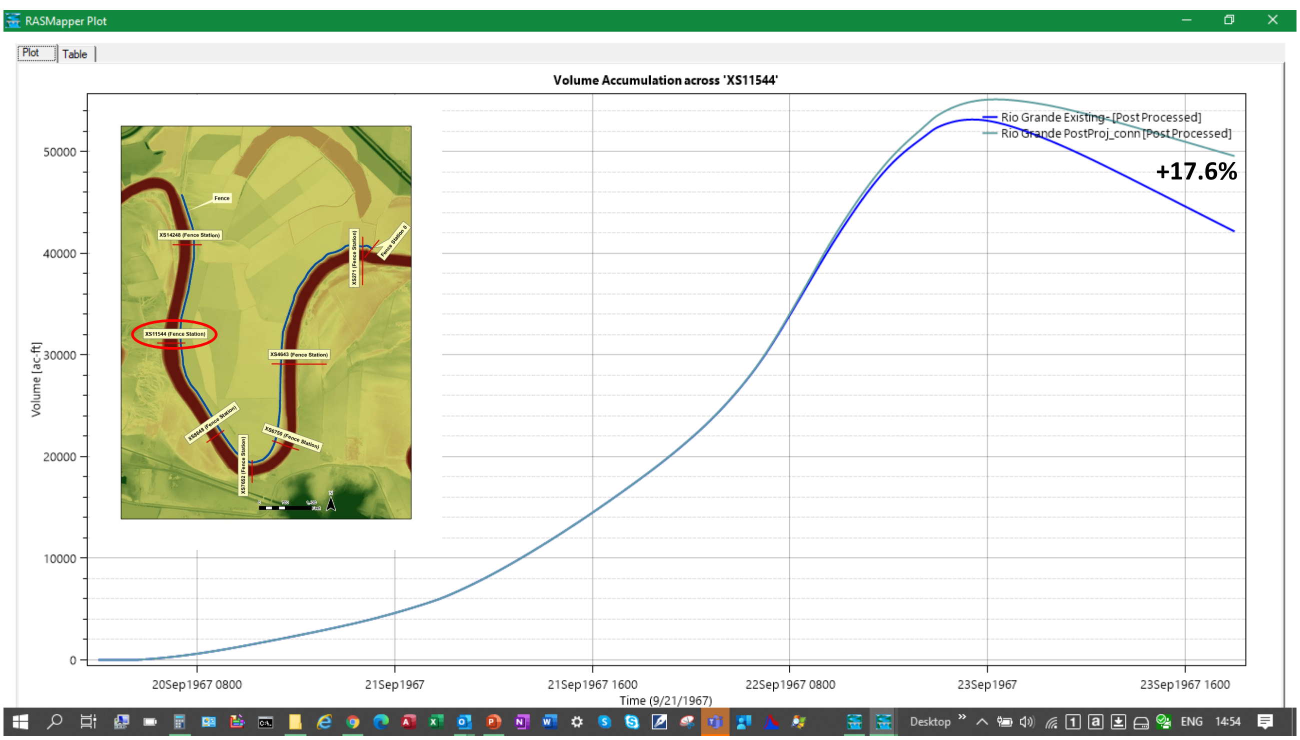

Figure 4.21. Rio Grande river channel cumulative flow at station 11544 (light blue – post-project, blue – pre-project)

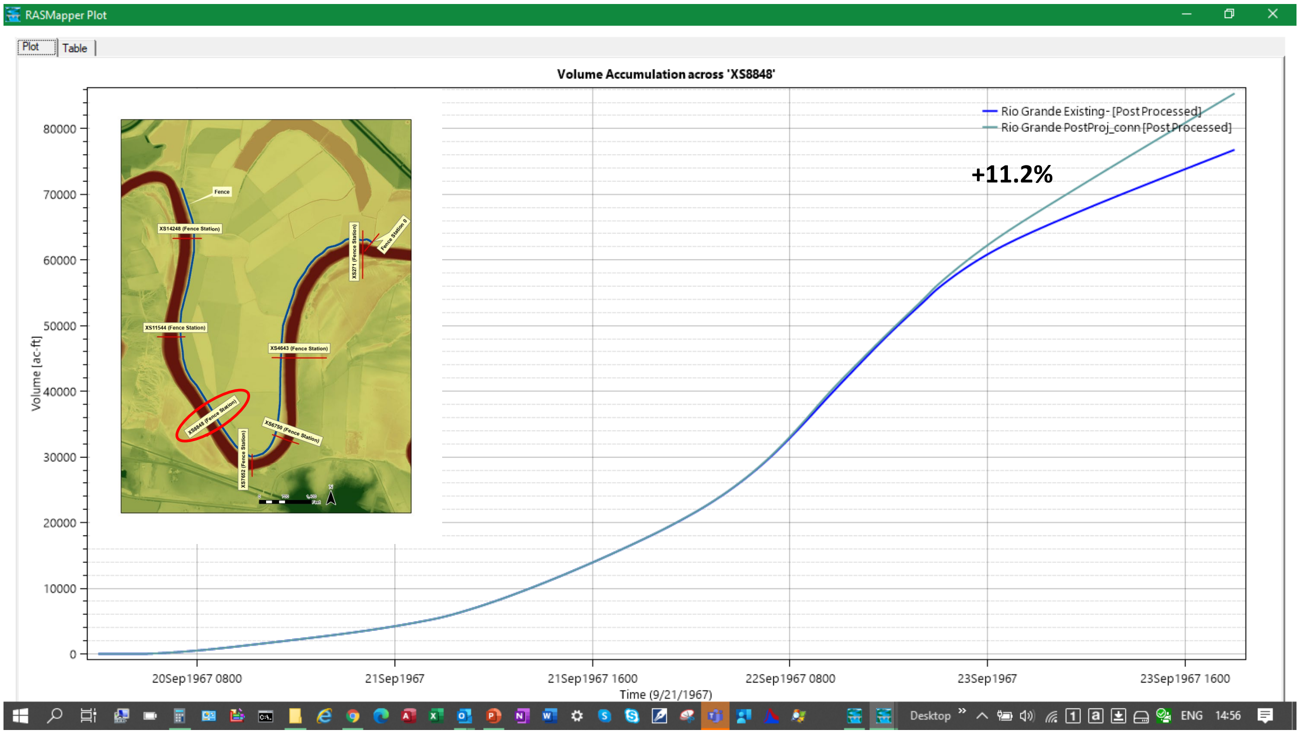

Figure 4.22. Rio Grande river channel cumulative flow at station 8848 (light blue – post-project, blue – pre-project)

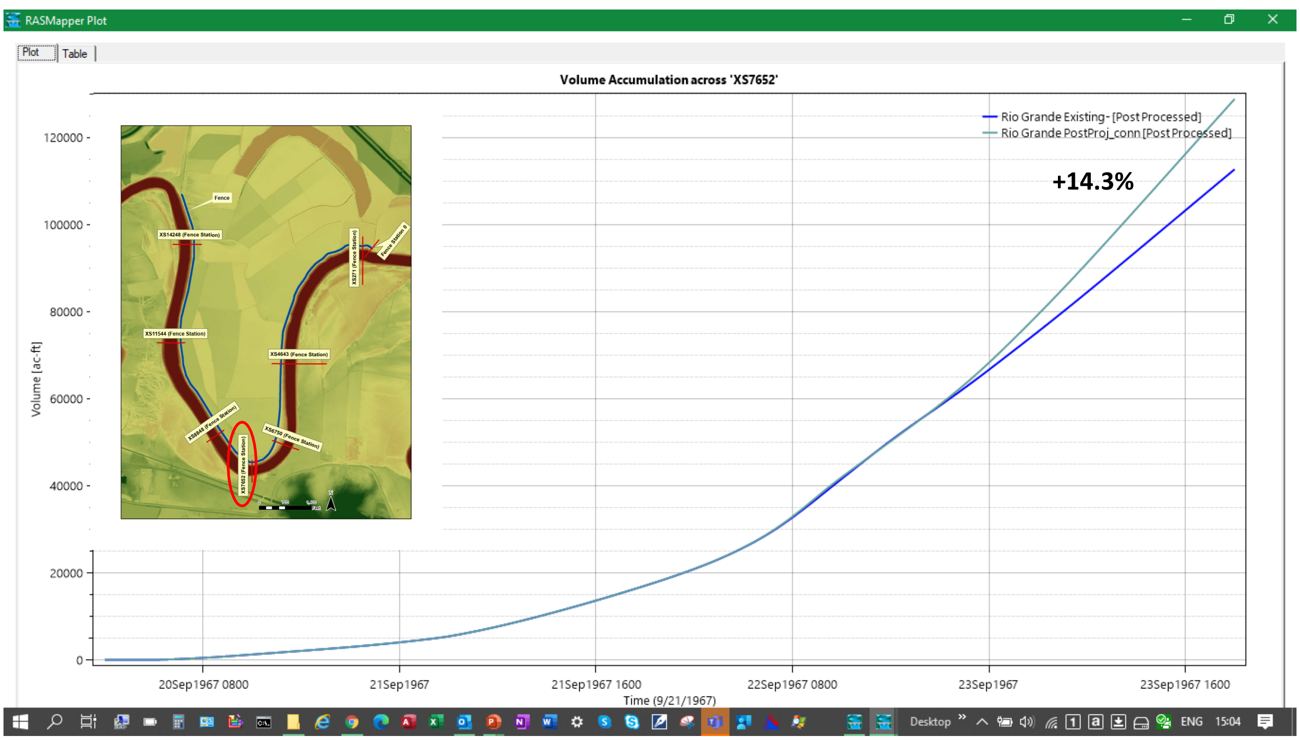

Figure 4.23. Rio Grande river channel cumulative flow at station 7652 (light blue – post-project, blue – pre-project)

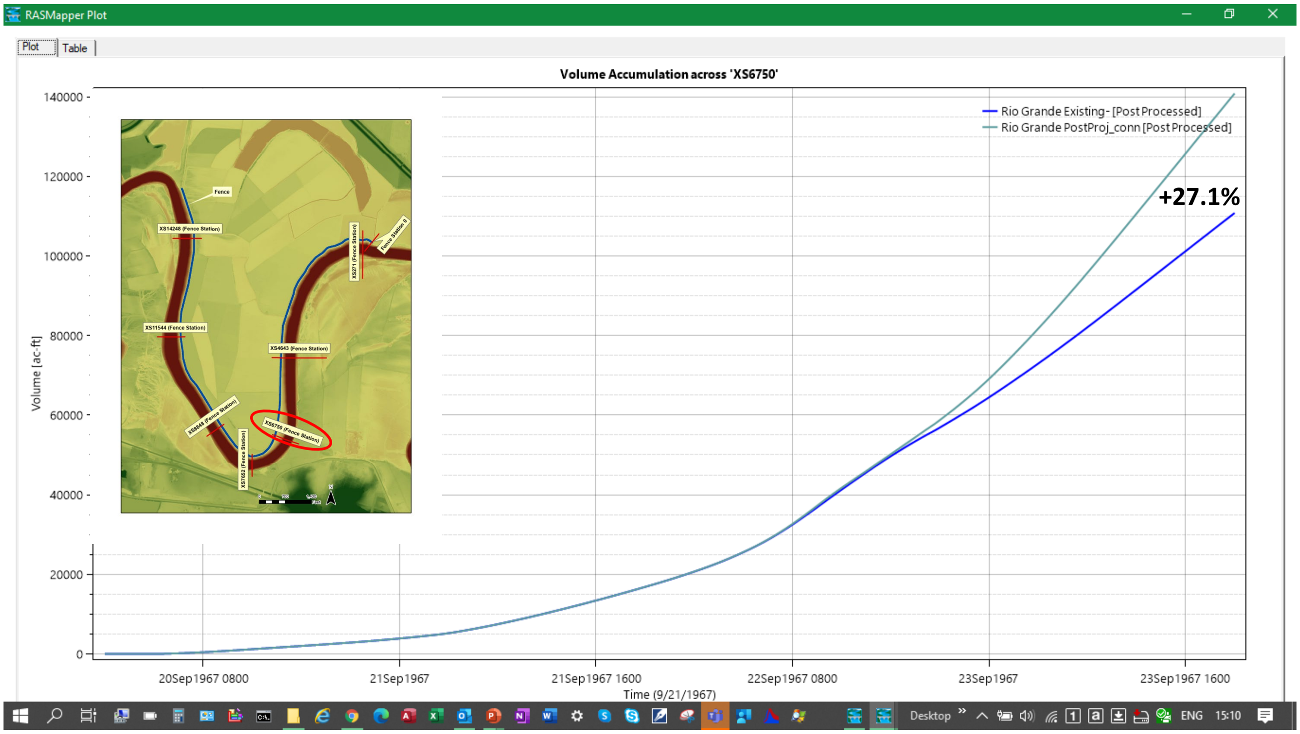

Figure 4.24. Rio Grande river channel cumulative flow at station 6750 (light blue – post-project, blue – pre-project)

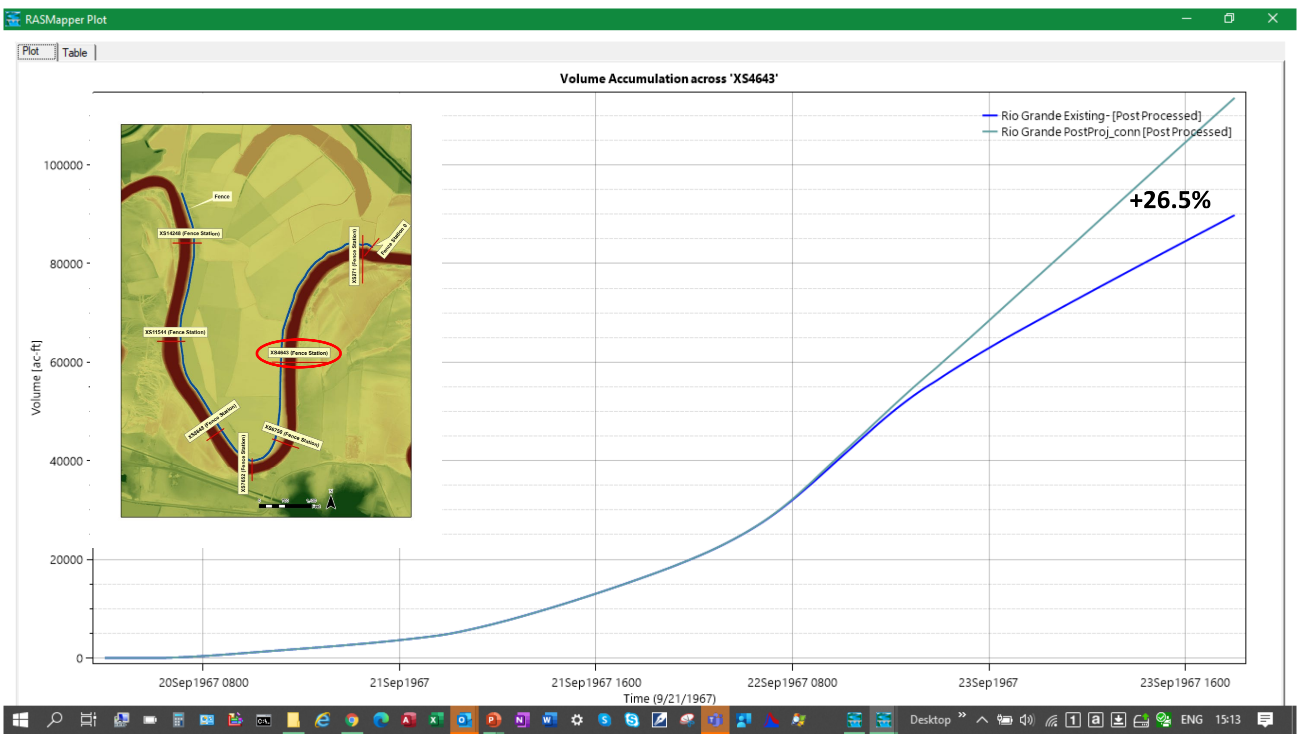

Figure 4.25. Rio Grande river channel cumulative flow at station 4643 (light blue – post-project, blue – pre-project)

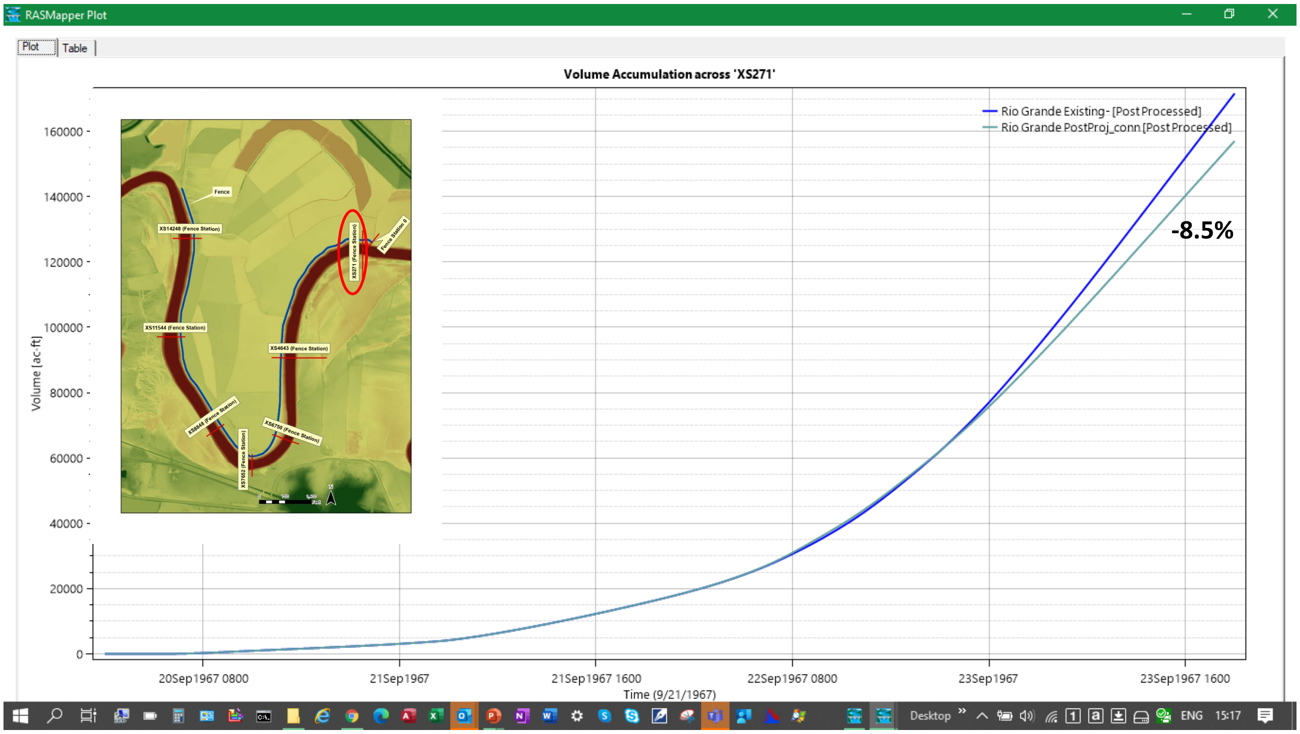

Figure 4.26. Rio Grande river channel cumulative flow at station 271 (light blue – post-project, blue – pre-project)

Other measures of flow deflection designated by IBWC guidelines are as follows:

- WSEL increases from the pre-project (without fence) to the post-project (with fence) condition.
- Percentage difference in maximum flows on the U.S. and Mexico sides of the border (the river channel centerline) from the pre-project (without fence) to the post-project (with fence) condition.

Differences in WSELs and maximum flows were derived from Arcadis model results using profile lines extending from the northern model boundary (the Mission Levee) to the river centerline on the U.S. side (left side floodplain and channel looking downstream), and from the river centerline to high ground on the Mexico side (right side river channel and floodplain looking downstream). Four profile cross sections were constructed intersecting cross sections 11544, 7652, 4643, and 271, as shown in Figure 4.27.

Changes in WSEL and maximum flows may not reliably indicate the magnitude of flow deflection for the following reasons:

- Due to specific energy considerations in open-channel flow hydraulics, WSEL may be relatively insensitive to change in flow, i.e., large increases in flow may produce only small changes or even negative changes in water surface elevation.
- With the wide floodplains and relatively narrow river channel in the study area, most of the flow from the upstream to downstream model boundaries is conveyed through the floodplains – in this case by factors of 2 to 5. Consequently, large changes in river channel flow might account for only small changes in total river and floodplain flow. However, increases in channel flow will have much greater potential to cause migration of the river and the U.S. Mexico border as a result.

- The Arcadis HEC-RAS model is fully two-dimensional and flow through any cross section is essentially one-dimensional. As a result, flow calculated across profile lines arbitrarily drawn across 2D floodplains may not accurately reflect the magnitude and direction of flow in a 2D flow field.

Notwithstanding these limitations, changes in WSEL and maximum flow from pre- to post-project conditions for the four profile lines are provided in Table 4.1.

Table 4.1. IBWC-designated flow deflection indicators

	Post-Proj – Pre-Proj change (+/-)		IBWC limits (feet)	Pre-Proj	Post-Proj	Post-Proj-Pre-Proj	% change (+/-)
	Max WSEL channel (feet)	Max WSEL floodplain (feet)		Max flow (cfs)	Max flow (cfs)	Max flow (cfs)	Max flow
XS11544 left (U.S.)	+0.29	+0.24	0.25-0.5	118,774	115,738	-3,036	-2.56%
XS11544 right (Mexico)	+0.29	+0.29	0.25-0.5	98,793	101,267	2,474	+2.50%
XS7652 left (U.S.)	+0.22	+0.25	0.25-0.5	199,696	192,179	-7,517	-3.76%
XS7652 right (Mexico)	+0.23	+0.25	0.25-0.5	28,730	35,089	6,359	+22.13%
XS4643 left (U.S.)	+0.06	+0.26	0.25-0.5	156,178	140,533	-15,645	-10.02%
XS4643 right (Mexico)	+0.05	+0.08	0.25-0.5	76,112	91,689	15,578	+20.47%
XS271 left (U.S.)	+0.02	+0.23	0.25-0.5	59,879	65,531	5,652	+9.44%
XS271 right (Mexico)	+0.02	+0.06	0.25-0.5	172,416	165,876	-6,540	-3.79%

The results of the analysis indicate, as expected, small increases in maximum WSEL on both sides of the border, generally falling within IBWC tolerances. However, the percentage change in maximum flow strongly indicates deflection toward the Mexico side of the river-floodplain system in three of the four profile lines.

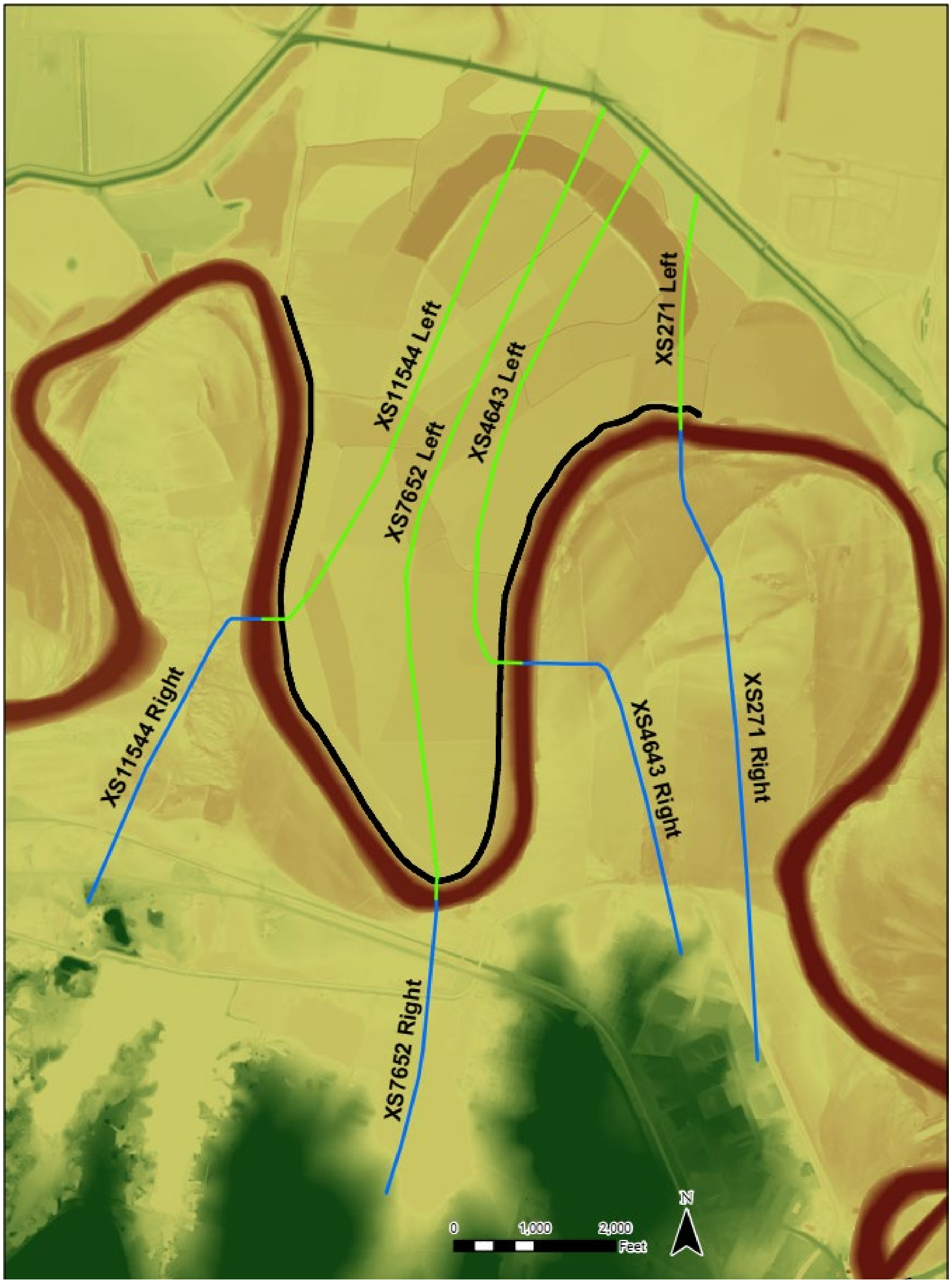

Figure 4.27. Profile lines for which changes in maximum water surface elevations and maximum flows were derived for use as IBWC-designated flow deflection indicators

4.5.3 Mission Levee freeboard reduction

A profile line was constructed along the northern model boundary, which follows the Phase I Mission Levee (Banker Weir to Inspiration Road) and Phase II Mission Levee (Inspiration Road to Abram Road), moving from east to west. Computed maximum WSELs and base terrain elevations along the entire Levee are shown in Figure 4.28.

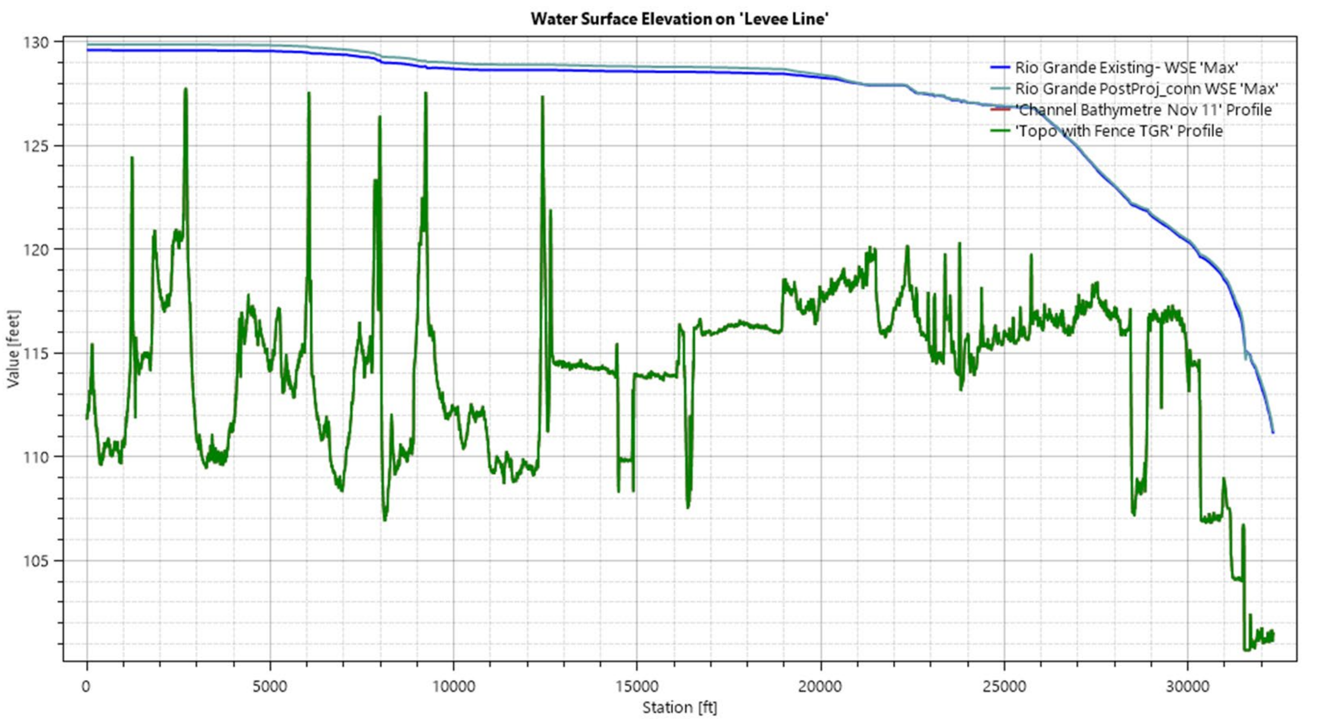

Figure 4.28. Maximum water surface elevation along Mission Levee (light blue – post-project, blue – pre-project, green – base elevation)

The data show that freeboard is reduced by up to 0.29 feet on the western and middle portions of the levee (Station 0 – 20000 in Figure 4.28) and by up to 0.06 feet on the eastern portion of the levee (Station 20000 – 32000) due to the fence.

4.5.4 Hydrodynamics of flow through fence

As shown by the previous section, flow through the bollard fence on the rising side of the design flood is generally west to east. The resistance of the fence to flow results in differential head from the outside to the inside of the fence on the western portion, and from the inside to the outside on the eastern portion, as shown in Figure 4.17. Flow resistance is reflected in higher velocities and turbulence through the constricted openings of the fence relative to ambient velocities in the adjacent river and floodplain. Higher velocities in comparison to natural, i.e., pre-project, conditions increase the potential for erosion of the fence foundation and produce structural loads and moments on the individual bollards.

4.5.4.1 Flow velocities through fence

Flow velocities in the river channel along the fence line range between 1 and 3 fps for the pre-project condition. For the post-project condition, however, maximum flow velocity through the unblocked fence openings on the western portion of the fence reaches 7.9 and exceeds 5 fps on the eastern portion of the fence. Figure 4.29 shows a color-coded map of maximum velocities through typical unblocked openings on the western portion of the fence. Figure 4.30 zooms out, showing the prevalence of high-velocity openings along most of the western fence line. Both figures show high-velocity plumes extending for several feet to the inside (land side) of the fence.

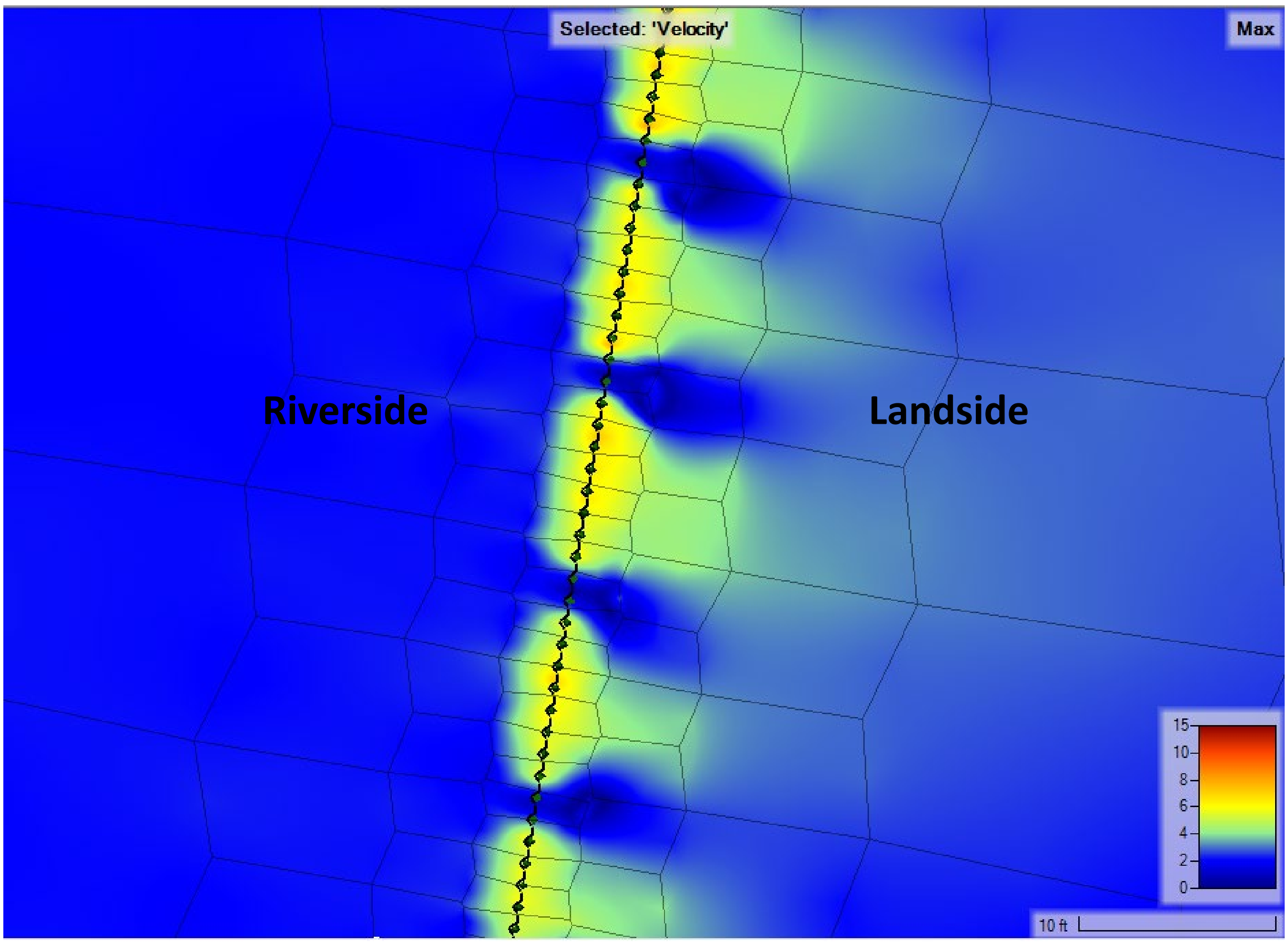

Figure 4.29. Maximum velocity plumes through typical unblocked openings on western portion of fence

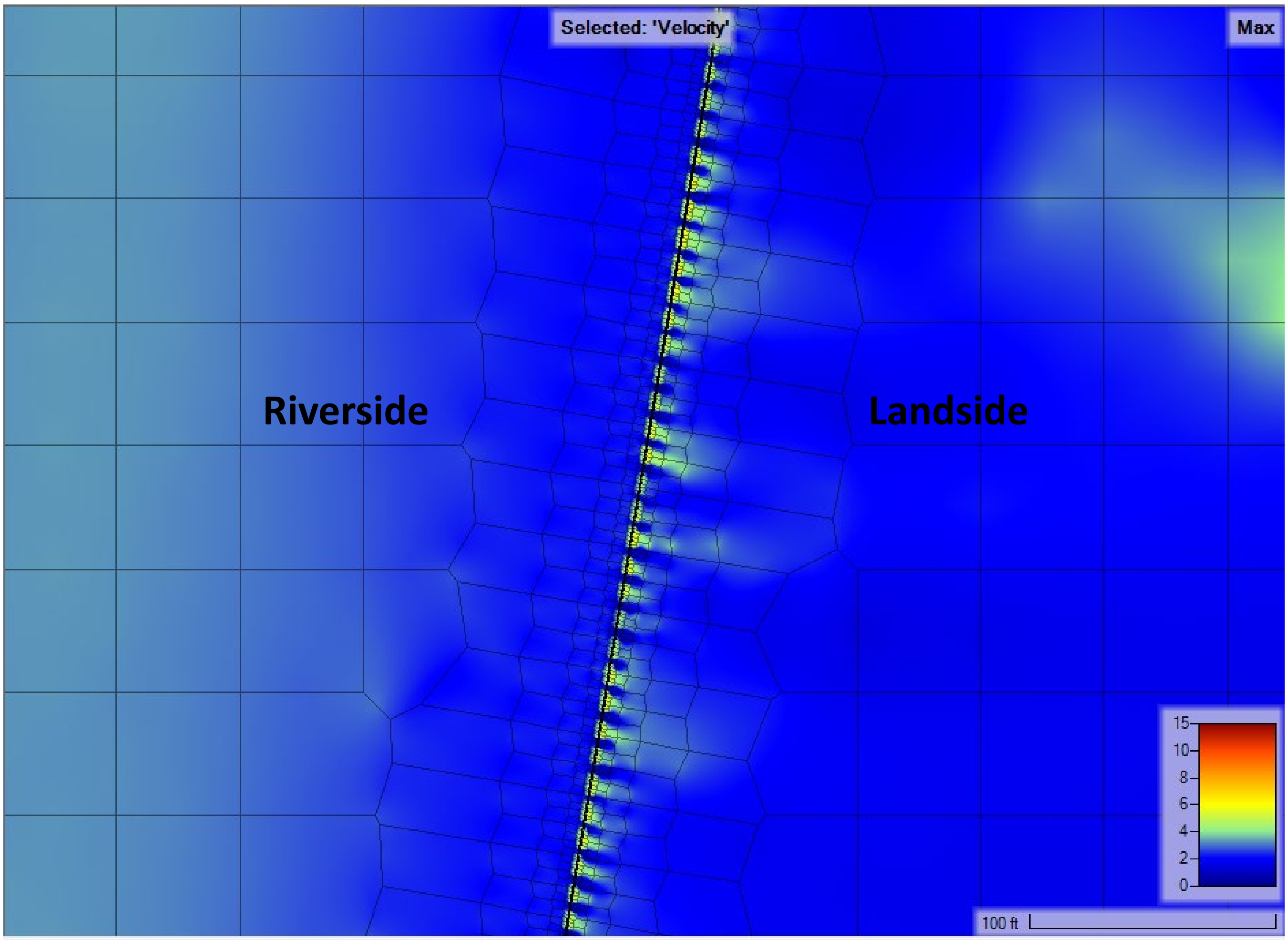

Figure 4.30. Maximum velocity plumes along western portion of fence

Figure 4.31 shows a color-coded map of maximum velocities through typical unblocked openings on the eastern portion of the fence. Figure 4.32 zooms out, showing the prevalence of high-velocity openings along the eastern fence line. Both figures show high-velocity plumes extending for several feet to the outside (river side) of the fence.

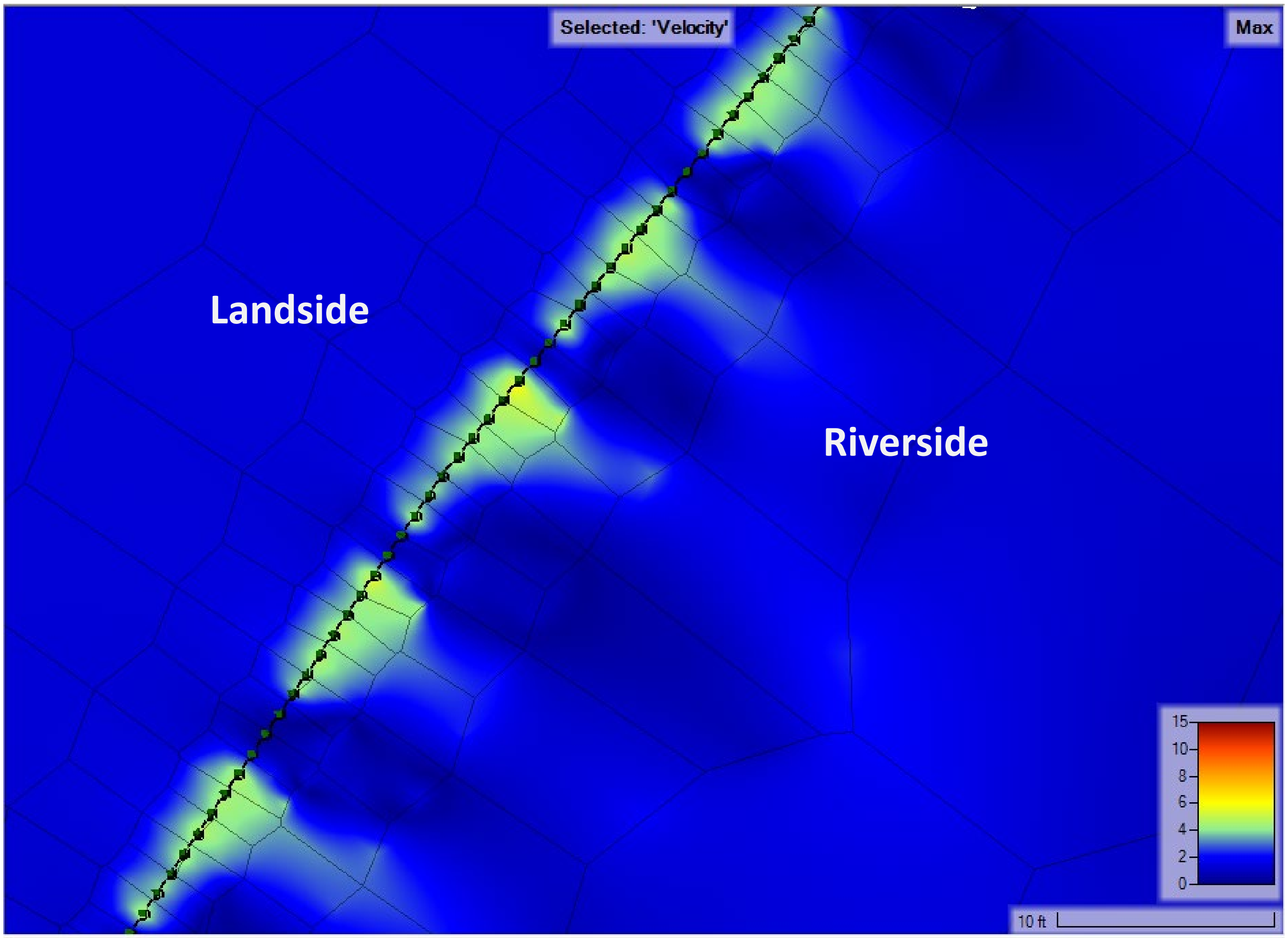

Figure 4.31. Maximum velocity plumes through typical unblocked openings on eastern portion of fence

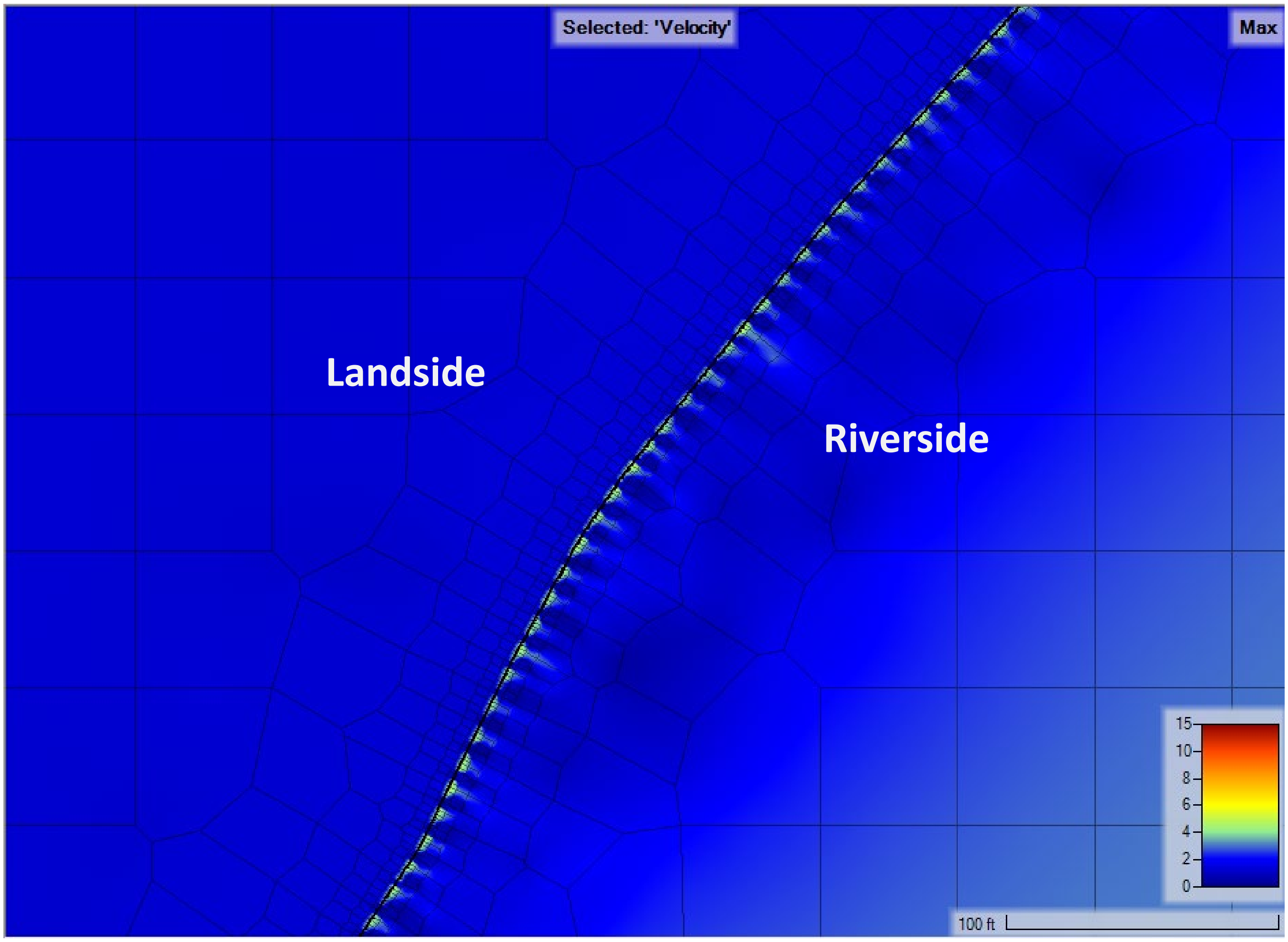

Figure 4.32. Maximum velocity plumes along eastern portion of fence

4.5.4.2 Hydraulic forces

In addition to velocity, other hydraulic parameters for determination of forces and moments on the bollard fence include head difference across the bollards (based on data displayed in Figure 4.17) and depth of flow on both sides of the fence. Figure 4.33 profiles base and top of the fence and maximum WSELs, showing that the fence is mostly submerged at the peak of the flood. Depth of flow and flow velocity affect bottom shear stress and erosion potential at the base of the fence as well.

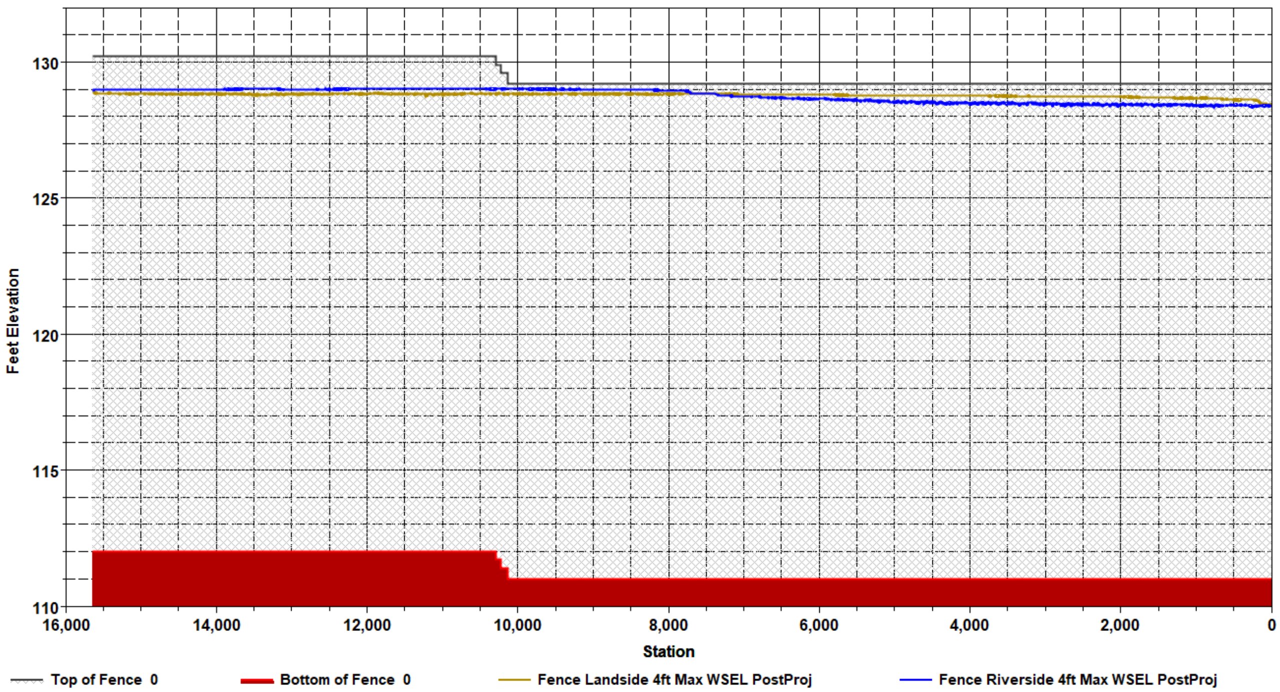

Figure 4.33. Profile showing fence base (dark red), top of bollards (gray), and maximum water surface elevations on land (orange) and river (blue) sides of fence

4.5.4.3 Erosion

Due to its location along the riverbank on the U.S. side of the border, erosion of the bank and base of the fence, whether caused or accelerated by the fence and associated features, is an important consideration in assessment of the potential for river meandering as well as to the geotechnical and structural stability of the fence itself. The paved road inside the fence likely affords some erosion protection on the landward side. Since construction, however, two kinds of erosion have been observed on the river side of the fence:

- Severe erosion of the base of the fence has occurred in some locations with rills and gullies as shown in Figure 4.34, indicative of high flow velocities through the fence openings from inside to the outside. As displayed in Figures 4.31 and 4.32, such conditions primarily occur along the eastern portion of the fence on the rising side of the flood hydrograph in this case. Without protective measures, for example armoring of the slope by riprap or soil reinforcement by natural vegetation, erosion of this type could expose and weaken the foundation of the fence over time.
- Large areas of vertical sloughing and caving of the bank have occurred, as shown in Figure 4.35. Bank erosion of this kind is not caused by high velocity flow, but by alternate and frequent saturation and drying of the riverbank, which can be caused by (1) average daily river stage fluctuations of 0.5 feet upstream of Anzalduas Dam, and (2) wakes generated by frequent high-speed river patrol boats, an example of which is shown in Figure 4.36. Riverbanks along both the western and eastern portions are equally subject to caving. Boat as well as wind-generated waves can also result in structural loading of the fence due to wave forces. The raising and steepening of the bank and removal of natural vegetation during fence construction may have

exacerbated bank erosion by some or all of the above-described mechanisms in comparison to pre-project conditions. Bank stabilization may be necessary to prevent river meandering and to ensure caving of the bank does not progress to the point of weakening the base of the bollard fence.

Figure 4.34. Severe erosion at base of fence

Figure 4.35. Severe bank caving on river side of fence

Figure 4.36. Rio Grande river patrol boat wake (Texas Department of Public Safety)

4.6 Hydraulic assessment – findings and conclusions

The TGR model is inadequate for simulation of 2D flows around and through the bollard fence, 30 percent of which is assumed to be obstructed by debris, specifically because (1) the fence is not represented with adequate resolution in the 2D terrain due to the 12:1 horizontal scale distortion, and (2) the virtual piers used in the model function more as guidewalls than as flow obstructions. The interpretation of the model results in the TGR report is not sufficiently comprehensive to accurately characterize the hydrodynamic impacts of the fence on flow exchange between the river and floodplain, deflection of river flow, and hydrodynamics of flow through the fence.

Designed to remedy the major deficiencies of the TGR model, the Arcadis hydraulic model was developed using the most recent version of HEC-RAS (version 6.0.0). Improvements to the model code, together with Arcadis-developed geographic information system (GIS) tools described in this report, enabled undistorted representation of the bollard fence at prototype scale within a variable 2D grid for more realistic simulation of flow hydrodynamics of the post-project condition. Due to the increased physical detail, model execution times were approximately 20 percent greater than the TGR model – on the order of 70 hours for simulation of the first 92 hours of the design flood – making simulation of both rising and recession sides of the flood impractical in this case.

The Arcadis model shows that the fence significantly impedes movement of water between the river and the floodplain behind the fence. Model results indicate that, overall, the fence significantly reduces conveyance from west to east within the model domain, creating as a result a large ineffective or ponding area behind the fence, head differentials between the river and the floodplain behind the fence, and high velocities of flow through the fence openings relative to flow velocity in the river channel.

The impedance to floodplain storage causes flow in the river along the fence to increase by up to 27 percent from pre-project conditions. Increased river channel flow indicates flow deflection and increased potential for migration of the river channel during high-flow events. The IBWC-designated measures of deflection shown in Table 4.1 also strongly indicate deflection toward the Mexico side of the river-floodplain system in three of the four profile lines.

The loss in floodplain conveyance capacity due to the fence causes reductions in freeboard along the Mission Levee to the north of the fence by up to 0.29 feet.

The Arcadis model indicates that maximum velocity of flow through unblocked bollards on the western portion of the fence reaches 7.9 fps, and exceeds 5 fps from the floodplain to the river on the eastern portion of the fence. These results contribute to structural loading of the bollards and indicate increased potential for scouring of the base of the fence in comparison to natural (pre-project) conditions.

Because the bollard fence is mostly submerged at the peak of the flood and produces head differentials of up to 0.25 feet across the fence, hydraulic loading on individual bollards could affect structural stability.

Severe erosion of the base of the fence has occurred in some locations with rills and gullies indicative of high flow velocities through the fence openings from inside to the outside, primarily occurring along the eastern portion of the fence on the rising side of the flood hydrograph. Without protective measures, erosion of this type could expose and weaken the foundation of the fence over time.

Since construction of the fence, large areas of vertical caving of the bank have been observed. With average daily river stage fluctuations of 0.5 feet upstream of Anzalduas Dam and wakes generated by frequent high-speed river patrol boats, the raising and steepening of the bank and removal of natural vegetation in construction of the fence may have exacerbated bank erosion. In addition, wind-generated waves during high-water conditions could add to structural loading of the fence due to hydraulic head and flow velocity.

5 Geotechnical Assessment of Bollard Fence

Bollard Fence, Mission, Texas

August 2021

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5.1 Geotechnical assessment – summary

5.1.1 Introduction

We prepared this geotechnical engineering assessment of design and construction of the foundation system of a bollard fence for the U.S. Attorney's Office, Southern District of Texas, U.S. Department of Justice. The 2.96-mile-long fence is located on the Texas bank of the Rio Grande between Anzalduas Park and Bentsen State Park south of Mission, Texas. The fence was constructed by Fisher in 2019-2020.

The location of the bollard fence and the site conditions are described in Section 2, and the purpose and scope of our engineering evaluation are presented in Section 3 of our report. The specific purpose of this geotechnical engineering assessment is to provide an expert opinion regarding design and construction of the fence's foundation system and to identify soil properties for use in the structural stability analysis of the fence. The assessment also includes providing an expert opinion regarding the fence's fitness for use considering the anticipated performance of the fence's foundation system.

5.1.2 Organization of this section

This section of the report is organized as follows:

- Section 5.2 describes site geology and soil conditions and summarizes our field exploration and laboratory testing results.
- Section 5.3 discusses general considerations for geotechnical issues and foundation design, presents applicable codes and standards, and provides a comparison of the Fisher fence with three similar fences.
- Section 5.4 discusses geotechnical and foundation considerations as specifically related to the Fisher fence.
- Section 5.5 presents our findings and conclusions regarding application of geotechnical considerations and foundation design to the Fisher fence.

5.2 Geology and soils

5.2.1 Regional geologic conditions

The University of Texas at Austin, Bureau of Economic Geology, mapped soils in the project area as Quaternary floodplain deposits consisting predominantly of silt and sand (Barnes et al. 1976). This description is consistent with more recent geological mapping including Moore and Richmond (1993) and Page et al. (2005). Figure 5.1 shows a portion of the Barnes et al. (1976) map that includes the project area.

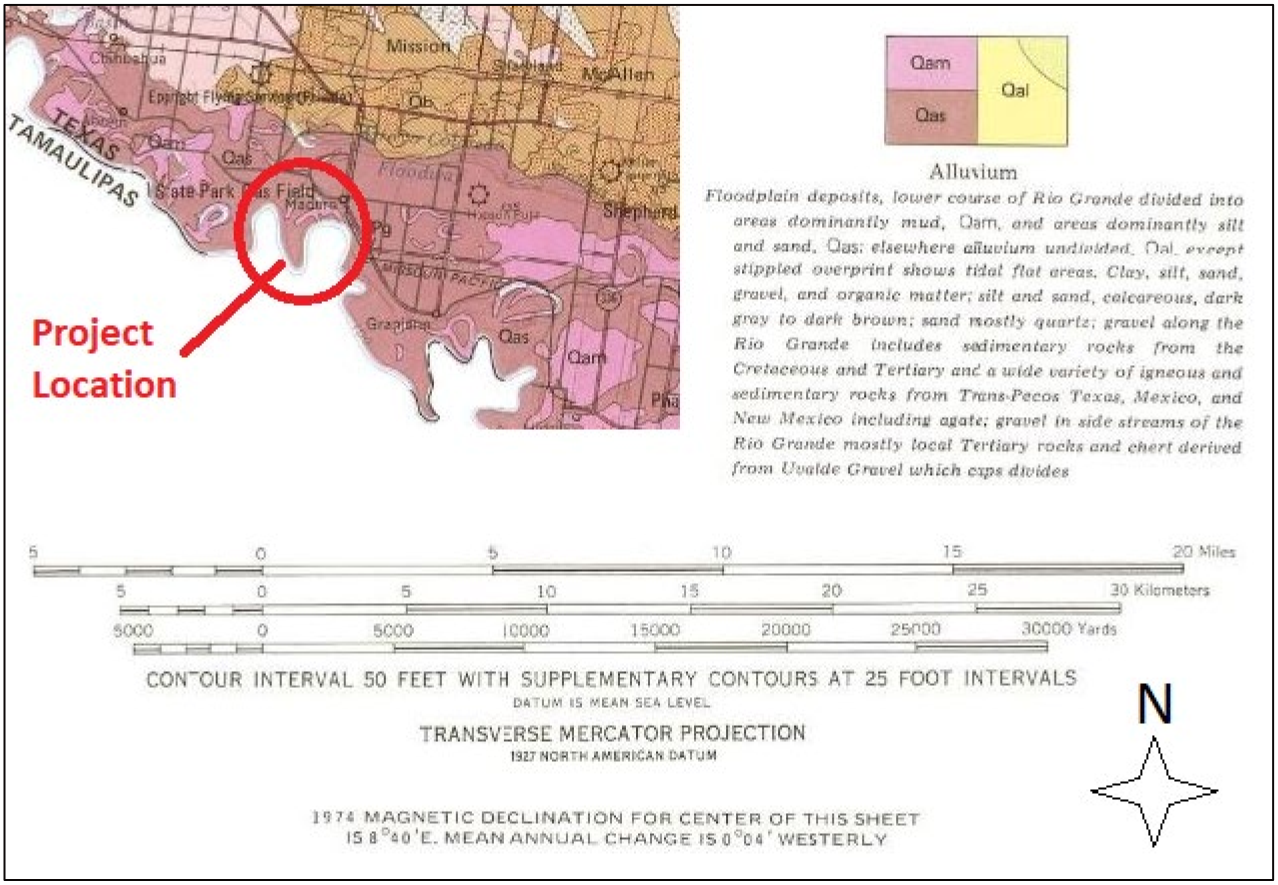

Figure 5.1. Geologic map showing surficial soils at and near the project location (modified from Barnes et al. 1976)

We prepared a site-specific soils map of the project area using USDA NRCS soil mapping data. This map and accompanying report are included in Appendix C. Results of the site-specific soil mapping indicate that most soils along the length of the fence comprise varying proportions of silt, clay, and fine sand.

The United State Geological Survey (USGS) 2018 Long-term National Seismic Hazard Map (USGS 2018) shows that the project location is mapped in the lowest seismic hazard zone for the U.S. Thus, earthquake effects are considered negligible. However, Page et al. (2005) mapped many faults near the project area. These faults are dip-slip growth faults that are generated when loose sediments slide into or toward the Gulf of Mexico basin. Accordingly, these faults are generally characterized by minimal displacement and are not associated with seismic activity. Figure 5.2 shows some of the mapped faults near the project location.

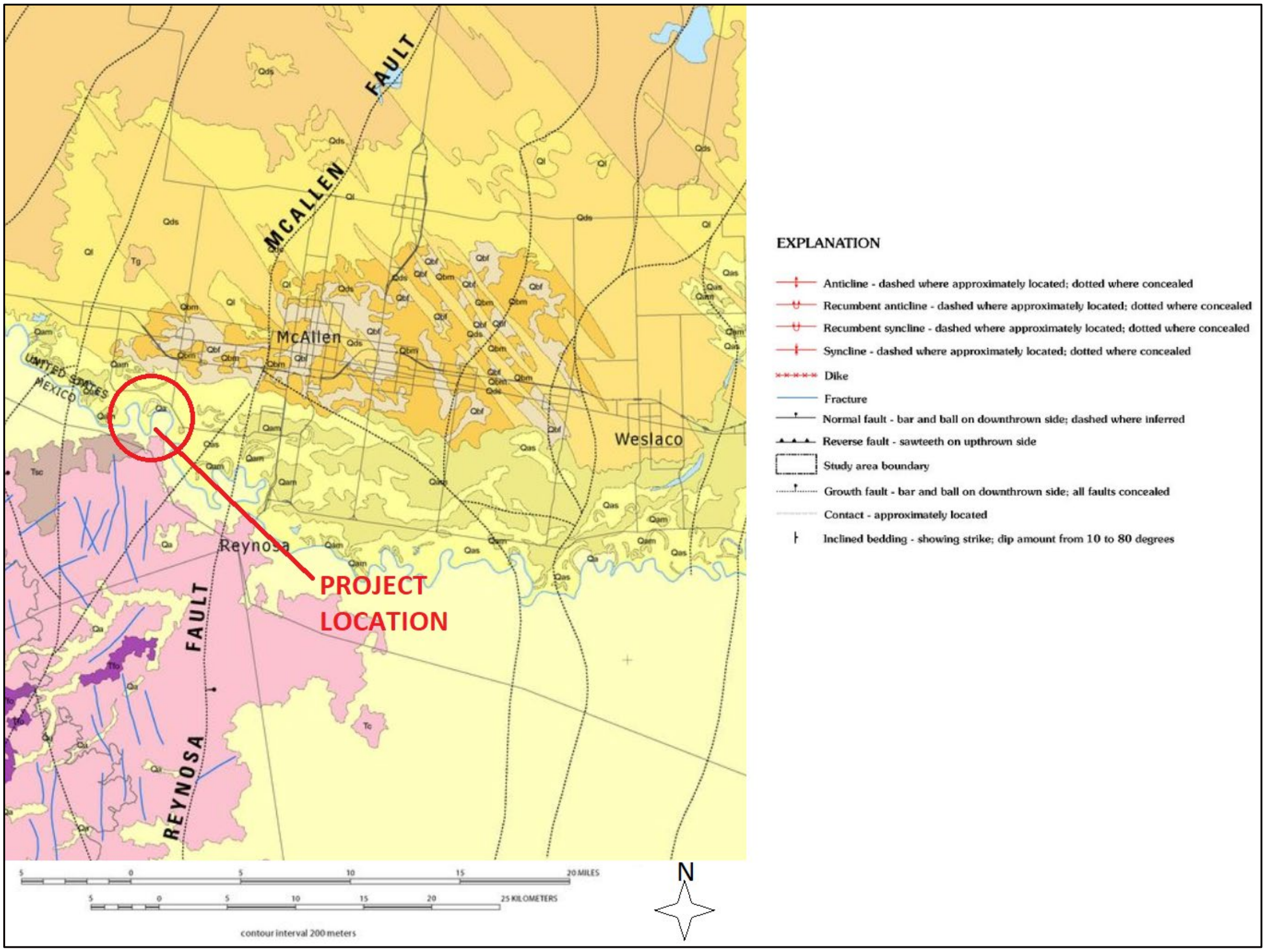

Figure 5.2. Faults near the project area (modified from Page et al. 2005)

5.2.2 Field exploration and testing

We completed a field investigation of the project site during the week of April 26, 2021. The field investigation included a site walk-through, measurement of key fence features, NDT of fence components, and excavation of 12 test pits between the fence and the river. Figure 5.3 shows approximate locations of the test pits. In general, we excavated the test pits immediately adjacent to the river side edge of the footing. Soil samples collected during excavation were tested in the laboratory for geotechnical properties. During excavation, the geotechnical testing subconsultant completed sand cone field density tests at depths of approximately 3 feet in each test pit. Details regarding the field investigation are provided in the Site and Subsurface Investigation Report included in Appendix B.

Figure 5.3. Approximate location of test pits excavated during site investigation in April 2021

5.2.3 Laboratory results and soil characterization

Soil samples obtained during the field investigation were tested for geotechnical index properties, strength, corrosivity, dispersivity, and compaction. The purpose of the laboratory test program was to generally characterize site soil conditions for identification of foundation design considerations and for use in the geotechnical and structural engineering analyses. The laboratory test results are summarized in Tables 5.1 through 5.4. Details regarding laboratory test results as well as ASTM International (ASTM) standard designations for laboratory tests conducted are included in Appendix B.

Site soil is generally a mixture of sand, silt, and clay. Soil encountered in the upstream half (approximately) of the project site generally has a greater sand content than soil in the downstream half (approximately) of the project site, which is generally fine-grained with less than 10 percent sand. Because dispersive clay is known to be present in the Rio Grande valley, we ran preliminary tests for dispersivity. The test results indicate that dispersive soil is present at locations along the fence alignment.

The field investigation and laboratory testing provided information for a geotechnical engineering assessment of the Fisher fence. However, a more comprehensive investigation and testing program is warranted for final design of a bollard fence like that constructed by Fisher. For example, additional exploration and testing are required to determine the areal limits of dispersive clay.

Table 5.1. Summary of geotechnical index properties

Test Pit ID	Depth (feet)	USCS	Moisture Content (%)	Grain Size Analysis			Atterberg Limits		
				% Gravel	% Sand	% Fines	LL	PL	PI
TP-1	3	CL	11.5	0.0	38.1	61.9	28	14	14
TP-2	3	SM	12.8	0.0	67.2	32.8	19	18	1
TP-3	3	SM	9.6	0.2	70.4	29.4	NP	NP	NP
P-4	3	CL	15.4	0.0	28.8	71.2	28	20	8
TP-5	3	SM	5.9	0.0	86.9	13.1	NP	NP	NP
TP-6	6	SC-SM	11.2	0.0	65.1	34.9	25	18	7
TP-7	3	CL	14.5	0.4	4.8	94.8	48	19	29
TP-8	3	CL	10.9	0.0	7.1	92.9	39	18	21
TP-9	3	CL	12.7	0.1	21.9	78.0	30	17	13
TP-10	3	CL	18.8	0.0	8.0	92.0	41	19	22
TP-11	3	ML	22.1	0.0	2.0	98.0	31	23	8
TP-12	3	SC-SM	12.7	0.0	57.6	42.4	24	19	5

Note: LL, PL, and PI are liquid limit, plastic limit, and plasticity index, respectively.

Table 5.2. Summary of density, strength, and compaction test results

Test Pit ID	Depth (feet)	Moisture Content (%)	Sand Cone Density (psf)	Proctor Max Density (psf)	Optimum Moisture Content (%)	Relative Compaction (%)	Direct Shear Phi (°)	C (psf)
TP-1	3	11.5	123.0	113.0	13.8	109	38.2	660
TP-2	3	12.8	88.9	107.4	14.4	83		
TP-3	3	9.6	87.3					
TP-4	3	15.4	84.7					
TP-5	3	5.9	90.2					
TP-6	3						38.8	419
TP-6	6	11.2	97.9	105.1	16.1	93		
TP-7	3	14.5	87.5	99.3	19.9	88	27.8	773
TP-8	3	10.9	62.0					
TP-9	3	12.7	92.4					
TP-10	3	18.8	76.1					
TP-11	3	22.1	95.5	105.1	16.1	91	35.5	662
TP-12	3	12.7	96.2	106.4	14.3	90		

Note: Refer to Appendix B for Proctor test results.

Table 5.3. Summary of corrosivity test results

Test Pit ID	Depth (feet)	USCS	Corrosivity Testing pH	Sulfates (ppm)	Sulfides (ppm)	Chlorides (ppm)	RedOx (mV)	Total Salts (ppm)	Resistivity (ohm-cm)
TP-1	3	CL	7.3	524	nil	119	+338	1605	1239
TP-5	3	SM	7.5	11	nil	14	+347	732	4337
TP-8	3	CL	7.5	349	nil	75	+335	1455	1342
TP-12	3	SC-SM	7.6	197	nil	19	+337	826	2994

Note: RedOx is oxidation-reduction potential by ASTM D1498.

Table 5.4. Summary of dispersivity test results

Test Pit ID	Depth (feet)	USCS	Dispersivity Testing		
			Crumb	Pinhole	Double Hydrometer
TP-2	3	CL		Moderate	
TP-4	3	CL			Non-dispersive
TP-5	3	SC-SM		Moderate	
TP-8	3	CL	Intermediate		
TP-9	3	CL		Non-dispersive	
TP-10	3	CL	Intermediate		
TP-11	3	ML			Non-dispersive

5.3 Foundation design considerations

5.3.1 General foundation design criteria

American Society of Civil Engineers (ASCE)/Structural Engineering Institute (SEI) 7-10 “provides minimum loads, hazard levels, associated criteria, and intended performance goals for buildings, other structures, and their non-structural components...” (ASCE/SEI 2010). ASCE 7 defines loads as follows:

> ***Loads:*** *Forces or other actions that result from the weight of all building materials, occupants and their possessions, environmental effects, differential movement, and restrained dimensional changes. Permanent loads are loads in which variations over time are rare or of small magnitude. All other loads are variable loads (see also “nominal loads”).*
>
> ***Nominal Loads:*** *The magnitudes of the loads specified in this standard for dead, live, soil, wind, snow, rain, flood, and earthquake loads.*
>
> ***Service Loads:*** *Loads imparted on a building or other structure because of (1) self-weight and superimposed dead load, (2) live loads assumed to be present during normal occupancy or use of the building or other structure, (3) environmental loads that are expected to occur during the defined service life of a building or other structure, and (4) self-straining forces and effects. Service live loads and environmental loads for a particular limit state are permitted to be less than the design loads specified in the standard. Service loads shall be identified for each serviceability state being investigated.*

For the purposes of this report, service loads include all dead, live, soil, wind, snow, rain, flood, and earthquake loads. Flood loads include hydrostatic, hydrodynamic, and impact from floating debris.

Two basic criteria serve to define successful foundation design: 1) meeting the standard of care; and 2) fitness for use. The standard of care is defined as follows: In the performance of services, a design professional is required to exercise the degree of care, skill, and diligence ordinarily exercised by other members of the profession performing under the same or similar circumstances as existing at the time the services are performed by the design professional (Hatem 1998). The standard of care is not absolute; it must be tailored to meet unique circumstances and conditions.

Because each project is characterized by unique features (e.g., location, site conditions, circumstances, time), the standard of care is likewise unique for each project. Determining the standard of care for a particular project can be a painstaking process involving detailed engineering analyses, codes and standards review, careful research into precedent with similar projects and similar circumstances, and extensive interviews with subject matter experts. Determining the standard of care was not included in the scope of this geotechnical assessment.

Fitness for use means that the performance of the foundation system will enable the structure to be safely used for its intended purpose, i.e., the foundation will perform its function of economically and efficiently transmitting service loads to the supporting soil without failure, unacceptable deformation, or need for extraordinary maintenance or repairs to preserve its integrity. Design and performance criteria to prevent failure and minimize deformation-related problems are typically defined by building codes and by commonly accepted industry standards and practices. Criteria defining maintenance requirements, anticipated need for repairs, and useful life are typically provided by the structure owner.

Our geotechnical engineering assessment is intended to form the basis of an expert opinion regarding the fitness for use of the fence’s foundation system with respect to design and performance criteria.

5.3.2 Foundations for the Fisher fence

The foundation transmits the weight of the structure as well as loads applied to the structure to the ground. If the near-surface soil can support the structure and applied loads, *shallow* foundations comprising spread, continuous, or mat footings placed a few feet below grade are typically used. If the near-surface soil is weak or compressible, or if applied loads exceed the load-carrying capacity of the near-surface soil, *deep* foundations such as piles or piers are used to transmit service loads to deeper, more competent strata. Deep foundations are also used if the near-surface soil may be removed by erosion. Some structures bear on rock using either shallow or deep foundations, and some structures bear on soil that has been strengthened using various soil improvement techniques.

The fence was constructed in 2019-2020. Fisher provided two drawings showing foundation design for the fence, the results of laboratory Proctor compaction tests performed during construction, and soil data. Fisher provided no documentation describing how soil data were developed, and no discussion of geotechnical considerations for foundation design.

The foundation system, shown in Figure 5.4, comprises a continuous, T-shaped reinforced concrete footing. The flanges of the T are 8 feet wide by 1 foot deep and the top elevation is at the ground surface. The stem is 1 foot 4 inches wide and extends 2 feet 2 inches deep below the flanges (i.e., 3 feet 2 inches below the existing ground surface). The stem of the T is poured integrally with the flanges and is asymmetrically placed as indicated in Figure 5.4. The bearing area of the shallow foundation is 8 square feet per foot of fence.

Key considerations for design of foundations, whether shallow or deep, depend on the structural capacity of the foundation elements as well as the behavior of the soil surrounding the foundations under loading. The objective of foundation design is to select an economical foundation system that will support service loads without causing shear failure of the soil, nor excessive deformations that will damage the structure’s fitness for use.

The soil supporting the foundation must be able to safely carry service loads without shear failure (referred to as the soil’s bearing capacity) and without detrimental deformation. Excessive foundation settlement may occur if soft, weak soil is compressed by service loads, or if cohesive soil is consolidated by service loads (a process of slowly squeezing water from the soil mass that causes settlement as the soil is compressed or consolidated). On the other hand, certain soil types can expand as water is added (e.g., changes in water content from seasonal wetting and drying) causing detrimental vertical deformation in the upward direction.

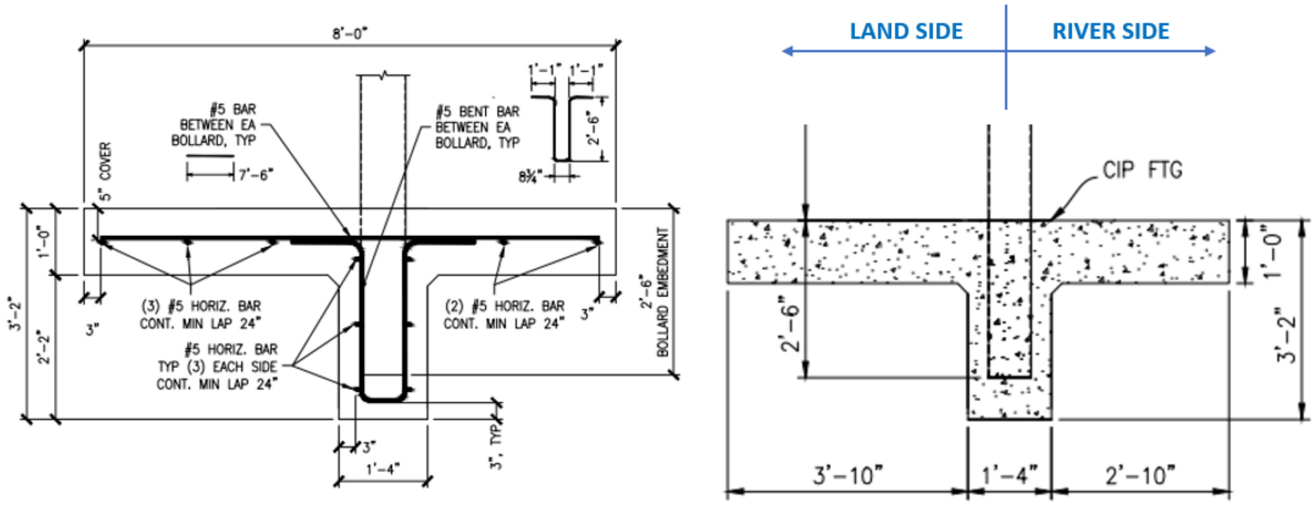

Figure 5.4. Sections of the fence foundation (Fisher 2019)

The soil surrounding the foundation also contributes to the performance of the foundation system. For example, the foundation must resist horizontal loads including loads from wind, flowing water, and debris impact. The applied horizontal loads are resisted by friction between the base of the foundation element and the supporting soil, and by the soil's lateral resistance (referred to as passive resistance) as the side of the foundation element is pushed against the soil face. In addition, the bearing capacity of the soil generally increases as the depth from the ground surface to the base of the foundation element increases. If soil alongside foundation elements is loosened (e.g., by burrowing animals or by seasonal moisture change), or if the soil is removed (e.g., by erosion), the performance of the foundation system can be compromised leading to excessive deformation or premature failure.

Appropriate foundation design requires:

- Information about site geology and soils.
- Identification of service loads and structural design criteria regarding issues such as tolerable deformation.
- An understanding of the anticipated behavior of site geotechnical conditions in response to construction activities, and to service loads from the planned structure and its use.
- Geotechnical engineering analyses to develop foundation design criteria that provide an adequate factor of safety[1] (FOS) against failure (typically defined by codes, design standards and guidance, and precedence) and ensure that structural deformations are within tolerable limits (typically defined by precedence and the owner's preference).
- Identification of other special geotechnical considerations including but not limited to fill placement and stability, presence of dispersive soil, corrosion, and protection against adverse environmental conditions such as frost action or erosion.

Depending on the complexity of site geology and soils and on the structural performance requirements, appropriate foundation design can entail thorough field investigations and laboratory testing to characterize site

[1] Factor of safety is defined as the ratio of forces tending to prevent failure divided by forces tending to cause failure. A factor of safety of 1.0 is considered incipient failure.

soil and its anticipated behavior, comprehensive geotechnical engineering analysis, and specialized construction. Foundation design is guided by experience derived from precedent, geotechnical engineering analysis and judgment, and application of guidance and requirements from building codes and standards. The following sections of this report describe issues typically considered in foundation design of structures such as the fence, beginning with a discussion of applicable codes and standards.

5.3.3 Design codes and standards

Section 2.2.1.1 of Version 5 of the *Tactical Infrastructure Design Standards* (U.S. Department of Homeland Security [DHS] 2020) specifies that the foundation system for a bollard fence must be designed based on site-specific geotechnical recommendations and that it must be at least 6 feet deep to meet the under-dig criterion. In addition, the fence must be protected from scour and erosion, although the 6-foot under-dig requirement may be sufficient for both stability and scour protection. We did not have access to earlier versions of the *Tactical Infrastructure Design Standards*; hence, we do not know if the 6-foot-under-dig requirement was in effect when the Fisher fence was designed and built.

We reviewed DHS and Texas design and construction practices to identify requirements and guidelines for bollard fence design and construction. Section 1.3 of the DHS Border Wall Program – Program Management Plan (undated) states "The Wall design shall meet all relevant codes and requirements associated with ASCE 7, ACI 318."

- ASCE 7 (2010) provides guidance for calculating forces and load combinations for designing structures. Guidance in ASCE 7 is relevant to analysis and design of structural elements and minimal guidance is provided specific to foundation design.
- American Concrete Institute (ACI) 318 (2014) describes design requirements for concrete.

In general, Texas uses the International Building Code (IBC) and allows local municipalities to adopt amendments to the IBC for specific local conditions. We found no amendments to the IBC adopted by the city of Mission, Texas, nor Hidalgo County. Chapter 18 of the IBC includes requirements for foundations and soils, which are discussed later in this section:

- Quality control during construction
- Expansive soils
- Presumptive allowable bearing pressure
- Coefficient of friction along the base of foundation

5.3.4 Review of similar fences

We compared geotechnical aspects of the Fisher fence with geotechnical reports and construction drawings for three other bollard fences in Texas serving the same general purpose. Our review is not comprehensive enough to establish a standard of care that Fisher or others might be required to meet. It does, however, provide valuable insight as to how foundations for other fences have been designed and constructed in Texas.

5.3.4.1 Segment K-2A

In a 2008 geotechnical report, Terrane Engineering Corporation (Terrane) described soil and site conditions and made geotechnical and foundation recommendations for the K-2A segment of a bollard fence near El Paso, Texas (Terrane 2008). Terrane's work included:

- Drilling 37 exploratory borings to depths of 26.5 feet along a 9-mile segment of the border.
- Conducting geotechnical laboratory analyses on samples collected from the borings (moisture content, dry density, gradation, plasticity index, standard Proctor, pH, resistivity, and soluble chlorides and sulfates).
- Preparing recommendations for foundation design including lateral earth pressures, earthworks (placement and compaction of fill, backfill, and roadway materials), construction observation and testing, and corrosivity.

In general, soil and site conditions for the K-2A fence are like those at the Fisher fence. Terrane:

- Provided recommendations for both shallow footings and deep foundations.
- Used soil strength of 34 degrees for analysis and recommended using a base friction factor of 0.4, reduced to 0.3 if used in conjunction with passive pressures.
- Recommended compacting fill and backfill to 95 percent relative compaction and compacting roadway materials to 100 percent relative compaction based on standard Proctor (ASTM D698).
- Reported that soils have high corrosion potential.

We also reviewed record drawings for Fence Project K-2A (RJM Architecture 2010). The K-2A fence varies in height with a minimum height of 18 feet. The foundation comprises drilled shafts 30 inches in diameter with a minimum depth of 10 feet 9 inches as shown in Figure 5.5. Shafts were constructed at each full height and intermediate post, which are generally 5 feet on center. Additional foundation details were provided at special features such as pedestrian and vehicle slide gates. The fence is located on the paved bank of an existing canal at the base of a paved slope, and the designers did not specify special erosion protection.

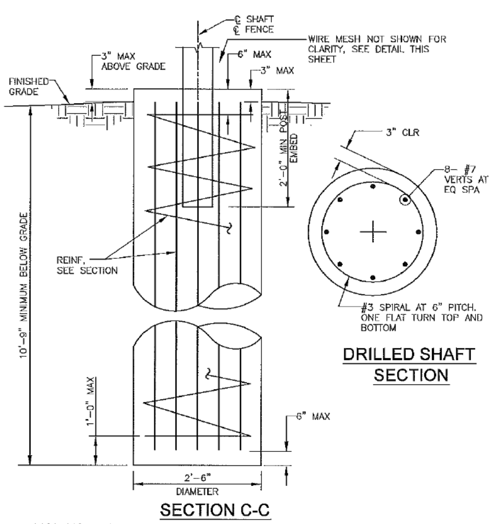

Figure 5.5. Typical K-2A fence foundation detail (RJM Architecture 2010)

5.3.4.2 El Paso pedestrian fence replacement

PSI prepared a geotechnical report for replacement for 17.4 miles of primary pedestrian fence at El Paso Segment D-3 (PSI 2020). PSI's work included:

- Drilling 184 soil borings spaced at approximately 500 feet to depths of 25 to 40 feet.
- Conducting geotechnical laboratory analyses on samples collected from the borings (moisture content, dry unit weight, modified Proctor, gradation, Atterberg limits, California Bearing Ratio [CBR], and corrosivity).
- Preparing recommendations for:
  - General site development and subgrade preparation.
  - Earthworks and fill compaction.
  - Foundation design including allowable bearing pressures, passive resistance, uplift capacities, and estimated movements.
  - Roadways.
  - Seismic design.
  - Excavation and drainage considerations.

In general, soil and site conditions at Segment D-3 are like those at the Fisher fence. PSI reported:

- That, based on laboratory testing of shrink/swell potential, potential vertical movement was estimated to be less than 1 inch.
- Recommendations for subgrade preparation and compaction of fill soil to at least 95 percent relative compaction using modified Proctor (ASTM D1557). For soil with a plasticity index greater than 25, PSI recommended achieving 94 to 98 percent relative compaction according to modified Proctor.
- That drilled shafts be designed for an allowable skin friction of 275 pounds per square foot (psf) and allowable end bearing of 3750 psf. PSI recommended neglecting skin friction in the upper 5 feet of the shaft, presumably to account for the possibility of weaker soil near the ground surface.

The new Segment D-3 fence is 30 feet high and will be built on a 2-foot-thick concrete pile cap placed on an existing concrete slab that is supported on drilled shaft foundations as shown in Figure 5.6. The existing drilled shaft foundations are generally 30 inches diameter spaced at 5 feet on center. The depths of the shafts are not shown on the plans. However, PSI reports that the existing shafts are 10 feet 9 inches below the existing ground surface (PSI 2020). The Segment D-3 fence is supported on a concrete footing located on a bench of a paved slope, and the designers did not specify special erosion protection.

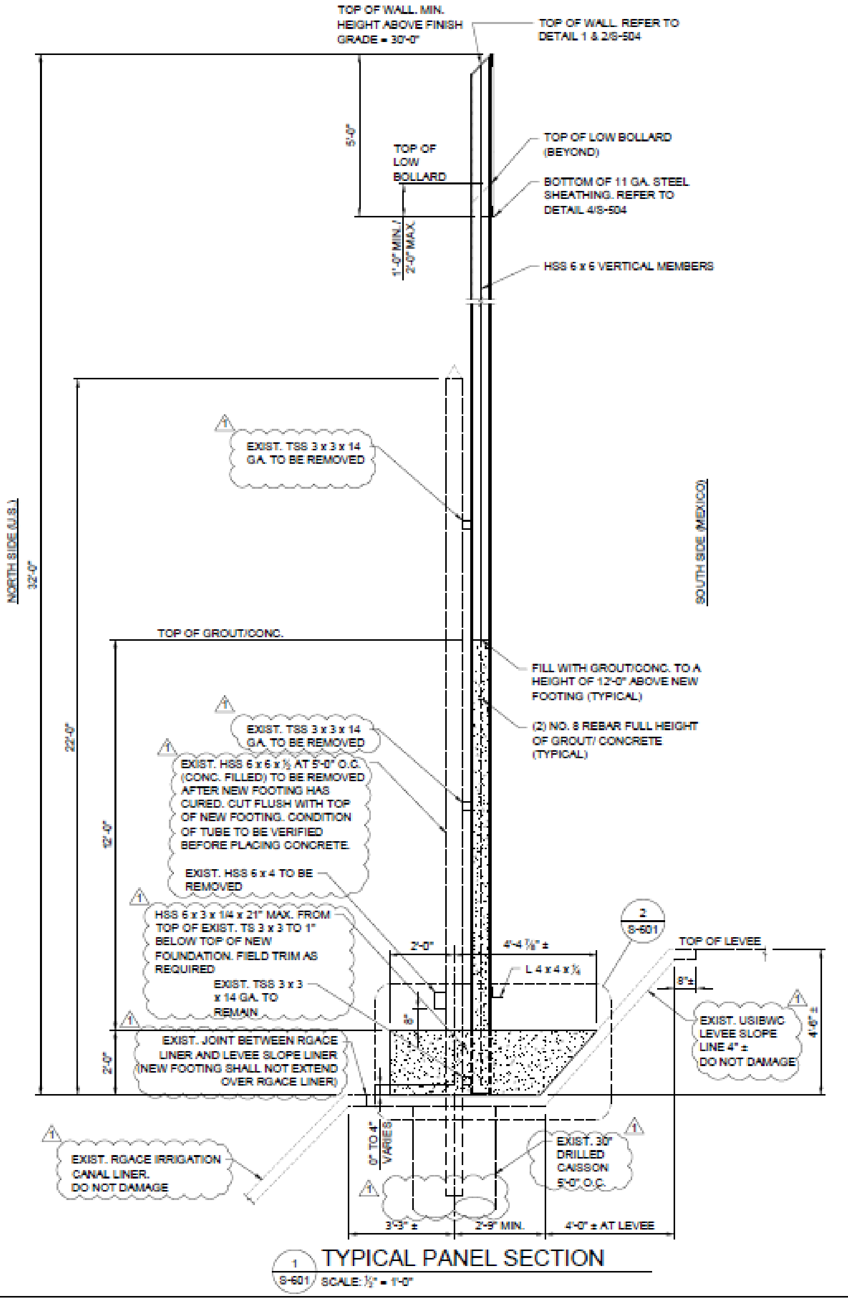

Figure 5.6. Fence foundation for Segment D-3, El Paso, TX (Benham 2020)

5.3.4.3 DHS Segment O-4 B, USIBWC levee

L&G Engineering Laboratory (L&G) prepared a geotechnical engineering report for the DHS Segment O-4 B, USIBWC Mission Levee Improvements Project in Hidalgo County, Texas (L&G 2009). The Segment O-4 B project comprises realignment of an existing levee, construction of retaining walls and box culverts, and installation of a bollard fence along the top of the levee. L&G's work included:

- Completing five borings ranging from 50 to 100 feet deep with three additional offset borings for sample collection ranging from 10 to 12 feet deep. In addition, the report includes four borings from previous investigations ranging from 35 to 105 feet deep.
- Performing soil index testing (moisture content, Atterberg limits, and sieve and hydrometer analyses) and strength and consolidation testing.
- Performing geotechnical engineering analysis for slope stability, bearing capacity, settlement, and seepage.
- Providing geotechnical engineering recommendations for drainage, site preparation, and fill placement.

Appendix G of the L&G report is a report prepared by PSI (dated November 7, 2009) on behalf of L&G for a portion of the Segment O-4B project that includes the bollard fence. Neither the L&G nor the PSI report includes specific geotechnical analyses and recommendations for foundation design of the bollard fence. Although no specific foundation recommendations were included, the fence designer presumably had access to the geotechnical information in the reports to guide the design of foundations. There is, however, no evidence that this was done.

Appendix A of the PSI report is a set of construction drawings titled "2009 USIBWC Mission Levee Improvements Project, DHS Segment O-4 B," prepared by Dannenbaum Engineering Company, McAllen, Texas, and DL Inc., Westlaco, Texas. The construction drawings include 74 sheets, which are undated. Sheets S02-1 through S02-5 include elevation views, sections, and details of the bollard fence. The bollard fence for this project is shown in Figure 5.7. The bollards are 6-inch square steel posts, 18 feet high, spaced 4 inches apart. The bollards are located at the shoulder of a 3H:1V (horizontal:vertical) slope and are supported on a continuous reinforced concrete footing that is 1 foot 8 inches wide by 10 feet deep. As the bollards are at the crest of the levee, the designers did not specify special erosion protection.

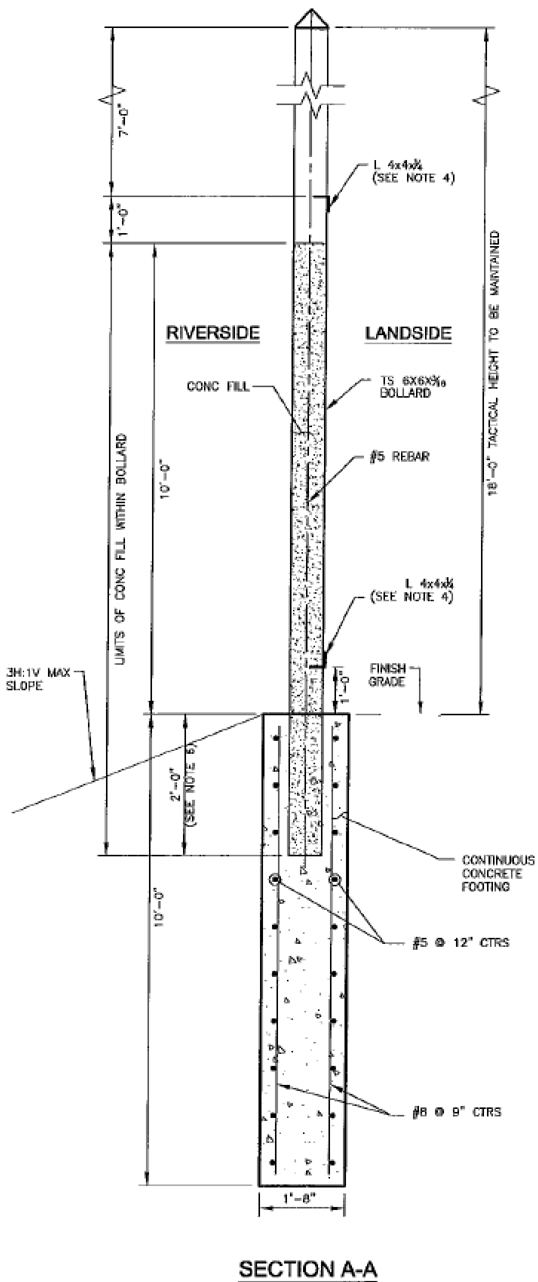

Figure 5.7. Fence foundation for Segment O-4 B, Mission Levee (L&G 2009)

5.3.5 Comparison of geotechnical considerations

A summary comparison of how geotechnical considerations were handled by Fisher versus those in the K-2A fence, Segment D-3 fence, and Segment 0-4 B fence is shown in Table 5.5.

Table 5.5. Summary comparison of geotechnical considerations

Geotechnical Considerations	Fence K-2A	Fence Segment D-3	DHS Segment 0-4 B	Fisher
Main project elements	New fence, gates, and roadway. Fence height varies, 18 feet minimum.	New 30-foot-high fence and roadway	Levee realignment, retaining walls, culverts, and new 18-foot-high fence	New 18-foot-high fence and roadway
Project length	9 miles	17.4 miles	Approximately 532 feet	2.96 miles
Geotechnical exploration	37 exploratory borings to depths of 26.5 feet	184 soil borings to depths of 25 to 40 feet	9 exploratory borings, 35 to 105 feet deep	None reported
Soil laboratory testing	Moisture content, dry density, gradation, plasticity index, and standard Proctor	Moisture content, dry density, modified Proctor, gradation, Atterberg limits, and CBR	Moisture content, Atterberg limits, sieve and hydrometer analysis, strength, and consolidation	Standard Proctor tests
Corrosivity testing	pH, resistivity, chloride, and sulfate testing	Soil resistivity, chloride, and sulfate testing	None reported	None reported
Geotechnical analysis and recommendations	Foundation design, lateral earth pressures, earthworks, roadway materials, construction observation and testing	Site development, subgrade preparation, earthworks, bearing pressures, passive resistance, uplift capacities, estimated movements, and roadways	None reported for the new fence	None reported
Foundation depth	10’-9”	10’-9”	10’	3’-2”
Foundation construction	Drilled shafts 30 inches in diameter, 5 feet on center	Fence is supported on a 2-foot-thick concrete pile cap founded on drilled shafts 30 inches in diameter, 5 feet on center	Continuous reinforced concrete footing 1 foot 8 inches wide	Shallow, continuous footing, no depth of burial
Seismic considerations	None reported	Included	None reported	None reported
Corrosivity recommendations	Included	Included	None reported	None reported
Check for dispersive soil	None reported	None reported	None reported	None reported
Fill and backfill compaction specification	Standard Proctor (ASTM D698)	Modified Proctor (ASTM D1557)	Standard Proctor (ASTM D698)	None reported
Foundation depth and protection from erosion	Drilled shafts are 10 feet 9 inches deep and are protected by existing concrete canal lining. No special erosion protection was provided.	Fence is supported on concrete pile cap supported on drilled shafts that are 10 feet 9 inches deep. Fence is on a paved slope; no special erosion protection was provided.	Drilled shafts are 10 feet deep. Bollards are located at the crest of a levee; no special erosion protection was provided.	Protected by concrete roadway on land side; no special protection provided on river side for initial construction.
Construction plans	17 sheets	24 sheets	5 sheets for the fence	2 sheets

5.4 Geotechnical engineering analysis

This section discusses geotechnical issues that, in our opinion, should be considered for the design and satisfactory long-term performance of foundations for a bollard fence like the fence designed and built by Fisher. This section also presents our analysis of geotechnical design criteria that should be used in structural stability analyses of the Fisher fence. We based our geotechnical engineering analyses on site observations, geotechnical investigation and laboratory results, and engineering judgment.

5.4.1 Embankment stability and soil considerations

Because the constructed foundation is essentially slab-on-grade, soil beneath the wall and the riverbank slope must be adequately stable for continued functionality. The following attributes of slope and subgrade stability are important for the bollard fence:

- **Embankment stability.** Stability of the riverbank slope is important because slope instability could cause fence failure. As needed, embankment stability is typically analyzed with specially designed computer software using site-specific geometry and groundwater conditions, and soil strength determinations based on the results of field and laboratory testing.
- **Compaction.** Compaction of underlying soils is important to provide strong, deformation-resistant support to the bollard fence structure. Section 1803.5.8 of the IBC requires that the maximum soil density and optimum water content be determined for the subgrade material. Common practice is to specify compacting the subgrade to approximately 90 to 95 percent of its maximum density as determined by a laboratory standard. Section 1803.5.8 of the IBC also requires that field tests be used to measure in-place dry density and relative compaction of subgrade materials.
- **Corrosivity.** The chemical composition of soil and porewater may cause corrosion and deterioration of concrete. The corrosion potential of soils can be determined in the laboratory and is related to pH, sulfate content, electrical resistivity, and/or chloride content. ACI 318 categorizes corrosion potential as a function of sulfate content.
- **Dispersivity.** Dispersive clay exhibits unique properties and can deflocculate and be rapidly eroded and carried away by waterflow. In some cases, dispersive clay can deflocculate in standing water. The potential for dispersivity is characterized using the crumb test, the pinhole test, and the double hydrometer test.
- **Settlement.** Settlement is often a critical factor for geotechnical design of a structure. Design and performance criteria to minimize settlement-related problems are typically defined by building codes, by precedence, by commonly accepted industry standards and practices, and by owner preference.
- **Potential vertical rise.** Expansive soils can damage structures. Texas has expansive soils that shrink and swell as a function of water content. Section 1803.5.3 of the IBC provides criteria to identify expansive soils.
- **Seismicity.** Special geotechnical and structural design details are often required in areas with moderate to high seismicity.

5.4.2 Foundation design considerations

We identified soil data for use in the structural analysis by review of field and laboratory test data (Appendix B); by information and guidance in codes and standards, particularly IBC; by observations made of the performance of the existing fence; and by engineering judgment.

Using field soil classifications, we selected samples for strength testing by direct shear in the laboratory. Direct shear tests of alluvial sediment indicate that internal friction angle and soil cohesion range from 39 to 28 degrees and 400 to 773 psf, respectively (Appendix B). Because the samples selected for strength testing were primarily fine-grained soil, they may not be representative of foundation soil that has a greater fraction of sand-sized particles. Using engineering judgment, we selected shear strength properties for structural analysis that are considered reasonable for use over the length of the bollard fence. We selected an angle of internal friction, 35 degrees, that is generally higher than friction angles usually observed in alluvial deposits, and disregarded cohesion because of the widespread presence of sandy (coarse-grained) material within the project limits.

5.4.2.1 Foundation bearing pressure and depth

Bearing capacity quantifies a soil's ability to support service loads that are transmitted to the foundation. The bearing capacity of a shallow foundation can be calculated and compared to actual loads to determine a bearing capacity FOS. More commonly, shallow foundations are designed using presumptive allowable bearing pressures such as those provided in Table 1806.2 of the IBC.

Because the fence is a lightly loaded structure and there is no evidence of bearing issues observed on site, we consider that the presumptive value cited in the IBC is reasonable and may be used for foundation design. Table 1806.2 of the IBC indicates that an allowable bearing pressure of 1500 psf may be used for the soil and conditions observed at this site.

The minimum depth of shallow foundations is typically governed by the following considerations:

- The foundation should be deep enough so that it bears on soil with adequate bearing capacity.
- The foundation should be deep enough so that adequate earth pressures are available to resist applied lateral loads.
- The foundation should have adequate soil cover to provide required uplift resistance, if needed.
- The foundation should be deep enough so that if soil adjacent to the foundation is removed or loosened by erosion, enough soil remains in place to provide adequate lateral and uplift resistance.
- The foundation should be located below the depth to which the soil is subject to seasonal volume changes caused by alternate wetting and drying, or that may be weakened by root holes or cavities produced by burrowing animals.
- The minimum foundation depth should conform to applicable codes and standards requirements, to common practice in the local area, and to experience gained from precedent.

5.4.2.2 Lateral resistance

The capacity of a shallow foundation system to resist applied lateral loads is provided by 1) frictional resistance along the base of the foundation, plus 2) passive resistance of the soil against the side of the foundation, less 3) the active pressure of the soil acting on the foundation opposite the side providing passive resistance. Using a soil strength of ø = 35 degrees as interpreted from laboratory test results, the friction factor, passive resistance, and active pressures appropriate for the Fisher foundation are as follows:

- The friction factor along the base of the foundation is tan ø = 0.70. It is common practice to include an FOS between 2 and 3 resulting in a friction factor of between about 0.25 to 0.35 for use in design. Table 1806.2 of the IBC suggests using a friction factor of 0.25 for the type of soils anticipated at this site.

- The passive resistance is given by K_P x ɣz per foot of fence, where K_P is the coefficient of passive earth pressure, ɣ is the density of soil, and z is the depth to the base of the footing. When computing passive resistance, the upper 1 foot to 2 feet of soil is usually neglected because the soil in this zone may be removed by erosion or may be compromised by loosening from seasonal moisture change or by animal burrows.

  The soil unit weight is 115 pounds per cubic foot (pcf) and for ø = 35 degrees, K_P = 3.69. Neglecting the upper 1 foot of soil, the passive resistance of the Fisher foundation is 920 psf per foot of fence. The resultant of the passive resistance is 997 pounds per foot of fence applied at a point that is 8.67 inches above the base of the foundation.

- The active pressure acting on the foundation is given by K_A x ɣz per foot of fence, where K_A is the coefficient of active earth pressure, ɣ is the density of soil, and z is the depth to the base of the footing (in the active case, the upper 1 to 2 feet of soil is not neglected).

  The soil unit weight is 115 pcf and for ø = 35 degrees, K_A = 0.271. The active pressure against the Fisher foundation is 99 psf per foot of fence. The resultant of the active pressure is 496 pounds per foot of fence applied at a point that is 12.67 inches above the base of the foundation.

5.4.2.3 Comparison with design information provided by Fisher

Fisher provided soil data shown in Table 5.6, which also shows data we recommend using in the structural analysis of the bollard fence. Fisher provided no discussion or reasoning by which the soil data were derived.

Table 5.6. Comparison of soil data

Soil Data	Fisher	Arcadis
Allowable bearing pressure (psf)	3,000	1,500
Soil friction angle (degrees)	32	35
Coefficient of passive pressure, K_P	3.25	3.69
Coefficient of active pressure, K_A	0.307	0.271
Soil density, heel (pcf)	107	115
Soil density, toe (pcf)	107	115
Friction coefficient	0.4	0.25
Soil height to ignore for passive pressure (inch)	12	12

5.4.3 Erosion protection considerations

The Fisher fence is located on the banks of the Rio Grande about 8 to 20 feet from the normal water's edge. In this location, the fence and its foundation will be affected by floods and high-water events on the river. The fence and its foundation will also be affected by precipitation runoff. The site soil is erodible, and laboratory testing indicates that dispersive soil is also present in some areas of the fence.

Because of soil properties and the presence of dispersive clay, erosion by flowing water from high-intensity rainfall, floods, and high-water events could remove soil and compromise the structural integrity or stability of the fence. To reduce the risk of soil removal, the fence should have properly designed erosion and scour protection using cobbles or riprap with appropriate filters, or other revetment to protect the foundation. In addition, erosion protection or other techniques should be used to contain and filter the dispersive soil and protect it from flowing water.

The permissible values of velocity should be determined so that damage exceeding normal maintenance will not result from any flood that could be reasonably expected to occur during the service life of the fence. The following table shows suggested maximum permissible mean channel velocities for various channel materials (USACE 1994), which may be used to guide design of erosion protection measures.

Table 5.7. Suggested maximum permissible mean channel velocities (USACE 1994)

Channel Material	Mean Channel Velocity, fps
Fine Sand	2.0
Coarse Sand	4.0
Fine Gravel[1]	6.0
Earth	
Sandy Silt	2.0
Silt Clay	3.5
Clay	6.0
Grass-lined Earth (slopes less than 5%)[2]	
Bermuda Grass	
Sandy Silt	6.0
Silt Clay	8.0
Kentucky Blue Grass	
Sandy Silt	5.0
Silt Clay	7.0

Channel Material	Mean Channel Velocity, fps
Poor Rock (usually sedimentary)	10.0
Soft Sandstone	8.0
Soft Shale	3.5
Good Rock (usually igneous or hard metamorphic)	20.0

Notes:
1. For particles larger than fine gravel (about 20 millimetres (mm) = 3/4 in.), see Plates 29 and 30.
2. Keep velocities less than 5.0 fps unless good cover and proper maintenance can be obtained.

5.5 Findings regarding the Fisher fence

Our findings and conclusions regarding the fence's foundation system are based on our geotechnical assessment of whether the foundation system satisfies fitness for use. Failure to meet fitness for use implies that the structure and its foundation system will be subject to unexpected maintenance needs, service interruptions, more rapid deterioration, or outright failure. Even though soil data were identified, Fisher provided no documentation indicating that the geotechnical issues identified earlier were analyzed or considered in design of foundation systems for the bollard fence.

5.5.1 Embankment stability and soils for the Fisher fence

The following is a summary of findings regarding geotechnical engineering attributes that are important for foundation design at the Fisher site:

- **Embankment stability.** In general, slopes flatter than about 3H:1V do not exhibit slope instability except in circumstances of unusual soil properties or adverse environmental conditions (e.g., weak soil or excessive seepage exiting the slope). Because we have limited topographic data, and there is limited information on soil strata, soil strength, and groundwater data, we did not perform a slope stability analysis for the Fisher fence. However, slope stability is not expected to be an issue at this site because slopes are generally about 5H:1V and no unusual conditions affecting stability (e.g., excessive seepage) appear to be present. However, as discussed below, it is possible that unprotected riverbank slopes could erode and eventually weaken the foundation or undermine the structure as discussed below.

- **Compaction.** With limited topographic data for before and after construction, it is not possible to determine if the fence foundation is on natural ground or fill. Though it is difficult to distinguish between natural ground and fill materials, we interpret that the upper 3 feet in most test pits comprise fill materials. Based on this observation, we conclude that the fence is supported, at least in part, by human-placed fill. Our field density test results indicate that, where performed, fill soils are generally inadequately compacted. We have no field density tests results for fill that exists directly below the footing.

  As described earlier, fill soil is generally compacted to 90 to 95 percent of its maximum dry density. Fisher provided three standard Proctor density tests (ASTM D 698) that were completed in November 2019, so the maximum soil dry density was known for comparison with measurements of in-place dry density. However, Fisher provided no results of field verification of in-place subgrade density at the site. Section 1803.5.8 of the IBC requires field verification of in-place density. Even though relative compaction was not measured, and our field density test results are generally below code values, inadequate compaction is not anticipated to cause issues at the bollard fence.

- **Corrosivity.** Based on sulfate content and Table 4.2.1 of ACI 318, soils are not considered corrosive. However, other references may consider soils corrosive based on soil resistivity results. Based on ACI 318, concrete corrosion is not expected to cause issues at the bollard fence.
- **Dispersivity.** Based on laboratory testing, dispersive soils were encountered along the fence. Dispersivity may exacerbate formation of erosion rills and gullies possibly undermining foundation elements as precipitation runoff or floodwaters are channelled between fence bollards. For example, see Figure 5.8, which illustrates erosion-caused rills and gullies that are characteristic of dispersive soil. We did not attempt to determine the extent of dispersive soils along the fence alignment. Dispersive soils, however, should be addressed with appropriate containment and erosion protection.
- **Settlement.** Excessive deformation of subgrade soil leading to settlement is not expected to be an issue at this site for this lightly loaded structure.
- **Potential vertical rise.** Laboratory test results do not indicate conditions for expansive soil according to Section 1803.5.3 of the IBC and expansive soils are not expected to cause issues at the bollard fence.
- **Seismicity.** South Texas is an area of low seismicity and seismic design is not considered an issue for the Fisher fence.

Figure 5.8. Rills and gullies characteristic of dispersive soil (source DOJ)

5.5.2 Foundation design for the Fisher fence

Fisher provided DOJ with two drawings prepared by TGR dated October 30, 2019. The drawings show fence sections, an elevation view, and design details. Soil data are provided; otherwise, there is no information regarding geotechnical design or construction considerations for the fence. Because only limited topographic information regarding the ground surface both before and after construction is available and because we could not take samples from beneath the footing, it was not possible to determine if the foundation was built on natural ground or on fill materials.

Fisher did not provide:

- Any background information about site geology and soils.
- Structural design criteria regarding issues such as tolerable settlement.
- Any background materials describing the anticipated behavior of site geotechnical conditions in response to construction activities, and to service loads from the fence (i.e., there are no documented design criteria).
- Any geotechnical engineering analyses used to develop foundation design criteria for bearing pressures, lateral support, uplift capacity, settlement predictions, or special provisions to deal with problem soils.
- Descriptions of other special geotechnical considerations including but not limited to site clearing, earthworks, dispersive soil, fill placement and compaction, bank stability, and corrosion.
- Information regarding soil testing and quality control during construction, although Fisher provided the results of three standard Proctor laboratory compaction tests.

Based on our review, Fisher failed to develop and provide documentation regarding development and discussion of geotechnical design and construction considerations for site geology and soils; structural performance requirements; foundation analysis and design; geotechnical engineering judgment applied to site conditions and earthworks; and quality control for geotechnical and foundation construction.

As part of our field investigation, we measured the thickness of the flange of the T-shaped footing at five locations along the fence alignment. Near the upstream end of the fence, we observed the flange to be 1 foot thick as indicated by the foundation design in Figure 5.4. However, at four other locations, we measured the flange to vary from 4.5 to 10 inches thick (refer the Site and Subsurface Investigation Report in Appendix B).

As the fence foundation was constructed at the ground surface, there is no soil cover available to contribute to uplift resistance. In addition, the ability of the soil adjacent to the foundation to resist lateral loads is compromised because:

- The thickness of the concrete in the flange of the T-shaped footing is less than 1 foot in locations along the fence alignment.
- The near-surface soil may be loosened by seasonal moisture changes or by root holes or animal burrows.
- The near-surface soil may be removed by erosion.

The foundation design is inconsistent with standard industry practice because it was built with no soil cover and its ability to resist lateral loads under all conditions including erosion is questionable. We conclude that the fence may not be fit for use under all anticipated loading conditions.

5.5.3 Erosion protection for the Fisher fence

The original design and construction of the fence did not include special provisions for protecting the fence and its foundation from erosion. Evidently the design assumed that natural grass volunteer vegetation on the riverbank would eventually provide adequate erosion protection. Following construction, erosion caused by storm runoff and the presence of dispersive soil in certain areas was observed around the fence (see Figure 5.8). In a letter to IBWC dated November 5, 2020, Fisher reported that riverbank erosion problems would be repaired, disturbed ground would be reseeded, and a 10-foot-wide gravel cover would be added to the river side bank for a portion of the fence. Our field work indicated that the gravel cover comprises 4-inch-minus, clean, angular stone generally less than 12 inches deep.

Figure 5-8 shows that there are areas of bare ground near the fence following construction. The maximum mean permissible channel velocity for bare earth (sandy silt) is 2 fps (USACE 1994). If volunteer vegetation could be assured, the maximum mean permissible channel velocity for grass-covered slopes is about 4 to 6 fps (USACE 1994). However, as indicated in Section 4 of this report, the fence could experience water velocities generally in the range of 5 to more than 7 fps during high-water events. Based on the erosion problems observed at the fence, and the possibility of high-water velocities, more robust erosion protection should be provided.

In addition, no provisions were made to handle the presence of dispersive soil at locations along the fence. Dispersive soils must be either removed or contained using appropriately designed granular soils capable of filtering small particles of soil as the dispersive soil deflocculates in the presence of water. Improperly designed containment of dispersive soil can hide its presence, potentially leading to more serious erosion problems. The 4-inch-minus, clean, angular stone placed in response to observed erosion will not provide satisfactory containment for dispersive soil without specially designed filters.

Figure 5.9 is a photograph showing severe bank erosion near the Fisher fence, which is caused by natural fluctuations in the river level, wave action, and boat wakes. The removal of existing bank vegetation without immediate replacement by properly designed erosion protection can lead to bank caving, which may eventually threaten the stability of the fence. Additional repair is warranted given the erodible nature of site soil; the presence of dispersive soil in certain areas; observed bank undermining by fluctuating river levels, waves, and boat wakes; and the possibility of high velocity water flow during floods. Repairs should include special details such as appropriately designed filters for dispersive soil.

Figure 5.9. Severe bank erosion caused by fluctuating river levels, wave action, and boat wakes (source DOJ)

Intervention to correct minor erosion problems is generally considered to be acceptable practice if 1) there is a predetermined plan for correction of problems; 2) there are resources available to make the corrections; and 3) there is a commitment by the owner to intervene when required. However, during major floods on the Rio Grande, access to the fence will be difficult and intervention may not be possible. Major flooding is likely to cause erosion from local turbulence around fence bollards, which could undermine the shallow foundations causing fence failure. For this reason, prudent practice would be to include adequate erosion protection during design and construction to reduce the likelihood of erosion and fence failure.

5.5.4 Conclusions regarding the Fisher fence

Our review of three other projects with well-developed scopes of work for geotechnical exploration, testing, and recommendations for similar projects in generally similar circumstances provides valuable insight into bollard fence construction in Texas but does not provide sufficient information to establish a standard of care. Our conclusions relative to the fitness for use of the Fisher bollard fence are presented below and are summarized in Table 5.8:

- The fence was constructed on a continuous, shallow reinforced concrete footing after clearing vegetation from the site. Site soil comprises mixtures of clay, silt, and sand. Up to about 3 feet of native material was used as fill at various locations. Where tested, the fill generally does not meet IBC compaction standards.
- There are no records of 1) field exploration and testing; 2) soil laboratory testing to support the geotechnical design; 3) geotechnical engineering analysis; 4) geotechnical recommendations for design; and 5) quality control of geotechnical aspects of design.
- The foundation for the Fisher fence extends to a depth of 3 feet 2 inches below finished grade. For the other three fences in Texas, one fence has a foundation depth of 10 feet, and two fences have foundation depths of 10 feet 9 inches. Because the foundation was constructed at the ground surface with no burial, it is unlikely to be capable of carrying service loads during floods on the Rio Grande (hydrostatic and hydrodynamic loads, and impact loads from floating debris). As such, the foundation system is likely not fit for use under all reasonably anticipated service loads. Increased depth of burial would have increased the lateral load-carrying capacity of the foundation and would have added weight for uplift resistance.
- The location of the fence near the riverbank and the presence of erodible soils require that the fence be protected from wind and water erosion. The other three Texas fences were built in areas that do not require specialized erosion protection. Though not provided in the original construction of the Fisher fence, some erosion protection has been added to repair local damage from precipitation runoff. Without additional erosion protection, satisfactory performance of the fence over the long-term is questionable and may create a situation where the fence is not fit for use.
- Dispersive soil is present at various locations along the fence alignment. No attempts were apparently made to either remove or contain dispersive soil. Erosion of dispersive soil may compromise fence integrity and its fitness for use.

Table 5.8. Summary of geotechnical assessment for the Fisher bollard fence

Geotechnical Consideration	Issue	Finding	Is Fitness for Use Compromised?
Embankment stability	Slope instability of the riverbank could cause fence failure.	Existing riverbank slopes are about 5H:1V. In general, slopes flatter than about 3H:1V do not exhibit slope instability except in unusual circumstances.	**Possibly.** Erosion of denuded and unprotected riverbanks could eventually compromise foundation performance. See Section 5.5.1.
Compaction	Compaction of underlying soils is important to the bollard fence to provide strong, deformation-resistant support to the structure.	Proctor tests were available during construction, but no field density tests were performed.	**Not likely.** See Section 5.5.1.
Corrosivity	The chemical composition of soil and porewater may cause corrosion and deterioration of concrete.	No documentation provided.	**Not likely.** See Section 5.5.1.
Dispersivity	Dispersive clay, known to be present in the Rio Grande valley, can be rapidly eroded and carried away by waterflow.	No documentation provided.	**Yes.** Dispersive clay must be removed or be contained to protect it from flowing water. See Section 5.5.1.
Settlement	Performance criteria to minimize settlement are typically defined by building codes, industry standards and practices, and owner preference.	No documentation provided.	**Not likely.** See Section 5.5.1.
Potential vertical rise	Texas has expansive soils that shrink and swell as a function of water content.	No documentation provided.	**Not likely.** See Section 5.5.1.
Seismicity	Not likely to be significant in this part of Texas.	No documentation provided.	**No.** See Section 5.5.1.
Foundation bearing pressure and depth	Adequate bearing is required for structural stability. Foundation should be deep enough to protect against soil loosened by moisture change, animal burrows, or erosion.	Foundation is at the ground surface. Soil data provided.	**Yes.** Limited uplift capacity is available and there is limited protection against soil loosened by moisture change, animal burrows, or erosion.
Lateral resistance	Adequate lateral resistance is required for structural stability.	Soil data provided.	**Yes.** Foundation may not provide adequate resistance to sliding and overturning. See Section 5.5.2.
Erosion Protection	Site soil is erodible and dispersive clay is present. Removal of existing vegetation exacerbated erosion problems.	None initially provided. Some added in response to observed erosion problems. The gravel layer added post-construction is inadequate for containment of dispersive soil.	**Yes.** Erosion protection and containment of dispersive soil are required. See Section 5.5.3.

6 Structural Assessment of Bollard Fence

Rio Grande, Hidalgo County, Texas

August 2021

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Licensed as a Professional Engineer in FL, TX, and NY

6.1 Structural assessment – summary

At the request of the U.S. Attorney's Office, Southern District of Texas, U.S. Department of Justice, we prepared this structural engineering assessment of the design and construction of a 2.96-mile-long bollard fence system located on the Texas bank of the Rio Grande between Anzalduas Park and Bentsen State Park south of Mission, Texas. The bollard fence system was constructed by Fisher in 2019-2020.

The specific location of the bollard fence and the site conditions are described in Section 2, and the purpose and scope of our engineering evaluation are presented in Section 3 of this report. The specific purpose of this structural engineering assessment is to provide an expert opinion regarding design and construction of the bollard fence system, and to present the findings of the external and internal stability analyses of the bollard fence and light/camera monopole if exposed to record flooding. In accordance with our Statement of Work, the record flood event is the design flood determined by IBWC based on the 1967 Hurricane Beulah. An opinion is rendered as to whether these components will maintain horizontal, vertical, and rotational equilibrium for the prescribed flood event, and have adequate strength during such flood condition.

6.2 Assessment of government-furnished information

The principal findings of the assessment of government-furnished information from the structural engineering standpoint are provided in the following subsections.

6.2.1 Plans

The plans prepared by TGR, and dated October 30, 2019, include a typical system cross section of the 18-foot-tall bollard fence, 20-foot-wide paved road, and 30-foot-tall light/camera monopole (sheet 1 of 2), and bollard fence typical wall section, reinforcement section, bollard section, and typical wall elevation (sheet 2 of 2). These plans are not signed and sealed by a licensed professional engineer in the State of Texas; however, as long as a representation that engineering services have been or will be offered to the public has not been made or implied, the bollard fence may be considered exempt from licensing requirements. Notwithstanding licensing requirements, the plans do not include design criteria, concrete notes, reinforcing and structural steel notes, and foundation notes, datum, benchmarks, and items requiring structural observation and inspection, among other contents considered to meet industry standards. The Standard of Care in Engineering is understood as the care and skill ordinarily used by members of the subject profession practicing under similar circumstances at the same time and in the same locality.

Other relevant information not included on the plans includes the reinforcing steel details and structural steel section details for the light/camera precast shaft supported monopole, as well as their respective material designations and grades.

The typical system cross section depicts the 3-foot-diameter 6-foot-long precast shaft protruding 2 feet above grade, as shown in Figure 6.1. The fence bollards are 1 foot and 1¼ inch on centers and have a clear space of 5 inches along the centerline of the fence, as shown in Figure 6.2.

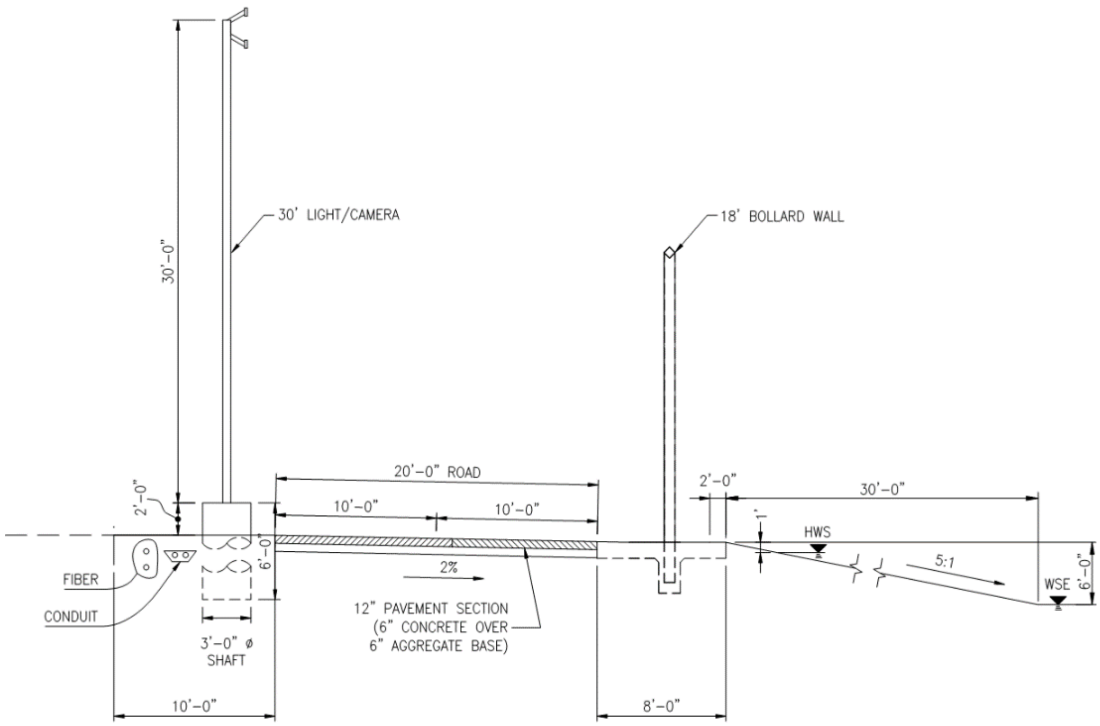


Figure 6.1. TGR typical system cross section

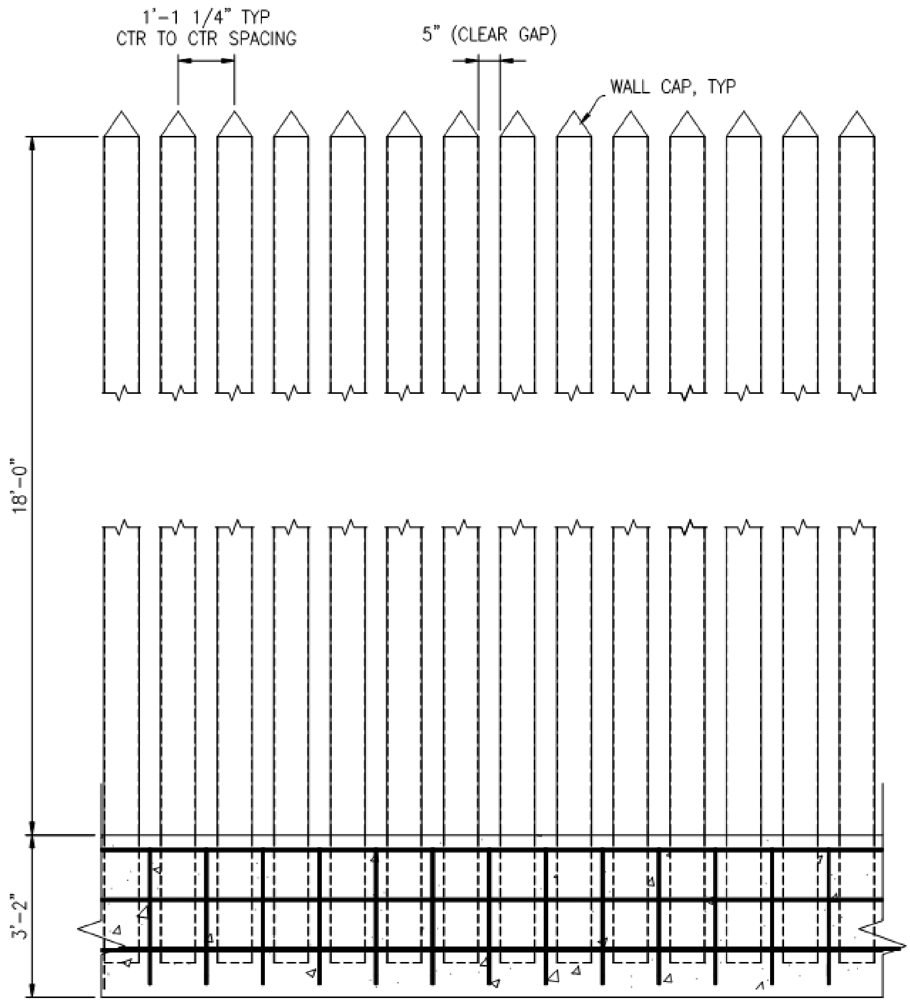


Figure 6.2. TGR typical wall elevation

The bollards are constructed of hollow structural section 6-inch by 6-inch by 1/8-inch-thick galvanized finish with a material designation of ASTM A80, Grade 75 kip/square inch. The bollards are rotated 45 degrees about their cross-sectional centroid and embedded 2 feet 6 inches into the reinforced concrete T-shaped footing as shown in Figures 6.3 and 6.4, respectively.

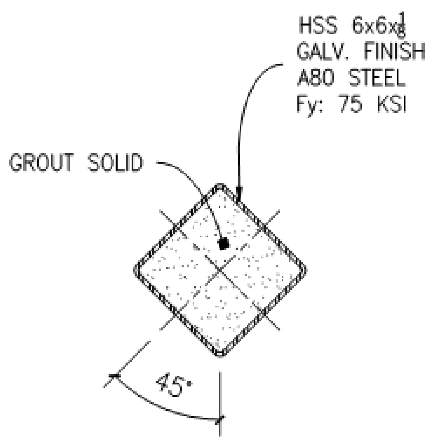


Figure 6.3. TGR typical bollard section

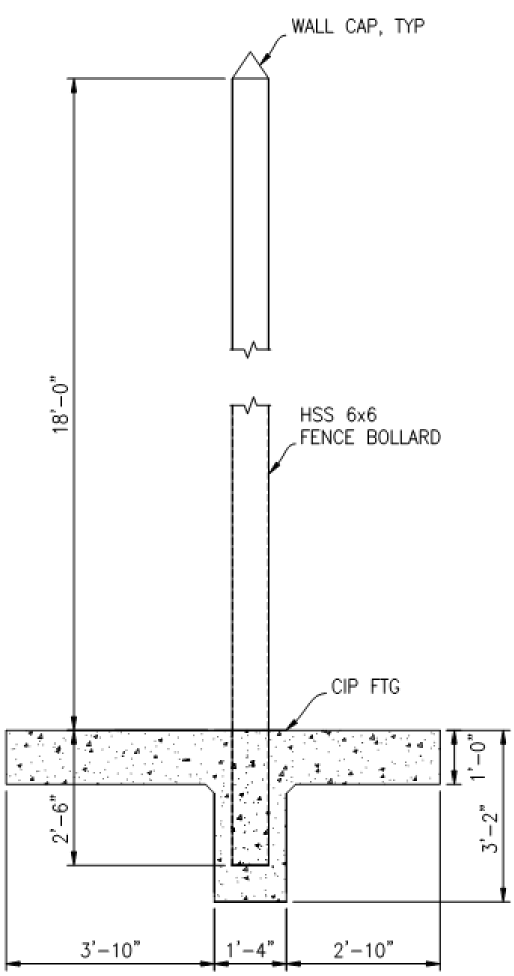


Figure 6.4. *TGR typical bollard fence section*

The T-shaped footing is reinforced for flexure with #5 bars at 1-foot 1¼-inch on-center spacing and 11 #5 bars for shrinkage and temperature in the longitudinal direction distributed as shown in Figure 6.5. U-shaped #5 bars at 1-foot 1¼-inch on-center spacing as well are provided for shear and torsional resistance, confinement for bollards, and constructability purposes. The adequacy of this reinforcement is assessed in Section 6.5.3; however, by inspection, the minimum lap of 24 inches (shown in Figure 6.5) does not meet the 31 inches required for a Class B splice, unless the lap is staggered to meet the ACI 318-14 Building Code Requirement for Structural Concrete, Section 25.5.2.1 (ACI 2014).

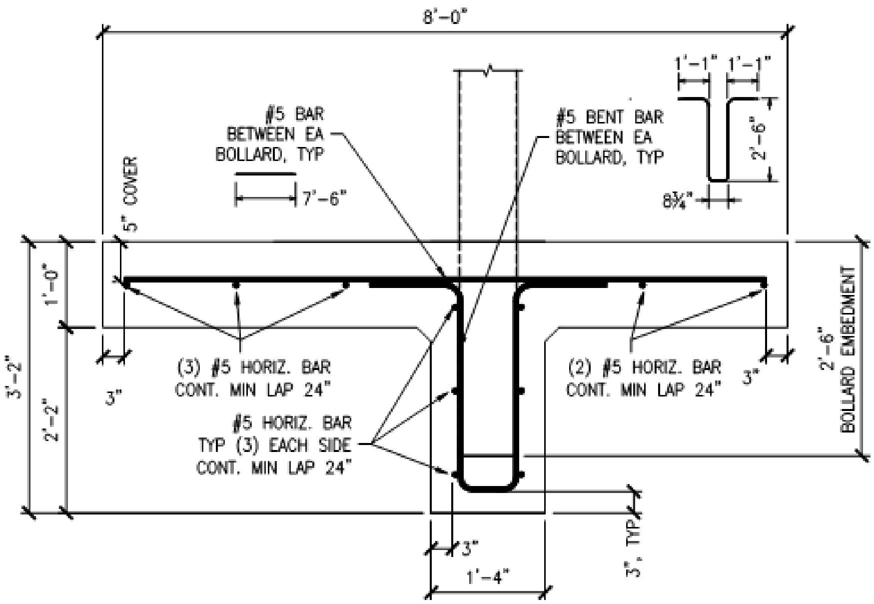


Figure 6.5. TGR typical reinforcement section

6.2.2 Calculations

The calculations prepared by Stinger Bridge & Iron, and dated November 21, 2019, used a software by ENERCALC, Inc. licensed to Fisher. The canned software reportedly complies with ACI 318-14, IBC 2015, and ASCE 7-10, which are the applicable codes adopted (or by reference) by the Texas Legislature. Noticeably, the calculations are not signed and sealed by a licensed professional in the State of Texas; however, as stated previously, as long as a representation that engineering services have been or will be offered to the public has not been made or implied, the bollard fence may be considered exempt from licensing requirements.

The stability analysis and design calculations account for dead loads, wind pressure, and earth pressure. Flood loads (e.g., hydrostatic, hydrodynamic, and floating debris) that may result from an unusual event such as the IBWC-designated design flood were entirely missed, even though the TGR computational fluid mechanics model indicated exposure to flood waters. The application of flood loads as required by IBC, ASCE 7-10, and Federal Emergency Management Agency (FEMA) P-55 are intended for the protection of life and property.

A basic mistake in computing earth pressures was identified, as follows. The active and passive pressures were computed based on saturated-soil density, which erroneously implies that the water has the same angle of repose of the soil. The correct methodology is to obtain the dry-soil density (subtracting the density of the water from the saturated-soil density) to which the respective active and passive coefficients would be applied, and the hydrostatic pressure accounted for separately assuming no angle of repose since the water is an isotropic material. The uplift pressure was also ignored in the TGR calculations prepared by Stinger Bridge & Iron.

6.2.3 Materials testing

Results of concrete testing conducted by MEG Engineers for TGR demonstrated adequate plastic and hardened properties. It is unclear why the minimum slump was specified as 8½ inches, perhaps due to the use of a high-range water-reducing admixture to render the concrete mix pumpable. This information is typically qualified in the technical specifications, which were not made available or do not exist. It is also not clear why the targeted freshly mixed concrete temperature and air entrainment plastic properties were not included in the test reports, again suggesting that technical specifications might not have been prepared for the project.

Results of reinforcing steel tests conducted by Western Technologies, Inc. for TGR demonstrated adequate cross-sectional properties, as well as adequate yielding and tensile strengths.

6.2.4 Operation and maintenance plan

A review of the Operation and Maintenance Plan (plan) prepared by TGR (updated on October 9, 2020) yielded the following findings:

- TGR stated that “uncontrolled growth of invasive species” “would further impede and redirect the flow” for the modified environment (post-construction of bollard fence). This statement is itself an acknowledgement of the potential impact of the modified environment on the natural flow of the river, hence posing further risks to the stability and integrity of the structure.
- The plan states that regular quarterly inspection supplemented with additional on-site visits after significant local or regional rainfall event are planned. It is understood that these inspections are incumbent on TGR.
- The plan states that clean-up of debris will be scheduled after inspection if a large amount of debris is found and after sugar cane is harvested. It is understood that these clean-up efforts are incumbent on TGR.
- The plan addresses vegetation control and erosion maintenance. Similarly, it is understood that these activities are incumbent on TGR.

6.3 Field visit

Arcadis was contracted by the DOJ, McAllen Division, to evaluate the site and subsurface conditions for the bollard fence constructed by Fisher along the Rio Grande near Mission, Texas. The fence consists of approximately 3 miles of 6-inch square hollow structural section bollards to a height of 18 feet above ground, a 20-foot-wide road section, and a 30-foot-tall light/camera at 200 feet on-center spacing (approximately) as described in Section 6.2.1. An important variance learned from Fisher during the field visit is that the bollards were filled with gravel instead of grout as shown in Figure 6.3. Arcadis documented the site conditions with photographs included in Appendix B.

6.3.1 Site observation and assessment

A surplus of steel bollards used to construct the fence section is stockpiled at the site as shown in Figure 6.6. These bollards were inspected during the investigation.

Figure 6.6. Surplus of steel bollards

The dimensions of the 6-inch by 6-inch tube sections were verified, and shop tags were included with record of galvanizing shipping details as shown in Figure 6.7. NDT was used to confirm the 1/8-inch thickness of bollards in place on the fence at locations every ¼ mile along the fence alignment as documented in Appendix B.

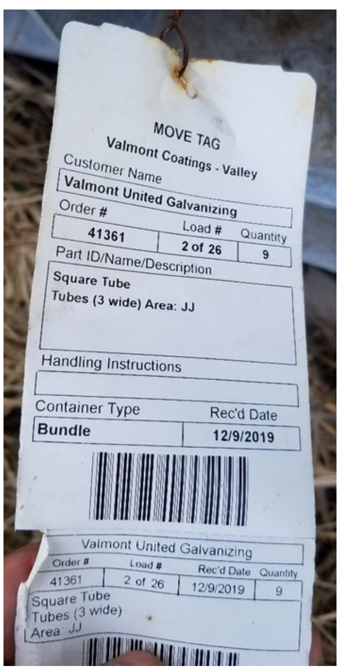


Figure 6.7. Shop tag with record of bollard galvanizing

A light/camera monopole founded in a precast concrete shaft foundation is shown in Figure 6.8. The light pole foundation is designed as 4 feet embedded into the ground with 2 feet exposed above ground as shown in Figure 6.1; however, the exposed height was at times higher due to variance in the finished grade with respect to the typical system cross section. The light/camera pole is also galvanized steel. No details were provided of the light pole anchor detail into the concrete foundation.

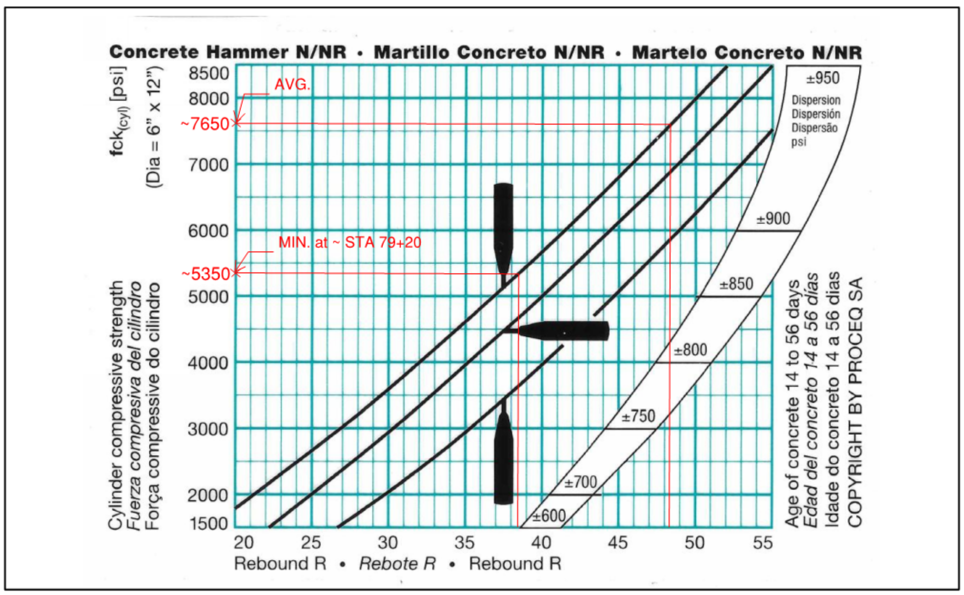
Figure 6.8. Precast shaft founded light/camera monopole

The overall width of the base foundation was verified to be 8 feet as shown on the plans (see Figures 6.1, 6.4, and 6.5). However, the thickness at some locations where the edge of the foundation was exposed due to erosion was less than the 1 foot shown on plans (see Figures 6.1 and 6.5). Figure 6.9 shows a non-conforming thickness of 4½ inches.

Figure 6.9. Non-conforming thickness of base foundation

6.3.2 Field data and materials testing

Terracon completed NDT to confirm the thickness of steel bollards, the configuration of reinforcing steel, and the compressive strength of the concrete mix used in the footing. A Schmidt rebound hammer was used to verify the compressive strength at 12 distinct locations ¼ mile apart along the 3-mile bollard fence alignment. The rebound hammer tests results are included in Appendix B. Based on the lowest R-value result, the minimum compressive strength is correlated to 5,350 pounds per square inch (psi); based on the average R-value result of all 12 locations, the average compressive strength is correlated to 7,650 psi, as shown in Figure 6.10. Even the minimum correlated value exceeds the specified compressive strength (at 28 days) reported by MEG Engineers as 4,500 psi.



Project No. 88215034	Site Plan: Rebound Hammer Chart Report Number: 0003	Terracon
Date: 5/6/2021	Technician: Ben Butler	1506 Mid Cities Drive
Reviewed: MR	Date: 4/27/2021	Pharr, Texas
Approved: AAS	Scale: Not to Scale	PH. (956) 283 8254 terracon.com

Figure 6.10. Rebound hammer converting chart

It should be noted that the chart converting rebound number (R-value) to a specific compressive strength does not account for differences in mix design, maturity, surface moisture, surface smoothness, and other factors and is primarily useful to indicate relative strengths between different test locations. Nevertheless, the rebound hammer test provides a practical and cost-effective solution for estimating in-place concrete strength.

The rebar size and spacing were verified satisfactorily at the same 12 locations with a Hilti PS200 Ferroscan, a ferrous detector scanner that allows verification with accuracy of rebar size and spacing.

6.4 Parameters from hydraulic engineering assessment

In accordance with key standard ASCE/SEI 7 2010 (ASCE 7-10), Section 5.3.1, structural systems of buildings or other structures shall be designed, constructed, connected, and anchored to resist flotation, collapse, and permanent lateral displacement due to action of flood loads associated with the design flood event and other loads prescribed in the load combinations of Chapter 2 of said standard. To this purpose, the bollard fence is in a noncoastal A-Zone. Flood loads applicable to this location include hydrostatic, hydrodynamic, and debris impact.

Per ASCE 7-10, Section 5.4.2, hydrostatic loads caused by a depth of water to the level of the design flood event shall be applied over all surfaces involved, both above and below ground.

Per ASCE 7-10, Section 5.4.3, hydrodynamic loads or the dynamic effects of moving water shall be determined by a detailed analysis utilizing principles of fluid mechanics; however, where water velocities do not exceed 10 fps, dynamic effects of moving waters shall be permitted to be converted into equivalent hydrostatic loads by increasing the design flood event for design purposes by an equivalent surcharge depth, d_h, on the headwater side and above the ground only, equal to:

$d_h = a*V^2/2g$,

where

a = coefficient of drag or shape factor (not less than 1.25)

V = average velocity of water (flow velocity)

g = acceleration due to gravity

Per ASCE 7-10, Section 5.4.5, impact loads are those that result from debris transported by floodwaters striking against structures, or parts thereof, and shall be determined using a rational approach as concentrated loads acting horizontally at the most critical location at or below the design flood event. Regarding ASCE 7-10, Section C.5.4.5, Special Impact Loads, USACE states that, absent a detailed analysis, special impact loads can be estimated as a uniform load of 100 pounds per foot (lb/ft). Guidance provided by FEMA P-55 2011 (FEMA P-55), Section 8.5.10, which is predicated on the same impulse-momentum approach discussed in ASCE 7-10, Section C5.4.5 Impact Loads, offers the following equation for debris impact as a concentrated load:

$F_i = W\ V\ C_D C_B C_{Str}$,

where

W = weight of the object = 1,000 lb (also recommended in ASCE 7-10, Section C5.4.5)

V = flow velocity

C_D, C_B, and C_{Str}, are the depth, blockage, and building structure coefficients, respectively, as provided by FEMA P-55, Section 8.5.10.

The relevant parameters (e.g., water surface elevations and average flow velocities) for the structural engineering assessment are provided in Section 4 of this report and summarized in the following subsections.

6.4.1 Water surface elevations

For the purpose of the structural engineering assessment, the WSELs shown in Table 6.1 were recommended by the Hydraulics Engineering Discipline Expert based on the large-domain 2D HEC-RAS fluid mechanics model output:

Table 6.1. Recommended Water Surface Elevations

Case	WSEL River Side (feet)	WSEL Land Side (feet)	Grade Elevation (feet)
Rising water from river side and maximum flow velocity of 7.9 fps	113.70	112.90	112.00
Rising water from river side and average velocity of 7 fps	129.03	128.70	112.74
Rising water from land side and average velocity of 6 fps	128.30	128.80	111.83

6.4.2 Flow velocity

For structural engineering assessment purposes, the following water flow velocities were recommended by the Hydraulics Engineering Discipline Expert based on the fluid mechanics model output of rising waters:

- Maximum flow velocity in the bollard fence at any given segment or time: 7.9 fps
- Average flow velocity in the western segment of bollard fence from river side (see Figure 6.11): 7 fps
- Average flow velocity in the eastern segment of bollard fence from land side (see Figure 6.12): 6 fps

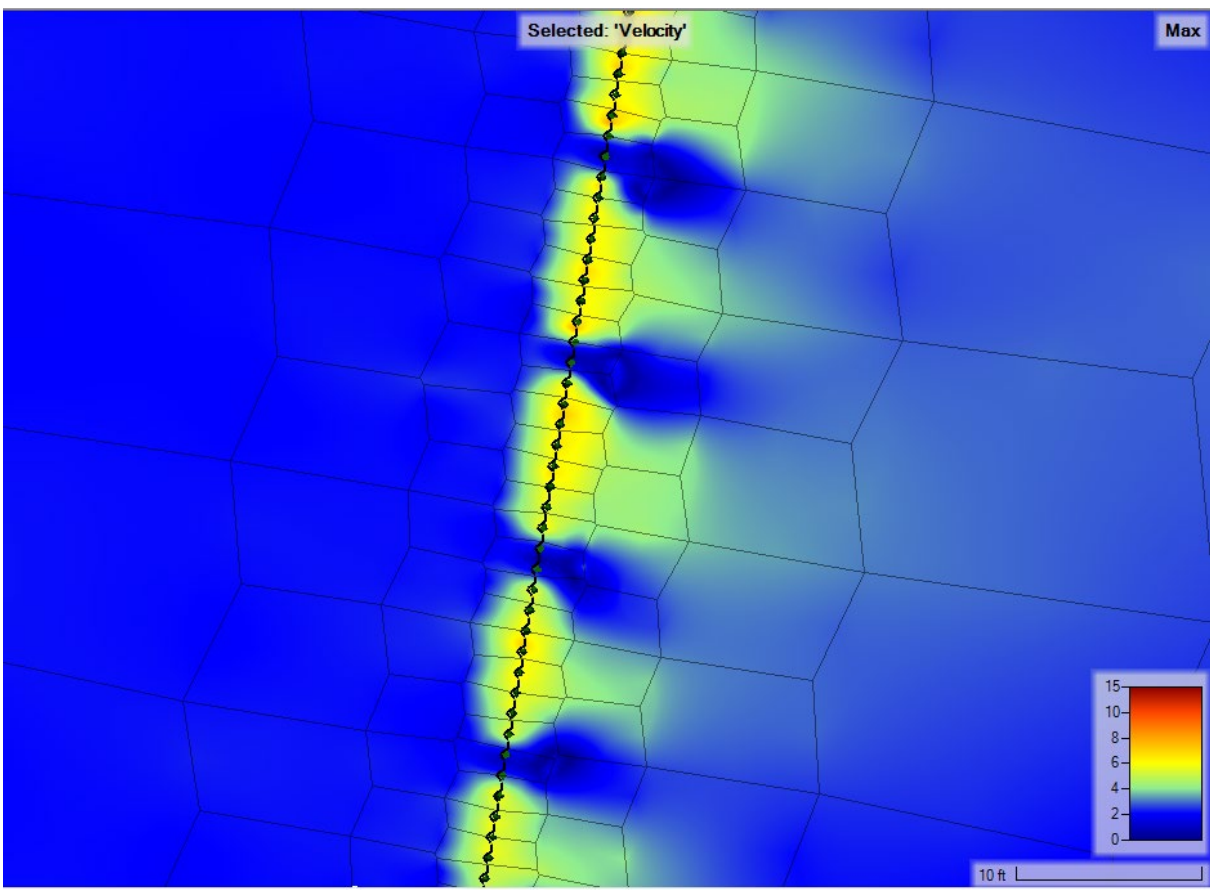


Figure 6.11. Maximum velocity plumes through typical unblocked openings on western portion of fence

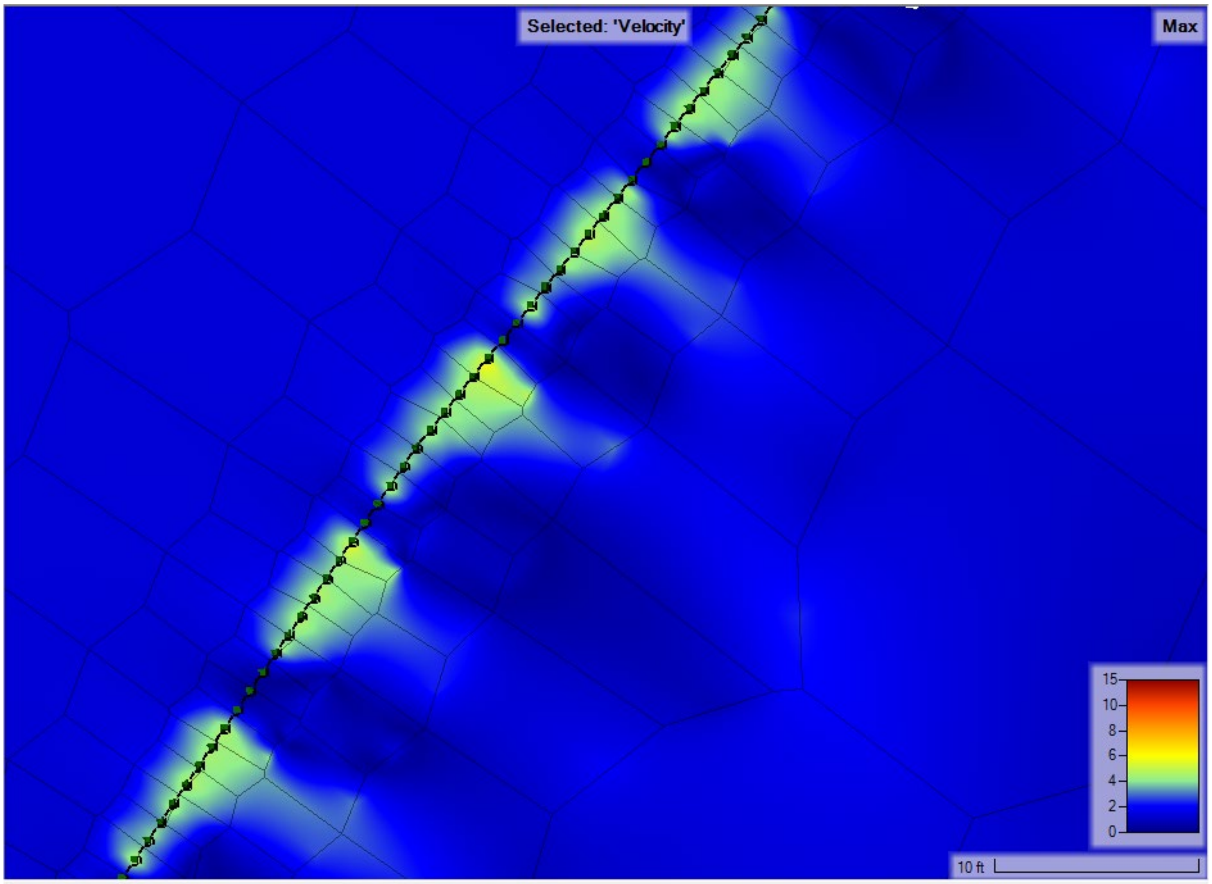


Figure 6.12. Maximum velocity plumes through typical unblocked openings on eastern portion of fence

6.5 Parameters from geotechnical engineering assessment

In accordance with key standard ASCE 7-10, Section 3.2.1, in the design of a structure below grade level, provisions shall be made for the lateral pressure of adjacent soil. When a portion or the whole of the adjacent soil is below a free-water surface, computations shall be based on the weight of the soil diminished by buoyancy, plus full hydrostatic pressure. In accordance with ASCE 7-10, Section 3.2.2, the upward pressure of water, where applicable, shall be taken as the full hydrostatic pressure applied over the entire area. The hydrostatic pressure shall be measured from the underside of the construction.

The relevant parameters (e.g., unit weight of soil, angle of internal friction, soil cohesion, allowable bearing pressure of soil, coefficient of friction with concrete, and active and passive coefficients) for the structural engineering assessment of the bollard fence system are provided in Section 5 and summarized in the following subsections.

6.5.1 Soil unit weight

Density, strength, and compaction test results are summarized in Table 5.2 in Section 5. For the purpose of the structural engineering assessment, a unit weight of soil (saturated) of 115 pcf was recommended by the Geotechnical Engineering Discipline Expert.

6.5.2 Angle of internal friction

Density, strength, and compaction test results are summarized in Table 5.2 in Section 5. For the purpose of the structural engineering assessment, an angle of internal friction (ø) equal to 35 degrees was recommended by the Geotechnical Engineering Discipline Expert.

6.5.3 Soil cohesion

For the purpose of the structural engineering assessment, a soil cohesion coefficient (c) equal to zero was recommended by the Geotechnical Engineering Discipline Expert for the type of soils identified at this site.

6.5.4 Coefficient of friction with concrete

For the purpose of the structural engineering assessment, a coefficient of friction with concrete (f) equal to 0.25 was recommended (Section 5), which was predicated in a prescribed friction factor in 2015 IBC Table 1806.2 for the type of soils identified at this site.

6.5.5 Active and passive earth coefficients

For the purpose of the structural engineering assessment, an active earth pressure coefficient (K_a) and a passive earth pressure coefficient (K_p) equal to 0.271 and 3.69, respectively, were recommended by the Geotechnical Engineering Discipline Expert (Section 5) based on the angle of internal friction discussed in Section 6.5.2 and using Rankine's formulae.

6.5.6 Allowable bearing capacity

For the purpose of the structural engineering assessment, an allowable bearing pressure equal to 1,500 psf was recommended by the Geotechnical Engineering Discipline Expert (Section 5), which was predicated in a prescribed vertical foundation pressure in 2015 IBC Table 1806.2 for the type of soils identified at this site.

6.6 Structural analysis of bollard fence system

The structural engineering assessment focuses on the external stability of the bollard fence system about its base, and on the internal stability or strength of its components. The approach, stability, and strength criteria, and findings, are presented in the following subsections.

The global stability aspect of the fence system or any of its components is not incumbent to the Structural Engineering Discipline Expert. Global stability may include, but is not limited to, deep-seated shear failure and long-term settlements.

6.6.1 Analysis approach

In accordance with ASCE 7-10, Section 1.3.4, the load effects on the bollard fence system and individual components shall be determined by methods of structural analysis that consider equilibrium, general stability, and both short- and long-term materials properties.

Previous subsections discuss the loads expected to occur during the service life of the bollard fence system such as flood loads (e.g., hydrostatic, hydrodynamic, and debris impact), earth and hydrostatic pressures acting below grade, and uplift pressure underside the foundation. Other load cases to be included in the analyses are the dead load components due to self-weight of materials and wind loads. The wind loads are computed based on ASCE 7-10, Chapter 27.

The analysis approach is to evaluate the external stability with loads at service (unfactored) level and to evaluate internal stability (strength) with loads at ultimate level (factored).

In terms of the Use and Occupancy of Buildings and Structures, ASCE 7-10, Table 1.5-1 would categorize the bollard fence system as Risk Category I because the structure represents a low risk to human life in the event of failure during normal conditions; however, consideration shall be given to the threat to human life and adverse impact in terms of economics in the event of failure during unusual conditions.

In USACE's Engineer Manual (EM) 1110-2-2100, 2005, Par. 3-2, the load conditions that a structure may encounter during its service life are grouped into the load condition categories of usual, unusual, and extreme. Per the Statement of Work, the flood of record, the design flood determined by IBWC based on the 1967 Hurricane Beulah (Beulah), shall be utilized in this assessment. Beulah is considered a storm with a return period of 300 years or an annual probability of occurrence of 0.0033. Based on Beulah's return period, EM 1110-2-2100, Table 3-1 would assign to Beulah a load condition category of Unusual. Par. 3-1 of the same EM explains that factors of safety that are specific to each loading condition are intended to keep the risk of failure at an acceptably low level and such that performance objectives are achieved.

For the purpose of this assessment, the bollard fence system is considered a "normal" (as opposed to "critical") structure subjected to an "unusual" loading condition.

6.6.2 External stability assessment

The objective of the external stability assessment of the bollard fence system is to confirm that its components, specifically the bollard fence and the light/camera, will maintain horizontal, vertical, and rotational equilibrium for the prescribed loading condition defined in Section 6.6.1 and demonstrate that prescribed factors of safety are met, such that the risk of failure is kept to an acceptably low level and that performance objectives are achieved, as stated before.

The stability criteria used to assess the bollard fence are based on recognizing this feature as a semi-gravity structure, which relies on its own weight and any water head resting on the base (foundation), as well as the soil lateral passive resistance, bearing support underside of the foundation, and reinforcement of the foundation for the optimized section. The specific stability criteria for gravity and semi-gravity structures are from EM 1110-2-2100, Chapter 3, and are detailed in the following subsections. The effects of scour and/or undermining due to erosion and under seepage are not included in this external stability assessment; however, the potential for occurrence and the associated risks should they occur, as well as mitigating measures, are discussed in Sections 4 and 5.

The stability criteria used to assess the light/camera monopole are based on recognizing this feature as a semi-gravity structure, as well.

6.6.2.1 Stability criteria against sliding

A factor of safety is required in sliding analyses to provide horizontal equilibrium with a suitable margin of safety between the loads that can cause instability and the strength of the materials along the base that can be mobilized to prevent instability:

$FS_{slidiing} = (N \tan ø + cL)/T$

where

N = force acting normal to the sliding failure plane under the structural wedge

ø = angle of internal friction of the foundation material under the structural wedge

c = cohesion strength of the foundation material under the structural wedge

L = length of the structural wedge in contact with the foundation

T = shear force acting parallel to the base of the structural wedge

The required factors of safety for sliding stability for normal structures are presented in the following excerpt from EM 1110-2-2100:

Table 3-3 Required Factors of Safety for Sliding - Normal Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
Well Defined	1.4	1.2	1.1
Ordinary	1.5	1.3	1.1
Limited	3.0	2.6	2.2

6.6.2.2 Stability criteria against flotation

A factor of safety is required for flotation to provide a suitable margin of safety between the loads that can cause instability and the weights of materials that resist flotation:

$FS_{flotation} = (W_S + W_C + S)/(U - W_G)$

where

W_S = weight of the structure, including weights of the fixed equipment or appurtenances, and soil above the structure (saturated and buoyant above and below the groundwater table, respectively)

W_C = weight of the water contained within the structure

S = surcharge loads

U = uplift forces acting on the base of the structure

W_G = weight of water above top surface of the structure

The required factors of safety for flotation stability for normal and critical structures are presented in the following excerpt from EM 1110-2-2100:

Table 3-4 Required Factors of Safety for Flotation – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	1.3	1.2	1.1

6.6.2.3 Stability criteria against overturning (location of resultant)

Rotational behavior is evaluated by determining the location of the resultant of all applied forces with respect to the potential failure plane. This location can be determined through static analysis. Limits on the location of the resultant force are presented in the following excerpt from EM 1110-2-2100:

Table 3-5 Requirements for Location of the Resultant – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	100% of Base in Compression	75% of Base in Compression	Resultant Within Base

6.6.2.4 Stability checks and findings of bollard fence

The structural engineering assessment of the bollard fence system stability included three loading conditions that may result from an unusual event such as 1967 Hurricane Beulah. These loading conditions are referred to as Cases A through C in the following findings. Calculations are included in Appendix D.

Case A1: This loading condition accounts for maximum flow velocity during rising water levels from the river side including 100 lb/ft debris impact:

Stability Criteria	**Required**	**Provided**	**Comment**
Factor of safety against sliding	1.2	1.02	FAILS
Factor of safety against flotation	1.2	2.22	PASSES
Location of resultant	75% base in compression	38.1% in compression[1]	FAILS
Bearing pressure	Less or equal than 1500 psf	510 psf	PASSES

[1]Ignoring the potential for higher uplift pressures to develop in a crack.

Case A2: This loading condition accounts for maximum flow velocity during rising water levels from the river side not including debris impact:

Stability Criteria	Required	Provided	Comment
Factor of safety against sliding	1.2	1.11	FAILS
Factor of safety against flotation	1.2	2.22	PASSES
Location of resultant	75% base in compression	41.4% in compression[1]	FAILS
Bearing pressure	Less or equal than 1500 psf	Judicious neglect	PASSES

[1]Ignoring the potential for higher uplift pressures to develop in a crack.

Case B1: This loading condition accounts for the maximum water surface during rising water levels from the river side in the western segment of the bollard fence including 100 lb/ft debris impact:

Stability Criteria	Required	Provided	Comment
Factor of safety against sliding	1.2	0.89	FAILS
Factor of safety against flotation	1.2	0.96	FAILS
Location of resultant	75% base in compression	Outside of the base	FAILS
Bearing pressure	Less or equal than 1500 psf	Buoyancy occurs	FAILS

Case B2: This loading condition accounts for the maximum water surface during rising water levels from the river side in the western segment of the bollard fence not including debris impact:

Stability Criteria	Required	Provided	Comment
Factor of safety against sliding	1.2	0.9	FAILS
Factor of safety against flotation	1.2	0.96	FAILS
Location of resultant	75% base in compression	Outside of the base	FAILS
Bearing pressure	Less or equal than 1500 psf	Buoyancy occurs	FAILS

Case C1: This loading condition accounts for the maximum water surface during rising water levels from the land side in the eastern segment of the bollard fence including 100 lb/ft debris impact:

Stability Criteria	Required	Provided	Comment
Factor of safety against sliding	1.2	0.85	FAILS
Factor of safety against flotation	1.2	1.06	FAILS
Location of resultant	75% base in compression	Outside of the base	FAILS
Bearing pressure	Less or equal than 1500 psf	Buoyancy occurs	FAILS

Case C2: This loading condition accounts for the maximum water surface during rising water levels from the land side in the eastern segment of the bollard fence not including debris impact:

Stability Criteria	Required	Provided	Comment
Factor of safety against sliding	1.2	0.85	FAILS
Factor of safety against flotation	1.2	1.06	FAILS
Location of resultant	75% base in compression	Outside of the base	FAILS
Bearing pressure	Less or equal than 1500 psf	Buoyancy occurs	FAILS

6.6.2.5 Stability checks and findings of light/camera monopole

The structural engineering assessment of the light/camera monopole stability included two loading conditions that similarly may result from an unusual event such as 1967 Hurricane Beulah. These loading conditions are referred to as Cases A and C in the following findings. Calculations are included in Appendix D.

Case A1: This loading condition accounts for maximum flow velocity during rising water levels from the river side including 100 lb/ft debris impact:

Stability Criteria	Required	Provided	Comment
Factor of safety against sliding	1.2	2.59	PASSES
Factor of safety against flotation	1.2	2.90	PASSES
Location of resultant	75% base in compression	Outside of the base	FAILS
Bearing pressure	Less or equal than 1500 psf	Significantly greater than 1500 psf as it	FAILS

Case A2: This loading condition accounts for maximum flow velocity during rising water levels from the river side not including debris impact:

Stability Criteria	Required	Provided	Comment
Factor of safety against sliding	1.2	2.75	PASSES
Factor of safety against flotation	1.2	2.90	PASSES
Location of resultant	75% base in compression	Outside of the base	FAILS
Bearing pressure	Less or equal than 1500 psf	Significantly greater than 1500 psf as it	FAILS

Remarks: *The **Case B*** loading condition accounts for the maximum water surface during rising water levels from the river side in the western segment of the bollard fence; however, as the bollard fence would shield the light/camera monopole from debris impact for this loading condition, Case C was deemed more stringent (and checks and findings follow).

Case C1: This loading condition accounts for the maximum water surface during rising water levels from the land side in the eastern segment of the bollard fence including 100 lb/ft debris impact:

Stability Criteria	**Required**	**Provided**	**Comment**
Factor of safety against sliding	1.2	2.62	PASSES
Factor of safety against flotation	1.2	3.40	PASSES
Location of resultant	75% base in compression	Outside of the base	FAILS
Bearing pressure	Less or equal than 1500 psf	Significantly greater than 1500 psf as it overturns	FAILS

Case C2: This loading condition accounts for the maximum water surface during rising water levels from the land side in the eastern segment of the bollard fence not including debris impact:

Stability Criteria	**Required**	**Provided**	**Comment**
Factor of safety against sliding	1.2	2.76	PASSES
Factor of safety against flotation	1.2	3.40	PASSES
Location of resultant	75% base in compression	Outside of the base	FAILS
Bearing pressure	Less or equal than 1500 psf	Significantly greater than 1500 psf as it overturns	FAILS

6.6.3 Internal stability (strength) assessment

All structural members and systems and all components in a building or other structure are designed to resist dead loads, soil loads, soil and hydrostatic pressures, flood loads (e.g., hydrostatic, hydrodynamic, and floating debris impact), environmental loads (e.g., wind), and self-straining forces of volume change due to temperature, as applicable. A continuous load path or transmitting these forces to the foundation shall be provided.

Live loads (e.g., uniform/concentrated occupancy loads, vehicular load), vehicular impact loads, other environmental loads (e.g., rain, earthquake), and differential settlement either are not applicable or are beyond the scope of this structural engineering assessment.

6.6.3.1 General design requirements

In accordance with 2015 IBC, Section 1604.2, buildings and other structures, and parts thereof, shall be designed and constructed to support safely the strength level (factored) loads in load combinations defined in this code without exceeding the appropriate strength limit states for the materials of construction. Alternatively, buildings and other structures, and parts thereof, shall be designed and constructed to support safely the service level (unfactored) loads in load combinations defined in this code without exceeding the appropriate specified allowable stresses for the materials of construction.

6.6.3.2 Loading criteria

From ASCE 7-10, applicable load combinations to this bollard fence system engineering assessment include the following:

For Strength Design,

1. 1.2D + 0.5W + 1.0F_a + 1.6H (based on ASCE 7-10 Eq. 4), where H adds to the primary variable load effect
2. 1.2D + 0.5W + 1.0F_a + 0.9H (based on ASCE 7-10 Eq. 4), where H resists the primary variable load effect
3. 0.9D + 0.5W + 1.0 F_a + 1.6 (based on ASCE 7-10 Eq. 6), where H adds to the primary variable load effect
4. 0.9D + 0.5W + 1.0 F_a + 0.9 (based on ASCE 7-10 Eq. 6), where H resists the primary variable load effect

For Allowable Stress Design,

5. 1.0D + 0.6W + 0.75 F_a + 1.0H (based on ASCE 7-10 Eq. 5), where H adds to the primary variable load effect
6. 1.0D + 0.6W + 0.75 F_a + 0.6H (based on ASCE 7-10 Eq. 5), where H resists the primary variable load effect

where

D = dead load, as defined in previous subsections

W = wind force, as defined in previous subsections

F_a = hydrostatic, hydrodynamic, and debris impact forces, as defined in previous subsections

H = load due to lateral earth pressure groundwater pressure

6.6.3.3 Strength and allowable strength design criteria

The basic requirements for strength and allowable stress design for this bollard fence system engineering assessment are as follows:

For Strength Design,

In accordance with ACI 318-14, Section 4.6.2, structures and members shall have design strength at all sections, ø S_n, greater than or equal to the required strength *U* calculated for the factored loads, forces, and moments in such combinations as required by this code (ACI) or the general building code (IBC).

where

S_n = nominal strength

ø = strength reduction factor

U = required strength determined by analysis at ultimate (factored) level

For Allowable Strength Design,

Allowable strength design is similar to what is known as allowable stress design in that they are both carried out with loads at service level. The difference is that for strength design, the primary provisions are given in terms of forces or moments rather than stresses. For this design approach, the allowable strength (R_n/Ω) must equal or exceed the required strength (R_a) calculated for the nominal loads, forces, and moments in such combinations as required by the applicable code or in its absence, the ASCE 7-10 Standard.

where

R_n = nominal strength

Ω = factor of safety given for a particular limit state

R_a = required strength determined by analysis at nominal (service) level

6.6.3.4 Flexural and shear strength checks and findings

The structural engineering assessment of the bollard fence system strength included two loading conditions that may result from an unusual event such as 1967 Hurricane Beulah. These loading conditions are referred to as Cases A and B in the following findings. Calculations are included in Appendix D.

Case A: This loading condition accounts for maximum flow velocity during rising water levels from the river side including 100 lb/ft debris impact:

Strength Criteria	Required	Provided	Comment
Flexure in foundation	1.39 kip*ft	8.98 kip*ft	PASSES
Shear in foundation	1.01 kip	8.32 kip	PASSES
Shrinkage and temperature reinforcement in foundation (longitudinal direction)	0.26 in^2	0.28 in^2	PASSES[1]
Shrinkage and temperature reinforcement in foundation (transverse direction)	2.07 in^2	2.17 in^2	PASSES[1]
Section moduli due to flexure in bollard	0.85 in^3	2.84 in^3	PASSES
Cross section due to shear in bollard	0.172 in^2	2.70 in^2	PASSES

[1] However, the lap splice provided does not meet the requirements of ACI 318-14, Section 25.5.2.1

Case B: This loading condition accounts for the maximum water surface during rising water levels from the river side in the western segment of the bollard fence including 100 lb/ft debris impact:

Strength Criteria	Required	Provided	Comment
Flexure in foundation	Not established because buoyancy occurs	8.98 kip*ft	FAILS
Shear in foundation	Not established because buoyancy occurs	8.32 kip	FAILS
Shrinkage and temperature reinforcement in foundation (longitudinal direction)	0.26 in^2	0.28 in^2	PASSES[1]
Shrinkage and temperature reinforcement in foundation (transverse direction)	2.07 in^2	2.17 in^2	PASSES[1]
Section moduli due to flexure in bollard	16.4 in^3	2.84 in^3	FAILS
Cross section due to shear in bollard	0.168 in^2	2.70 in^2	PASSES

[1] However, the lap splice provided does not meet the requirements of ACI 318-14, Section 25.5.2.1

Remarks: Case C (loading condition that accounts for the maximum water surface during rising water levels from the land side in the eastern segment of the bollard fence) was not investigated because the structure becomes buoyant, failing due to external stability without testing the flexural and shear strength of the foundation. Case C is also less imposing on the bollard than Case B in terms of strength.

6.7 Findings and conclusions

The following are the main findings and conclusions derived from the assessment of government-furnished information (plans, calculations, materials testing, and maintenance plan) and the field visit:

- The plans prepared by TGR, dated October 30, 2019, were not signed and sealed by a licensed professional engineer in the State of Texas; however, as long as a representation that engineering services have been or will be offered to the public has not been made or implied, the bollard fence may be considered exempt from licensing requirements. Notwithstanding licensing requirements, the plans do not include design criteria, concrete notes, reinforcing and structural steel notes, and foundation notes, datum, benchmarks, and items requiring structural observation and inspection, among other contents considered to meet industry standards.
- The minimum lap of 24 inches for shrinkage and temperature reinforcement does not meet the 31-inch required for a Class B splice, unless the lap is staggered to meet the requirements of ACI 318-14 Building Code Requirement for Structural Concrete, Section 25.5.2.1.
- The calculations prepared by Stinger Bridge & Iron, dated November 21, 2019, are not signed and sealed by a licensed professional in the State of Texas; however, as stated before, as long as a representation that engineering services have been or will be offered to the public has not been made or implied, the bollard fence may be considered exempt from licensing requirements.
- The stability analysis and design calculations account for dead loads, wind pressure, and earth pressure. Flood loads (e.g., hydrostatic, hydrodynamic, and floating debris) that may result from an unusual event such as 1967 Hurricane Beulah were entirely missed, even though the TGR hydraulic model indicated exposure to flood waters.
- The operation and maintenance plan by TGR acknowledges that "uncontrolled growth of invasive species would further impede and redirect the flow" for the modified environment (post-construction of bollard fence). This statement is itself an acknowledgement of the potential impact of the modified environment on the natural flow of the river, hence posing further risks to the stability and integrity of the structure.
- The light/camera pole foundation is designed as 4 feet embedded into the ground with 2 feet exposed above ground; however, the exposed height was at times higher due to variance in finished grade with respect to the typical system cross section, which results in greater exposure to lateral loads coupled with less axial and lateral geotechnical capacities of the foundation.
- At some locations where the edge of the foundation was exposed due to erosion, the thickness of the footing was less than the 1 foot shown on plans. This non-conformance has an adverse impact on the external and internal stabilities by design (and even for the purpose of this engineering assessment) of the bollard fence. Similarly, any present erosion would have an adverse impact on the passive resistance assumed by design and for the purpose of this assessment, unless effectively mitigated.
- The structural engineering assessment of the bollard fence system external stability included the following three loading conditions that may result from an unusual event such as 1967 Hurricane Beulah:

  - Case A: This loading condition accounts for maximum flow velocity during rising water levels from the river side as the floodplain begins to fill. The bollard does not meet sliding and location of resultant criteria. Noteworthy, this condition occurs early in the design flood, with relatively shallow depths of flow over the base of the fence and the floodplain behind the fence just beginning to fill, it would likely occur during much smaller and more frequent floods than a Beulah-magnitude event.
  - Case B: This loading condition accounts for the maximum water surface during rising water levels from the river side in the western segment of the bollard fence. The bollard not only does not meet sliding, flotation, location of resultant, and bearing pressure criteria, but would effectively slide, overturn, and become buoyant.
  - Case C: This loading condition accounts for the maximum water surface during rising water levels from the land side in the western segment of the bollard fence. The bollard not only does not meet sliding, flotation, location of resultant, and bearing pressure criteria, but would effectively slide, overturn, and become buoyant.
- The structural engineering assessment of the light/camera monopole external stability included the following two loading conditions that may result from an unusual event such as 1967 Hurricane Beulah:
  - Case A: This loading condition accounts for maximum flow velocity during rising water levels from the river side. The monopole does not meet sliding and location of resultant criteria, and it would effectively slide and/or overturn.
  - Case C: This loading condition accounts for the maximum water surface during rising water levels from the land side in the western segment of the bollard fence. The monopole does not meet sliding and location of resultant criteria, and it would effectively slide and/or overturn.
- The structural engineering assessment of the bollard fence system internal stability (strength) included the following two loading conditions that may result from an unusual event such as 1967 Hurricane Beulah:
  - Case A: This loading condition accounts for maximum flow velocity during rising water levels from the river side. For this loading condition, the foundation and bollard have adequate flexural and shear strength.
  - Case B: This loading condition accounts for the maximum water surface during rising water levels from the river side in the western segment of the bollard fence. For this loading condition, the flexural and shear strength of the foundation will not be tested because the bollard fence would become buoyant prior to reaching any strength limit, and the bollard itself would experience inelastic (permanent) deformations if impacted by the 1,000-pound mass outlined in the strength criterion unless the impact is distributed over six or more bollards.

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Appendix A

Existing Information Provided by Department of Justice (provided separately)

A.1 TGR HEC-RAS Model

A.2 Daily Pool Elevations for Anzalduas Dam

Appendix B

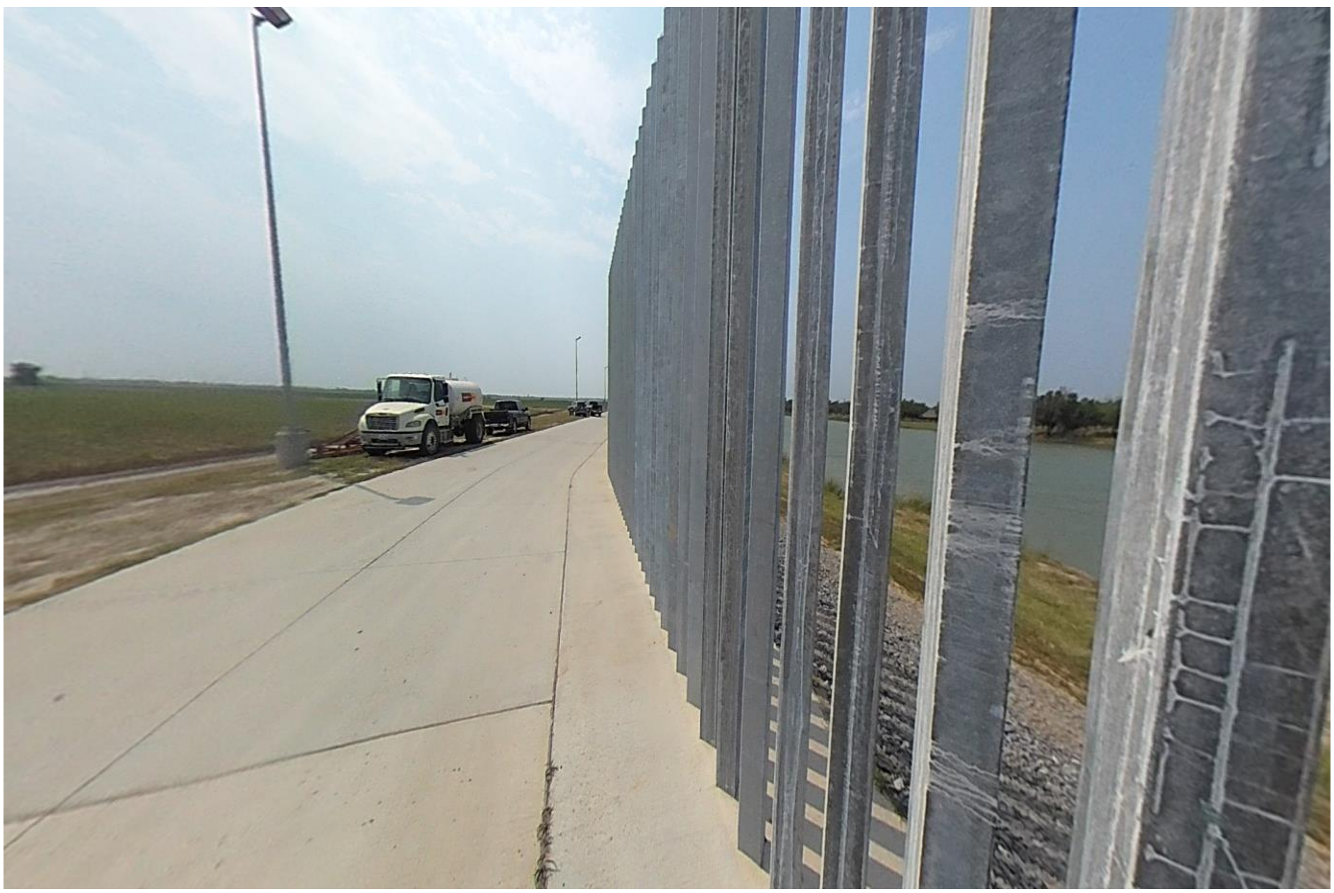
Arcadis Site and Subsurface Investigation Report

United States Department of Justice

Site and Subsurface Investigation

Border Fence
Mission, Texas

July 2021



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Acronyms and Abbreviations

Arcadis	Arcadis U.S., Inc.
ASTM	ASTM International
bgs	below ground surface
DOJ	United States Department of Justice
Fisher	Fisher Industries
GPR	ground-penetrating radar
LL	liquid limit
MEG	Millennium Engineering Group
NDT	non-destructive testing
pcf	pounds per cubic foot
PI	plasticity index
PL	plastic limit
psi	pounds per square inch
STA	station
TP	test pit

1 Introduction

The United States Department of Justice (DOJ), McAllen Division retained Arcadis U.S., Inc. (Arcadis) to evaluate the site and subsurface conditions associated with the border fence (fence) constructed by Fisher Industries (Fisher) along the Rio Grande River near Mission, Texas. Plan sheets prepared by TGR Construction, Inc. (TGR), a subsidiary of Fisher, in 2019 as used for construction of the fence were provided to Arcadis for review and are included in Attachment 1.

Jason Vazquez and John Sparks of Arcadis completed a site visit on April 27, 2021, accompanied by Paxton Warner of DOJ and Tommy Fisher of Fisher. During the site visit, Mr. Fisher described the fence materials and construction methods, as well as fence performance and maintenance conducted since construction. Arcadis documented the site conditions with photographs. Non-destructive testing (NDT) was completed during the site visit by Terracon to measure the thickness of steel bollards, estimate the configuration of reinforcing steel, and measure the compressive strength of the concrete footing. NDT results are provided in Attachment 2.

Subsurface conditions were investigated by excavating test pits (TPs) on April 28 and 29, 2021, along the riverside of the fence. Terracon contractor excavated twelve TPs using a JCB 8069 mini-excavator to depths of 7 to 9 feet below ground surface (bgs). Soil samples were collected at depths of 3 feet or 6 feet bgs for laboratory testing to confirm material properties. Terracon completed sand cone density tests per ASTM International (ASTM) D-1556 at a depth of 3 feet bgs and bulk samples were collected from depths of either 3 feet or 6 feet bgs for Standard Proctor testing per ASTM D-698 and recorded on the test pit logs in Attachment 3. Photographs of TP excavations and field testing are provided in Attachment 4, and soil testing summaries and results are provided in Attachment 5. Site and subsurface details are described as follows.

2 Background Information

The fence was constructed in 2019–2020 and consists of approximately 3 miles of 6-inch by 6-inch square tube steel bollards spaced at 1.125 feet on center to a height of 18 feet above ground. The fence includes a 20-foot-wide road section and 30-foot-tall light poles with security cameras on 6-foot-tall, 3-foot-diameter, pre-cast concrete foundations spaced approximately every 200 feet along the fence. The bollards are 1/8-inch-thick galvanized steel with 5 inches of clear space between bollards and are embedded into a reinforced concrete T-shaped footing as shown on Figure 1. The fence was constructed along the Rio Grande riverbank, approximately 8 to 20 feet from the shoreline for normal water levels. The fence alignment begins at Station (STA) 0+00 near the downstream limits and increases every 100 feet upstream to terminate near STA 156+00 as shown on Figure 2.

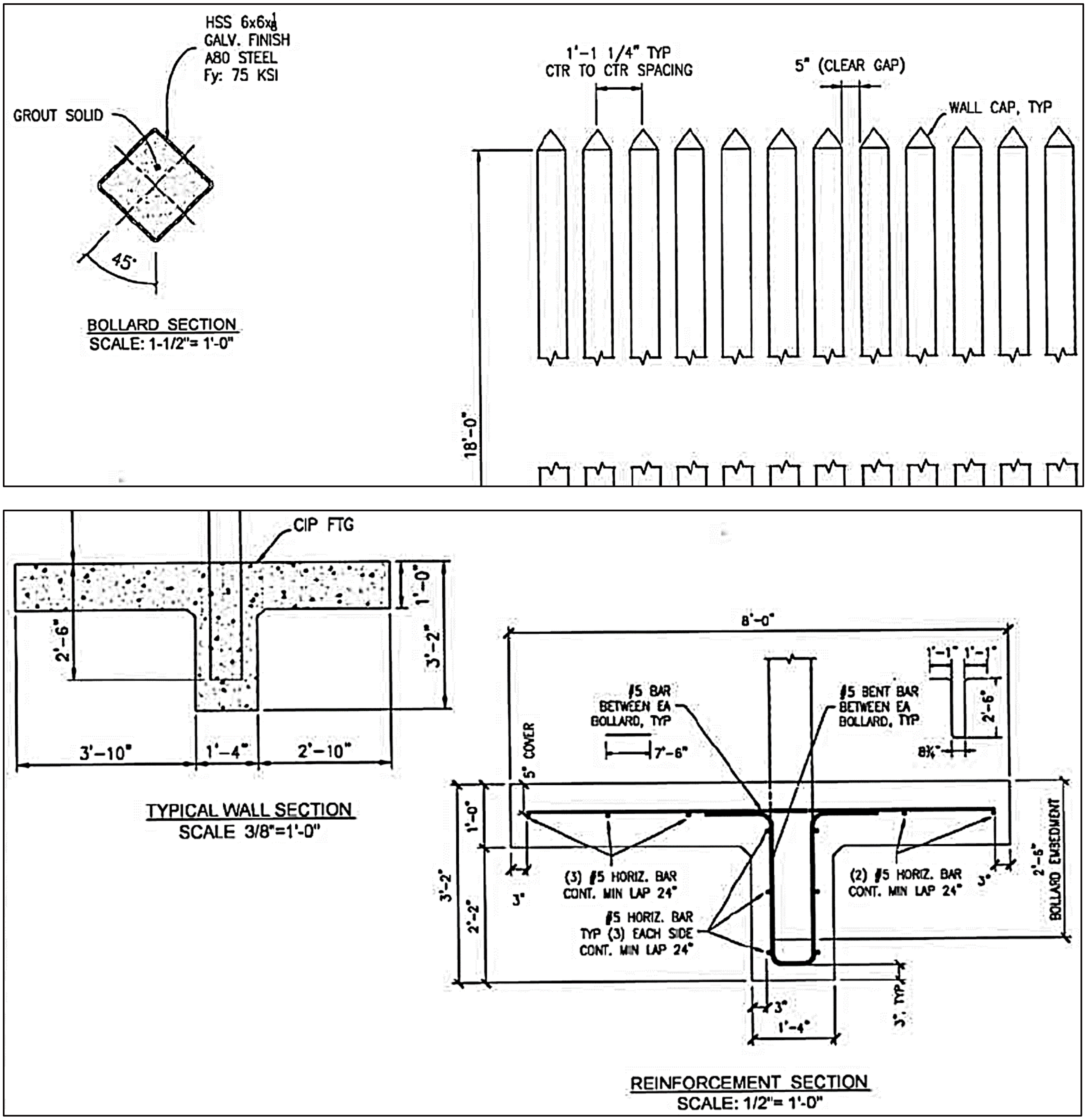


Figure 1: Construction Details from Plans by TGR Dated October 30, 2019

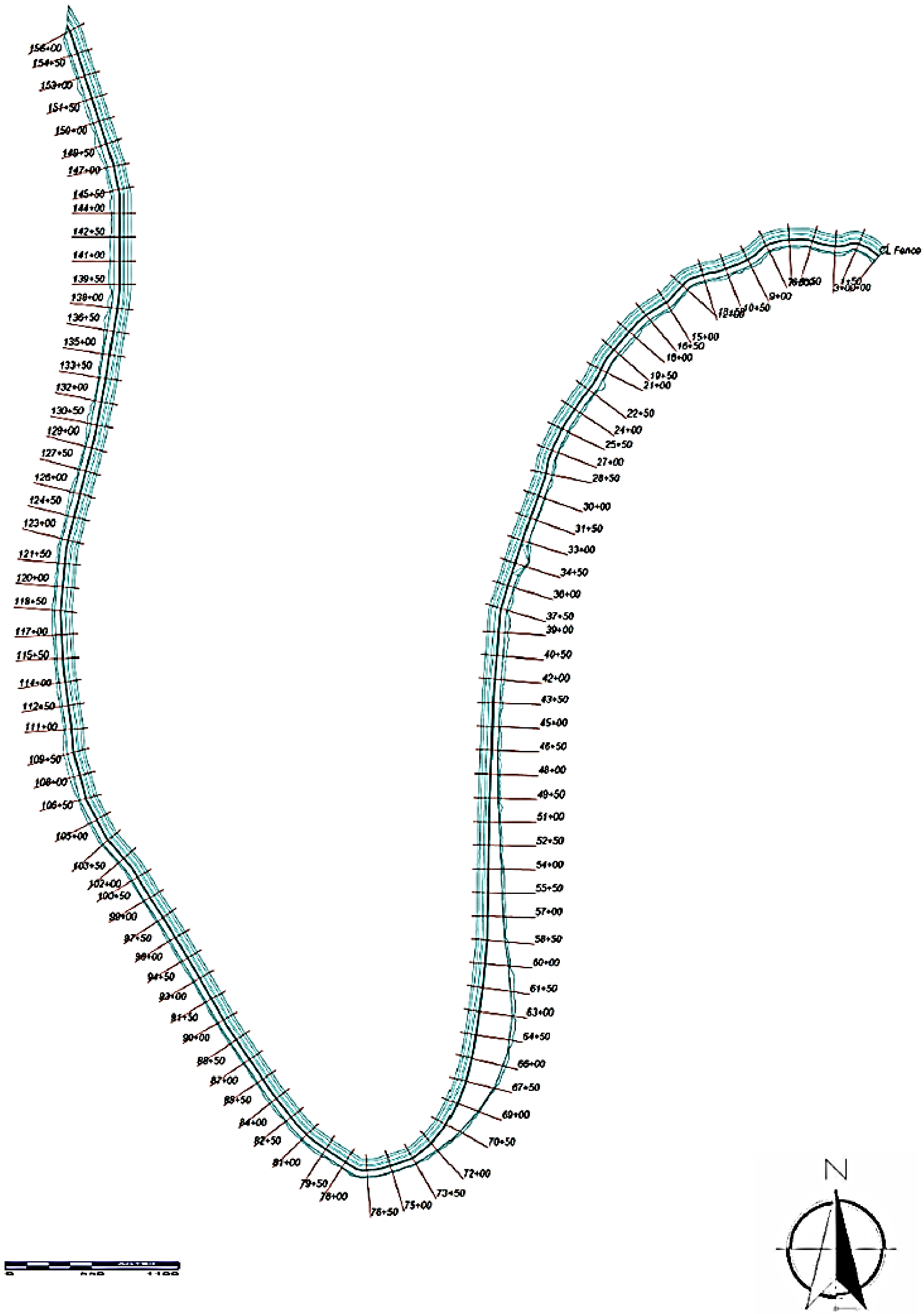


Figure 2: Fence Alignment from McAllen Border Fence Cross Section Plan Overview by Fisher 2020

2.1 Fence Materials and Construction

Mr. Fisher reported that the fence was constructed as shown on the plans provided (details shown on Figure 1). An exception is that the bollards were not grouted solid as shown on the plans, and instead were backfilled with an unspecified grade “pea” gravel. No material specifications, quality control test results, or field reports were provided regarding the backfill of the bollards and no field verification was completed for this investigation. Pre-cast concrete caps were grouted to the top of the fence bollards using a rebar dowel. Shop tags providing records of galvanizing and photographs of example materials are shown on Figure 3.

Surplus steel bollards used to construct the fence section are stockpiled at the site as shown on Figure 4, and the dimensions of the 6-inch by 6-inch tube sections were verified. A photograph of typical completed fence near STA 156+00 at the upstream limits is shown on Figure 5.

Figure 3: Shop Tag from Galvanizing, Tube Steel, and Concrete Cap with Dowel

Figure 4: Surplus Steel Bollards Stockpiled on Site

Figure 5: Constructed Bollard Fence Near STA 156+00 at Upstream Limits

Concrete mix designs for the wall footing and pavement sections were not provided. Mr. Fisher reported that the concrete was designed for a compressive strength of at least 4,500 pounds per square inch (psi) at 28 days. Results of concrete compressive strength testing conducted by Millennium Engineering Group (MEG) in February 2020 are summarized in Table 1. Results indicate that compressive strengths at test locations met the criteria of greater than 4,500 psi at 28 days.

Table 1: Concrete Compressive Strength Test Summary (MEG, February 2020)

Report No.	Date	STA	Cylinder ID	Days	Strength (psi)
8-4	2/19/2020	26+00	13-A	28	5950
8-4	2/19/2020	26+00	13-B	28	6260
8-1A	2/19/2020	28+00	14-A	7	3600
8-1A	2/19/2020	28+00	14-B	28	5350
8-1A	2/19/2020	28+00	14-C	28	5320
8-2A	2/19/2020	30+00	15-A	3	2740
8-2A	2/19/2020	30+00	15-B	7	3280
8-2A	2/19/2020	30+00	15-C	28	4910
8-2A	2/19/2020	30+00	15-D	28	4800
8-3A	2/19/2020	31+60	16-A	3	3820
8-3A	2/19/2020	31+60	16-B	7	4310
8-3A	2/19/2020	31+60	16-C	28	5190
8-3A	2/19/2020	31+60	16-D	28	5180
8-5	2/19/2020	33+10	17-A	28	4800
8-5	2/19/2020	33+10	17-B	28	5340
9-2A	2/19/2020	39+00	19-A	7	3300
9-2A	2/19/2020	39+00	19-B	28	5140
9-2A	2/19/2020	39+00	19-C	28	4910
10-20	2/26/2020	73+50	36-A	28	5310
10-20	2/26/2020	73+50	36-B	28	5370
10-24	2/26/2020	83+50	45-A	28	6200
10-24	2/26/2020	83+50	45-B	28	6510
10-26	2/26/2020	96+20	47-A	28	5970
10-26	2/26/2020	96+20	47-B	28	5830
10-23	2/26/2020	-	44-A	28	5620
10-23	2/26/2020	-	44-B	28	5560

2.2 Road and Light Pole Construction

The service road that adjoins the fence footing consists of 6-inch-thick reinforced concrete rigid pavement over 6 inches of aggregate base on top of earth subgrade, with a 2% grade toward the fence. The light pole foundation is designed as a 4-foot embedment into the ground with 2 feet exposed above ground as shown on Figure 6. The pre-cast concrete foundation as used for the light poles is shown on Figure 7 and Figure 8. The light pole is galvanized steel. No details were provided of the light pole anchor into the concrete foundation.

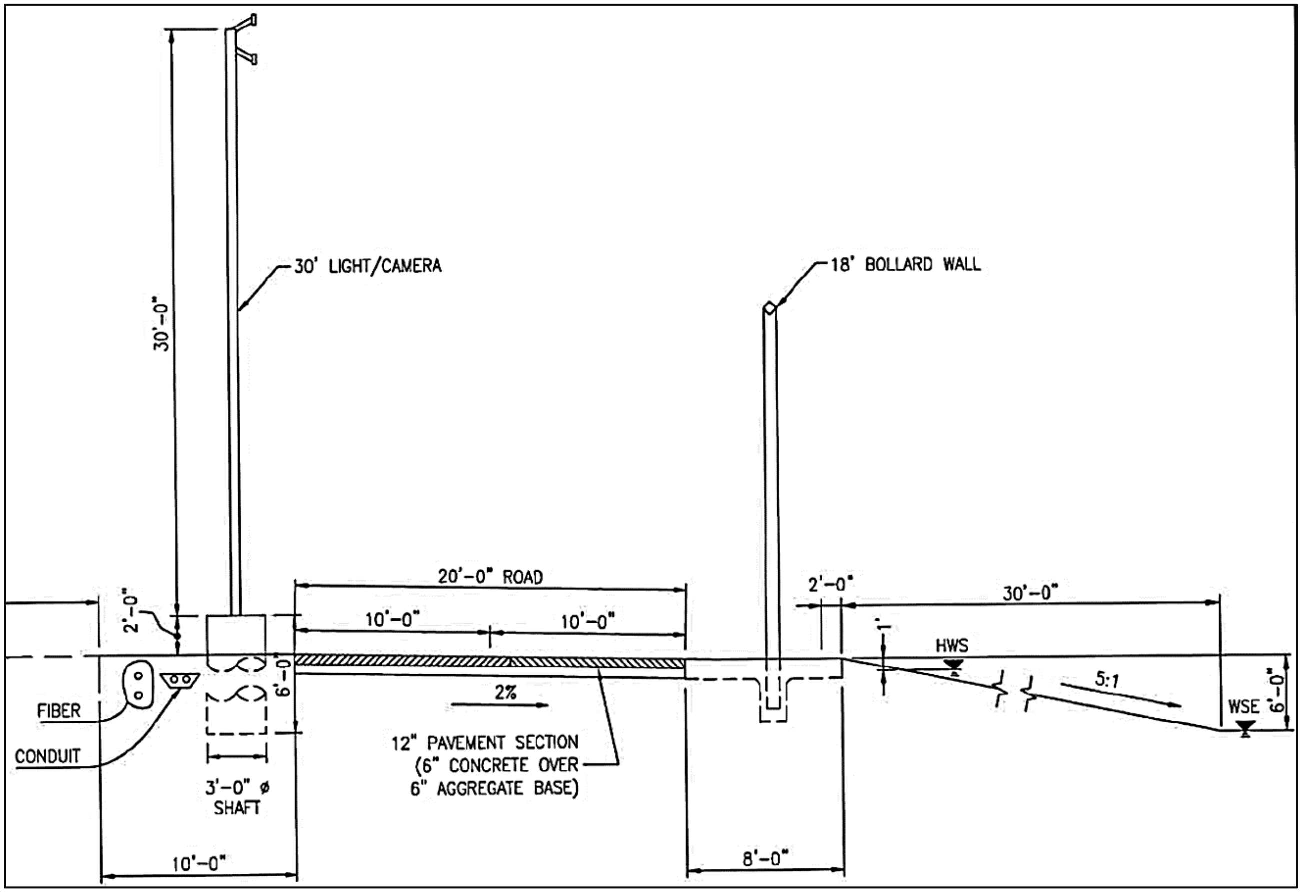


Figure 6: Typical Fence Section with Road and Utility Poles from Plans by TGR Dated October 30, 2019

Figure 7: Pre-cast Concrete Footing for Utility Pole Installation

Figure 8: Utility Pole at the Edge of the Road Near STA 0+00

3 Site Conditions

General observations were made of the site conditions and fence structures. NDT was used to measure the thickness of bollards in place on the fence at random locations approximately every ¼ mile along the fence alignment. The measurements are provided in the field logs included in Attachment 2.

3.1 Concrete Surfaces

The surface condition of the concrete was observed to be fair to good, with minor cracking and joint separation detected at various locations along the fence alignment. Cracks in the fence footing appeared to be surficial only as shown on Figure 9, which is typical of shrinkage cracking that starts near the angle of the bollard and extends to the edge of footing in both directions as shown on Figure 10 and Figure 11. Cracks and Joint separation were also observed on the access road that abuts the fence footing as shown in Figure 12 and Figure 13.

Figure 9: Surface Cracking Near STA 152+50 from Bollard to Edge of Footing

Figure 10: Surface Cracking Near STA 36+50 All the Way Across Footing and Around Bollard

Figure 11: Surface Cracking Near STA 152+50 Around Bollard and to Edge of Footing

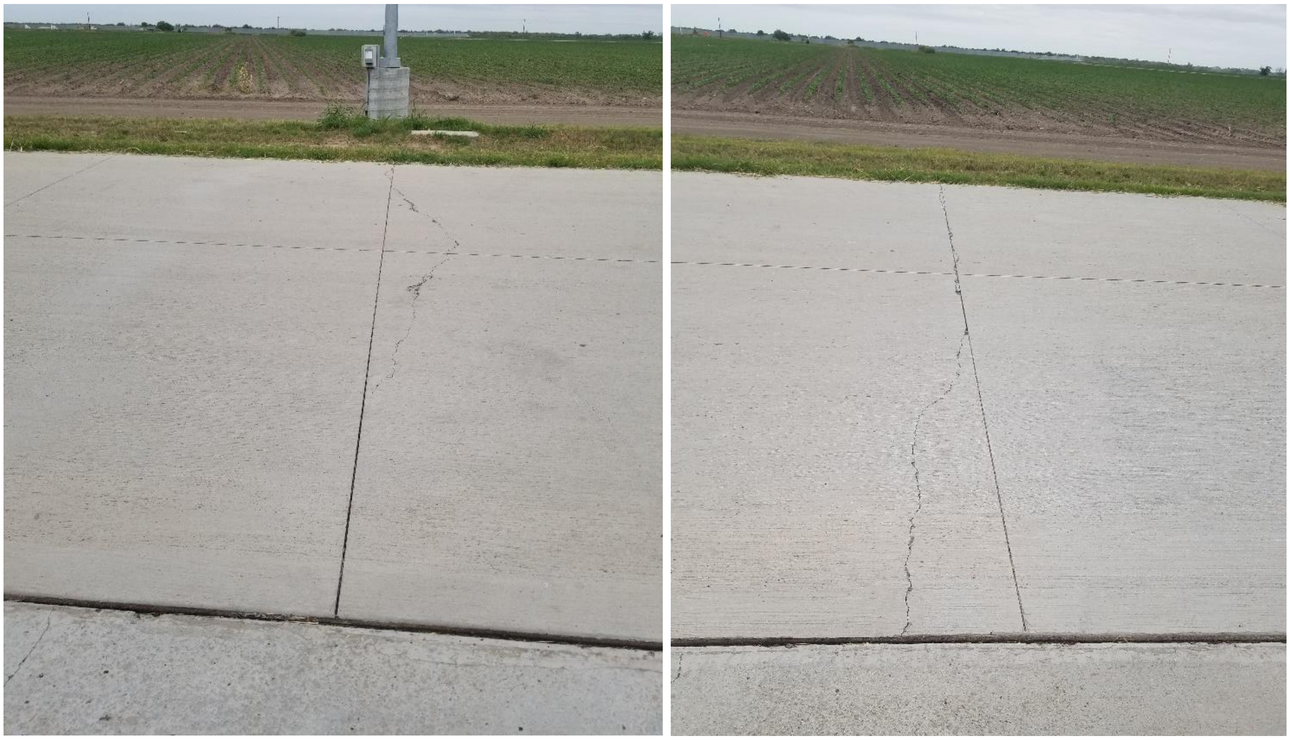


Figure 12: Typical Cracking of Concrete Surface along Access Road

Figure 13: Typical Joint Separation along Access Road Abutment to Concrete Footing

3.2 Footing Dimensions

The depth of the concrete footing shown in the design (Figure 1) is 1 foot up to the steel bollards, where the depth increases to 3 feet for the embedment of the bollards. The edge of the footing on the riverside of the fence was evaluated at five locations along the alignment to confirm the depth of the concrete. This evaluation indicated that the edge of the concrete is not a uniform depth of 1 foot, with one location measuring as little as 4.5 inches as shown on Figure 14. One location evaluated near STA 156+00 did measure a full 12 inches as shown on Figure 15.

Figure 14: Footing Depth Measurement of Riverside Edge Near STA 0+00

Figure 15: Footing Depth Measurement of Riverside Edge Near STA 156+00

3.3 Non-Destructive Testing

Terracon used NDT to measure the concrete strength with a rebound hammer and the spacing of reinforcement with a ground-penetrating radar (GPR) as described in Attachment 2. Tests were conducted at approximately every ¼ mile, for a total of 12 test locations. Results indicate that the concrete at test locations has a compressive strength of approximately 4,500 psi per the design. GPR scans indicate that the reinforcement is #5 rebar at approximately 5 inches on center in both directions per the plans, as shown on Figures 16 through 18 and reported in Attachment 2.

Terracon contracted with a certified welding inspector from BRL NDT Services for ultrasonic gauge testing to confirm the thickness of fence bollards. Testing was also conducted at 12 locations along the fence and all tested locations had a bollard thickness of at least 1/8 inch per the plans, as reported in Attachment 2.

Figure 16: Typical GPR Test Location for Rebar Spacing and Concrete Strength

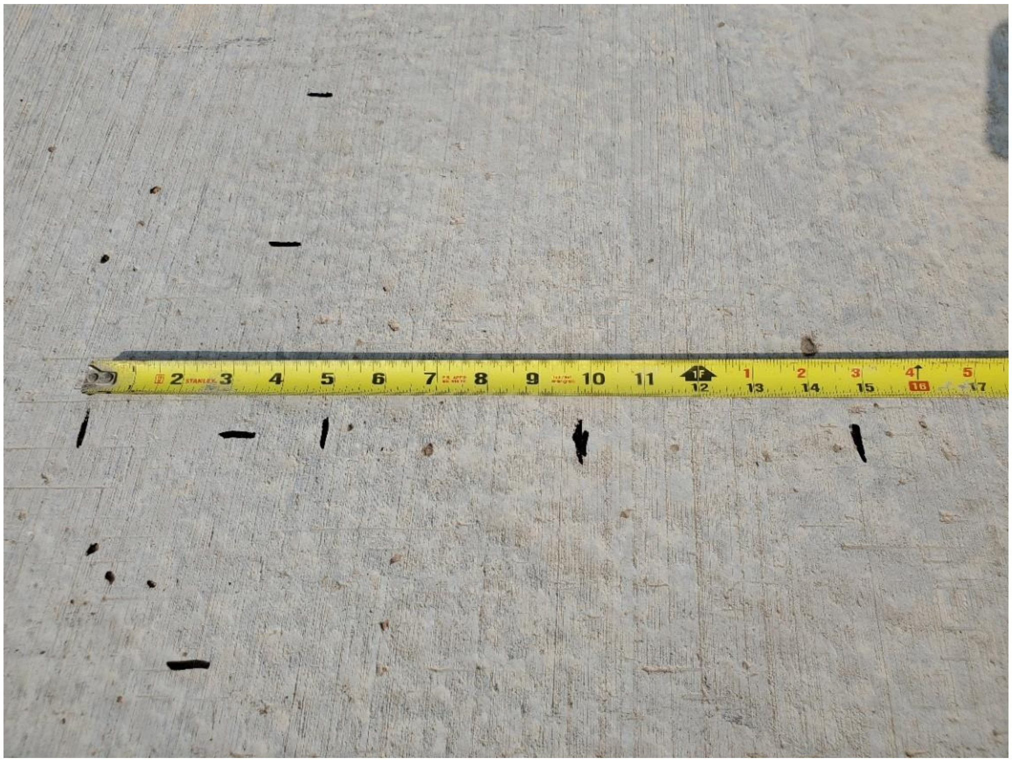


Figure 17: Rebar Spacing along footing at Approximately Every 5 Inches on Center as Determined by GPR

Figure 18: Rebar Spacing across footing at Approximately Every 5 Inches on Center as Determined by GPR

3.4 Surface Drainage

Surface water is conveyed underneath the fence foundation via corrugated high-density polyethylene (HDPE) pipes as shown on Figure 19. The design details and locations for the drainpipes were not provided for this investigation. Surface water travels as sheet flow across the road and fence footing to drain on the ground surface to the river. This surface flow has caused rills to be formed as observed along the edge of the fill material on the riverside of the fence, as shown on Figure 20. Historical information from August 2020 (included in Attachment 1) documented major rill erosion damage on the riverside of the fence following major storms that caused flooding along the river. At the time of the current investigation, most of the damage had been repaired and covered with grass; however, the rills could be seen as ground surface rutting and bare areas with limited grass cover.

Figure 19: Drainage Culverts Installed for Fence Construction

Figure 20: Typical Surface Erosion Rills on Riverside of Fence

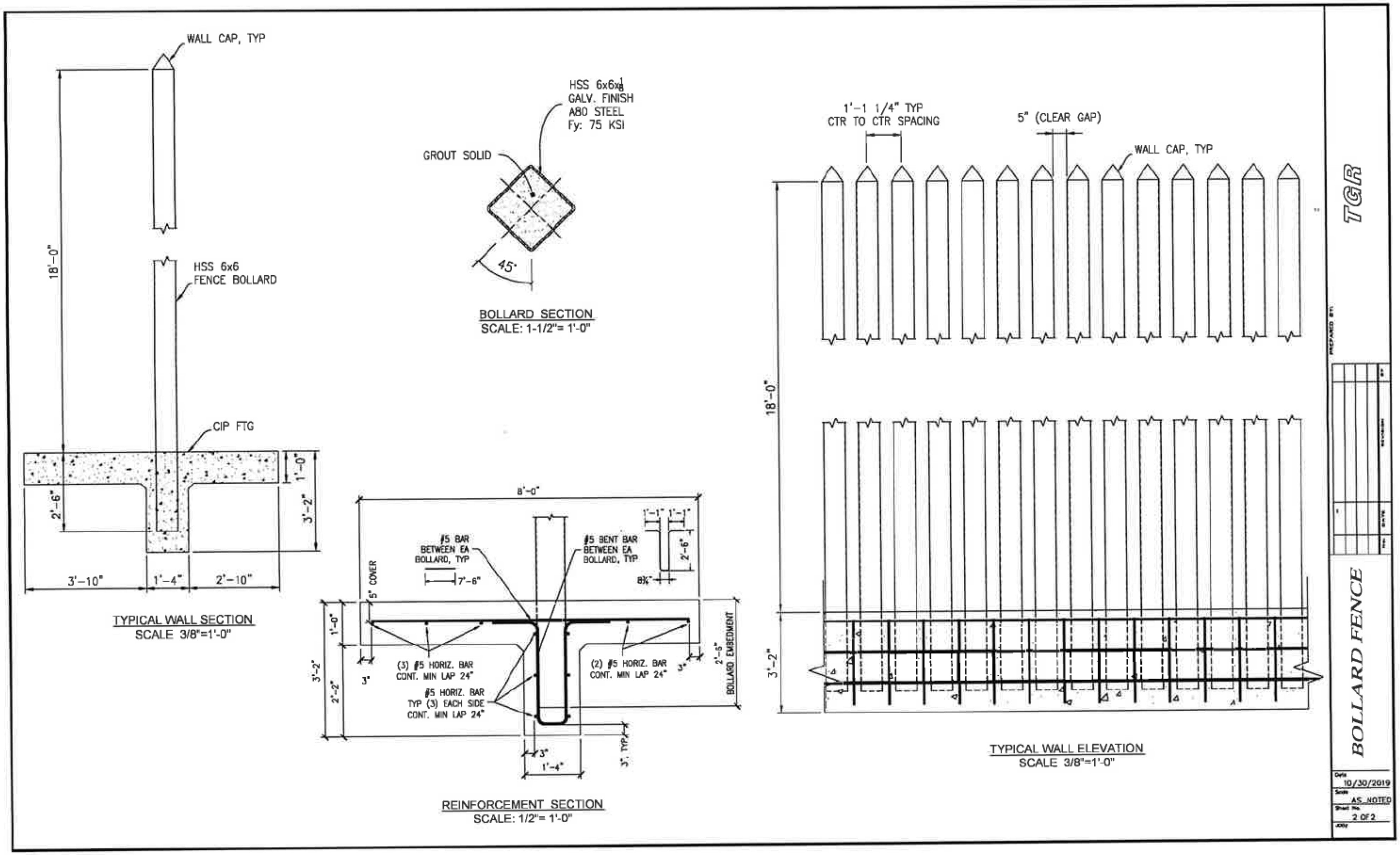
Attachment 1

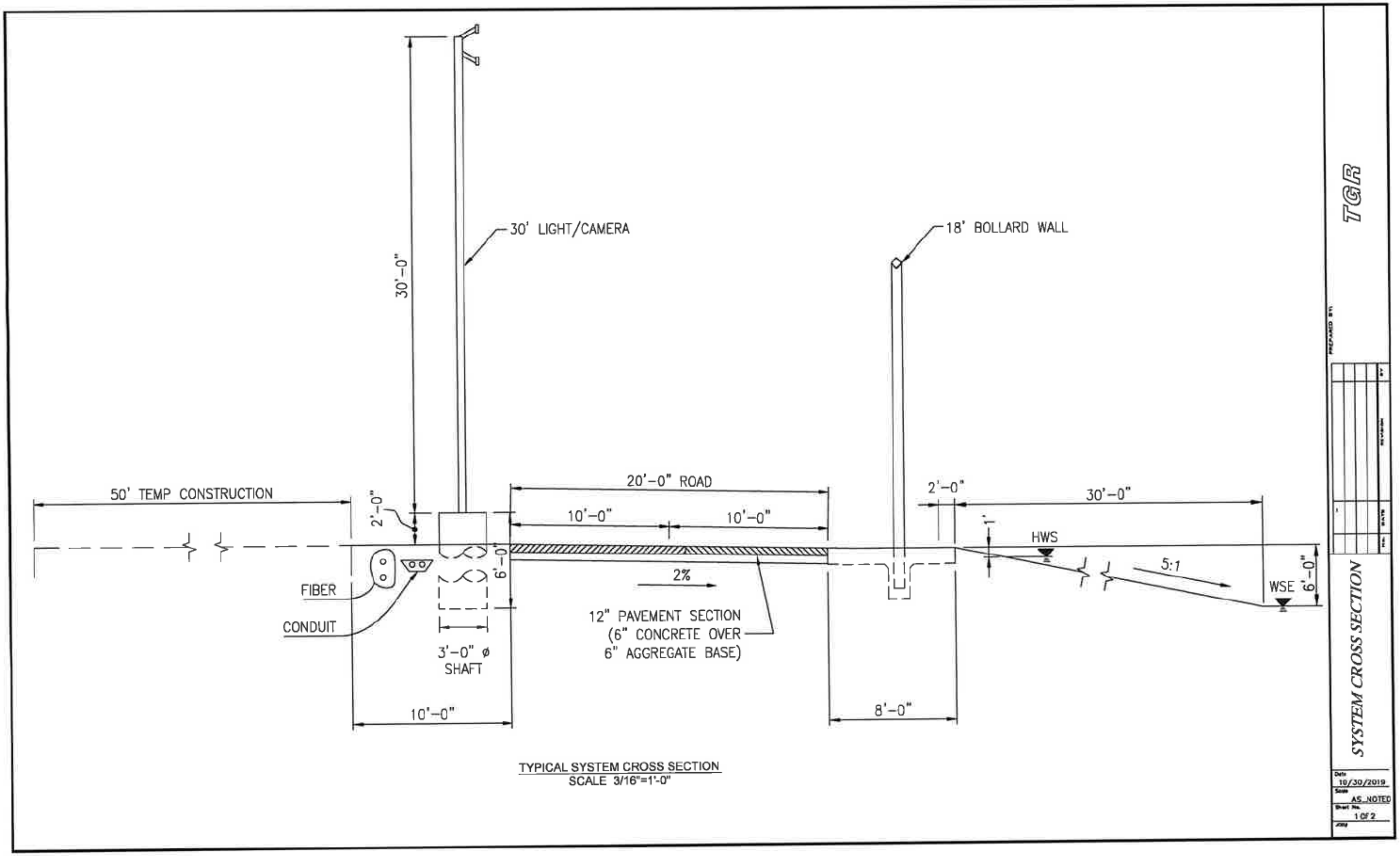
Existing Information Provided by DOJ

Plans by TRG, dated 10/30/2019 (2 pages)

Quality Control Reports by MEG, Jan-Feb 2020 (21 pages)

Erosion Damage Photos, August 2020 (4 pages)


WALL CAP, TYP
18'-0"
HSS 6x6
FENCE BOLLARD
CIP FTG
1'-0"
2'-6"
3'-2"
3'-10"
1'-4"
2'-10"
TYPICAL WALL SECTION
SCALE 3/8"=1'-0"
HSS 6x6x
GALV. FINISH
A80 STEEL
Fy: 75 KSI
GROUT SOLID
45°
BOLLARD SECTION
SCALE: 1-1/2"= 1'-0"
8'-0"
#5 BAR
BETWEEN EA
BOLLARD, TYP
7'-6"
#5 BENT BAR
BETWEEN EA
BOLLARD, TYP
1'-1"
1'-1"
2'-6"
5" COVER
(3) #5 HORIZ. BAR
CONT. MIN LAP 24"
#5 HORIZ. BAR
TYP (3) EACH SIDE
CONT. MIN LAP 24"
(2) #5 HORIZ. BAR
CONT. MIN LAP 24"
2'-6"
BOLLARD EMBEDMENT
3'-2"
2'-2"
3"
1'-4"
3", TYP
REINFORCEMENT SECTION
SCALE: 1/2"= 1'-0"
1'-1 1/4" TYP
CTR TO CTR SPACING
5" (CLEAR GAP)
WALL CAP, TYP
18'-0"
3'-2"
TYPICAL WALL ELEVATION
SCALE 3/8"=1'-0"
TGR
PREPARED BY:
BOLLARD FENCE
Date
10/30/2019
Scale
AS NOTED
Sheet No.
2 OF 2



TYPICAL SYSTEM CROSS SECTION
SCALE 3/16"=1'-0"

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Consultants - Geotechnical - Testing

TBPE FIRM No. F-3913
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5918 McPherson Rd., Ste. 5

Area Offices
Pharr, Texas 78577 956-702-8500
Harlingen, Texas 78550 956-454-8832
Laredo, Texas 78041 956-568-1664

Report On: Proctor - Soils

Lab No: 9584
Report No: 1-1

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

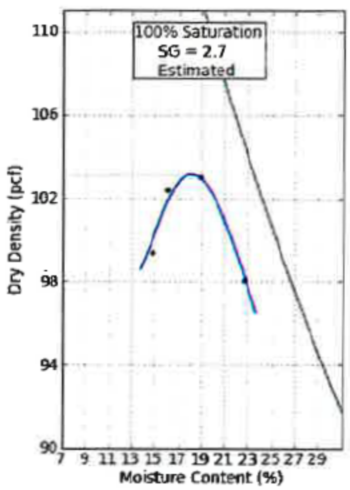
Project: McAllen Border Fence

Report Date: 11/22/2019

Location: STA 11+50, At Center Line of Road

Sample Date: 11/15/2019
Sampled By: Client

Material: Subgrade



% Moisture	Dry Density Lbs./Cu.Ft.
14.8	99.40
16.1	102.4
18.9	103.0
22.7	98.10
18.2 Optimum	103.2 Maximum

Sieve	% Passing
No. 4	100
No. 10	100
No. 40	100
No. 200	40

Color: Brown
Description: Clayey Sand

Liquid Limit: 26
Plastic Limit: 22
Plasticity Index: 4

Desc of Rammer:Manual
Preparation Method:Moist

Test Method (As Applicable):ASTM D 1140, ASTM D 2487, ASTM D 4318, ASTM D-698 Method-B

Respectfully Submitted,
Millennium Engineers Group, Inc.

STATE OF TEXAS
JUAN M. BORJON
121570
LICENSED
PROFESSIONAL ENGINEER

Juan Borjon, P.E.

11/24/2019

MEG ENGINEERS
Millennium Engineers Group
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Report On: Proctor - Soils

Lab No: 9584
Report No: 1-1

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

Report Date: 11/22/2019

Location: STA 11+50, At Center Line of Road

Sample Date: 11/15/2019
Sampled By: Client

Material: Subgrade

Orig: Civil Solutions Engineering & Mgmt. LLC
Attn: Timoth C. Fish (1-cc copy)
1-ec Civil Solutions Engineering & Mgmt. LLC
Attn: Timothy C. Fish
1-ec Civil Solutions Engineering & Mgmt. LLC
Attn: John Halvarson
1-ec Civil Solutions Engineering & Mgmt. LLC
Attn: Bruce Meyer
1-ec Millennium Engineers Group
Attn: Humberto Palma
1-ec Millennium Engineers Group
Attn: Juan M. Borjon
1-ec Millennium Engineers Group Attn: Andres Palma
1-ec Millennium Engineers Group Attn: Sergio Tovar

Respectfully Submitted,
Millennium Engineers Group, Inc.

STATE OF TEXAS
JUAN M. BORJON
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Juan Borjon

Juan Borjon, P.E.

11/24/2019

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Area Offices

Report On: Proctor - Soils

Lab No: 9584-1
Report No: 2-1

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

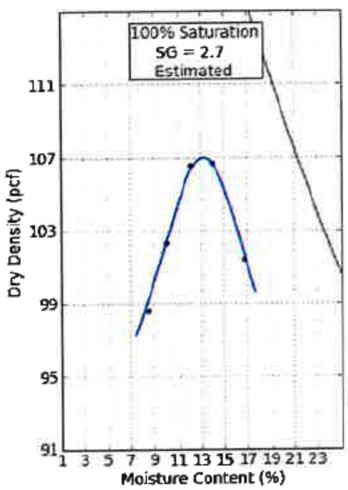
Project: McAllen Border Fence

Report Date: 11/22/2019

Location: STA 12+00, Left Water Edge

Sample Date: 11/15/2019
Sampled By: Client
Field ID: 5768

Material: Subgrade



% Moisture		Dry Density Lbs./Cu.Ft.	
8.5		98.60	
10.1		102.3	
12.1		106.6	
14.0		106.7	
16.7		101.4	
13.3	Optimum	107.0	Maximum

Sieve	% Passing
No. 4	100
No. 10	100
No. 40	99
No. 200	89

Color: Brown
Description: Fat Clay

Liquid Limit: 50
Plastic Limit: 22
Plasticity Index: 28

Desc of Rammer:Manual
Preparation Method:Moist

Test Method (As Applicable):ASTM D 1140, ASTM D 2487, ASTM D 4318, ASTM D-698 Method-B

Respectfully Submitted,
Millennium Engineers Group, Inc.

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Report On: Proctor - Soils

Lab No: 9584-1
Report No: 2-1

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

Report Date: 11/22/2019

Location: STA 12+00, Left Water Edge

Sample Date: 11/15/2019
Sampled By: Client
Field ID: 5768

Material: Subgrade

Orig: Civil Solutions Engineering & Mgmt. LLC
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1-ec Millennium Engineers Group Attn: Sergio Tovar

Respectfully Submitted,
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Report On: **Proctor - Soils**

Lab No: 9584-2
Report No: 3-1

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

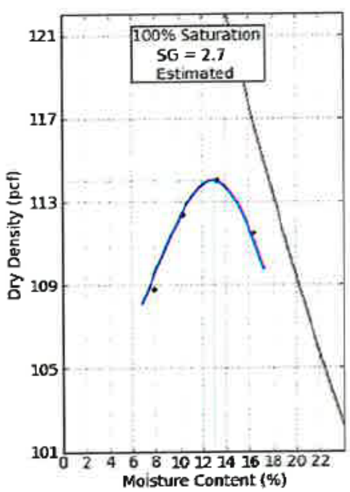
Project: McAllen Border Fence

Report Date: 11/21/2019

Location: STA 12+00, Left Water Edge

Sample Date: 11/15/2019
Sampled By: Client

Material: Subgrade



% Moisture	Dry Density Lbs./Cu.Ft.
7.9	108.8
10.3	112.4
13.3	114.0
16.3	111.5
13.0 Optimum	114.0 Maximum

Sieve	% Passing
No. 4	100
No. 10	100
No. 40	100
No. 200	73

Color: Brown
Description: Lean Clay with Sand

Liquid Limit: 31
Plastic Limit: 18
Plasticity Index: 13

Desc of Rammer:Manual
Preparation Method:Moist

Test Method (As Applicable):ASTM D 1140, ASTM D 2487, ASTM D 4318, ASTM D-698 Method-B

Respectfully Submitted,
Millennium Engineers Group, Inc.

STATE OF TEXAS
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11/24/2019

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Report On: Proctor - Soils

Lab No: 9584-2
Report No: 3-1

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

Report Date: 11/21/2019

Location: STA 12+00, Left Water Edge

Sample Date: 11/15/2019
Sampled By: Client

Material: Subgrade

Orig: Civil Solutions Engineering & Mgmt. LLC
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1-ec Millennium Engineers Group Attn: Sergio Tovar

Respectfully Submitted,
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Report On: Concrete Compression

Lab No: 10664-3
Report No: 8-4

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

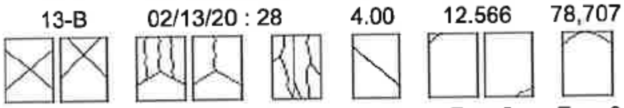
Report Date: 02/19/2020

Location: Mission, Hidalgo County, Texas

Sample Date: 01/16/2020
Sampled By: Client
By Order Of: Bruce
Field ID: 01-19-19300

Cylinder Marked	Age Tested (date : days)	Diameter (in)	Area (in²)	Max Load (lbs)	Break Type	Cure Loc	Compressive Strength (PSI)	Average Strength (PSI)	Tested By
13-A	02/13/20 : 28	4.00	12.566	74,784	Type 3	Lab	5,950		
13-B	02/13/20 : 28	4.00	12.566	78,707	Type 3	Lab	6,260	6,110	

MEETS REFERENCE VALUE



	Measurement	Specification
Temp.: Ambient:	76ºF	NAºF
Mix:	84ºF	NAºF
Slump:	8.5	Min. 8.5
Air Content:		

Specification: 4,500 psi @ 28 days
Source: Fisher
Plant:
TruckNo: 1407
Mix Code: TEXBF6.5
Ticket No: NA
Sampled At: Truck

Weather: NA
Transported By: Client
Placement Date: 01/16/2020
Time Batched: 8:04 am
Time Sampled: 8:12 am
Curing Method: Standard

Quantity Represented: 33 cu. yds.
Placement Location: STA 26+00, 3' R FT CL
Sample Location: Bollard Wall Footer
Remarks:
Test Method (As Applicable): Unless noted, concrete was sampled and tested in accordance with ASTM C172, C143, C231 or C173, C1054 and C138. Compressive strength tests per ASTM C39, C31.

Respectfully Submitted,
Millennium Engineers Group, Inc.

STATE OF TEXAS
JUAN M. BORJON
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02/20/2020

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Area Offices
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Harlingen, Texas 78550 956-454-8832
Laredo, Texas 78041 956-568-1664

Report On: **Concrete Compression**

Lab No: **10664**
Report No: 8-1A

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

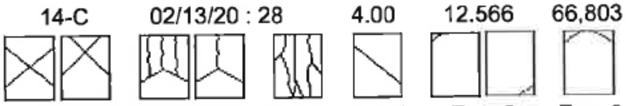
Project: McAllen Border Fence

Report Date: 02/19/2020 **Revised Test Report**
Prev. Rpt. Date: 02/04/2020
Sample Date: 01/16/2020
Sampled By: Humberto Palma
By Order Of: Bruce
Field ID: 01-19-19300

Location: Mission, Hidalgo County, Texas

Cylinder Marked	Age Tested (date : days)	Diameter (in)	Area (in²)	Max Load (lbs)	Break Type	Cure Loc	Compressive Strength (PSI)	Average Strength (PSI)	Tested By
14-A	01/23/20 : 7	4.00	12.566	45,270	Type 5	Lab	3,600	3,600	
14-B	02/13/20 : 28	4.00	12.566	67,198	Type 3	Lab	5,350		
14-C	02/13/20 : 28	4.00	12.566	66,803	Type 3	Lab	5,320	5,330	

MEETS REFERENCE VALUE



Type 1 Type 2 Type 3 Type 4 Type 5 Type 6

	Measurement	Specification
Temp.: Ambient:	83°F	NA°F
Mix:	83°F	NA°F
Slump:	8.5	Min. 8.5
Air Content:	*	NA

Specification: 4,500 psi @ 28 days
Source: Fisher
Plant:
TruckNo: NA
Mix Code: TEXBF6.5
Ticket No: NA
Sampled At: Truck

Weather: NA
Transported By: Palma, Humberto
Placement Date: 01/16/2020
Time Batched: 11:03 am
Time Sampled: 11:10 am
Curing Method: Standard

Quantity Represented: 121 cu. yds.
Placement Location: STA 28+00, 3' R FT CL
Sample Location: Bollard Wall Footer
Remarks:
Test Method (As Applicable): Unless noted, concrete was sampled and tested in accordance with ASTM C172, C143, C231 or C173, C1054 and C138. Compressive strength tests per ASTM C39, C31.

Respectfully Submitted,
Millennium Engineers Group, Inc.

STATE OF TEXAS
JUAN M. BORJON
121570
LICENSED
PROFESSIONAL ENGINEER

Juan Borjon, P.E.
02/21/2020

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Area Offices
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Harlingen, Texas 78550 956-454-8832
Laredo, Texas 78041 956-568-1664

Report On: Concrete Compression

Lab No: 10664-1
Report No: 8-2A

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

Report Date: 02/19/2020 **Revised Test Report**
Prev. Rpt. Date: 02/04/2020
Sample Date: 01/16/2020
Sampled By: Humberto Palma
By Order Of: Bruce
Field ID: 01-19-19300

Location: Mission, Hidalgo County, Texas

Cylinder Marked	Age Tested (date : days)	Diameter (in)	Area (in²)	Max Load (lbs)	Break Type	Cure Loc	Compressive Strength (PSI)	Average Strength (PSI)	Tested By
15-A	01/19/20 : 3	4.00	12.566	34,381	Type 5	Lab	2,740	2,740	
15-B	01/23/20 : 7	4.00	12.566	41,158	Type 5	Lab	3,280	3,280	
15-C	02/13/20 : 28	4.00	12.566	61,756	Type 3	Lab	4,910		
15-D	02/13/20 : 28	4.00	12.566	60,275	Type 5	Lab	4,800	4,860	

MEETS REFERENCE VALUE


Type 1 Type 2 Type 3 Type 4 Type 5 Type 6

	Measurement	Specification
Temp.: Ambient:	83ºF	NAºF
Mix:	83ºF	NAºF
Slump:	8.5	Min. 8.5
Air Content:	*	NA

Specification: 4,500 psi @ 28 days
Source: Fisher
Plant:
TruckNo: NA
Mix Code: TEXBF6.5
Ticket No: NA
Sampled At: Truck

Weather: NA
Transported By: Palma, Humberto
Placement Date: 01/16/2020
Time Batched: 1:50 pm
Time Sampled: 1:59 pm
Curing Method: Standard

Quantity Represented: 198 cu. yds.
Placement Location: STA 30+00, 3' R FT CL
Sample Location: Bollard Wall Footer
Remarks:
Test Method (As Applicable): Unless noted, concrete was sampled and tested in accordance with ASTM C172, C143, C231 or C173, C1054 and C138. Compressive strength tests per ASTM C39, C31.

Respectfully Submitted,
Millennium Engineers Group, Inc.

STATE OF TEXAS
JUAN M. BORJON
121570
LICENSED
PROFESSIONAL ENGINEER

Juan Borjon, P.E.
02/21/2020

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Area Offices

Pharr, Texas 78577	956-702-8500
Harlingen, Texas 78550	956-454-8832
Laredo, Texas 78041	956-568-1664

Report On: Concrete Compression

Lab No: 10664-2
Report No: 8-3A

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

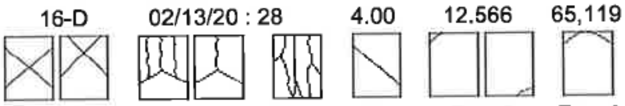
Project: McAllen Border Fence

Report Date: 02/19/2020 **Revised Test Report**
Prev. Rpt. Date: 02/04/2020
Sample Date: 01/16/2020
Sampled By: Humberto Palma
By Order Of: Bruce
Field ID: 01-19-19300

Location: Mission, Hidalgo County, Texas

Cylinder Marked	Age Tested (date : days)	Diameter (in)	Area (in²)	Max Load (lbs)	Break Type	Cure Loc	Compressive Strength (PSI)	Average Strength (PSI)	Tested By
16-A	01/19/20 : 3	4.00	12.566	48,028	Type 5	Lab	3,820	3,820	
16-B	01/23/20 : 7	4.00	12.566	54,143	Type 5	Lab	4,310	4,310	
16-C	02/13/20 : 28	4.00	12.566	65,165	Type 3	Lab	5,190		
16-D	02/13/20 : 28	4.00	12.566	65,119	Type 5	Lab	5,180	5,180	

MEETS REFERENCE VALUE



Type 1 Type 2 Type 3 Type 4 Type 5 Type 6

	Measurement	Specification
Temp.: Ambient:	84°F	NA°F
Mix:	83°F	NA°F
Slump:	8.5	Min. 8.5
Air Content:	*	NA

Specification: 4,500 psi @ 28 days
Source: Fisher
Plant:
TruckNo: NA
Mix Code: TEXBF6.5
Ticket No: NA
Sampled At: Truck

Weather: NA
Transported By: Palma, Humberto
Placement Date: 01/16/2020
Time Batched: 3:41 pm
Time Sampled: 3:50 pm
Curing Method: Standard

Quantity Represented: 253 cu. yds.
Placement Location: STA 31+60, 3' R FT CL
Sample Location: Bollard Wall Footer
Remarks:
Test Method (As Applicable): Unless noted, concrete was sampled and tested in accordance with ASTM C172, C143, C231 or C173, C1054 and C138. Compressive strength tests per ASTM C39, C31.

Respectfully Submitted,
Millennium Engineers Group, Inc.

Juan Borjon, P.E.

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121570
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PROFESSIONAL ENGINEER

02/21/2020

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1221 E. Tyler Ave.	Harlingen, Texas 78550	956-454-8832
5918 McPherson Rd., Ste. 5	Laredo, Texas 78041	956-568-1664

Report On: Concrete Compression

Lab No: 10664-4
Report No: 8-5

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

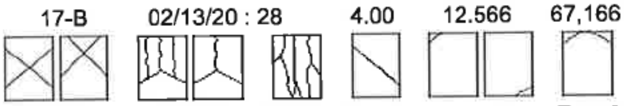
Report Date: 02/19/2020

Sample Date: 01/16/2020
Sampled By: Humberto Palma
By Order Of: Bruce
Field ID: 01-19-19300

Location: Mission, Hidalgo County, Texas

Cylinder Marked	Age Tested (date : days)	Diameter (in)	Area (in^2)	Max Load (lbs)	Break Type	Cure Loc	Compressive Strength (PSI)	Average Strength (PSI)	Tested By
17-A	02/13/20 : 28	4.00	12.566	60,357	Type 3	Lab	4,800		
17-B	02/13/20 : 28	4.00	12.566	67,166	Type 3	Lab	5,340	5,070	

MEETS REFERENCE VALUE



	Measurement	Specification
Temp.: Ambient:	73°F	NA°F
Mix:	81°F	NA°F
Slump:	8.5	Min. 8.5
Air Content:		

Specification: 4,500 psi @ 28 days
Source: Fisher
Plant:
TruckNo: 1408
Mix Code:
Ticket No: NA
Sampled At: Truck

Weather: NA
Transported By: Client
Placement Date: 01/16/2020
Time Batched: 6:48 pm
Time Sampled: 6:55 pm
Curing Method: Standard

Quantity Represented: 330 cu. yds.
Placement Location: STA 33+10, 3' RT CL
Sample Location: Bollard Wall Footer
Remarks:
Test Method (As Applicable): Unless noted, concrete was sampled and tested in accordance with ASTM C172, C143, C231 or C173, C1054 and C138. Compressive strength tests per ASTM C39, C31.

Respectfully Submitted,
Millennium Engineers Group, Inc.

STATE OF TEXAS
JUAN M. BORJON
121570
LICENSED
PROFESSIONAL ENGINEER

Juan Borjon, P.E.
02/20/2020

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Harlingen, Texas 78550 956-454-8832
Laredo, Texas 78041 956-568-1664

Report On: Concrete Compression

Lab No: 10665-1
Report No: 9-2A

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

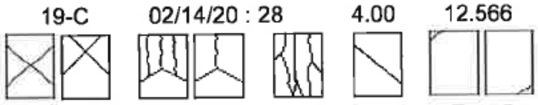
Project: McAllen Border Fence

Report Date: 02/19/2020 **Revised Test Report**
Prev. Rpt. Date: 02/04/2020
Sample Date: 01/17/2020
Sampled By: Humberto Palma
By Order Of: Bruce
Field ID: 01-19-19300

Location: Mission, Hidalgo County, Texas

Cylinder Marked	Age Tested (date : days)	Diameter (in)	Area (in²)	Max Load (lbs)	Break Type	Cure Loc	Compressive Strength (PSI)	Average Strength (PSI)	Tested By
19-A	01/24/20 : 7	4.00	12.566	41,429	Type 5	Lab	3,300	3,300	
19-B	02/14/20 : 28	4.00	12.566	64,547	Type 3	Lab	5,140		
19-C	02/14/20 : 28	4.00	12.566	61,656	Type 5	Lab	4,910	5,020	

MEETS REFERENCE VALUE


Type 1 Type 2 Type 3 Type 4 Type 5 Type 6

	Measurement	Specification
Temp.: Ambient:	77°F	NA°F
Mix:	83°F	NA°F
Slump:	8.5	Min. 8.5
Air Content:	*	NA

Specification: 4,500 psi @ 28 days
Source: Fisher
Plant:
TruckNo: NA
Mix Code: TEXBF6.5
Ticket No: NA
Sampled At: Truck

Weather: NA
Transported By: Palma, Humberto
Placement Date: 01/17/2020
Time Batched: 1:00 pm
Time Sampled: 1:06 pm
Curing Method: Standard

Quantity Represented: 165 cu. yds.
Placement Location: STA 39+00, 3' R FT CL
Sample Location: Bollard Wall Footer
Remarks:
Test Method (As Applicable): Unless noted, concrete was sampled and tested in accordance with ASTM C172, C143, C231 or C173, C1054 and C138. Compressive strength tests per ASTM C39, C31.

Respectfully Submitted,
Millennium Engineers Group, Inc.

STATE OF TEXAS
JUAN M. BORJON
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Juan Borjon, P.E.
02/21/2020

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5918 McPherson Rd., Ste. 5	Laredo, Texas 78041	956-568-1664

Report On: Concrete Compression

Lab No: 11193-3
Report No: 10-20

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

Report Date: 02/26/2020

Sample Date: 01/21/2020
Sampled By: Humberto Palma
By Order Of: Bruce
Field ID: 01-19-19300

Location: Mission, Hidalgo County, Texas

Cylinder Marked	Age Tested (date : days)	Diameter (in)	Area (in²)	Max Load (lbs)	Break Type	Cure Loc	Compressive Strength (PSI)	Average Strength (PSI)	Tested By
36-A	02/18/20 : 28	4.00	12.566	66,711	Type 5	Lab	5,310		
36-B	02/18/20 : 28	4.00	12.566	67,455	Type 3	Lab	5,370	5,340	

MEETS REFERENCE VALUE


Type 1 Type 2 Type 3 Type 4 Type 5 Type 6

	Measurement	Specification
Temp.: Ambient:	59°F	NA°F
Mix:	67°F	NA°F
Slump:	8.5	Min. 8.5
Air Content:	*	NA

Specification: 4,500 psi @ 28 days
Source: Fisher
Plant:
TruckNo: NA
Mix Code: TEXBF6.5
Ticket No: NA
Sampled At: Truck

Weather: NA
Transported By: Palma, Humberto
Placement Date: 01/21/2020
Time Batched: 7:39 pm
Time Sampled: 7:47 pm
Curing Method: Standard

Quantity Represented: 297 cu. yds.
Placement Location: STA 73+50, 3' R FT CL
Sample Location: Bollard Wall Fence
Remarks: Sample by Client.
Test Method (As Applicable): Unless noted, concrete was sampled and tested in accordance with ASTM C172, C143, C231 or C173, C1054 and C138. Compressive strength tests per ASTM C39, C31.

Respectfully Submitted,
Millennium Engineers Group, Inc.

STATE OF TEXAS
JUAN M. BORJON
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Juan Borjon, P.E.
02/27/2020

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1221 E. Tyler Ave.
5918 McPherson Rd., Ste. 5

Area Offices

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Harlingen, Texas 78550	956-454-8832
Laredo, Texas 78041	956-568-1664

Report On: **Concrete Compression**

Lab No: 11194-3
Report No: 10-24

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

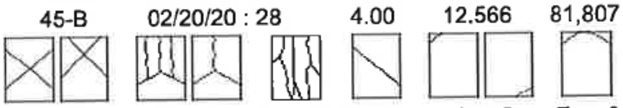
Report Date: 02/26/2020

Sample Date: 01/23/2020
Sampled By: Humberto Palma
By Order Of: Bruce
Field ID: 01-19-19300

Location: Mission, Hidalgo County, Texas

Cylinder Marked	Age Tested (date : days)	Diameter (in)	Area (in²)	Max Load (lbs)	Break Type	Cure Loc	Compressive Strength (PSI)	Average Strength (PSI)	Tested By
45-A	02/20/20 : 28	4.00	12.566	77,861	Type 3	Lab	6,200		
45-B	02/20/20 : 28	4.00	12.566	81,807	Type 3	Lab	6,510	6,350	

MEETS REFERENCE VALUE



Type 1 Type 2 Type 3 Type 4 Type 5 Type 6

	Measurement	Specification
Temp.: Ambient:	72°F	NA°F
Mix:	76°F	NA°F
Slump:	8.5	Min. 8.5
Air Content:	*	NA

Specification: 4,500 psi @ 28 days
Source: Fisher
Plant:
TruckNo: 1407
Mix Code: TEXFB6.5
Ticket No: NA
Sampled At: Truck

Weather: NA
Transported By: Palma, Humberto
Placement Date: 01/23/2020
Time Batched: 8:26 pm
Time Sampled: 8:35 pm
Curing Method: Standard

Quantity Represented: 182 cu. yds.
Placement Location: STA 83+50, 3' R FT CL
Sample Location: Bollard Wall Fence
Remarks: Sample by Client.
Test Method (As Applicable): Unless noted, concrete was sampled and tested in accordance with ASTM C172, C143, C231 or C173, C1054 and C138. Compressive strength tests per ASTM C39, C31.

Respectfully Submitted,
Millennium Engineers Group, Inc.

STATE OF TEXAS
JUAN M. BORJON
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Juan Borjon, P.E.
02/27/2020

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Laredo, Texas 78041 956-568-1664

Report On: Concrete Compression

Lab No: 11196-1
Report No: 10-26

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

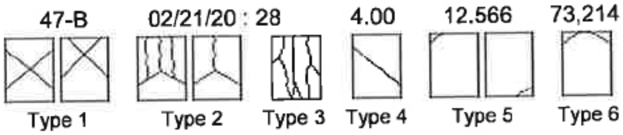
Report Date: 02/26/2020

Sample Date: 01/24/2020
Sampled By: Humberto Palma
By Order Of: Bruce
Field ID: 01-19-19300

Location: Mission, Hidalgo County, Texas

Cylinder Marked	Age Tested (date : days)	Diameter (in)	Area (in²)	Max Load (lbs)	Break Type	Cure Loc	Compressive Strength (PSI)	Average Strength (PSI)	Tested By
47-A	02/21/20 : 28	4.00	12.566	75,072	Type 3	Lab	5,970		
47-B	02/21/20 : 28	4.00	12.566	73,214	Type 3	Lab	5,830	5,900	

MEETS REFERENCE VALUE



	Measurement	Specification
Temp.: Ambient:	73°F	NA°F
Mix:	74°F	NA°F
Slump:	8.5	Min. 8.5
Air Content:	*	NA

Specification: 4,500 psi @ 28 days
Source: Fisher
Plant:
TruckNo: 1408
Mix Code: TEXFB6.5
Ticket No: NA
Sampled At: Truck

Weather: NA
Transported By: Palma, Humberto
Placement Date: 01/24/2020
Time Batched: 3:52 pm
Time Sampled: 4:04 pm
Curing Method: Standard

Quantity Represented: 154 cu. yds.
Placement Location: STA 96+20, 3' R FT CL
Sample Location: Bollard Wall Fence
Remarks: Sample by Client.
Test Method (As Applicable): Unless noted, concrete was sampled and tested in accordance with ASTM C172, C143, C231 or C173, C1054 and C138. Compressive strength tests per ASTM C39, C31.

Respectfully Submitted,
Millennium Engineers Group, Inc.

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02/27/2020

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Harlingen, Texas 78550 956-454-8832
Laredo, Texas 78041 956-568-1664

Report On: Concrete Compression

Lab No: 11194-2
Report No: 10-23

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

Report Date: 02/26/2020

Sample Date: 01/23/2020
Sampled By: Humberto Palma
By Order Of: Bruce
Field ID: 01-19-19300

Location: Mission, Hidalgo County, Texas

Cylinder Marked	Age Tested (date : days)	Diameter (in)	Area (in²)	Max Load (lbs)	Break Type	Cure Loc	Compressive Strength (PSI)	Average Strength (PSI)	Tested By
44-A	02/20/20 : 28	4.00	12.566	70,576	Type 5	Lab	5,620		
44-B	02/20/20 : 28	4.00	12.566	69,816	Type 5	Lab	5,560	5,590	

MEETS REFERENCE VALUE

	Measurement	Specification
Temp.: Ambient:	NA	NA°F
Mix:		NA°F
Slump:		NA
Air Content:	*	NA

Specification: 4,500 psi @ 28 days
Source: Fisher
Plant:
TruckNo: NA
Mix Code: TEXBF6.5
Ticket No: NA
Sampled At: Truck

Weather: NA
Transported By: Palma, Humberto
Placement Date: 01/23/2020
Time Batched: NA
Time Sampled: NA
Curing Method: Standard

Quantity Represented:
Placement Location: NA
Sample Location: Bollard Wall Fence
Remarks: Sample by Client.
Test Method (As Applicable): Unless noted, concrete was sampled and tested in accordance with ASTM C172, C143, C231 or C173, C1054 and C138. Compressive strength tests per ASTM C39, C31.

Respectfully Submitted,
Millennium Engineers Group, Inc.

STATE OF TEXAS
JUAN M. BORJON
121570
LICENSED
PROFESSIONAL ENGINEER

Juan Borjon, P.E.
02/27/2020

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Harlingen, Texas 78550 956-454-8832
Laredo, Texas 78041 956-568-1664

Report On: Proctor - Soils

Lab No: 10401
Report No: 9-5

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

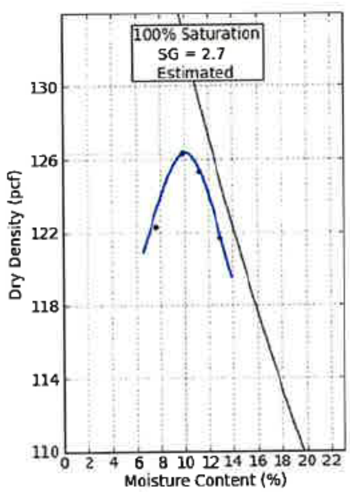
Project: McAllen Border Fence

Report Date: 01/23/2020

Sample Date: 01/17/2020
Sampled By: Angel Cano
By Order Of: Bruce
Field ID: 5889

Location: STA 2+00

Material: Flexible Base



% Moisture	Dry Density Lbs./Cu.Ft.
7.6	122.3
9.8	126.3
11.2	125.3
12.9	121.7
10.1 Optimum	126.4 Maximum

Color: Brown
Description: Caliche With Gravel

Liquid Limit: 24
Plastic Limit: 12
Plasticity Index: 12

Desc of Rammer:Manual
Preparation Method:Moist
Oversized Material:

Test Method (As Applicable):ASTM D 1140, ASTM D 2487, ASTM D 4318, ASTM D-698 Method-C

Respectfully Submitted,
Millennium Engineers Group, Inc.

STATE OF TEXAS
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02/04/2020

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Laredo, Texas 78041 956-568-1664

Report On: Proctor - Soils

Lab No: 10401
Report No: 9-5

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

Report Date: 01/23/2020

Sample Date: 01/17/2020
Sampled By: Angel Cano
By Order Of: Bruce
Field ID: 5889

Location: STA 2+00

Material: Flexible Base

Orig: Civil Solutions Engineering & Mgmt. LLC
Attn: Timoth C. Fish (1-cc copy)
1-ec Civil Solutions Engineering & Mgmt. LLC
Attn: Timothy C. Fish
1-ec Civil Solutions Engineering & Mgmt. LLC
Attn: John Halvarson
1-ec Civil Solutions Engineering & Mgmt. LLC
Attn: Bruce Meyer
1-ec Millennium Engineers Group
Attn: Humberto Palma
1-ec Millennium Engineers Group
Attn: Juan M. Borjon
1-ec Millennium Engineers Group Attn: Andres Palma
1-ec Millennium Engineers Group Attn: Sergio Tovar

Respectfully Submitted,
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Juan Borjon, P.E.

02/04/2020

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Area Offices
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Harlingen, Texas 78550 956-454-8832
Laredo, Texas 78041 956-568-1664

Report On: Sieve Analysis

Lab No: 10401-1
Report No: 9-5

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

Report Date: 01/25/2020

Location: STA 2+00

Sample Date: 01/17/2020
Sampled By: Angel Cano
By Order Of: Bruce

Material: Flexible Base

Description: Brown Caliche With Gravel

Sieve	% Passing	% Retained
2 1/2 in	100	0
2 in	95	5
1 3/4 in	94	6
7/8 in	84	16
3/4 in	80	20
1/2 in	70	30
3/8 in	65	35
No. 4	53	47
No. 40	36	64

Test Method (As Applicable):Tex-110-E

Respectfully Submitted,
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Area Offices

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Laredo, Texas 78041	956-568-1664

Report On: Sieve Analysis

Lab No: 10401-1
Report No: 9-5

Project No: 01-19-19300 **Acct. No.:** CSEM2019

Client: Civil Solutions Engineering & Mgmt. LLC
Timoth C. Fish
P.O. Box 262
Joseph City, AZ 86032

Project: McAllen Border Fence

Report Date: 01/25/2020

Location: STA 2+00

Sample Date: 01/17/2020
Sampled By: Angel Cano
By Order Of: Bruce

Material: Flexible Base

Orig: Civil Solutions Engineering & Mgmt. LLC
Attn: Timoth C. Fish (1-cc copy)
1-ec Civil Solutions Engineering & Mgmt. LLC
Attn: Timothy C. Fish
1-ec Civil Solutions Engineering & Mgmt. LLC
Attn: John Halvarson
1-ec Civil Solutions Engineering & Mgmt. LLC
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Attn: Juan M. Borjon
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1-ec Millennium Engineers Group Attn: Sergio Tovar

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02/04/2020

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Phoenix, Arizona 85040-2921
(602) 437-3737 • wt-us.com

TENSION & BEND TESTS ON STEEL

Date of Report: 1/7/20

Client: CIVIL SOLUTIONS ENGINEERING & MGMT
PO BOX 262
ST, JOSEPH CITY, AZ 86032

Job No. 2169XE375
Event: 1

Project: TENSILE TESTING
Contractor: N/A
Type / Use of Material: REIFORCING STEEL
Supplier / Source: N/A
Referenced Standard ASTM A615

Authorized By: TC FISH Date: 12/18/19
Sampled By: TC FISH Date: 12/18/19
Submitted By: TC FISH Date: 12/18/19
Location: WT/PHX

TEST DATA

SAMPLE NO.		1	2	3					
SIZE		5	5	5					
MILL									
HEAT NO.		3087797	6011017	6011016					
GRADE		60	60	60					
AREA, SQ. IN.		.31	.31	.31					
YIELD POINT	LBF	25,703	20,007	20,024					
	PSI	82,900	64,500	64,600					
TENSILE STRENGTH	LBF	31,866	32,323	32,128					
	PSI	102,800	104,300	103,600					
GAUGE LENGTH, IN.		8.0	8.0	8.0					
ELONGATION, %		18.0	17.0	13.0					
MEETS REQUIREMENTS OF REFERENCED STANDARD	YES	X	X	X					
	NO								

COMMENTS:

COPIES TO: CLIENT (1)

REVIEWED BY: RF

Photographs of Erosion Damage from August 2020
Provided by DOJ as received from Butterfly Center

Photographs of Erosion Damage from August 2020
Provided by DOJ as received from Butterfly Center

Photographs of Erosion Damage from August 2020
Provided by DOJ as received from Butterfly Center

Arcadis 000162

Photographs of Erosion Damage from August 2020
Provided by DOJ as received from Butterfly Center

Attachment 2

Non-Destructive Testing Results

Rebound Hammer and GPR Scan by Terracon (3 pages)

Ultrasonic Thickness by BRL (3 pages)

Report: 88215034.0001
Service Date: 4/27/2021
Report Date: 5/6/2021

Terracon
1506 Mid Cities Drive
Pharr, Texas 78577
TX Reg. No. F-3272

Client

Arcadis
3850 North Causeway Boulevard, Suite 990
Metairie, LA

Project

Border Wall Geotechnical Services
Mission, Texas

Project Number: 88215034

On April 27th, 2021, a Terracon representative visited the above referenced site. The following items were observed or discussed:

Border wall bollard footing was tested for compressive strength of the concrete and scanned to determine approximate rebar size and location.

Equipment used: Hilti PS200 Ferroscan, Schmidt Rebound Hammer

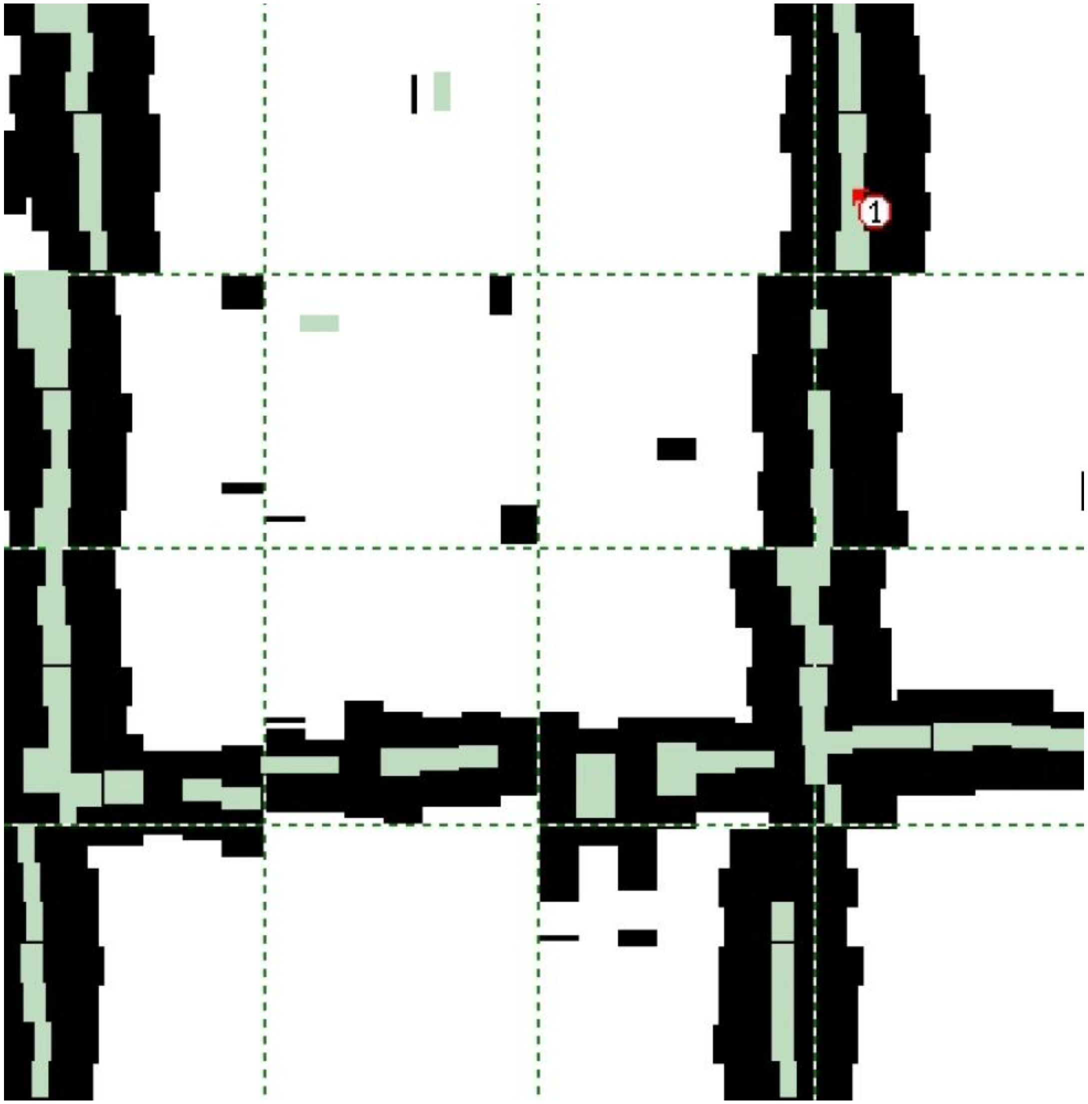
Locations scanned: Every ¼ mile on 3-mile span of border wall for a total of 12 locations.

Rebars were marked and found to be #5 bars every 5 to 6 inches on center in all locations in a single mat. See attached photos for examples. Rebound hammer testing was also performed at each location

Results were reported to Jason Vasquez w/ Arcadis prior to Terracon representative leaving site.

Services:

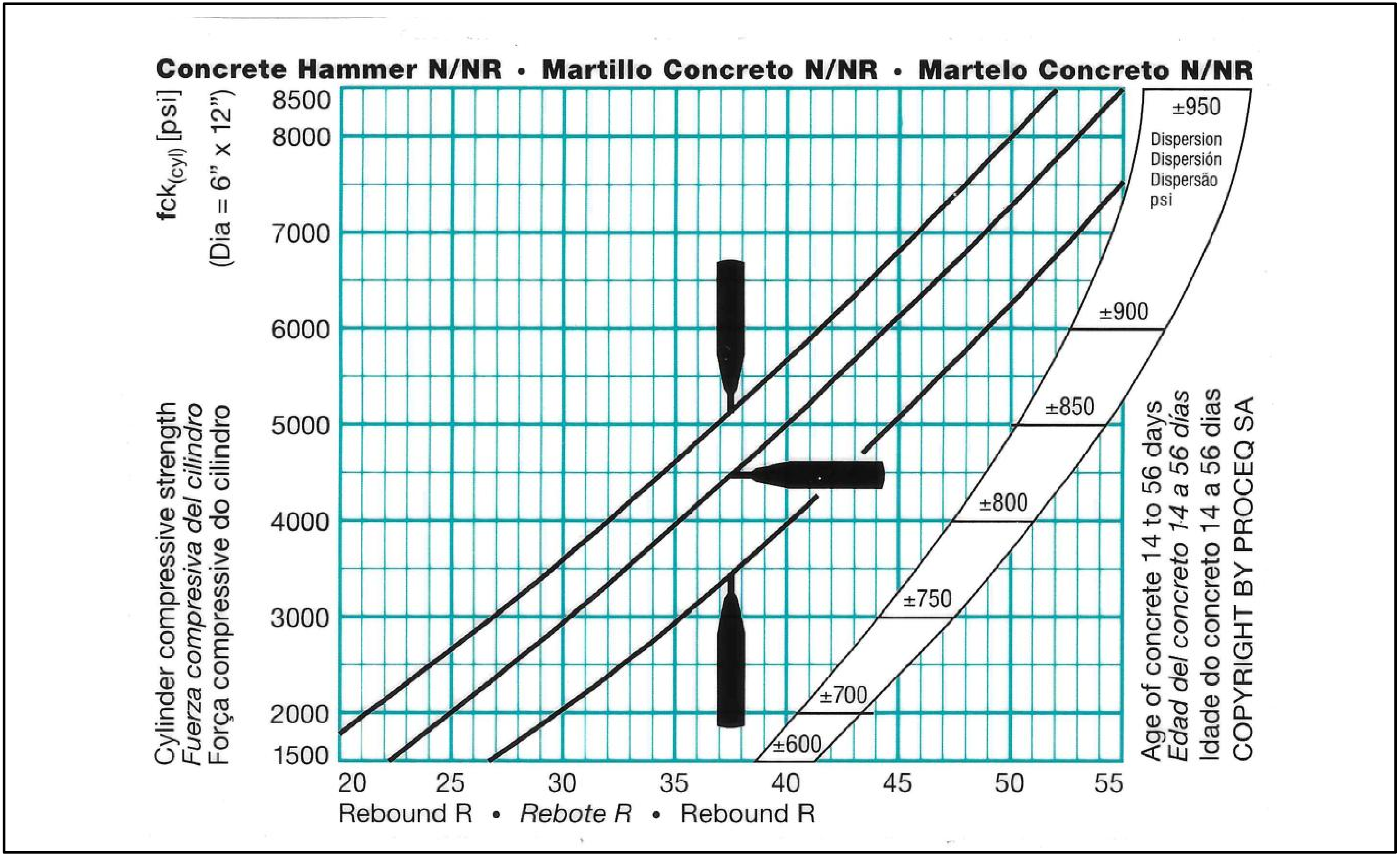
Reported To:
Contractor:
Report Distribution:



Marker 1: Bar size verified to be #5 bar by Ferroscan

Project No. 88215034	Site Plan: Bar Size Verification (Marker 1)	Terracon
	Report Number: 0002	
Date: 5/6/2021	Technician: Ben Butler	1506 Mid Cities Drive
Reviewed: MR	Date: 4/27/2021	Pharr, Texas
Approved: AAS	Scale: Not to Scale	PH. (956) 283 8254 terracon.com

BD000L, 08-24-13, Rev.1



Project No. 88215034	Site Plan: Rebound Hammer Chart	Terracon
	Report Number: 0003	
Date: 5/6/2021	Technician: Ben Butler	1506 Mid Cities Drive
Reviewed: MR	Date: 4/27/2021	Pharr, Texas
Approved: AAS	Scale: Not to Scale	PH. (956) 283 8254 terracon.com

BRL NDT SERVICES

Structural Steel Inspection Report

CLIENT:	TERRACON CONSULTANTS	**DATE:**	APRIL 27, 2021
ADDRESS:	6911 Blanco Rd. SATX 78216	**PROJECT:**	DOJ Mission Border Fence
OFFICE P.O.C.:	Jeremy Moreno	**SITE P.O.C.**	Jason Vasquez
PURCHASE ORDER:	Job Number 88215034	**LOCATION:**	Mission, TX

SCOPE

Perform ultrasonic thickness testing (UTT) of border fence.

SUMMARY OF ITEMS OBSERVED

A. 6” X 6” Bollards
B. Gate Structure
C. Gate Column

REFERENCED DOCUMENTATION

BRL NDT SERVICES UT-01

Visual Observation Elevation(s)/ Location(s):
6” X 6” Fence Bollards
Observation Results:
Arrived onsite and met with client field representative Jason Vasquez. Mr. Vasquez requested an ultrasonic thickness verification of the 6” X 6” fence bollards that were located at the site staging area. Mr. Vasquez then requested that a thickness verification be performed every quarter mile along the three mile stretch of fence. Thickness verification also performed on the gate structure. All 6” X 6” fence bollards tested were found to be .125” thick. Gate Structure thickness is .186”. Gate Column thickness is .365”.
Expectations:
No further action required.

CERTIFIED WELDING INSPECTOR:	Virgil Martinez			**SIGNATURE:**	AWS Virgil T Martinez CWI 13001271 QC1 EXP. 9/1/2021		
CERTIFIED WELDING INSPECTOR:				**SIGNATURE:**			
INITIAL INSPECTION HOURS:	10	**REINSPECTION HOURS:**	N/A	**MILEAGE:**	540 RT	**CONSUMABLES:**	1

BRL NDT SERVICES

Structural Steel Inspection Report

PICTURES

Staging Area

End View of Fence Bollard

Fence Bollard Thickness Reading .125"

Gate Structure .186"

CERTIFIED WELDING INSPECTOR:	Virgil Martinez			SIGNATURE:	AWS Virgil T Martinez CWI 13001271 QC1 EXP. 9/1/2021		
CERTIFIED WELDING INSPECTOR:				SIGNATURE:			
INITIAL INSPECTION HOURS:	10	REINSPECTION HOURS:	N/A	MILEAGE:	540 RT	CONSUMABLES:	1

BRL NDT SERVICES

Structural Steel Inspection Report

PICTURES	
Gate Column .365”	**N/A**
	N/A
N/A	**N/A**
N/A	N/A

CERTIFIED WELDING INSPECTOR:	Virgil Martinez			**SIGNATURE:**	AWS® Virgil T Martinez CWI 12091271 QC1 EXP. 9/1/2021		
CERTIFIED WELDING INSPECTOR:				**SIGNATURE:**			
INITIAL INSPECTION HOURS:	10	**REINSPECTION HOURS:**	N/A	**MILEAGE:**	540 RT	**CONSUMABLES:**	1

Arcadis 000170

Attachment 3

Test Pit Field Logs (12 pages)

ARCADIS

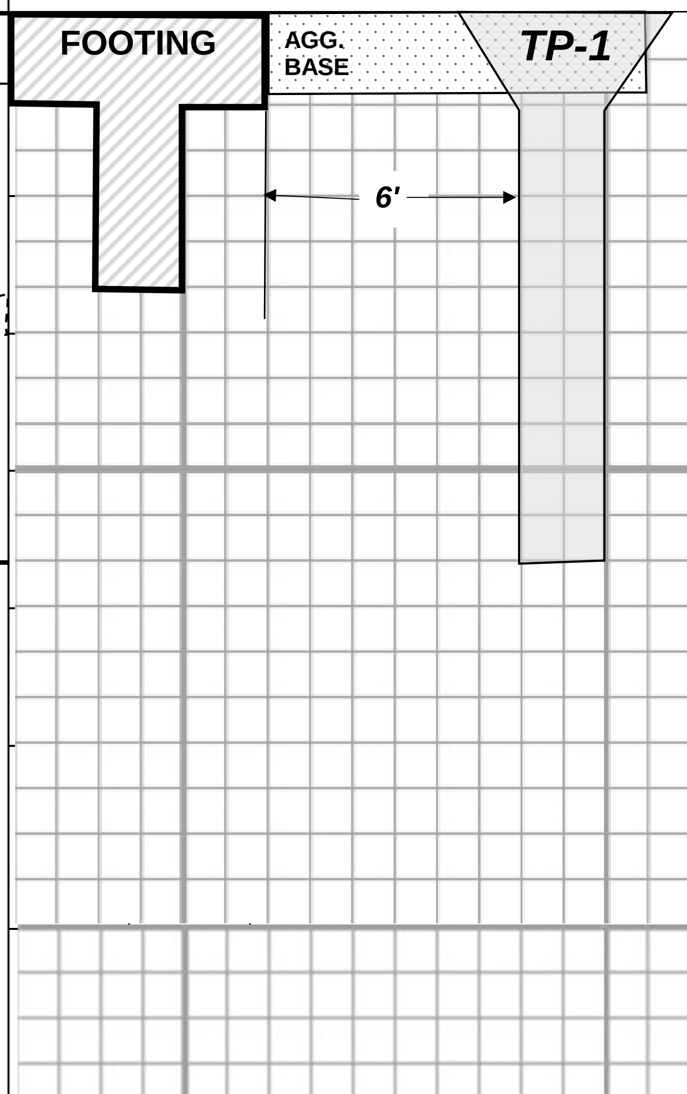
TEST PIT LOG

TEST PIT ID: TP-1
SHEET: 1
OF: 1

	INSTALLATION LOWER RIO GRANDE VALLEY
PROJECT DOJ Mission Border Fence	**MODEL** JCB 8069 **BUCKET** 12 inches
	DIG METHOD Excavator trenching
LOCATION (Coordinates or Station) Riverside of Fence ~ STA 156+00	**DATUM FOR ELEVATION SHOWN (TBM or MSL)**
EXCAVATION AGENCY Terracon	**TOTAL NUMBER OF SAMPLES TAKEN** — **DISTURBED** 2 — **UNDISTURBED** -
TEST PIT ID (as shown above) TP-1	**DATE OF TRENCHING** 28-Apr-21 — **STARTED** 9am — **COMPLETED** 3pm
DEPTH OF TRENCH 6 feet bgs	**WATER DEPTH AT TRENCHING (feet bgs)** - feet bgs
NAME OF OPERATOR — **NAME OF TECHNICIAN** Alfonso Soto	**NAME OF LOGGER** Jason Vazquez

DEPTH (feet)	MATERIAL DESCRIPTION AND NOTES
1	6"-10" Aggregate Base
2	Lean clay with sand and silt brown, moist, relatively stiff,
3	plastic with variable silt and sand content
4	*Sand Cone Test / Proctor Sample* moisture increases with depth,
5	soil material gets darker and less stiff
6	trench did not experience caving during excavation
7	Termination Depth ~ 6'
8	
9	
10	
11	
12	

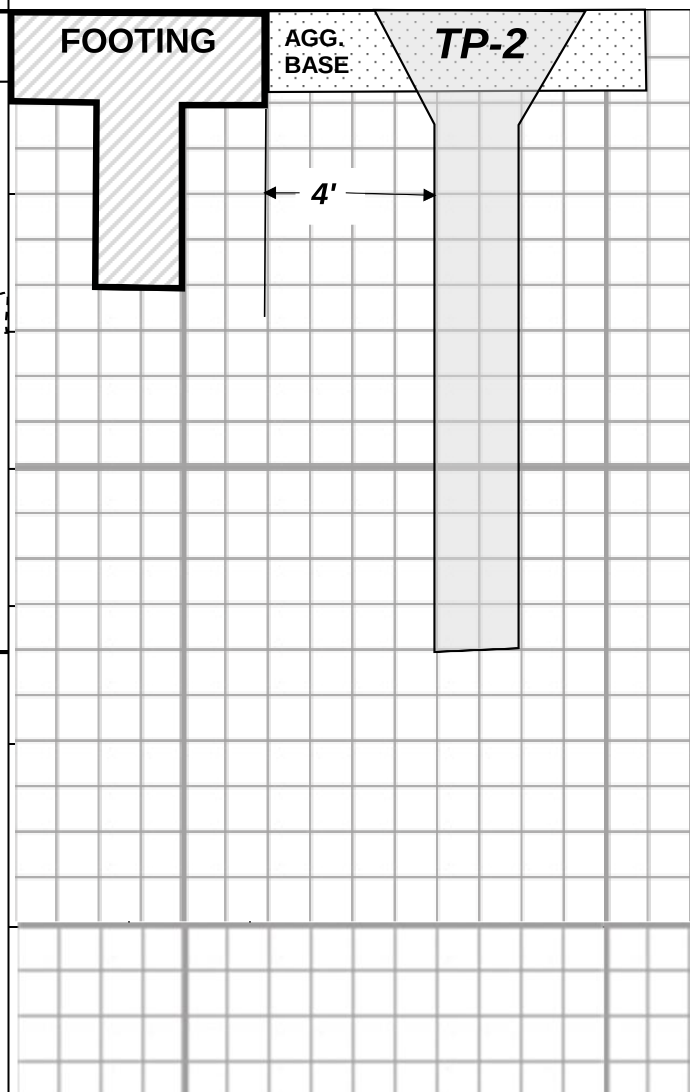
TRENCH SKETCH AND DETAILS



	INSTALLATION LOWER RIO GRANDE VALLEY
PROJECT DOJ Mission Border Fence	**MODEL** JCB 8069 **BUCKET** 12 inches
	DIG METHOD Excavator trenching
LOCATION (Coordinates or Station) Riverside of Fence ~ STA 139+50	**DATUM FOR ELEVATION SHOWN (TBM or MSL)**
EXCAVATION AGENCY Terracon	**TOTAL NUMBER OF SAMPLES TAKEN** — **DISTURBED** 2 — **UNDISTURBED** -
TEST PIT ID (as shown above) TP-2	**DATE OF TRENCHING** 28-Apr-21 — **STARTED** 9:30am — **COMPLETED** 3:30pm
DEPTH OF TRENCH 7 feet bgs	**WATER DEPTH AT TRENCHING (feet bgs)** 7 feet bgs
NAME OF OPERATOR — **NAME OF TECHNICIAN** Alfonso Soto	**NAME OF LOGGER** Jason Vazquez

DEPTH (feet)	MATERIAL DESCRIPTION AND NOTES
1	6"-10" Aggregate Base
2	Silty sand brown, moist, relatively stiff,
3	slightly plastic
4	*Sand Cone Test / Proctor Sample* moisture increases with depth,
5	soil material gets darker and less stiff
6	
7	trench did not experience caving during excavation Groundwater detected at bottom of trench
8	Termination Depth ~ 7'
9	
10	
11	
12	

TRENCH SKETCH AND DETAILS



ARCADIS

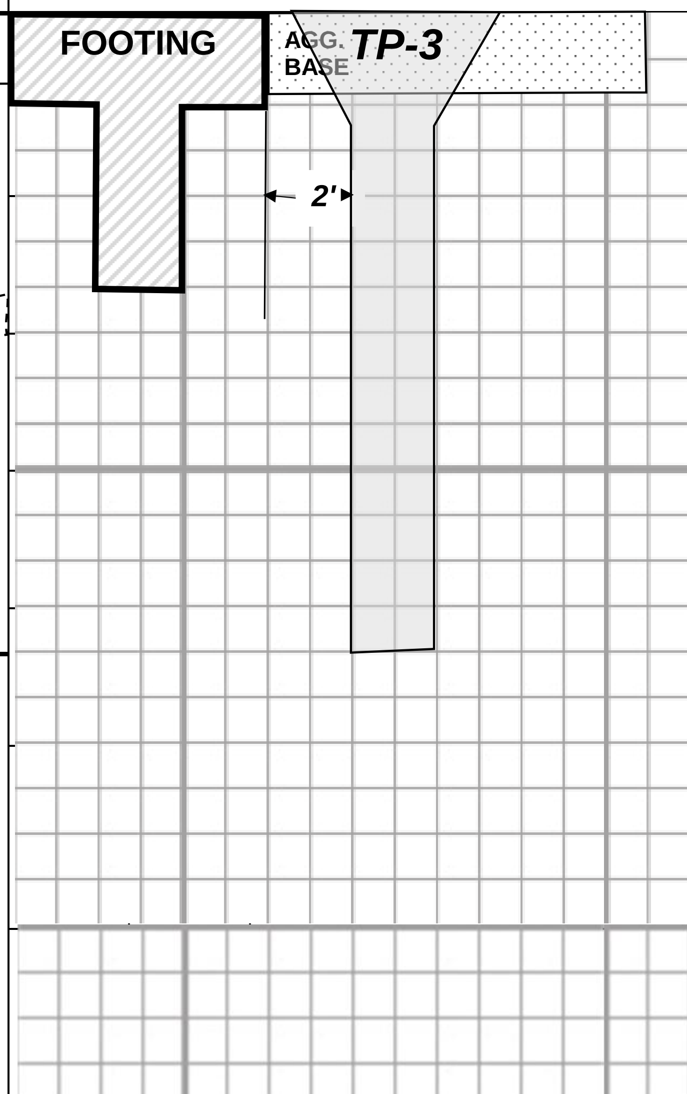
TEST PIT LOG

TEST PIT ID: TP-3
SHEET: 1
OF: 1

PROJECT DOJ Mission Border Fence	**INSTALLATION** LOWER RIO GRANDE VALLEY
	MODEL JCB 8069 **BUCKET** 12 inches
	DIG METHOD Excavator trenching
LOCATION (Coordinates or Station) Riverside of Fence ~ STA 125+00	**DATUM FOR ELEVATION SHOWN (TBM or MSL)**
EXCAVATION AGENCY Terracon	**TOTAL NUMBER OF SAMPLES TAKEN** — **DISTURBED:** 2 — **UNDISTURBED:** -
TEST PIT ID (as shown above) TP-3	**DATE OF TRENCHING:** 28-Apr-21 — **STARTED:** 10am — **COMPLETED:** 3pm
DEPTH OF TRENCH 7 feet bgs	**WATER DEPTH AT TRENCHING (feet bgs)** - feet bgs
NAME OF OPERATOR — **NAME OF TECHNICIAN** Alfonso Soto	**NAME OF LOGGER** Jason Vazquez

DEPTH (feet)	MATERIAL DESCRIPTION AND NOTES
1	6"-10" Aggregate Base
2	Silty sand brown, moist, relatively dense,
3	slightly plastic
4	*Sand Cone Test / Proctor Sample* transitions to non plastic
5	soil material gets darker and less dense
6	
7	trench did not experience caving during excavation
8	Termination Depth ~ 7'
9	
10	
11	
12	

TRENCH SKETCH AND DETAILS



ARCADIS

TEST PIT LOG

TEST PIT ID: TP-4
SHEET: 1
OF: 1

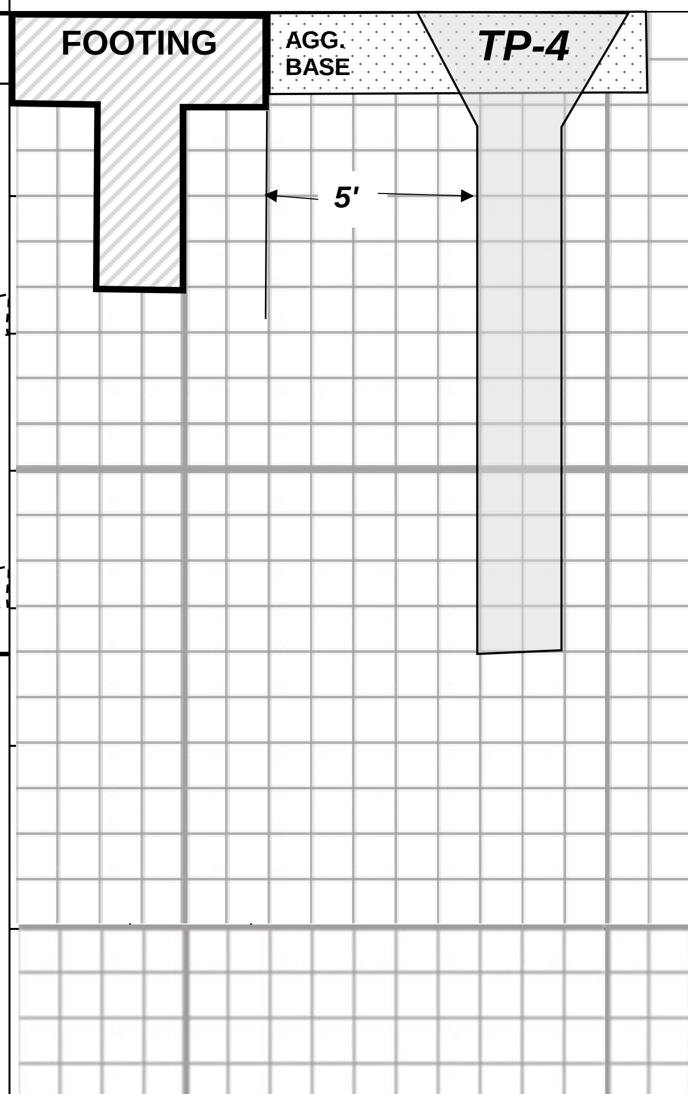
INSTALLATION	LOWER RIO GRANDE VALLEY
PROJECT	DOJ Mission Border Fence
MODEL	JCB 8069
BUCKET	12 inches
DIG METHOD	Excavator trenching
LOCATION (Coordinates or Station)	Riverside of Fence ~ STA 115+00
DATUM FOR ELEVATION SHOWN (TBM or MSL)	
EXCAVATION AGENCY	Terracon
TEST PIT ID (as shown above)	TP-4
DEPTH OF TRENCH	7 feet bgs
WATER DEPTH AT TRENCHING (feet bgs)	7 feet bgs
NAME OF OPERATOR	
NAME OF TECHNICIAN	Alfonso Soto
NAME OF LOGGER	Jason Vazquez

TOTAL NUMBER OF SAMPLES TAKEN	DISTURBED	UNDISTURBED
	2	-

DATE OF TRENCHING	STARTED	COMPLETED
28-Apr-21	10:30am	2:30pm

DEPTH (feet)	MATERIAL DESCRIPTION AND NOTES
1	6"-10" Aggregate Base
2	Lean clay with sand and silt brown, moist, relatively stiff, plastic with variable silt and sand content
3	
4	*Sand Cone Test* roots encountered below 3'
5	moisture increases with depth, soil material gets darker and less stiff
6	
7	*Proctor Sample* Groundwater detected at bottom of trench
8	Termination Depth ~ 7'
9	
10	
11	
12	

TRENCH SKETCH AND DETAILS



ARCADIS

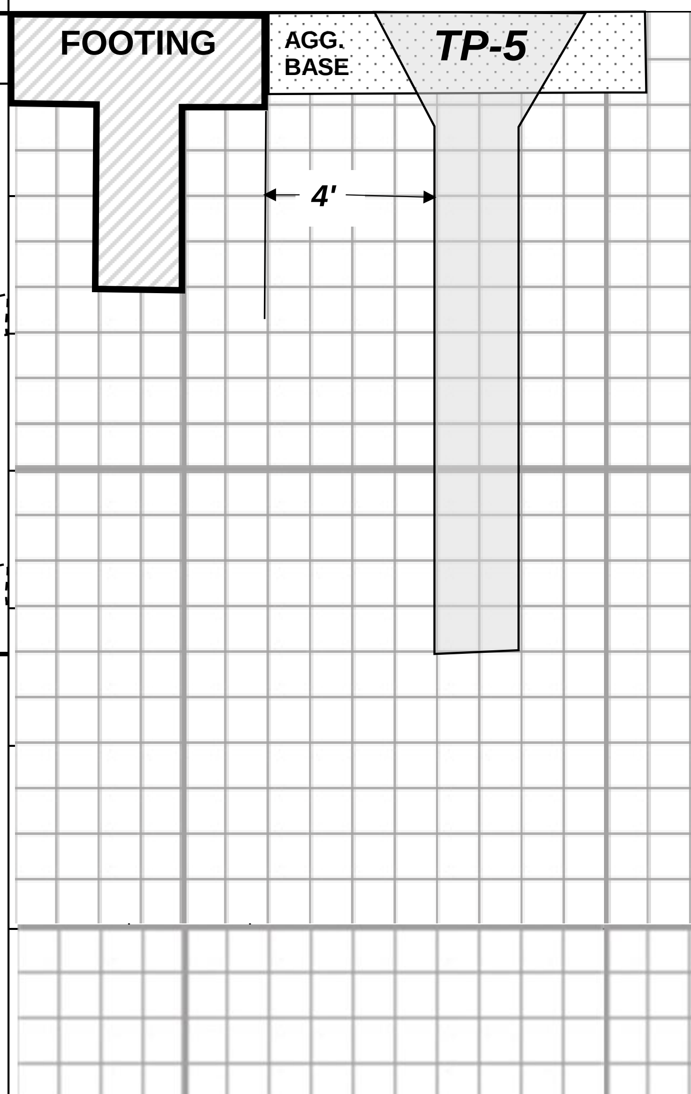
TEST PIT LOG

TEST PIT ID: TP-5
SHEET: 1
OF: 1

	INSTALLATION LOWER RIO GRANDE VALLEY
PROJECT DOJ Mission Border Fence	**MODEL** JCB 8069 **BUCKET** 12 inches
	DIG METHOD Excavator trenching
LOCATION (Coordinates or Station) Riverside of Fence ~ STA 101+00	**DATUM FOR ELEVATION SHOWN (TBM or MSL)**
EXCAVATION AGENCY Terracon	**TOTAL NUMBER OF SAMPLES TAKEN** — **DISTURBED** 2 — **UNDISTURBED** -
TEST PIT ID (as shown above) TP-5	**DATE OF TRENCHING** 28-Apr-21 — **STARTED** 11am — **COMPLETED** 2pm
DEPTH OF TRENCH 7 feet bgs	**WATER DEPTH AT TRENCHING (feet bgs)** 7 feet bgs
NAME OF OPERATOR — **NAME OF TECHNICIAN** Alfonso Soto	**NAME OF LOGGER** Jason Vazquez

DEPTH (feet)	MATERIAL DESCRIPTION AND NOTES
1	6"-10" Aggregate Base
2	Silty sand brown, moist, relatively dense,
3	slightly plastic
4	*Sand Cone Test* transitions to non plastic
5	moisture increases with depth, soil material gets darker and less dense
6	
7	*Proctor Sample* trench did not experience caving during excavation
8	Termination Depth ~ 7'
9	
10	
11	
12	

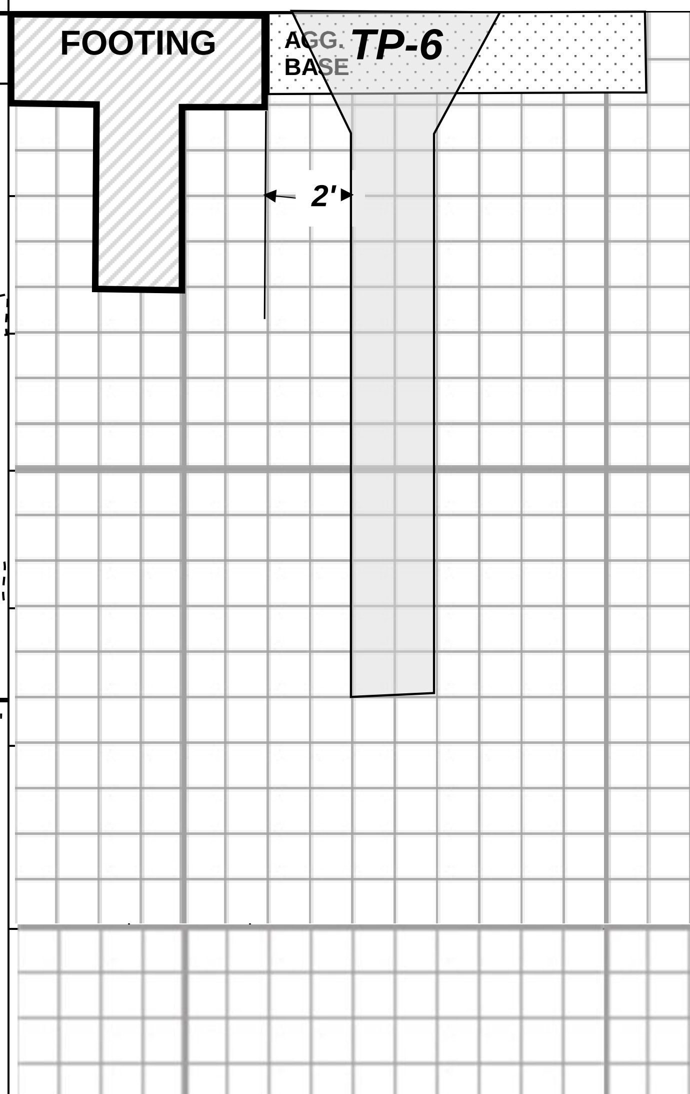
TRENCH SKETCH AND DETAILS



	INSTALLATION LOWER RIO GRANDE VALLEY
PROJECT DOJ Mission Border Fence	**MODEL** JCB 8069 **BUCKET** 12 inches
	DIG METHOD Excavator trenching
LOCATION (Coordinates or Station) Riverside of Fence ~ STA 88+50	**DATUM FOR ELEVATION SHOWN (TBM or MSL)**
EXCAVATION AGENCY Terracon	**TOTAL NUMBER OF SAMPLES TAKEN** — **DISTURBED** 2 — **UNDISTURBED** -
TEST PIT ID (as shown above) TP-6	**DATE OF TRENCHING** 28-Apr-21 — **STARTED** 12pm — **COMPLETED** 1pm
DEPTH OF TRENCH 7.5 feet bgs	**WATER DEPTH AT TRENCHING (feet bgs)** 7.5 feet bgs
NAME OF OPERATOR — **NAME OF TECHNICIAN** Alfonso Soto	**NAME OF LOGGER** Jason Vazquez

DEPTH (feet)	MATERIAL DESCRIPTION AND NOTES
1	6"-10" Aggregate Base
2	Silty clayey sand brown, moist, relatively dense,
3	slightly plastic
4	*Sand Cone Test* transitions to non plastic
5	soil material gets darker and less dense
6	
7	*Proctor Sample* trench did not experience caving during excavation
8	Groundwater detected at bottom of trench
	Termination Depth ~ 7.5'
9	
10	
11	
12	

TRENCH SKETCH AND DETAILS



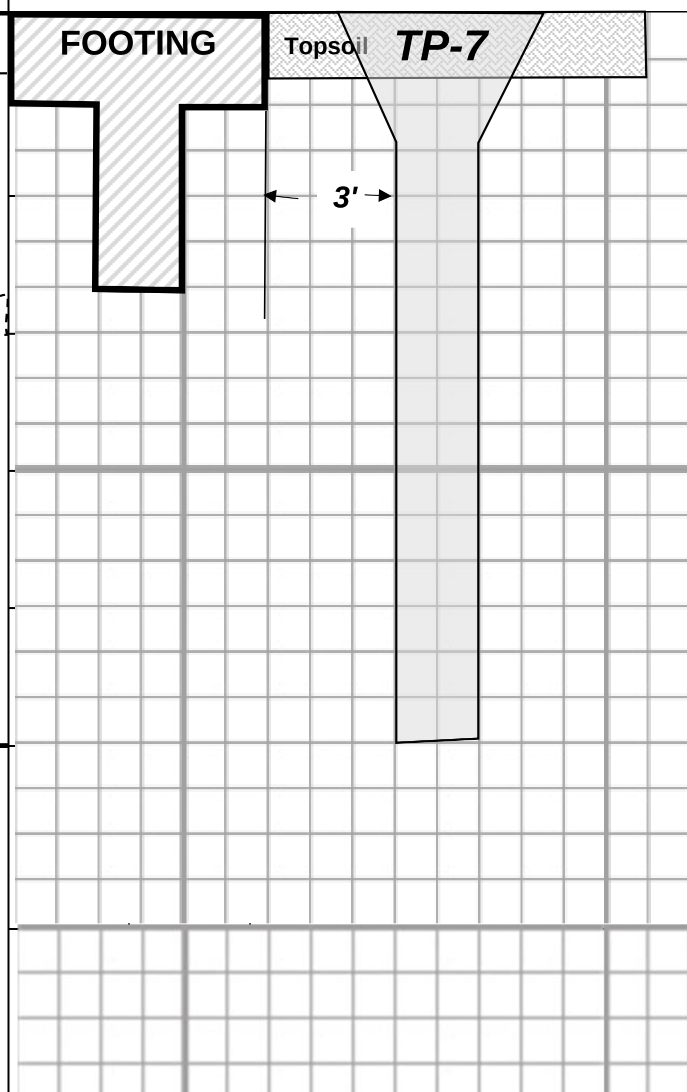
ARCADIS
TEST PIT LOG

TEST PIT ID: TP-7
SHEET: 1
OF: 1

	INSTALLATION LOWER RIO GRANDE VALLEY
PROJECT DOJ Mission Border Fence	**MODEL** JCB 8069 **BUCKET** 12 inches
	DIG METHOD Excavator trenching
LOCATION (Coordinates or Station) Riverside of Fence ~ STA 0+00	**DATUM FOR ELEVATION SHOWN (TBM or MSL)**
EXCAVATION AGENCY Terracon	**TOTAL NUMBER OF SAMPLES TAKEN** — DISTURBED: 2; UNDISTURBED: -
TEST PIT ID (as shown above) TP-7	**DATE OF TRENCHING** 29-Apr-21 — STARTED: 9am; COMPLETED: 10am
DEPTH OF TRENCH 8 feet bgs	**WATER DEPTH AT TRENCHING (feet bgs)** 8 feet bgs
NAME OF OPERATOR / **NAME OF TECHNICIAN** Alfonso Soto	**NAME OF LOGGER** Jason Vazquez

DEPTH (feet)	MATERIAL DESCRIPTION AND NOTES
1	6"-8" Topsoil with grass
2	Lean clay with sand and silt brown, moist, relatively stiff,
3	plastic with variable silt and sand content
4	*Sand Cone Test / Proctor Sample* roots encountered below 3'
5	moisture increases with depth, soil material gets darker and less stiff
6	
7	
8	trench did not experience caving during excavation Groundwater detected at bottom of trench
9	Termination Depth ~ 8'
10	
11	
12	

TRENCH SKETCH AND DETAILS



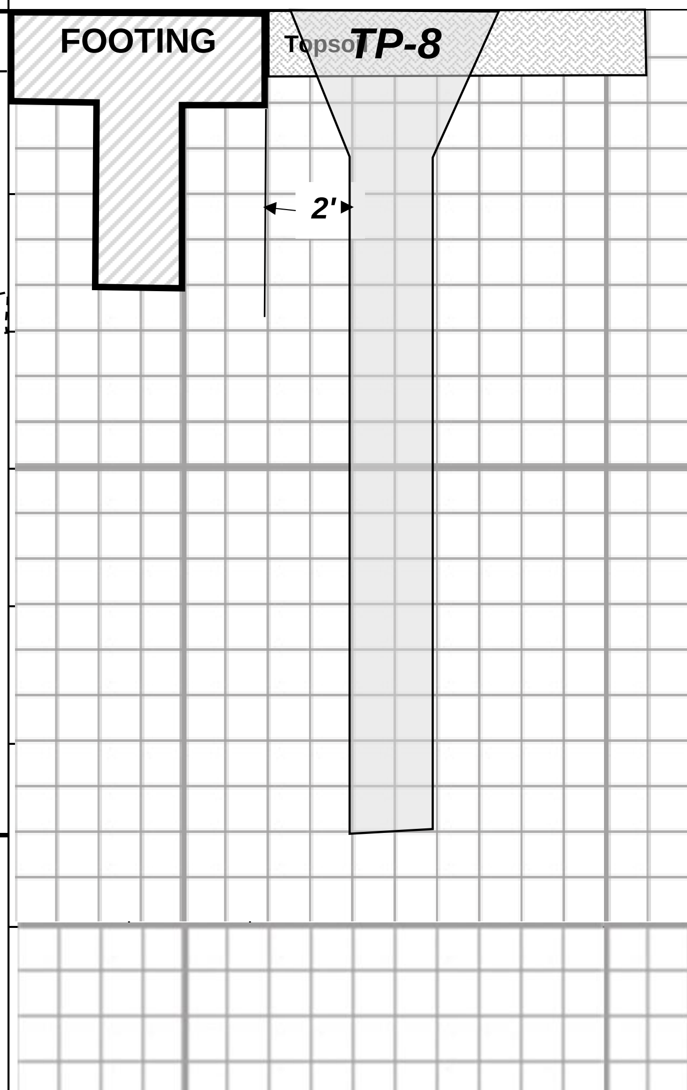
ARCADIS
TEST PIT LOG

TEST PIT ID: TP-8
SHEET: 1
OF: 1

	INSTALLATION LOWER RIO GRANDE VALLEY
PROJECT DOJ Mission Border Fence	**MODEL** JCB 8069 **BUCKET** 12 inches
	DIG METHOD Excavator trenching
LOCATION (Coordinates or Station) Riverside of Fence ~ STA 13+00	**DATUM FOR ELEVATION SHOWN (TBM or MSL)**
EXCAVATION AGENCY Terracon	**TOTAL NUMBER OF SAMPLES TAKEN** — **DISTURBED** 2; **UNDISTURBED** -
TEST PIT ID (as shown above) TP-8	**DATE OF TRENCHING** 29-Apr-21 — **STARTED** 10am; **COMPLETED** 11am
DEPTH OF TRENCH 9 feet bgs	**WATER DEPTH AT TRENCHING (feet bgs)** 8 feet bgs
NAME OF OPERATOR / **NAME OF TECHNICIAN** Alfonso Soto	**NAME OF LOGGER** Jason Vazquez

DEPTH (feet)	MATERIAL DESCRIPTION AND NOTES
1	6"-8" Topsoil with grass
2	Lean clay with sand and silt brown, moist, relatively stiff,
3	plastic with variable silt and sand content
4	*Sand Cone Test / Proctor Sample* roots encountered below 3'
5	moisture increases with depth, soil material gets darker and less stiff
6	
7	
8 ▽	Groundwater detected at 8'
9	water flowed into trench below 8' slight caving of trench with water intrusion
10	Termination Depth ~ 9'
11	
12	

TRENCH SKETCH AND DETAILS



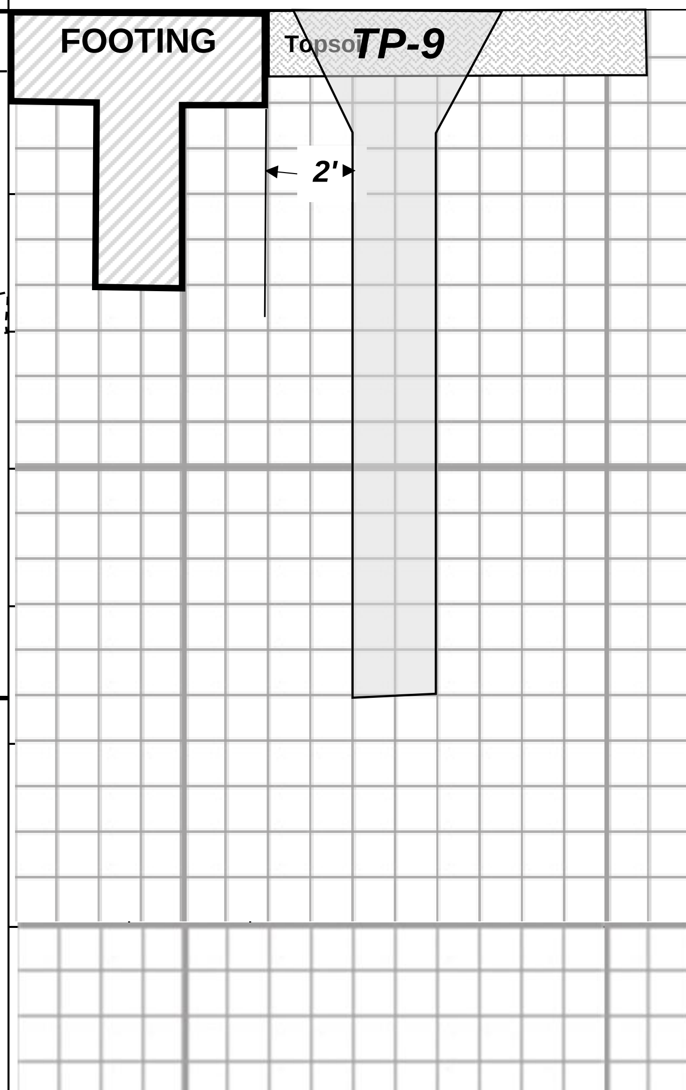
ARCADIS

TEST PIT LOG

	INSTALLATION LOWER RIO GRANDE VALLEY
PROJECT DOJ Mission Border Fence	**MODEL** JCB 8069 **BUCKET** 12 inches
	DIG METHOD Excavator trenching
LOCATION (Coordinates or Station) Riverside of Fence ~ STA 24+00	**DATUM FOR ELEVATION SHOWN (TBM or MSL)**
EXCAVATION AGENCY Terracon	**TOTAL NUMBER OF SAMPLES TAKEN** — **DISTURBED** 2; **UNDISTURBED** -
TEST PIT ID (as shown above) TP-9	**DATE OF TRENCHING** 29-Apr-21 — **STARTED** 11am; **COMPLETED** 12pm
DEPTH OF TRENCH 7.5 feet bgs	**WATER DEPTH AT TRENCHING (feet bgs)** 7.5 feet bgs
NAME OF OPERATOR / **NAME OF TECHNICIAN** Alfonso Soto	**NAME OF LOGGER** Jason Vazquez

DEPTH (feet)	MATERIAL DESCRIPTION AND NOTES
1	6"-8" Topsoil with grass
2	Lean clay with sand and silt brown, moist, relatively stiff,
3	plastic with variable silt and sand content
4	*Sand Cone Test / Proctor Sample* roots encountered below 3'
5	moisture increases with depth, soil material gets darker and less stiff
6	
7	trench did not experience caving during excavation
▽ 8	Groundwater detected at bottom of trench Termination Depth ~ 7.5'
9	
10	
11	
12	

TRENCH SKETCH AND DETAILS



ARCADIS

TEST PIT LOG

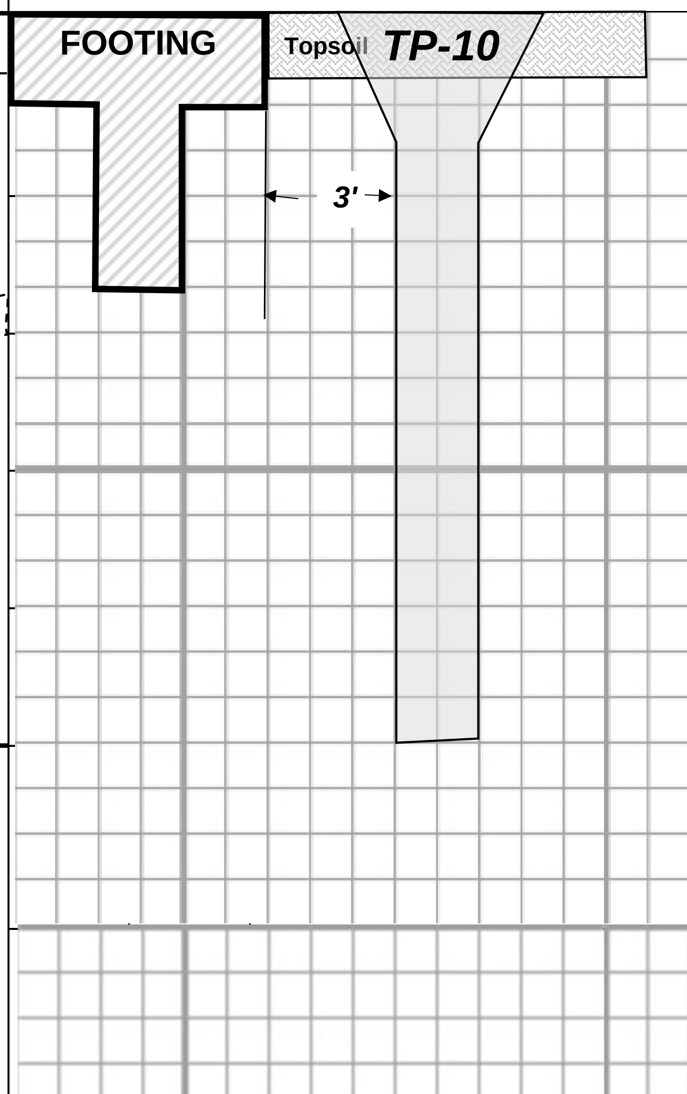
INSTALLATION	LOWER RIO GRANDE VALLEY
PROJECT	DOJ Mission Border Fence
MODEL	JCB 8069
BUCKET	12 inches
DIG METHOD	Excavator trenching
LOCATION (Coordinates or Station)	Riverside of Fence ~ STA 36+50
DATUM FOR ELEVATION SHOWN (TBM or MSL)	
EXCAVATION AGENCY	Terracon
TEST PIT ID (as shown above)	TP-10
DEPTH OF TRENCH	8 feet bgs
WATER DEPTH AT TRENCHING (feet bgs)	8 feet bgs
NAME OF OPERATOR	
NAME OF TECHNICIAN	Alfonso Soto
NAME OF LOGGER	Jason Vazquez

TOTAL NUMBER OF SAMPLES TAKEN	**DISTURBED**	**UNDISTURBED**
	2	-

DATE OF TRENCHING	**STARTED**	**COMPLETED**
29-Apr-21	1pm	2pm

DEPTH (feet)	MATERIAL DESCRIPTION AND NOTES
1	6"-8" Topsoil with grass
2	Lean clay with sand and silt brown, moist, relatively stiff,
3	plastic with variable silt and sand content
4	*Sand Cone Test / Proctor Sample* roots encountered below 3'
5	moisture increases with depth, soil material gets darker and less stiff
6	
7	
8	trench did not experience caving during excavation Groundwater detected at bottom of trench
9	Termination Depth ~ 8'
10	
11	
12	

TRENCH SKETCH AND DETAILS



ARCADIS

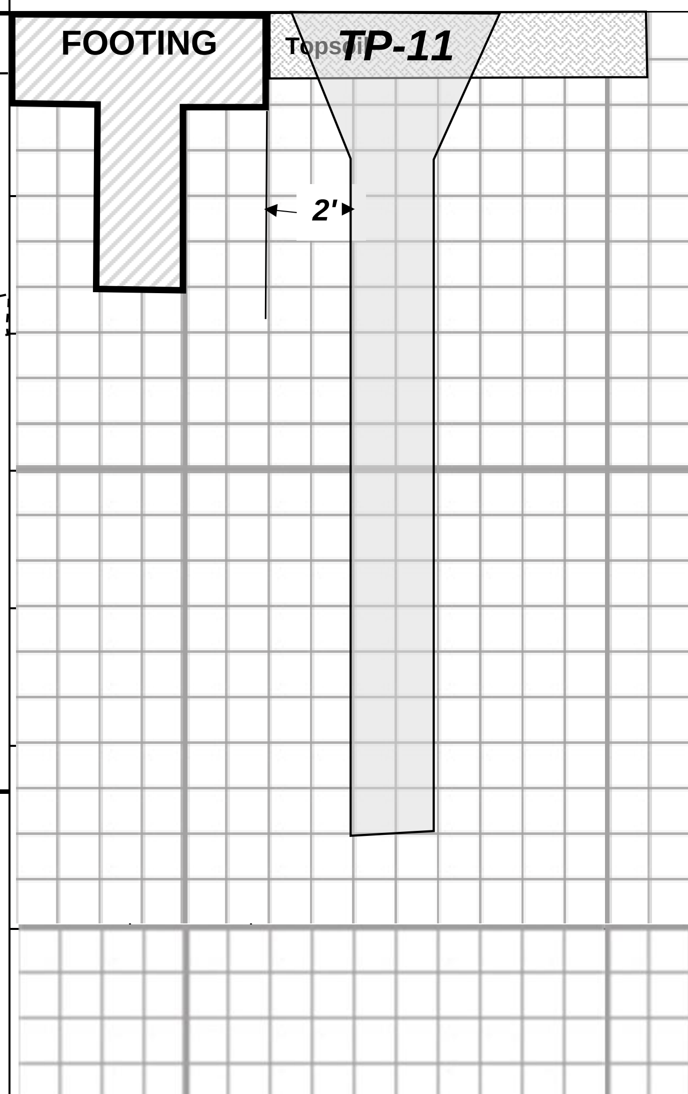
TEST PIT LOG

TEST PIT ID: TP-11
SHEET: 1
OF: 1

	INSTALLATION LOWER RIO GRANDE VALLEY
PROJECT DOJ Mission Border Fence	**MODEL** JCB 8069 **BUCKET** 12 inches
	DIG METHOD Excavator trenching
LOCATION (Coordinates or Station) Riverside of Fence ~ STA 56+50	**DATUM FOR ELEVATION SHOWN (TBM or MSL)**
EXCAVATION AGENCY Terracon	**TOTAL NUMBER OF SAMPLES TAKEN** — **DISTURBED:** 2 — **UNDISTURBED:** -
TEST PIT ID (as shown above) TP-11	**DATE OF TRENCHING** 29-Apr-21 — **STARTED:** 2pm — **COMPLETED:** 3pm
DEPTH OF TRENCH 8.5 feet bgs	**WATER DEPTH AT TRENCHING (feet bgs)** 8 feet bgs
NAME OF OPERATOR — **NAME OF TECHNICIAN** Alfonso Soto	**NAME OF LOGGER** Jason Vazquez

DEPTH (feet)	MATERIAL DESCRIPTION AND NOTES
1	6"-8" Topsoil with grass
2	Lean silt with clay and sand brown, moist, relatively stiff,
3	slightly plastic with variable clay and sand content
4	*Sand Cone Test / Proctor Sample* roots encountered below 3'
5	moisture increases with depth, soil material gets darker and less stiff
6	
7	
8	Groundwater detected at 8' water flowed into trench below 8'
9	slight caving of trench with water intrusion Termination Depth ~ 8.5'
10	
11	
12	

TRENCH SKETCH AND DETAILS



ARCADIS

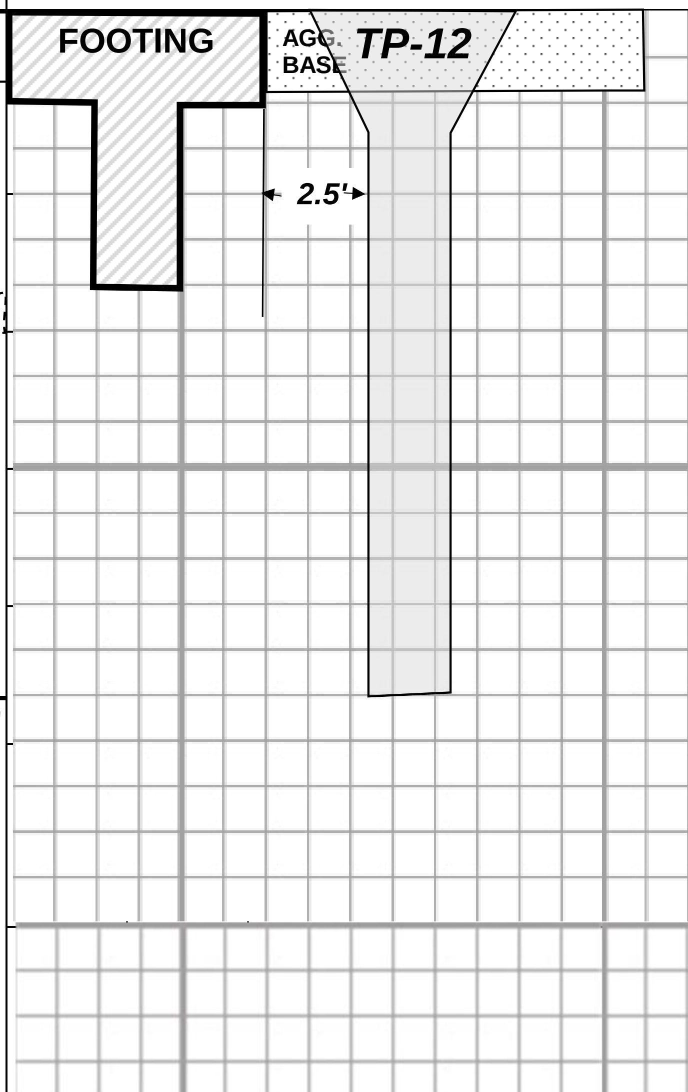
TEST PIT LOG

TEST PIT ID: TP-12
SHEET: 1
OF: 1

	INSTALLATION LOWER RIO GRANDE VALLEY
PROJECT DOJ Mission Border Fence	**MODEL** JCB 8069 **BUCKET** 12 inches
	DIG METHOD Excavator trenching
LOCATION (Coordinates or Station) Riverside of Fence ~ STA 76+50	**DATUM FOR ELEVATION SHOWN (TBM or MSL)**
EXCAVATION AGENCY Terracon	**TOTAL NUMBER OF SAMPLES TAKEN** — DISTURBED: 2; UNDISTURBED: -
TEST PIT ID (as shown above) TP-12	**DATE OF TRENCHING** 29-Apr-21 — STARTED: 3pm; COMPLETED: 4pm
DEPTH OF TRENCH 7.5 feet bgs	**WATER DEPTH AT TRENCHING (feet bgs)** 7.5 feet bgs
NAME OF OPERATOR / **NAME OF TECHNICIAN** Alfonso Soto	**NAME OF LOGGER** Jason Vazquez

DEPTH (feet)	MATERIAL DESCRIPTION AND NOTES
1	6"-10" Aggregate Base
2	Silty clayey sand brown, moist, relatively dense,
3	slightly plastic
4	*Sand Cone Test / Proctor Sample* transitions to non plastic
5	soil material gets darker and less dense
6	
7	trench did not experience caving during excavation
8	Groundwater detected at bottom of trench Termination Depth ~ 7.5'
9	
10	
11	
12	

TRENCH SKETCH AND DETAILS



Attachment 4

Test Pit Photo Log (55 pages)

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 1

Description:
Upstream Wall limit near STA 156+00 where TP-1 was excavated, looking north at fence footing and aggregate base.

Date: 4/28/2021

Photograph: 2

Description:
TP-1 excavation showing typical soil conditions.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 3

Description:
TP-1 excavation showing measurement of depth (sounding) and typical soil conditions.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 4

Description:
TP-1 excavation showing depth sounding and details of soil and base materials.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 5

Description:
TP-1 looking south at excavated trench.

Date: 4/28/2021

Photograph: 6

Description:
TP-1 looking southeast at backfilled trench.

Date: 4/28/2021

Photograph Log

ARCADIS

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 7

Description:
TP-2 excavation near STA 139+50 looking northwest at footing and aggregate base.

Date: 4/28/2021

Photograph: 8

Description:
TP-2 excavation showing sand cone test at depth of 3 feet bgs.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 9

Description:
TP-2 excavation showing typical soil conditions at 3 feet after sand cone test.

Date: 4/28/2021

Photograph: 10

Description:
TP-2 excavation looking southwest at typical soil conditions.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 11

Description:
TP-2 excavation showing soil conditions and depth sounding.

Date: 4/28/2021

Photograph: 12

Description:
TP-2 excavation showing typical conditions of trench.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 13

Description:
TP-2 excavation at termination depth of 7 feet bgs showing soils and trace groundwater entering trench.

Date: 4/28/2021

Photograph: 14

Description:
TP-3 excavation near STA 125+00 looking north at footing and aggregate base.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 15

Description:
TP-3 excavation at 3' depth showing typical conditions after sand cone test.

Date: 4/28/2021

Photograph: 16

Description:
TP-3 excavation looking south showing typical soil materials.

Date: 4/28/2021

Photograph: 17

Description:
TP-3 excavation showing typical subgrade conditions.

Date: 4/28/2021

Photograph: 18

Description:
TP-3 excavation showing roots encountered at depths below 3 feet bgs.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 19

Description:
TP-4 excavation near STA 115+00 looking northwest at typical soil conditions at 3 feet bgs.

Date: 4/28/2021

Photograph: 20

Description:
TP-4 excavation at 3' depth showing roots encountered near sand cone test location.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 21

Description:
TP-4 excavation showing roots encountered at depths below 3 feet bgs.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 22

Description:
TP-4 excavation showing soil conditions and groundwater at bottom of trench.

Date: 4/28/2021

Photograph: 23

Description:
TP-4 showing typical conditions for excavated trench.

Date: 4/28/2021

Photograph: 24

Description:
TP-4 looking southeast at backfilled trench.

Date: 4/28/2021

Photograph: 25

Description:
TP-5 excavation near STA 101+00 looking northwest at typical site and soil conditions.

Date: 4/28/2021

Photograph: 26

Description:
TP-5 excavation showing sand cone test at depth of 3’ bgs.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 27

Description:
TP-5 excavation showing depth sounding and typical soil conditions.

Date: 4/28/2021

Arcadis 000201

Photograph: 28

Description:
TP-5 excavation showing depth sounding and details of soil and base materials.

Date: 4/28/2021

Photograph: 29

Description:
TP-5 showing typical conditions for excavated trench.

Date: 4/28/2021

Photograph: 30

Description:
TP-5 excavated materials consisting of silty/sand clay and root matter below fill materials.

Date: 4/28/2021

Photograph: 31

Description:
TP-5 looking southeast at backfilled trench.

Date: 4/28/2021

Photograph: 32

Description:
TP-6 excavation near STA 88+50 looking northwest showing typical conditions at 3 feet bgs after sand cone test.

Date: 4/28/2021

Photograph: 33

Description:
TP-6 excavation at depth sounding showing details of soil materials at 5 feet bgs.

Date: 4/28/2021

Photograph: 34

Description:
TP-6 excavation sounding at 6 feet bgs for Proctor sample.

Date: 4/28/2021

Photograph: 35

Description:
TP-6 Proctor sample collection at 6 feet bgs.

Date: 4/28/2021

Photograph: 36

Description:
TP-6 excavation sounding showing typical soil conditions.

Date: 4/28/2021

Photograph: 37

Description:
TP-6 excavation sounding at 7 feet bgs.

Date: 4/28/2021

Photograph: 38

Description:
TP-6 groundwater entry at bottom of trench, around 7 feet bgs.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 39

Description:
TP-6 excavation showing typical soil conditions with roots and groundwater intrusion to the trench.

Date: 4/28/2021

Photograph: 40

Description:
TP-6 excavation showing typical conditions for subgrade soil with roots below fill.

Date: 4/28/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 41

Description:
TP-6 excavation showing typical conditions for fill and subgrade soils.

Date: 4/28/2021

Photograph: 42

Description:
TP-7 excavation near STA 0+00 looking southeast at sand cone test depth of 3 feet bgs.

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 43

Description:
TP-7 excavation showing sand cone test at 3 feet bgs.

Date: 4/29/2021

Photograph: 44

Description:
TP-7 showing soil conditions after sand cone test at depth of Proctor sample collection

Date: 4/29/2021

Photograph: 45

Description:
TP-7 excavation looking southeast at clayey soils from trench

Date: 4/29/2021

Photograph: 46

Description:
TP-7 excavation at groundwater intrusion at bottom of trench

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 47

Description:
TP-7 excavation sounding at bottom of trench at 8 feet bgs

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 48

Description:
TP-7 excavation sounding at bottom of trench at 8 feet bgs

Date: 4/29/2021

Photograph: 49

Description:
TP-7 looking southeast at backfilled trench

Date: 4/29/2021

Photograph: 50

Description:
TP-8 location looking northwest at erosion gulley that occurs due to surface drainage

Date: 4/29/2021

Photograph: 51

Description:
TP-8 excavation looking northeast showing approximate depth of fill materials.

Date: 4/29/2021

Photograph: 52

Description:
TP-8 excavation looking at sand cone test at 3 feet bgs.

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 53

Description:
TP-8 excavation looking northeast showing soil conditions

Date: 4/29/2021

Photograph: 54

Description:
TP-8 excavation sounding showing groundwater at bottom of trench at 9 feet bgs.

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 55

Description:
TP-8 excavation showing soil and groundwater conditions at 9 feet bgs.

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 56

Description:
TP-8 excavation showing soil conditions at 9 feet bgs.

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 57

Description:
TP-8 excavation sounding at 9 feet bgs.

Date: 4/29/2021

Photograph: 58

Description:
TP-8 excavation showing soil conditions at 9 feet bgs.

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 59

Description:
TP-9 excavation looking northwest showing soil conditions at 3 feet bgs

Date: 4/29/2021

Photograph: 60

Description:
TP-9 excavation at 3 feet bgs showing the sand cone test

Date: 4/29/2021

Photograph: 61

Description:
TP-9 excavation at 3 feet bgs showing the soil condition for Proctor sample

Date: 4/29/2021

Photograph: 62

Description:
TP-9 excavation at sounding depth of 7.5 feet bgs showing root matter from grass

Date: 4/29/2021

Photograph: 63

Description:
TP-9 excavation at sounding depth of 7.5 feet bgs showing soil conditions.

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 64

Description:
TP-9 excavation showing soil conditions.

Date: 4/29/2021

Photograph: 65

Description:
TP-9 excavation showing soil conditions.

Date: 4/29/2021

Photograph: 66

Description:
TP-10 excavation at 3 feet bgs showing the soil condition for Proctor sample

Date: 4/29/2021

Photograph: 67

Description:
TP-10 excavation looking northwest at proctor sampling from depth of 3 feet bgs

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 68

Description:
TP-10 excavation at sounding depth of 8 feet bgs showing soil and groundwater conditions

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 69

Description:
TP-10 excavation at sounding depth of 8 feet bgs

Date: 4/29/2021

Photograph: 70

Description:
TP-10 excavation showing soil and groundwater conditions

Date: 4/29/2021

Photograph: 71
Description:
TP-10 excavation soil materials from trench excavation.

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 72

Description:
TP-10 looking northwest at backfilled trench

Date: 4/29/2021

Photograph: 73

Description:
TP-11excavation looking northeast

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 74

Description:
TP-11

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 75

Description:
TP-11 excavation at sounding depth of 8.5 feet bgs showing soil conditions

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 76

Description:
TP-11 excavation looking northwest showing soil conditions.

Date: 4/29/2021

Photograph: 77

Description:
TP-11 excavation soil materials show wet condition when groundwater intrudes into trench

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 78

Description:
TP-11 excavation showing soil and groundwater conditions.

Date: 4/29/2021

Photograph: 79

Description:
TP-12 excavation at 3 feet bgs showing sand cone test and soil conditions.

Date: 4/29/2021

Photograph: 80

Description:
TP-12 excavation at 3 feet bgs showing sand cone test and soil conditions.

Date: 4/29/2021

Photograph: 81

Description:
TP-12 excavation at sounding depth of 7.5 feet bgs showing soil conditions.

Date: 4/29/2021

Photograph: 82

Description:
TP-12 excavation at sounding depth of 7.5 feet bgs showing soil conditions.

Date: 4/29/2021

Photograph Log

DOJ Mission Border Fence
Site Investigation April 2021

Photograph: 83

Description:
TP-12 excavation showing soil conditions.

Date: 4/29/2021

Attachment 5

Laboratory Test Results (55 pages)

Project: Border Wall Geotechnical Services
Location:
Number: 88215034

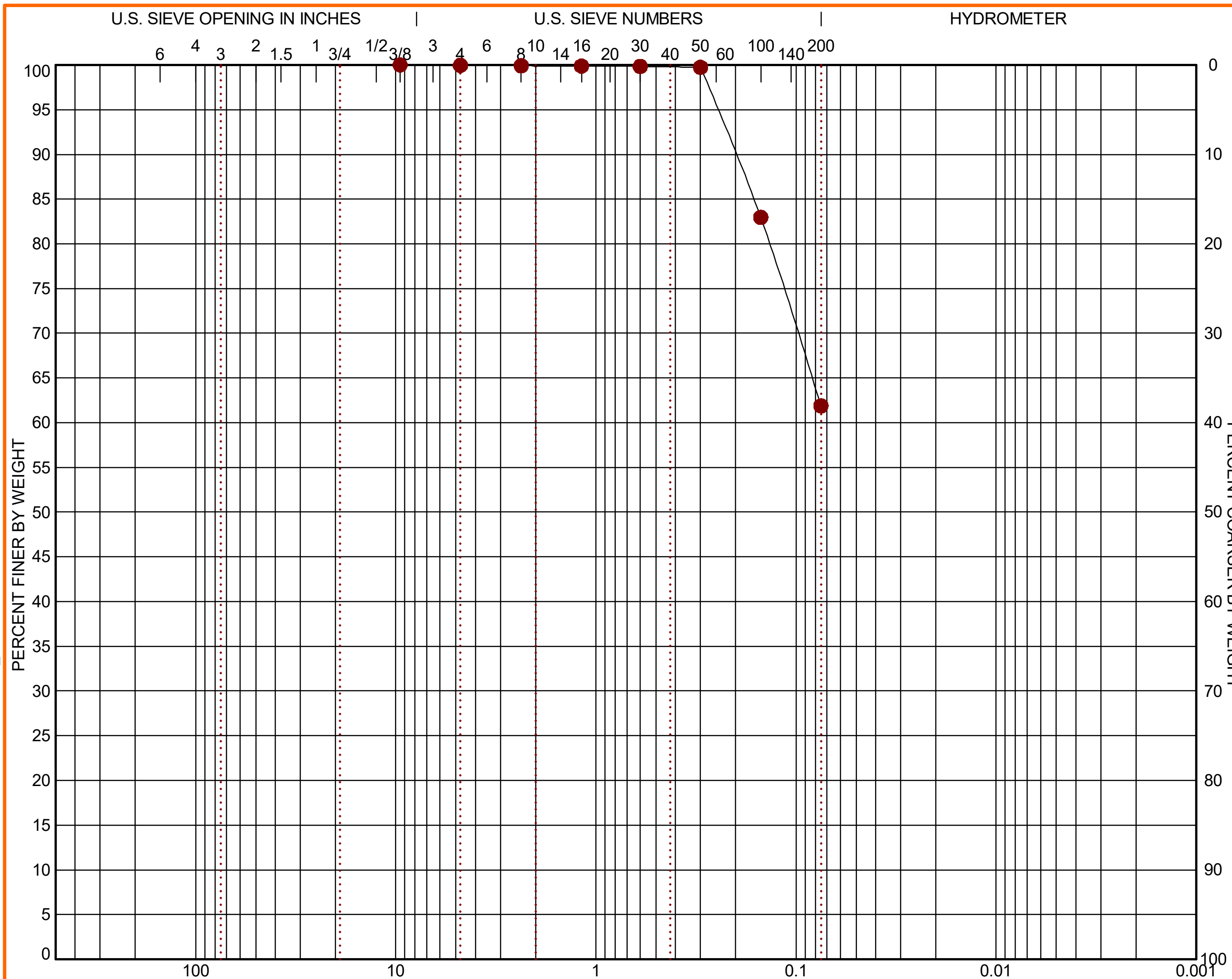
Bore-hole	Depth	N-Value	Pocket Pen	Water Content (%)	W - PL (%)	Dry Density (pcf)	Liquid Limit	Plastic Limit	PI	%<#200 Sieve	%<0.002 mm	% Clay Passing #200	Strength	Failure Strain	Conf Pressure	USCS	Group Name
TP-1	3.0			11.6	-2		28	14	14	61						CL	
TP-1	3.1									62							
TP-2	3.0			12.8	-5		19	18	1	33						SM	
TP-2	3.1									33							
TP-3	3.0			9.6			NP	NP	NP	24						SM	
TP-3	3.1									29							
TP-4	3.0			15.4	-5		28	20	8	83						CL	
TP-4	3.1									71							
TP-5	3.0			5.9			NP	NP	NP	14						SM	
TP-5	3.1									13							
TP-6	3.1									35							
TP-6	6.0			11.2	-7		25	18	7	46						SC-SM	
TP-7	3.0			14.5	-5		48	19	29	95						CL	
TP-7	3.1									95							
TP-8	3.0			10.9	-7		39	18	21	95						CL	
TP-8	3.1									93							
TP-9	3.0			12.7	-4		30	17	13	79						CL	
TP-9	3.1									78							
TP-10	3.0			18.8	0		41	19	22	92						CL	
TP-10	3.1									92							
TP-11	3.0			22.1	-1		31	23	8	98						ML	
TP-11	3.1									98							
TP-12	3.0			12.7	-6		24	19	5	38						SC-SM	
TP-12	3.1									42							

Project: Border Wall Geotechnical Services
Location:
Number: 88215034

Bore-hole	Depth	N-Value	Pocket Pen	Water Content (%)	W - PL (%)	Dry Density (pcf)	Liquid Limit	Plastic Limit	PI	%<#200 Sieve	%<0.002 mm	% Clay Passing #200	Strength	Failure Strain	Conf Pressure	USCS	Group Name
TP-1	3.0			11.6	-2		28	14	14	61						CL	
TP-1	3.1									62							
TP-2	3.0			12.8	-5		19	18	1	33						SM	
TP-2	3.1									33							
TP-3	3.0			9.6						24							
TP-3	3.1									29							
TP-4	3.0			15.4	-5		28	20	8	83						CL	
TP-4	3.1									71							
TP-5	3.0			5.9						14							
TP-5	3.1									13							
TP-6	3.1									35							
TP-6	6.0			11.2	-7		25	18	7	46						SC-SM	
TP-7	3.0			14.5	-5		48	19	29	95						CL	
TP-7	3.1									95							
TP-8	3.0			10.9	-7		39	18	21	95						CL	
TP-8	3.1									93							
TP-9	3.0			12.7	-4		30	17	13	79						CL	
TP-9	3.1									78							
TP-10	3.0			18.8	0		41	19	22	92						CL	
TP-10	3.1									92							
TP-11	3.0			22.1	-1		31	23	8	98						ML	
TP-11	3.1									98							
TP-12	3.0			12.7	-6		24	19	5	38						SC-SM	
TP-12	3.1									42							

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	TP-1	3.1	0.0	0.0	38.1		61.9		

GRAIN SIZE

	●		
D_{60}			
D_{30}			
D_{10}			

COEFFICIENTS

	●		
C_C			
C_U			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
3/8"	100.0				
#4	99.96				
#8	99.91				
#16	99.88				
#30	99.84				
#50	99.73				
#100	82.96				
#200	61.88				

SOIL DESCRIPTION

●	Sandy Lean Clay (CL)

REMARKS

●	

PROJECT: Border Wall Geotechnical Services

PROJECT NUMBER: 88215034

SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

Terracon
1506 Mid Cities Dr
Pharr, TX

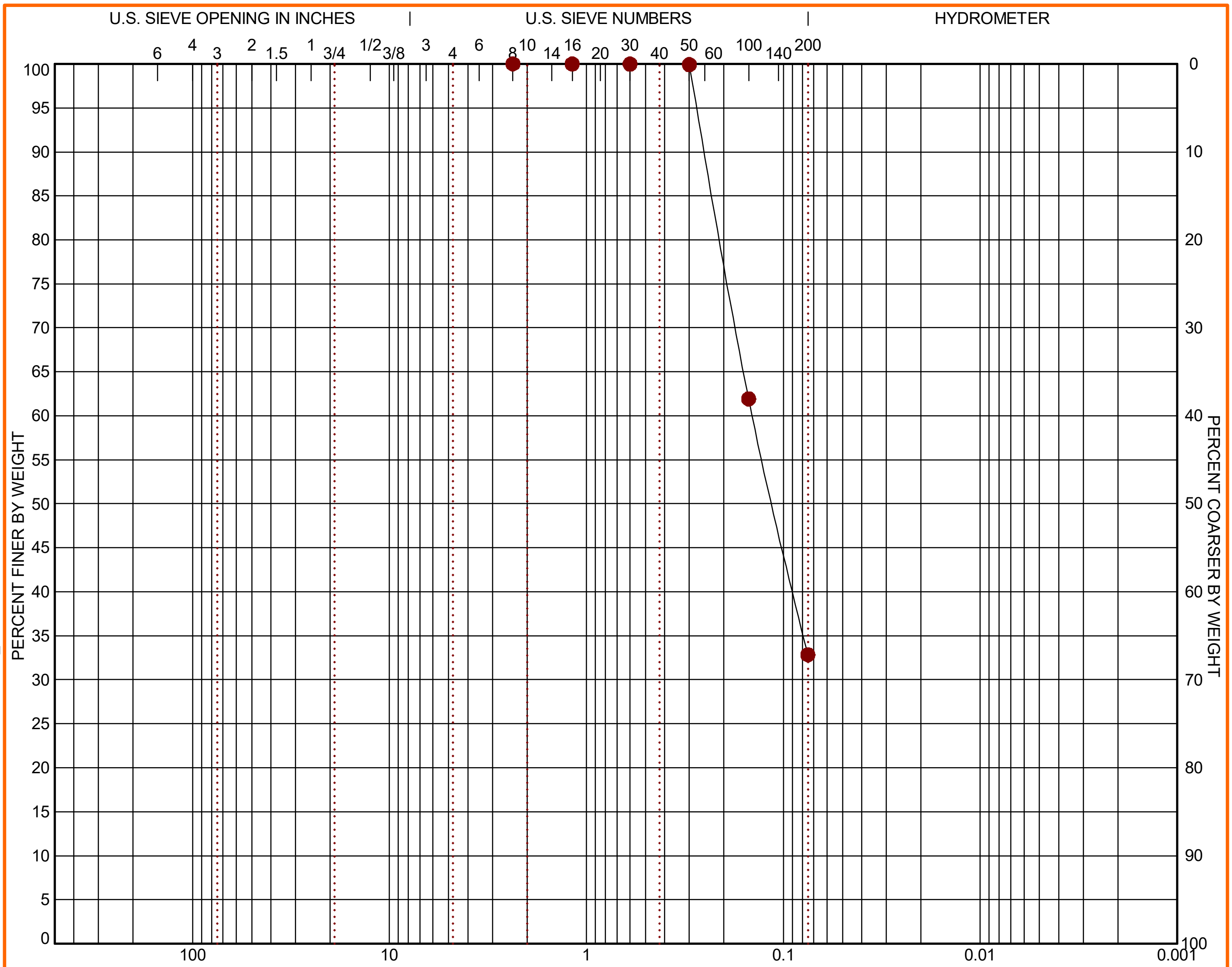
CLIENT: ARCADIS US, Inc.
Metairie, LA

Arcadis 000243

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 88215034 BORDER WALL GEOTE.GPJ TERRACON_DATATEMPLATE.GDT 6/29/21

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	TP-2	3.1	0.0	0.0	67.2		32.8		

GRAIN SIZE			
	●		
D_{60}	0.143		
D_{30}			
D_{10}			

COEFFICIENTS			
	●		
C_C			
C_U			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#8	100.0				
#16	99.99				
#30	99.98				
#50	99.92				
#100	61.92				
#200	32.83				

SOIL DESCRIPTION	
●	Silty Sand (SM)

REMARKS	
●	

PROJECT: Border Wall Geotechnical Services

SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

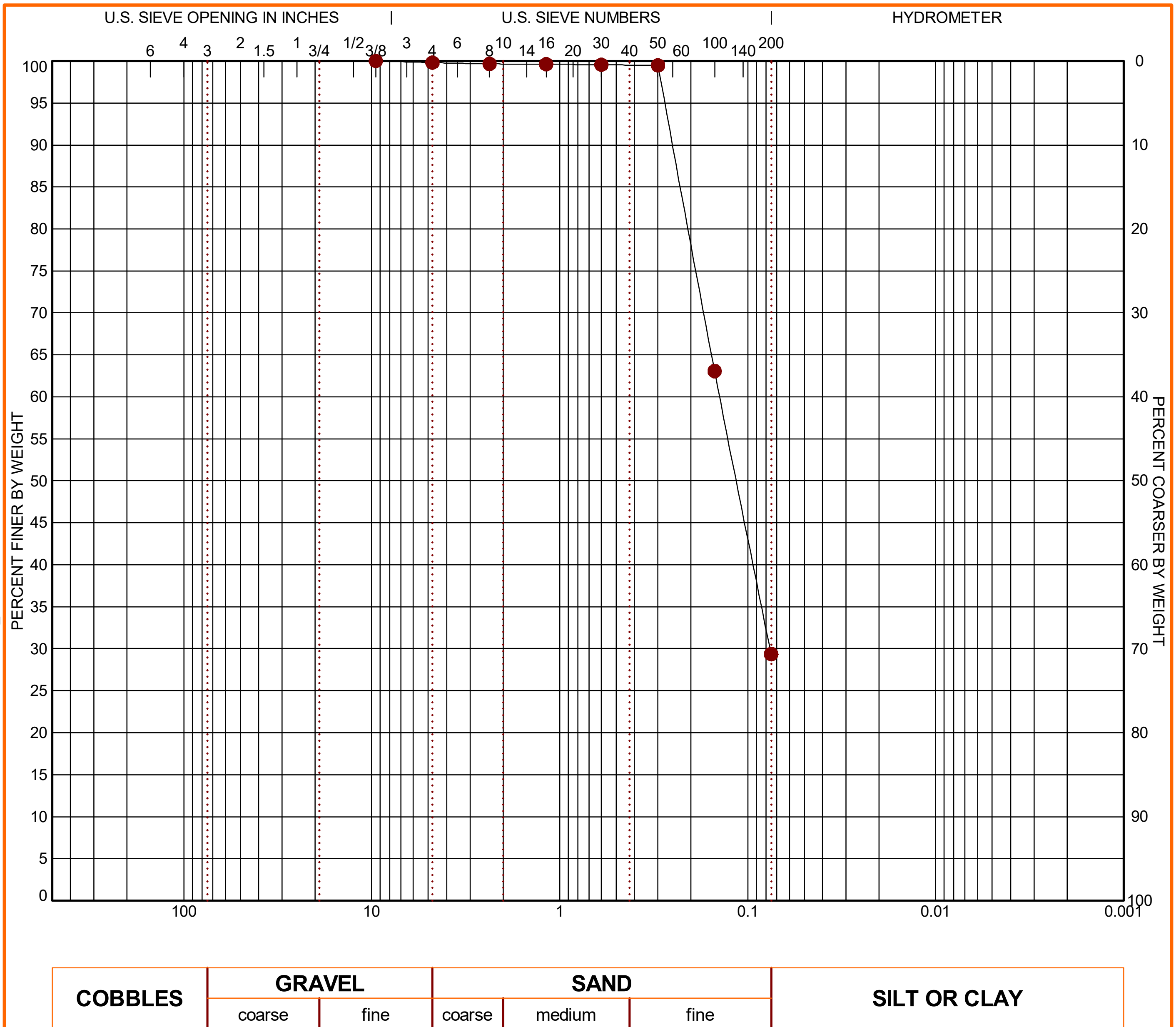
Terracon
1506 Mid Cities Dr
Pharr, TX

PROJECT NUMBER: 88215034

CLIENT: ARCADIS US, Inc.
Metairie, LA

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
● TP-3	3.1	0.0	0.2	70.4		29.4		

GRAIN SIZE

	●		
D_{60}	0.141		
D_{30}	0.076		
D_{10}			

COEFFICIENTS

	●		
C_C			
C_U			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
3/8"	100.0				
#4	99.81				
#8	99.67				
#16	99.61				
#30	99.55				
#50	99.48				
#100	63.07				
#200	29.36				

SOIL DESCRIPTION

● Silty Sand (SM)

REMARKS

●

PROJECT: Border Wall Geotechnical Services

SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

Terracon
1506 Mid Cities Dr
Pharr, TX

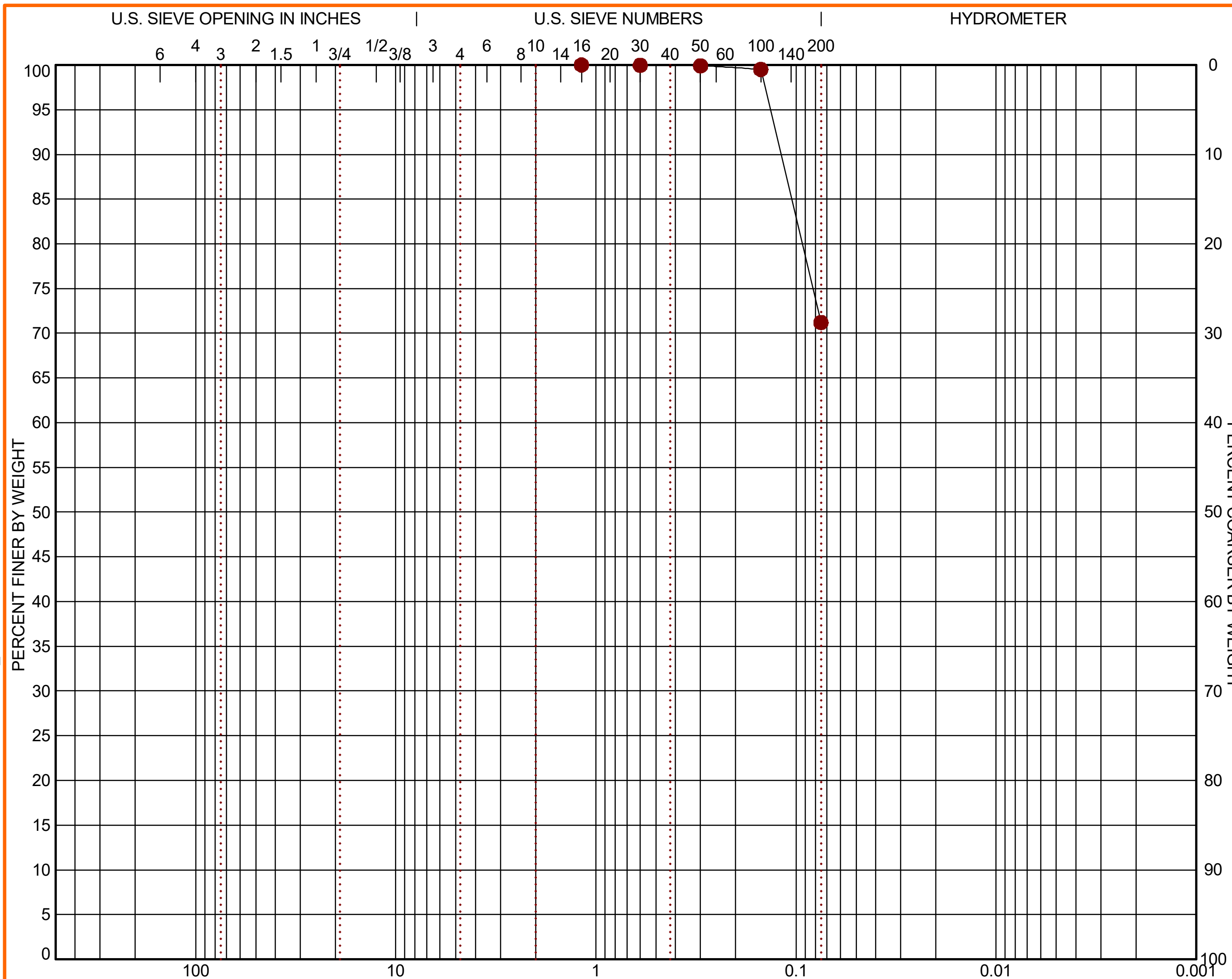
PROJECT NUMBER: 88215034

CLIENT: ARCADIS US, Inc.
Metairie, LA

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 88215034 BORDER WALL GEOTE.GPJ TERRACON_DATATEMPLATE.GDT 6/29/21

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	TP-4	3.1	0.0	0.0	28.8		71.2		

GRAIN SIZE	●		
D_{60}			
D_{30}			
D_{10}			

COEFFICIENTS	●		
C_C			
C_U			

Sieve ●	% Finer	Sieve	% Finer	Sieve	% Finer
#16	100.0				
#30	99.96				
#50	99.9				
#100	99.5				
#200	71.2				

SOIL DESCRIPTION	
●	Lean Clay with Sand (CL)

REMARKS	
●	

PROJECT: Border Wall Geotechnical Services

SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

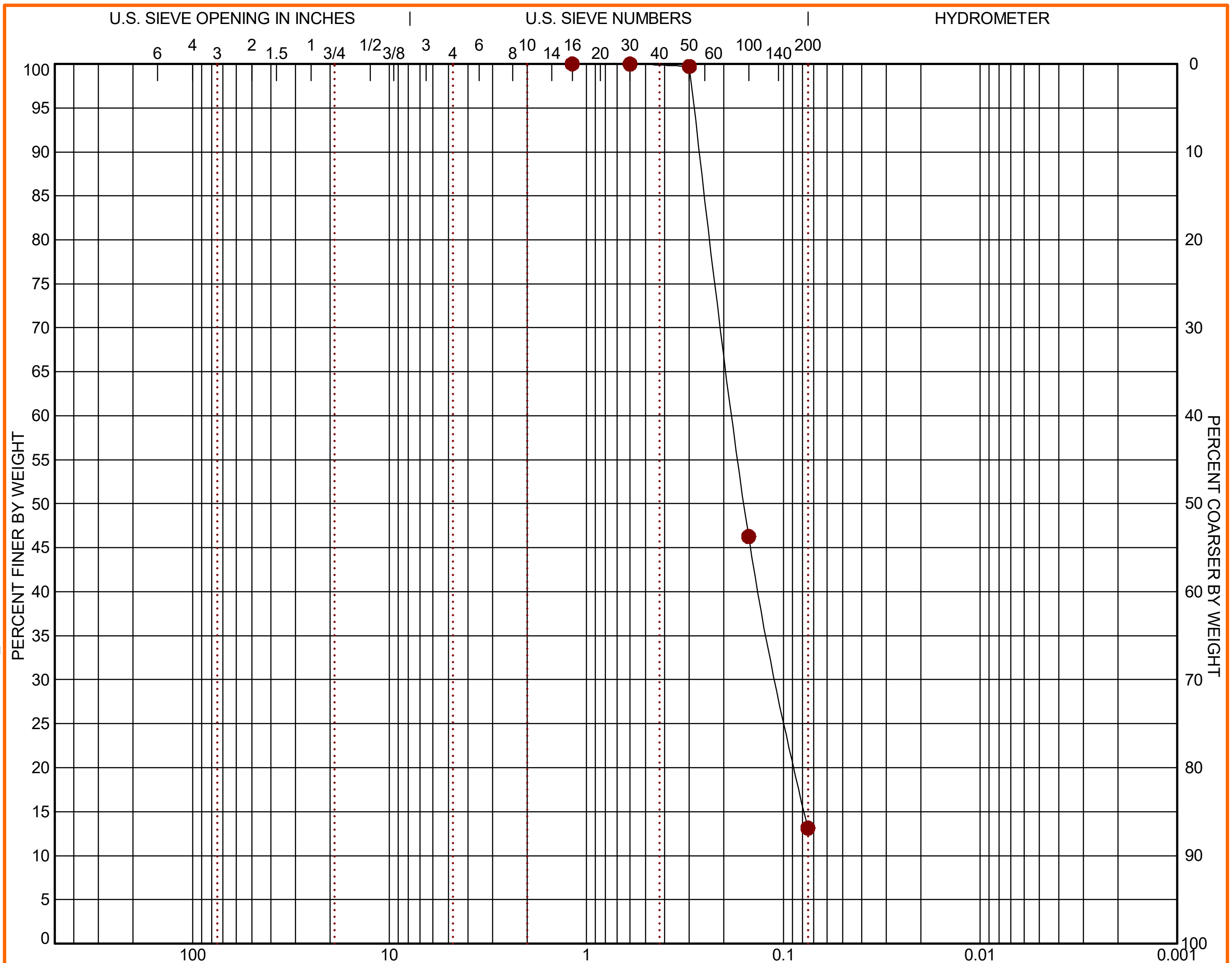
Terracon
1506 Mid Cities Dr
Pharr, TX

PROJECT NUMBER: 88215034

CLIENT: ARCADIS US, Inc.
Metairie, LA

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	TP-5	3.1	0.0	0.0	86.9		13.1		

GRAIN SIZE			
	●		
D_{60}	0.179		
D_{30}	0.107		
D_{10}			

COEFFICIENTS			
	●		
C_C			
C_U			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#16	100.0				
#30	99.99				
#50	99.71				
#100	46.27				
#200	13.13				

SOIL DESCRIPTION	
●	Silty Sand (SM)

REMARKS	
●	

PROJECT: Border Wall Geotechnical Services

SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

Terracon
1506 Mid Cities Dr
Pharr, TX

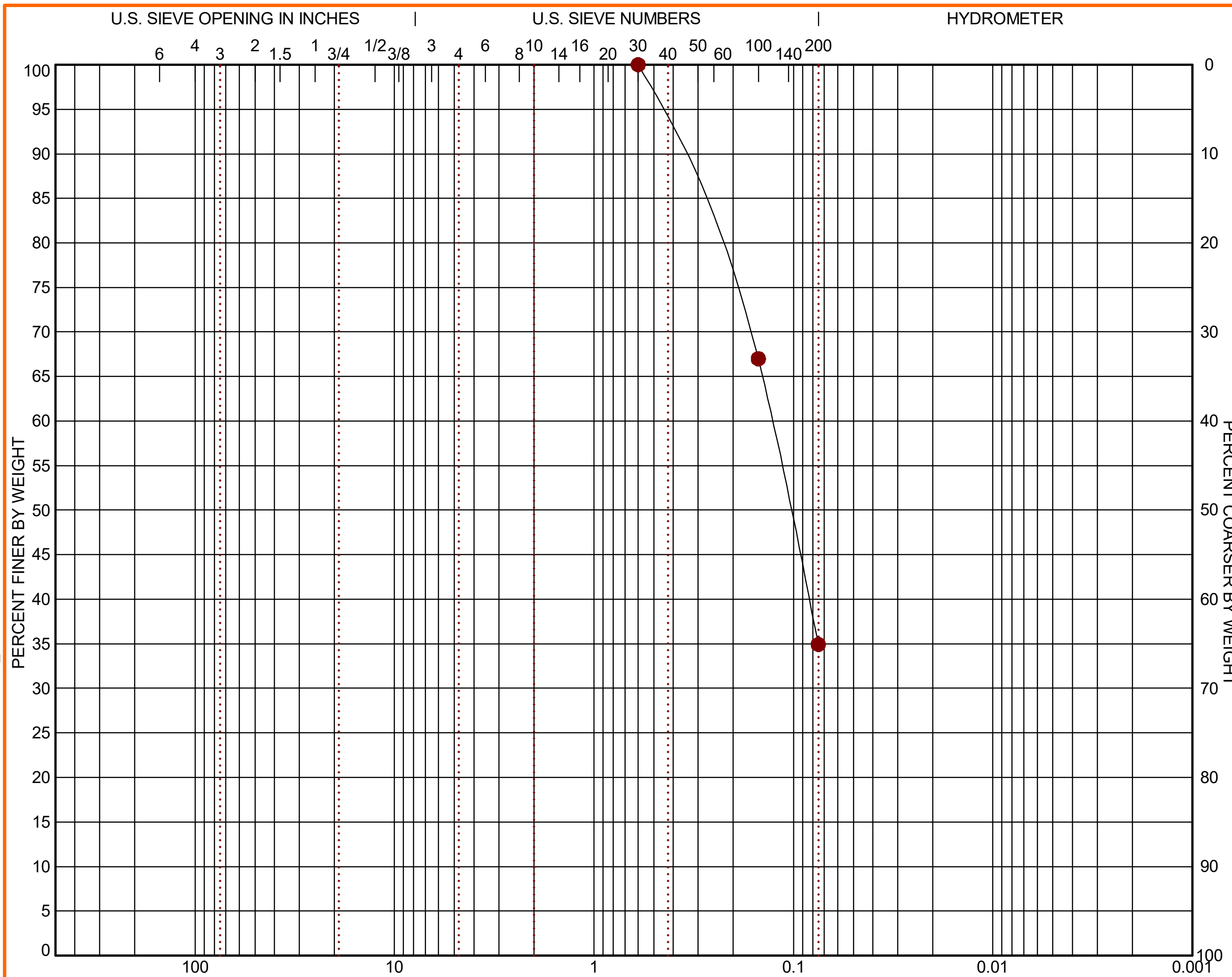
Arcadis 000247

PROJECT NUMBER: 88215034

CLIENT: ARCADIS US, Inc.
Metairie, LA

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	TP-6	3.1	0.0	0.0	65.1		34.9		

GRAIN SIZE

	●		
D_{60}	0.129		
D_{30}			
D_{10}			

COEFFICIENTS

	●		
C_c			
C_u			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#30	100.0				
#100	67.0				
#200	34.92				

SOIL DESCRIPTION

●	Silty Clayey Sand (SC-SM)

REMARKS

●	

PROJECT: Border Wall Geotechnical Services

SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

Terracon
1506 Mid Cities Dr
Pharr, TX

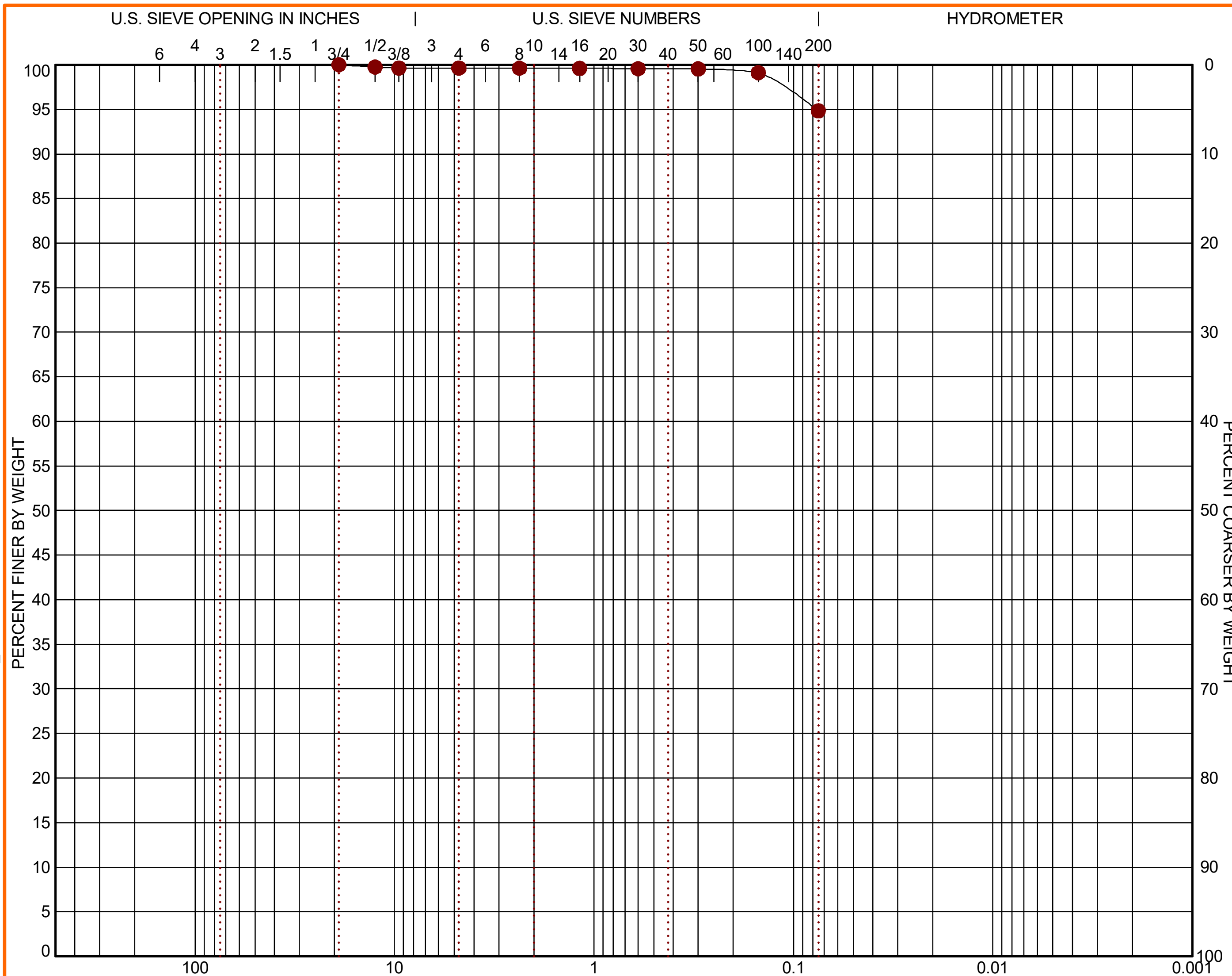
PROJECT NUMBER: 88215034

CLIENT: ARCADIS US, Inc.
Metairie, LA

Arcadis 000248

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	TP-7	3.1	0.0	0.4	4.8		94.8		

GRAIN SIZE

	●		
D_{60}			
D_{30}			
D_{10}			

COEFFICIENTS

	●		
C_c			
C_u			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
3/4"	100.0				
1/2"	99.74				
3/8"	99.63				
#4	99.63				
#8	99.61				
#16	99.59				
#30	99.56				
#50	99.54				
#100	99.12				
#200	94.83				

SOIL DESCRIPTION

● Lean Clay (CL)

REMARKS

●

PROJECT: Border Wall Geotechnical Services

PROJECT NUMBER: 88215034

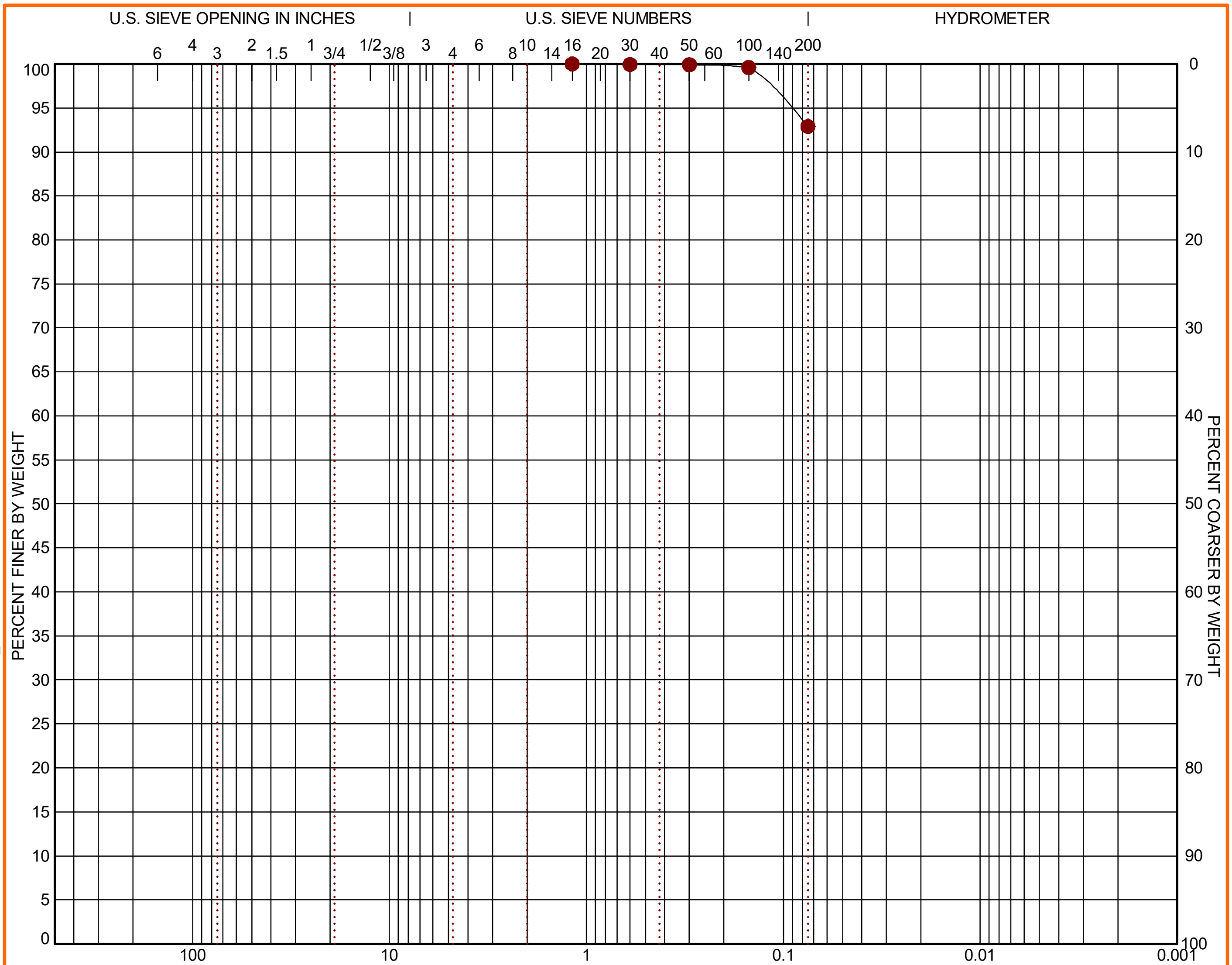
SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

Terracon
1506 Mid Cities Dr
Pharr, TX

CLIENT: ARCADIS US, Inc.
Metairie, LA

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	TP-8	3.1	0.0	0.0	7.1		92.9		

GRAIN SIZE	●		
D_{60}			
D_{30}			
D_{10}			

COEFFICIENTS	●		
C_c			
C_u			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#16	100.0				
#30	99.94				
#50	99.91				
#100	99.6				
#200	92.89				

	SOIL DESCRIPTION
●	Lean Clay (CL)

	REMARKS
●	

PROJECT: Border Wall Geotechnical Services

PROJECT NUMBER: 88215034

SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

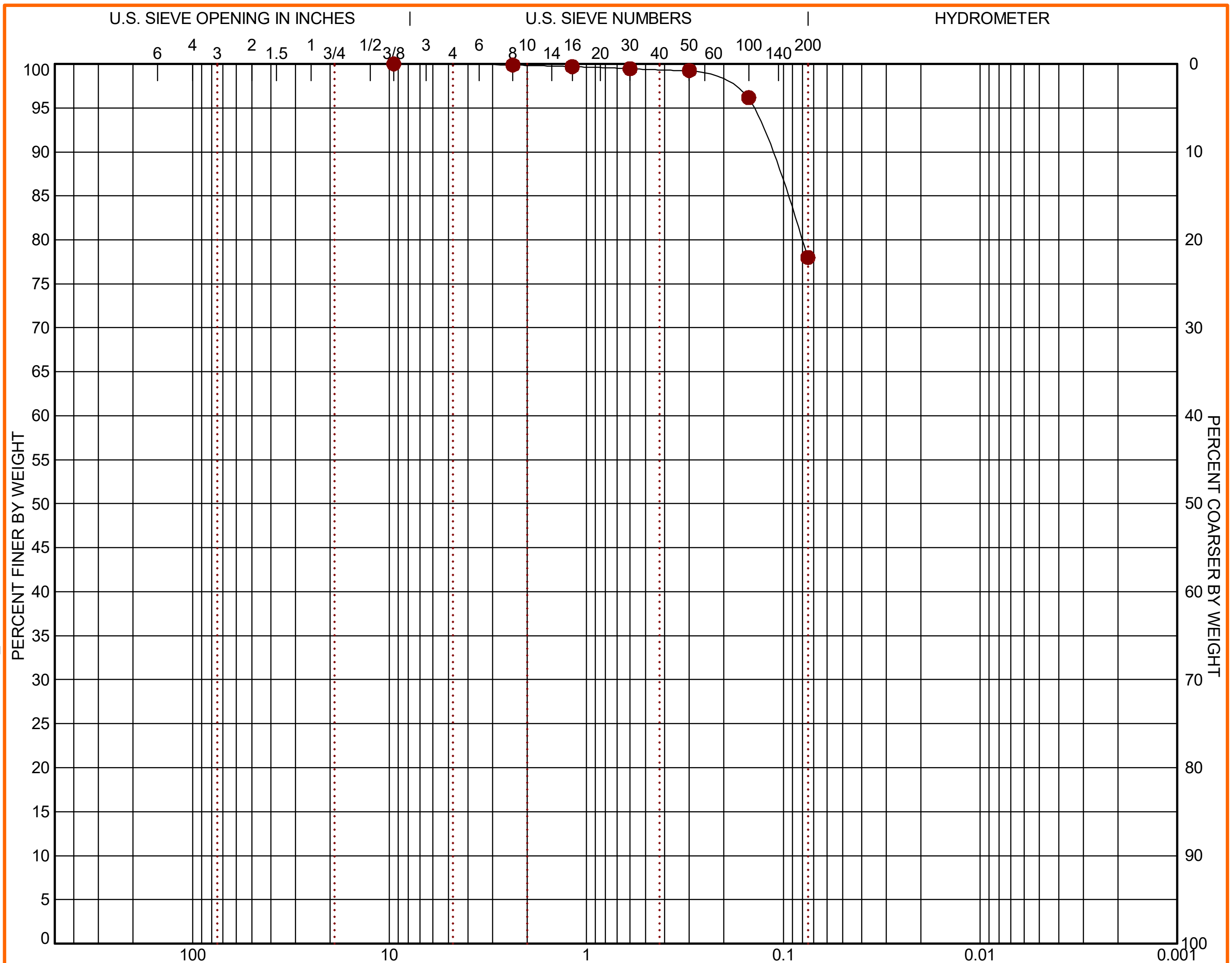
Terracon
1506 Mid Cities Dr
Pharr, TX

CLIENT: ARCADIS US, Inc.
Metairie, LA

Arcadis 000250

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	TP-9	3.1	0.0	0.1	21.9		78.0		

GRAIN SIZE			
	●		
D_{60}			
D_{30}			
D_{10}			

COEFFICIENTS			
	●		
C_C			
C_U			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#200	78.0				
3/8"	100.0				
#8	99.87				
#16	99.69				
#30	99.45				
#50	99.25				
#100	96.16				

SOIL DESCRIPTION	
●	Lean Clay with Sand (CL)

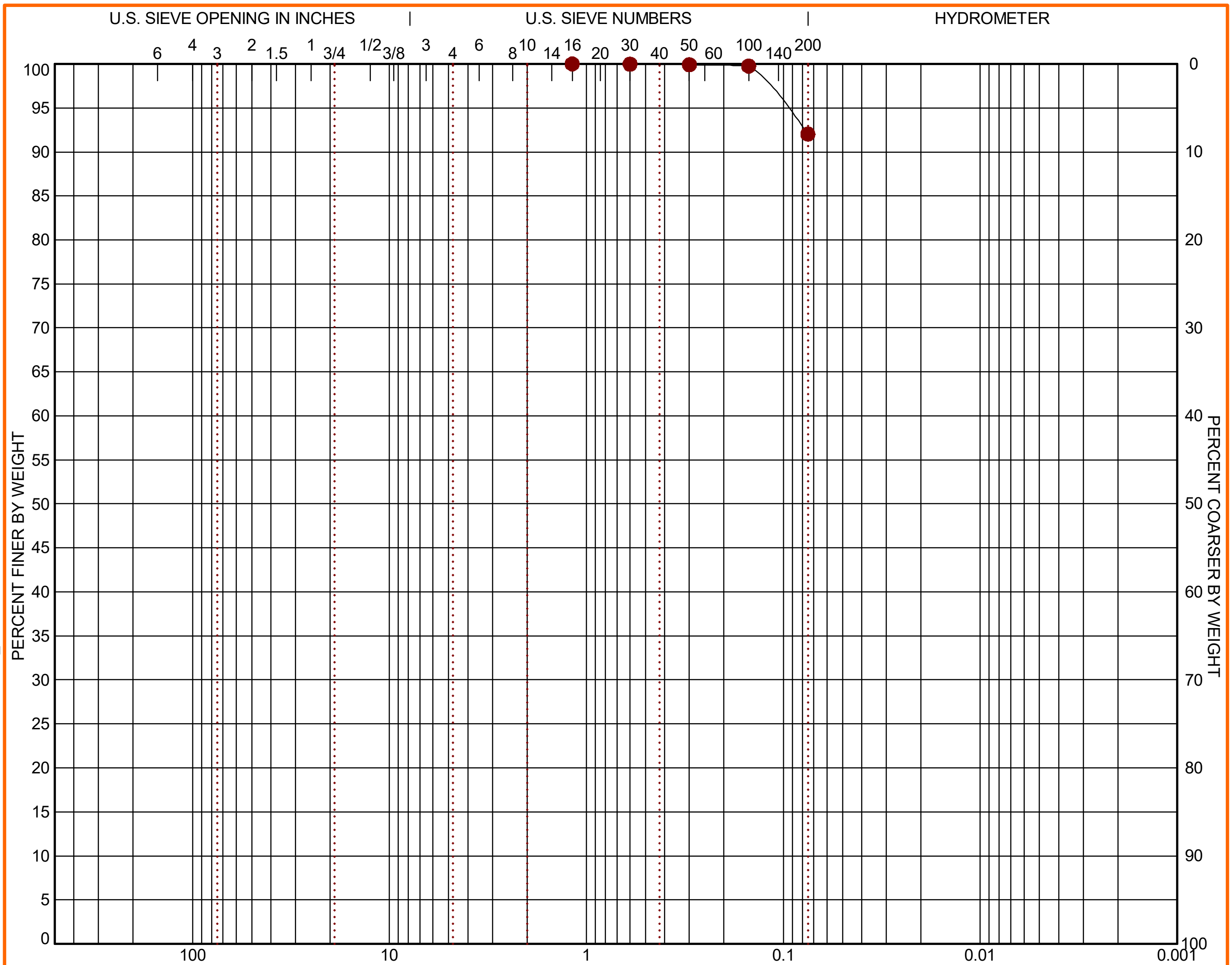
REMARKS	
●	

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 88215034 BORDER WALL GEOTE.GPJ TERRACON_DATATEMPLATE.GDT 6/29/21

PROJECT: Border Wall Geotechnical Services	Terracon 1506 Mid Cities Dr Pharr, TX	PROJECT NUMBER: 88215034
SITE: 1.75 Miles SW of Madero,Texas Mission, Texas		CLIENT: ARCADIS US, Inc. Metairie, LA

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	TP-10	3.1	0.0	0.0	8.0		92.0		

GRAIN SIZE

	●		
D_{60}			
D_{30}			
D_{10}			

COEFFICIENTS

	●		
C_C			
C_U			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#16	100.0				
#30	99.98				
#50	99.91				
#100	99.75				
#200	92.02				

SOIL DESCRIPTION

●	Lean Clay (CL)

REMARKS

●	

PROJECT: Border Wall Geotechnical Services

SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

Terracon
1506 Mid Cities Dr
Pharr, TX

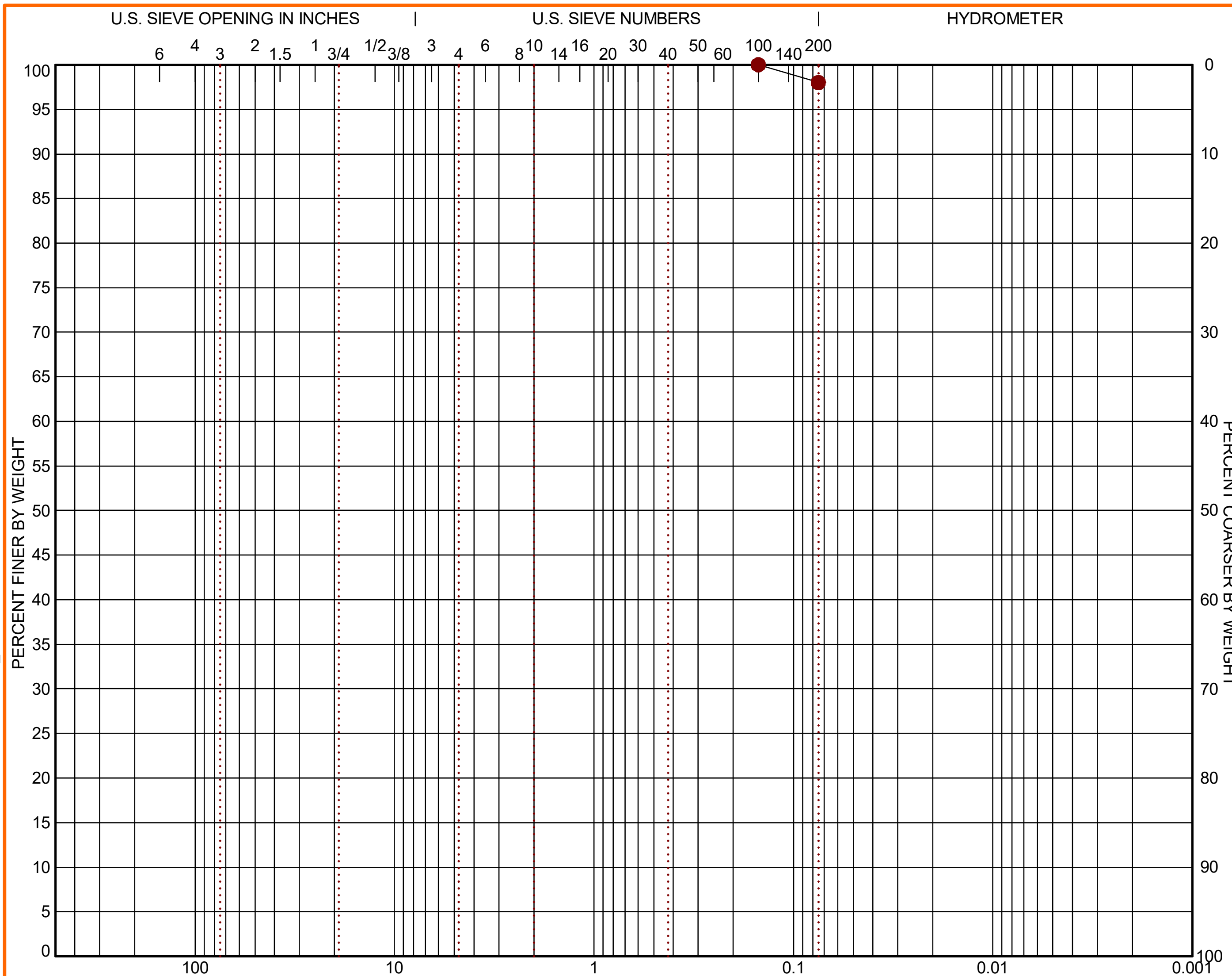
PROJECT NUMBER: 88215034

CLIENT: ARCADIS US, Inc.
Metairie, LA

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 88215034 BORDER WALL GEOTE.GPJ TERRACON_DATATEMPLATE.GDT 6/29/21

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	TP-11	3.1	0.0	0.0	2.0		98.0		

GRAIN SIZE

	●		
D_{60}			
D_{30}			
D_{10}			

COEFFICIENTS

	●		
C_C			
C_U			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#100 #200	100.0 98.01				

SOIL DESCRIPTION

●	Silt (ML)

REMARKS

●	

PROJECT: Border Wall Geotechnical Services

SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

Terracon
1506 Mid Cities Dr
Pharr, TX

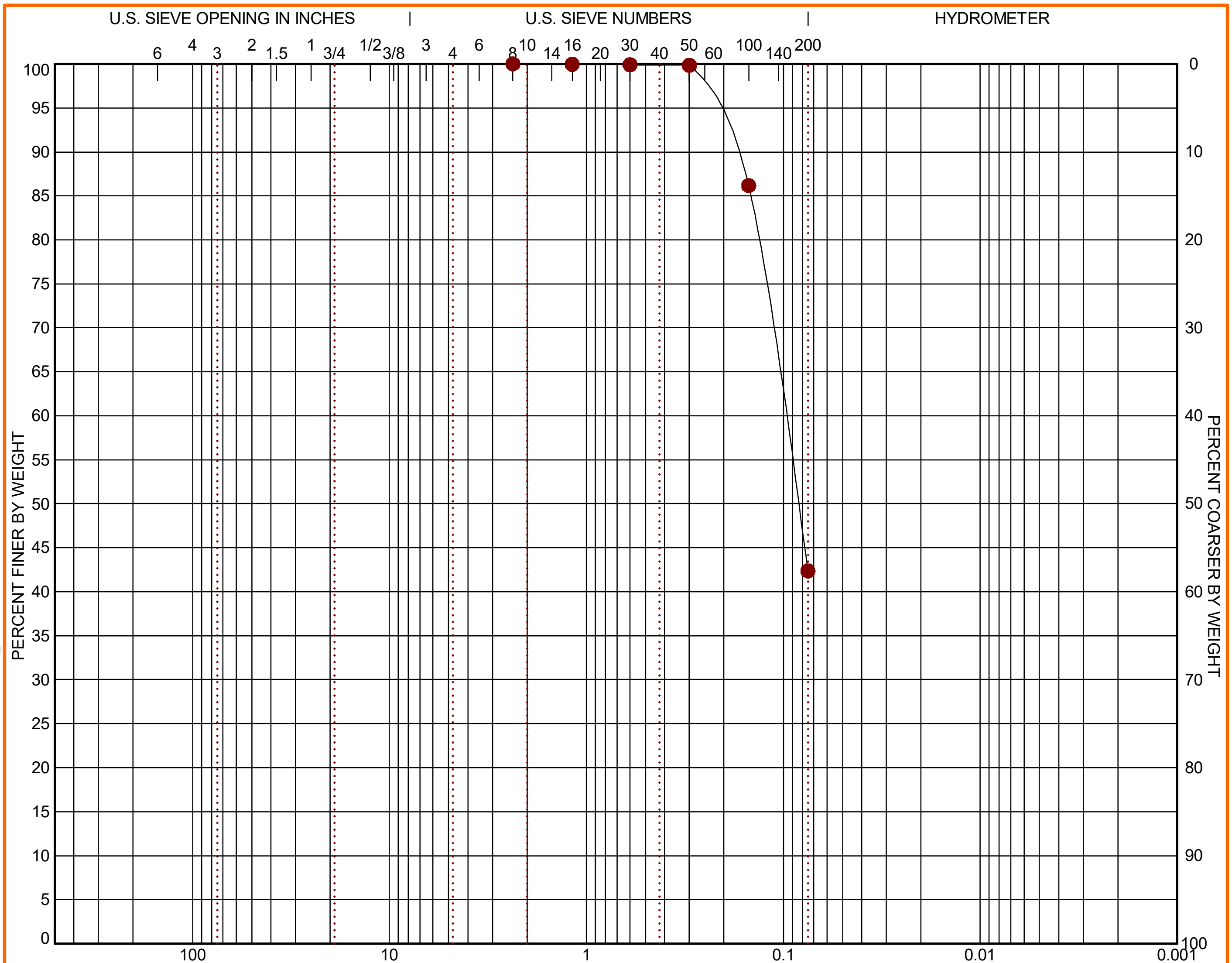
Arcadis 000253

PROJECT NUMBER: 88215034

CLIENT: ARCADIS US, Inc.
Metairie, LA

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	TP-12	3.1	0.0	0.0	57.6		42.4		

GRAIN SIZE

	●		
D_{60}	0.099		
D_{30}			
D_{10}			

COEFFICIENTS

	●		
C_C			
C_U			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#8	100.0				
#16	99.95				
#30	99.91				
#50	99.84				
#100	86.17				
#200	42.37				

SOIL DESCRIPTION

●	Silty Clayey Sand (SC-SM)

REMARKS

●	

PROJECT: Border Wall Geotechnical Services	Terracon 1506 Mid Cities Dr Pharr, TX	PROJECT NUMBER: 88215034
SITE: 1.75 Miles SW of Madero,Texas Mission, Texas		CLIENT: ARCADIS US, Inc. Metairie, LA

SAND CONE DENSITY TESTING REPORT

Report Number: 88215034.0007
Service Date: 04/28/21
Report Date: 06/28/21
Task: Labor

Terracon
1506 Mid Cities Dr
Pharr, TX 78577-2128
956-283-8254 Reg No: F-3272

Client

ARCADIS US, Inc.
Attn: Charlie Wildman
3850 N Causeway Blvd
Suite 990
Metairie, LA 70002

Project

Border Wall Geotechnical Services
1.75 Miles SW of Madero ,Texas
Mission, TX

Project Number: 88215034

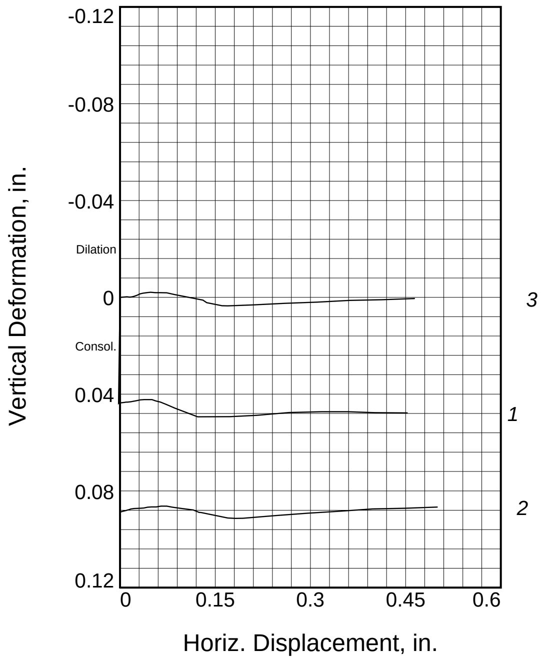
Test Pit No.	Field Test Results			Max. Dry Density, pcf	Opt. Moisture Content, %	Soil Classification
	Dry Density, pcf	Moisture Content, %	Compaction, %			
1	123.0	11.5	107.9	114.0	13.8	Sandy Lean Clay (CL)
2	88.9	12.8	82.8	107.4	14.4	Silty Sand (SM)
3	87.3	9.6	81.3	107.4	14.4	Silty Sand (SM)
4	84.7	15.4	74.3	114.0	13.8	Sandy Lean Clay (CL)
5	90.2	5.9	84.0	107.4	14.4	Silty Sand (SM)
6	97.9	11.2	93.1	105.1	16.1	Silty, Clayey Sand (SC-SM)
7	87.5	14.5	88.1	99.3	19.9	Lean Clay (CL)
8	62.0	10.9	62.4	99.3	19.9	Lean Clay (CL)
9	92.4	12.7	81.0	114.0	13.8	Sandy Lean Clay (CL)
10	76.1	18.8	76.7	99.3	19.9	Lean Clay (CL)
11	95.5	22.1	90.9	105.1	16.1	Silt (ML)
12	96.2	12.7	90.4	106.4	14.3	Silty, Clayey Sand (SC-SM)

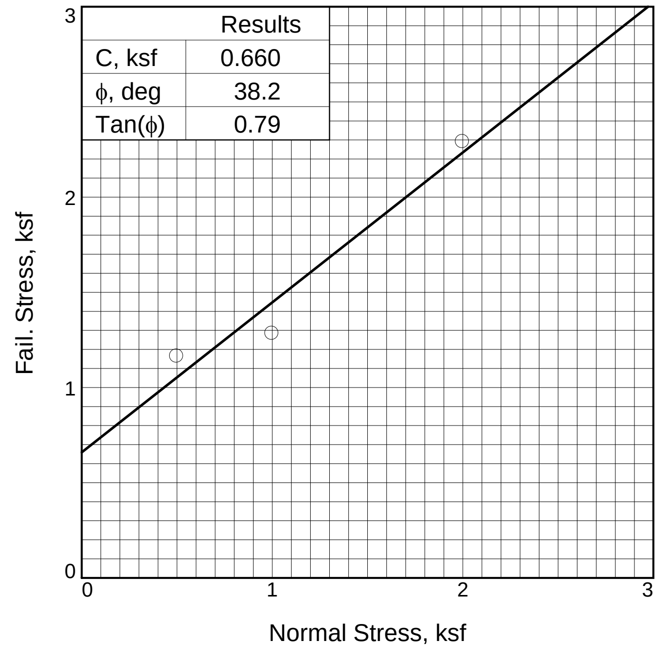
Services:

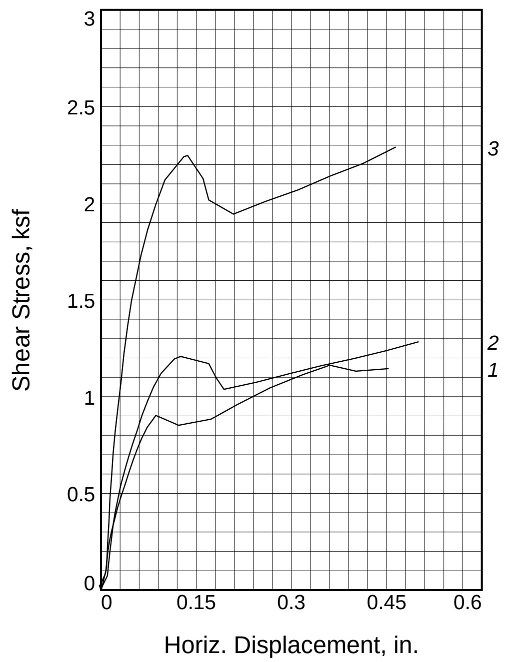
Terracon Rep.: Adrian E.Leal
Reported To:
Contractor:
Report Distribution:
(1) ARCADIS US, Inc., Charlie Wildman

Reviewed By: ______________________

Martin Reyes
Senior Staff Engineer







	Sample No.	1	2	3
Initial	Water Content, %	13.6	13.6	13.6
	Dry Density, pcf	109.5	108.9	109.7
	Saturation, %	67.9	66.8	68.2
	Void Ratio	0.5391	0.5485	0.5365
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	20.7	20.9	19.0
	Dry Density, pcf	109.5	108.9	109.7
	Saturation, %	103.5	102.9	95.4
	Void Ratio	0.5391	0.5485	0.5365
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.000	1.000	1.000
	Normal Stress, ksf	0.500	1.000	2.000
	Fail. Stress, ksf	1.163	1.283	2.290
	Displacement, in.	0.359	0.500	0.464
	Ult. Stress, ksf			
	Displacement, in.			
	Strain rate, in./min.	0.001	0.001	0.001

Sample Type: Laboratory Molded
Description: Sandy Lean Clay (CL)

Assumed Specific Gravity= 2.7
Remarks:

Client: Arcadis, Inc.

Project: Border Wall Geotechnical Services

Location: TP-1

Depth: 3 ft.

Proj. No.: 88215034 **Date Sampled:**

DIRECT SHEAR TEST REPORT
Terracon Consultants, Inc.
Houston, TX

DIRECT SHEAR TEST

5/25/2021

Date:
Client: Arcadis, Inc.
Project: Border Wall Geotechnical Services
Project No.: 88215034
Location: TP-1
Depth: 3 ft.
Description: Sandy Lean Clay (CL)
Remarks:
Type of Sample:
Assumed Specific Gravity= 2.7 **LL=** 28 **PL=** 14 **PI=** 14

Parameters for Specimen No. 1

Specimen Parameter	Initial	Consolidated	Final
Moisture content: Moist soil+tare, gms.	214.730		205.500
Moisture content: Dry soil+tare, gms.	196.020		179.580
Moisture content: Tare, gms.	58.050		54.200
Moisture, %	13.6	20.7	20.7
Moist specimen weight, gms.	160.25		
Diameter, in.	2.500	2.500	
Area, in.²	4.909	4.909	
Height, in.	1.000	1.000	
Net decrease in height, in.		0.000	
Wet density, pcf	124.4	132.2	
Dry density, pcf	109.5	109.5	
Void ratio	0.5391	0.5391	
Saturation, %	67.9	103.5	

Test Readings for Specimen No. 1

Normal stress = 0.5 ksf
Strain rate, in./min. = 0.001
Strength calculations use strain adjusted areas
Fail. Stress = 1.163 ksf **at reading no.** 24

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
0	0.0000	0.000	0.0	0.000	0.7503
1	-0.0023	0.758	0.8	0.022	0.7065
2	-0.0018	0.750	0.8	0.022	0.7066
3	0.0072	2.951	3.0	0.087	0.7069
4	0.0092	5.108	5.1	0.151	0.7070
5	0.0112	7.070	7.1	0.209	0.7070
6	0.0137	8.766	8.8	0.259	0.7070
7	0.0172	10.571	10.6	0.313	0.7072
8	0.0217	12.410	12.4	0.368	0.7074
9	0.0262	14.406	14.4	0.428	0.7076
10	0.0322	16.428	16.4	0.490	0.7079
11	0.0382	18.339	18.3	0.549	0.7081
12	0.0442	20.402	20.4	0.612	0.7080
13	0.0502	22.220	22.2	0.669	0.7080

Test Readings for Specimen No. 1

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
14	0.0562	23.927	23.9	0.723	0.7075
15	0.0637	25.825	25.8	0.783	0.7070
16	0.0727	27.606	27.6	0.841	0.7061
17	0.0863	29.435	29.4	0.903	0.7045
18	0.1223	27.242	27.2	0.852	0.7009
19	0.1733	27.459	27.5	0.883	0.7010
20	0.2153	29.145	29.1	0.960	0.7016
21	0.2664	30.821	30.8	1.046	0.7027
22	0.3174	31.821	31.8	1.113	0.7030
23	0.3564	32.319	32.3	1.157	0.7030
24	0.3594	32.414	32.4	1.163	0.7030
25	0.4015	30.726	30.7	1.132	0.7026
26	0.4525	30.087	30.1	1.145	0.7025

Parameters for Specimen No. 2

Specimen Parameter	Initial	Consolidated	Final
Moisture content: Moist soil+tare, gms.	214.730		228.820
Moisture content: Dry soil+tare, gms.	196.020		200.250
Moisture content: Tare, gms.	58.050		63.580
Moisture, %	13.6	20.9	20.9
Moist specimen weight, gms.	159.28		
Diameter, in.	2.500	2.500	
Area, in.^{2}	4.909	4.909	
Height, in.	1.000	1.000	
Net decrease in height, in.		0.000	
Wet density, pcf	123.6	131.6	
Dry density, pcf	108.9	108.9	
Void ratio	0.5485	0.5485	
Saturation, %	66.8	102.9	

Test Readings for Specimen No. 2

Normal stress = 1.0 ksf

Strain rate, in./min. = 0.001

Strength calculations use strain adjusted areas

Fail. Stress = 1.283 ksf **at reading no.** 30

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
0	0.0000	0.000	0.0	0.000	0.7503
1	0.0000	0.526	0.5	0.015	0.6612
2	0.0010	0.467	0.5	0.014	0.6617
3	0.0100	2.509	2.5	0.074	0.6622
4	0.0120	4.689	4.7	0.138	0.6624
5	0.0140	6.806	6.8	0.201	0.6625
6	0.0165	9.080	9.1	0.269	0.6628
7	0.0190	11.422	11.4	0.338	0.6629
8	0.0220	13.441	13.4	0.399	0.6630
9	0.0270	16.112	16.1	0.479	0.6631
10	0.0315	18.398	18.4	0.549	0.6631

Test Readings for Specimen No. 2

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
11	0.0375	20.792	20.8	0.622	0.6633
12	0.0435	22.959	23.0	0.689	0.6636
13	0.0495	25.048	25.0	0.754	0.6637
14	0.0570	27.348	27.3	0.826	0.6637
15	0.0645	29.758	29.8	0.903	0.6640
16	0.0736	32.142	32.1	0.980	0.6640
17	0.0826	34.245	34.2	1.049	0.6636
18	0.0946	36.339	36.3	1.120	0.6631
19	0.1156	38.344	38.3	1.195	0.6624
20	0.1246	38.529	38.5	1.207	0.6614
21	0.1276	38.442	38.4	1.206	0.6613
22	0.1696	36.477	36.5	1.171	0.6591
23	0.1816	33.912	33.9	1.096	0.6589
24	0.1936	31.906	31.9	1.038	0.6590
25	0.2447	32.072	32.1	1.075	0.6601
26	0.2957	32.386	32.4	1.118	0.6611
27	0.3467	32.586	32.6	1.160	0.6619
28	0.3978	32.563	32.6	1.197	0.6628
29	0.4488	32.586	32.6	1.237	0.6631
30	0.4998	32.675	32.7	1.283	0.6636

Parameters for Specimen No. 3

Specimen Parameter	Initial	Consolidated	Final
Moisture content: Moist soil+tare, gms.	214.730		211.190
Moisture content: Dry soil+tare, gms.	196.020		183.940
Moisture content: Tare, gms.	58.050		40.150
Moisture, %	13.6	19.0	19.0
Moist specimen weight, gms.	160.52		
Diameter, in.	2.500	2.500	
Area, in.²	4.909	4.909	
Height, in.	1.000	1.000	
Net decrease in height, in.		0.000	
Wet density, pcf	124.6	130.5	
Dry density, pcf	109.7	109.7	
Void ratio	0.5365	0.5365	
Saturation, %	68.2	95.4	

Test Readings for Specimen No. 3

Normal stress = 2.0 ksf

Strain rate, in./min. = 0.001

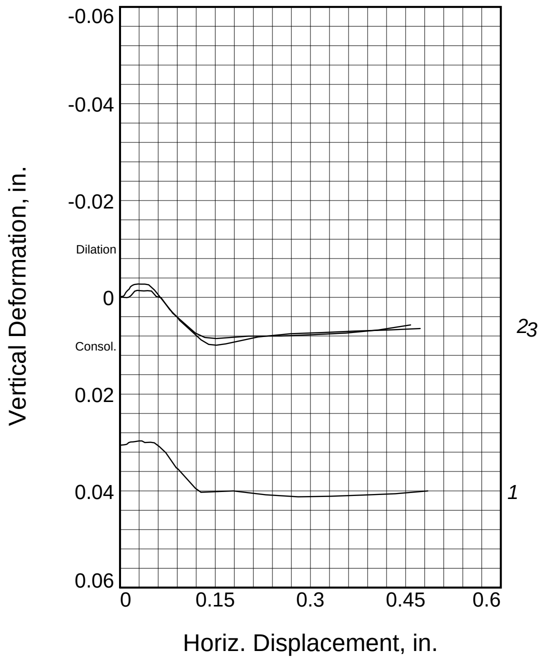
Strength calculations use strain adjusted areas

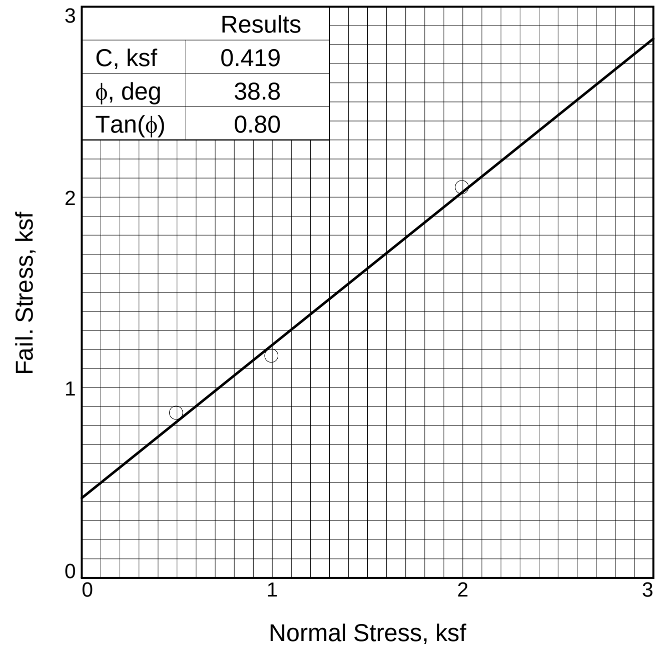
Fail. Stress = 2.290 ksf **at reading no.** 28

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
0	0.0000	0.028	0.0	0.000	0.7503
1	0.0005	0.079	0.1	0.002	0.7503
2	0.0085	3.914	3.9	0.114	0.7505
3	0.0105	8.264	8.2	0.243	0.7505
4	0.0125	12.267	12.2	0.361	0.7505
5	0.0140	16.086	16.1	0.474	0.7504
6	0.0165	19.839	19.8	0.586	0.7504
7	0.0190	23.935	23.9	0.708	0.7505
8	0.0225	27.844	27.8	0.825	0.7508
9	0.0270	32.189	32.2	0.957	0.7512
10	0.0315	36.160	36.1	1.077	0.7517
11	0.0360	40.941	40.9	1.223	0.7520
12	0.0420	45.662	45.6	1.368	0.7522
13	0.0480	49.849	49.8	1.498	0.7524
14	0.0556	53.587	53.6	1.617	0.7523
15	0.0631	57.171	57.1	1.732	0.7522
16	0.0736	61.176	61.1	1.864	0.7522
17	0.0856	64.849	64.8	1.988	0.7514
18	0.1006	68.604	68.6	2.120	0.7507
19	0.1306	71.362	71.3	2.242	0.7491
20	0.1366	71.277	71.2	2.246	0.7481
21	0.1606	66.620	66.6	2.127	0.7469
22	0.1696	62.838	62.8	2.017	0.7468
23	0.2087	59.252	59.2	1.944	0.7471
24	0.2597	59.506	59.5	2.010	0.7478
25	0.3107	59.431	59.4	2.069	0.7483
26	0.3618	59.628	59.6	2.142	0.7490
27	0.4128	59.477	59.4	2.206	0.7493

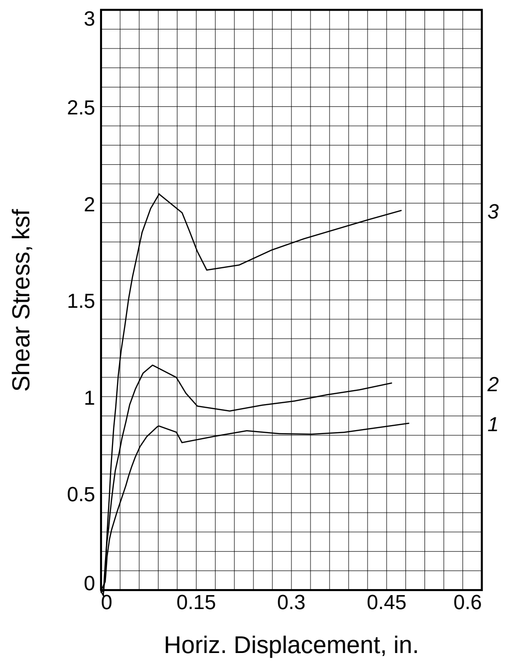
Test Readings for Specimen No. 3

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
28	0.4638	59.745	59.7	2.290	0.7498





Results	
C, ksf	0.419
ϕ, deg	38.8
Tan(ϕ)	0.80



	Sample No.	1	2	3
Initial	Water Content, %	15.9	15.9	15.9
	Dry Density, pcf	100.5	100.3	100.3
	Saturation, %	63.5	63.2	63.3
	Void Ratio	0.6780	0.6812	0.6801
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.020	1.010	1.030
At Test	Water Content, %	22.5	21.1	21.6
	Dry Density, pcf	100.5	100.3	100.3
	Saturation, %	89.4	83.7	85.8
	Void Ratio	0.6780	0.6812	0.6801
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.020	1.010	1.030
	Normal Stress, ksf	0.500	1.000	2.000
	Fail. Stress, ksf	0.862	1.163	2.048
	Displacement, in.	0.485	0.081	0.092
	Ult. Stress, ksf			
	Displacement, in.			
	Strain rate, in./min.	0.001	0.001	0.001

Sample Type: Laboratory Molded
Description: Silty Clayey Sand (SC-SM

Assumed Specific Gravity= 2.70
Remarks:

Client: Arcadis, Inc.

Project: Border Wall Geotechnical Services

Location: TP-6

Depth: 3 ft.

Proj. No.: 88215034 **Date Sampled:**

DIRECT SHEAR TEST REPORT
Terracon Consultants, Inc.
Houston, TX

DIRECT SHEAR TEST

5/25/2021

Date:
Client: Arcadis, Inc.
Project: Border Wall Geotechnical Services
Project No.: 88215034
Location: TP-6
Depth: 3 ft.
Description: Silty Clayey Sand (SC-SM)
Remarks:
Type of Sample: Laboratory Molded
Assumed Specific Gravity= 2.70 **LL= 25** **PL= 18** **PI= 7**

Parameters for Specimen No. 1

Specimen Parameter	Initial	Consolidated	Final
Moisture content: Moist soil+tare, gms.	180.750		214.880
Moisture content: Dry soil+tare, gms.	163.970		186.260
Moisture content: Tare, gms.	58.720		58.780
Moisture, %	15.9	22.5	22.5
Moist specimen weight, gms.	153.07		
Diameter, in.	2.500	2.500	
Area, in.^{2}	4.909	4.909	
Height, in.	1.020	1.020	
Net decrease in height, in.		0.000	
Wet density, pcf	116.5	123.0	
Dry density, pcf	100.5	100.5	
Void ratio	0.6780	0.6780	
Saturation, %	63.5	89.4	

Test Readings for Specimen No. 1

Normal stress = 0.50 ksf
Strain rate, in./min. = 0.001
Strength calculations use strain adjusted areas
Fail. Stress = 0.862 ksf **at reading no.** 30

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
0	0.0000	0.000	0.0	0.000	0.7212
1	0.0000	-0.088	-0.1	-0.003	0.6906
2	0.0065	1.444	1.4	0.043	0.6907
3	0.0075	2.872	2.9	0.085	0.6907
4	0.0085	4.466	4.5	0.132	0.6907
5	0.0095	5.867	5.9	0.173	0.6907
6	0.0115	7.540	7.5	0.222	0.6909
7	0.0135	8.967	9.0	0.265	0.6911
8	0.0165	10.520	10.5	0.311	0.6913
9	0.0205	11.965	12.0	0.355	0.6913
10	0.0250	13.526	13.5	0.402	0.6914
11	0.0300	15.170	15.2	0.452	0.6915
12	0.0345	16.549	16.5	0.494	0.6915
13	0.0390	18.048	18.0	0.540	0.6912

Test Readings for Specimen No. 1

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
14	0.0435	19.663	19.7	0.590	0.6912
15	0.0480	21.161	21.2	0.636	0.6912
16	0.0540	22.818	22.8	0.688	0.6911
17	0.0615	24.517	24.5	0.742	0.6903
18	0.0720	26.064	26.1	0.794	0.6891
19	0.0886	27.485	27.5	0.844	0.6859
20	0.0901	27.554	27.6	0.847	0.6858
21	0.0916	27.547	27.5	0.848	0.6856
22	0.1186	26.142	26.1	0.816	0.6817
23	0.1276	24.296	24.3	0.762	0.6809
24	0.1786	24.626	24.6	0.795	0.6811
25	0.2297	24.790	24.8	0.823	0.6803
26	0.2807	23.627	23.6	0.808	0.6799
27	0.3317	22.836	22.8	0.806	0.6800
28	0.3828	22.394	22.4	0.815	0.6803
29	0.4338	22.305	22.3	0.839	0.6806
30	0.4848	22.178	22.2	0.862	0.6811

Parameters for Specimen No. 2

Specimen Parameter	Initial	Consolidated	Final
Moisture content: Moist soil+tare, gms.	180.750		216.930
Moisture content: Dry soil+tare, gms.	163.970		189.600
Moisture content: Tare, gms.	58.720		60.220
Moisture, %	15.9	21.1	21.1
Moist specimen weight, gms.	151.28		
Diameter, in.	2.500	2.500	
Area, in.²	4.909	4.909	
Height, in.	1.010	1.010	
Net decrease in height, in.		0.000	
Wet density, pcf	116.2	121.4	
Dry density, pcf	100.3	100.3	
Void ratio	0.6812	0.6812	
Saturation, %	63.2	83.7	

Test Readings for Specimen No. 2

Normal stress = 1.0 ksf

Strain rate, in./min. = 0.001

Strength calculations use strain adjusted areas

Fail. Stress = 1.163 ksf **at reading no.** 17

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
0	0.0000	8.741	0.0	0.000	0.7147
1	0.0005	8.410	-0.3	-0.010	0.7148
2	0.0035	7.787	-1.0	-0.028	0.7149
3	0.0055	11.374	2.6	0.077	0.7150
4	0.0070	13.861	5.1	0.151	0.7152
5	0.0090	16.847	8.1	0.239	0.7157
6	0.0120	19.619	10.9	0.321	0.7161
7	0.0140	21.980	13.2	0.391	0.7164
8	0.0165	24.582	15.8	0.469	0.7169
9	0.0195	27.219	18.5	0.548	0.7172
10	0.0225	29.564	20.8	0.618	0.7174
11	0.0285	32.581	23.8	0.710	0.7175
12	0.0330	35.050	26.3	0.785	0.7174
13	0.0390	37.750	29.0	0.868	0.7174
14	0.0451	40.673	31.9	0.959	0.7173
15	0.0541	43.215	34.5	1.040	0.7162
16	0.0661	45.680	36.9	1.121	0.7143
17	0.0811	46.740	38.0	1.163	0.7118
18	0.0826	46.634	37.9	1.160	0.7115
19	0.1186	43.961	35.2	1.100	0.7073
20	0.1336	41.078	32.3	1.018	0.7064
21	0.1517	38.666	29.9	0.951	0.7062
22	0.2027	37.044	28.3	0.926	0.7067
23	0.2537	37.117	28.4	0.956	0.7067
24	0.3047	36.894	28.2	0.977	0.7070
25	0.3558	36.947	28.2	1.010	0.7073
26	0.4068	36.747	28.0	1.035	0.7080
27	0.4578	36.762	28.0	1.070	0.7090

Parameters for Specimen No. 3

Specimen Parameter	Initial	Consolidated	Final
Moisture content: Moist soil+tare, gms.	180.750		218.690
Moisture content: Dry soil+tare, gms.	163.970		190.900
Moisture content: Tare, gms.	58.720		62.250
Moisture, %	15.9	21.6	21.6
Moist specimen weight, gms.	154.38		
Diameter, in.	2.500	2.500	
Area, in.²	4.909	4.909	
Height, in.	1.030	1.030	
Net decrease in height, in.		0.000	
Wet density, pcf	116.3	122.0	
Dry density, pcf	100.3	100.3	
Void ratio	0.6801	0.6801	
Saturation, %	63.3	85.8	

Test Readings for Specimen No. 3

Normal stress = 2.0 ksf

Strain rate, in./min. = 0.001

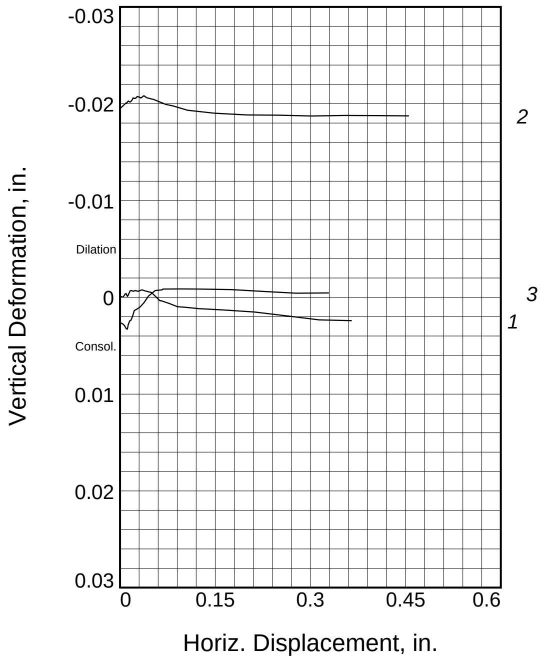
Strength calculations use strain adjusted areas

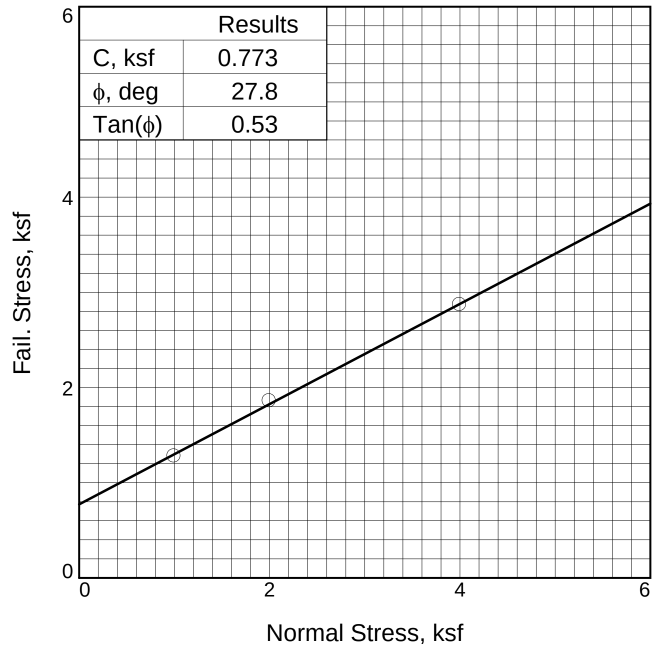
Fail. Stress = 2.048 ksf **at reading no.** 19

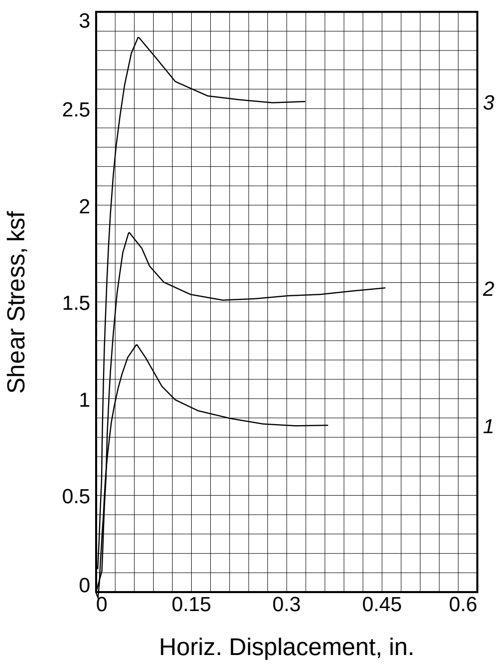
No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
0	0.0000	2.239	0.0	0.000	0.7212
1	0.0005	2.103	-0.1	-0.004	0.7211
2	0.0040	1.930	-0.3	-0.009	0.7212
3	0.0070	5.428	3.2	0.094	0.7211
4	0.0085	9.996	7.8	0.229	0.7211
5	0.0105	14.087	11.8	0.349	0.7211
6	0.0130	18.420	16.2	0.478	0.7212
7	0.0150	22.849	20.6	0.609	0.7213
8	0.0175	26.901	24.7	0.730	0.7216
9	0.0200	30.493	28.3	0.837	0.7219
10	0.0230	34.023	31.8	0.943	0.7224
11	0.0270	39.458	37.2	1.107	0.7226
12	0.0315	43.635	41.4	1.234	0.7225
13	0.0375	47.977	45.7	1.368	0.7225
14	0.0436	52.441	50.2	1.506	0.7225
15	0.0496	56.057	53.8	1.620	0.7224
16	0.0571	59.625	57.4	1.734	0.7213
17	0.0646	63.206	61.0	1.849	0.7211
18	0.0781	66.812	64.6	1.973	0.7187
19	0.0916	68.797	66.6	2.048	0.7169
20	0.0931	68.573	66.3	2.043	0.7165
21	0.1276	64.411	62.2	1.951	0.7124
22	0.1396	60.921	58.7	1.853	0.7114
23	0.1516	57.317	55.1	1.751	0.7113
24	0.1667	53.853	51.6	1.654	0.7115
25	0.2177	53.193	51.0	1.681	0.7130
26	0.2687	53.970	51.7	1.758	0.7136
27	0.3198	54.095	51.9	1.816	0.7139

Test Readings for Specimen No. 3

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
28	0.3708	53.881	51.6	1.866	0.7142
29	0.4218	53.579	51.3	1.916	0.7144
30	0.4729	53.113	50.9	1.962	0.7147







Sample No.		1	2	3
Initial	Water Content, %	20.4	20.4	20.4
	Dry Density, pcf	95.2	94.8	94.8
	Saturation, %	70.6	69.9	69.8
	Void Ratio	0.7899	0.7983	0.7984
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.090	1.110	1.090
At Test	Water Content, %	26.9	24.0	23.0
	Dry Density, pcf	95.2	94.8	94.8
	Saturation, %	93.1	82.1	78.6
	Void Ratio	0.7899	0.7983	0.7984
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.090	1.110	1.090
Normal Stress, ksf		1.000	2.000	4.000
Fail. Stress, ksf		1.278	1.858	2.868
Displacement, in.		0.063	0.053	0.066
Ult. Stress, ksf				
Displacement, in.				
Strain rate, in./min.		0.001	0.001	0.001

Sample Type: Laboratory Molded
Description: Lean Clay (CL)

Assumed Specific Gravity= 2.73
Remarks:

Client: Arcadis, Inc.

Project: Border Wall Geotechnical Services

Location: TP-7

Depth: 3 ft.

Proj. No.: 88215034 **Date Sampled:**

DIRECT SHEAR TEST REPORT
Terracon Consultants, Inc.
Houston, TX

DIRECT SHEAR TEST

5/25/2021

Date:
Client: Arcadis, Inc.
Project: Border Wall Geotechnical Services
Project No.: 88215034
Location: TP-7
Depth: 3 ft.
Description: Lean Clay (CL)
Remarks:
Type of Sample: Laboratory Molded
Assumed Specific Gravity=2.73 **LL= 48** **PL= 19** **PI= 29**

Parameters for Specimen No. 1

Specimen Parameter	Initial	Consolidated	Final
Moisture content: Moist soil+tare, gms.	226.080		229.700
Moisture content: Dry soil+tare, gms.	198.200		194.260
Moisture content: Tare, gms.	61.710		62.660
Moisture, %	20.4	26.9	26.9
Moist specimen weight, gms.	161.05		
Diameter, in.	2.500	2.500	
Area, in.^{2}	4.909	4.909	
Height, in.	1.090	1.090	
Net decrease in height, in.		0.000	
Wet density, pcf	114.7	120.9	
Dry density, pcf	95.2	95.2	
Void ratio	0.7899	0.7899	
Saturation, %	70.6	93.1	

Test Readings for Specimen No. 1

Normal stress = 1.00 ksf
Strain rate, in./min. = 0.001
Strength calculations use strain adjusted areas
Fail. Stress = 1.278 ksf **at reading no.** 18

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
0	0.0000	4.760	0.0	0.000	0.6370
1	0.0005	4.390	-0.4	-0.011	0.6371
2	0.0030	3.806	-1.0	-0.028	0.6370
3	0.0050	6.284	1.5	0.045	0.6371
4	0.0065	9.605	4.8	0.143	0.6372
5	0.0080	12.165	7.4	0.218	0.6374
6	0.0095	15.031	10.3	0.303	0.6374
7	0.0115	18.316	13.6	0.400	0.6371
8	0.0125	20.673	15.9	0.470	0.6372
9	0.0140	23.537	18.8	0.555	0.6374
10	0.0160	26.532	21.8	0.644	0.6377
11	0.0180	28.886	24.1	0.714	0.6377
12	0.0210	31.653	26.9	0.797	0.6376
13	0.0240	34.322	29.6	0.878	0.6377

Test Readings for Specimen No. 1

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
14	0.0285	37.093	32.3	0.963	0.6376
15	0.0345	39.972	35.2	1.051	0.6378
16	0.0405	42.334	37.6	1.125	0.6377
17	0.0495	45.044	40.3	1.212	0.6375
18	0.0630	46.914	42.2	1.278	0.6367
19	0.0645	46.879	42.1	1.278	0.6367
20	0.0780	44.366	39.6	1.210	0.6364
21	0.0900	41.856	37.1	1.141	0.6361
22	0.1035	39.110	34.3	1.064	0.6360
23	0.1245	36.487	31.7	0.994	0.6358
24	0.1605	34.112	29.4	0.938	0.6357
25	0.2115	32.050	27.3	0.897	0.6355
26	0.2625	30.425	25.7	0.869	0.6351
27	0.3135	29.397	24.6	0.860	0.6347
28	0.3645	28.719	24.0	0.862	0.6346

Parameters for Specimen No. 2

Specimen Parameter	Initial	Consolidated	Final
Moisture content: Moist soil+tare, gms.	226.080		231.380
Moisture content: Dry soil+tare, gms.	198.200		199.280
Moisture content: Tare, gms.	61.710		65.590
Moisture, %	20.4	24.0	24.0
Moist specimen weight, gms.	163.24		
Diameter, in.	2.500	2.500	
Area, in.²	4.909	4.909	
Height, in.	1.110	1.110	
Net decrease in height, in.		0.000	
Wet density, pcf	114.1	117.5	
Dry density, pcf	94.8	94.8	
Void ratio	0.7983	0.7983	
Saturation, %	69.9	82.1	

Test Readings for Specimen No. 2

Normal stress = 2.00 ksf

Strain rate, in./min. = 0.001

Strength calculations use strain adjusted areas

Fail. Stress = 1.858 ksf **at reading no.** 19

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
0	0.0000	0.000	0.0	0.000	0.6370
1	0.0000	0.019	0.0	0.001	0.6565
2	0.0090	3.661	3.7	0.108	0.6571
3	0.0105	7.634	7.6	0.225	0.6571
4	0.0115	11.039	11.0	0.326	0.6572
5	0.0130	15.096	15.1	0.446	0.6573
6	0.0155	20.092	20.1	0.594	0.6572
7	0.0165	24.256	24.3	0.718	0.6572
8	0.0175	27.528	27.5	0.815	0.6573
9	0.0190	31.433	31.4	0.931	0.6574
10	0.0205	34.877	34.9	1.034	0.6576
11	0.0225	38.375	38.4	1.139	0.6576
12	0.0245	41.458	41.5	1.232	0.6576
13	0.0270	45.005	45.0	1.339	0.6578
14	0.0300	48.456	48.5	1.444	0.6577
15	0.0330	51.850	51.8	1.547	0.6576
16	0.0375	55.344	55.3	1.655	0.6578
17	0.0420	58.571	58.6	1.756	0.6576
18	0.0510	61.639	61.6	1.856	0.6575
19	0.0525	61.639	61.6	1.858	0.6575
20	0.0720	58.342	58.3	1.777	0.6569
21	0.0840	54.991	55.0	1.685	0.6568
22	0.1065	51.650	51.7	1.602	0.6563
23	0.1485	48.497	48.5	1.539	0.6560
24	0.1995	46.226	46.2	1.509	0.6559
25	0.2505	45.123	45.1	1.517	0.6558
26	0.3015	44.209	44.2	1.532	0.6557
27	0.3525	43.064	43.1	1.539	0.6558

Test Readings for Specimen No. 2

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
28	0.4036	42.214	42.2	1.557	0.6558
29	0.4546	41.269	41.3	1.573	0.6558

Parameters for Specimen No. 3

Specimen Parameter	Initial	Consolidated	Final
Moisture content: Moist soil+tare, gms.	226.080		226.840
Moisture content: Dry soil+tare, gms.	198.200		196.080
Moisture content: Tare, gms.	61.710		62.260
Moisture, %	20.4	23.0	23.0
Moist specimen weight, gms.	160.29		
Diameter, in.	2.500	2.500	
Area, in.²	4.909	4.909	
Height, in.	1.090	1.090	
Net decrease in height, in.		0.000	
Wet density, pcf	114.1	116.6	
Dry density, pcf	94.8	94.8	
Void ratio	0.7984	0.7984	
Saturation, %	69.8	78.6	

Test Readings for Specimen No. 3

Normal stress = 4.0 ksf

Strain rate, in./min. = 0.001

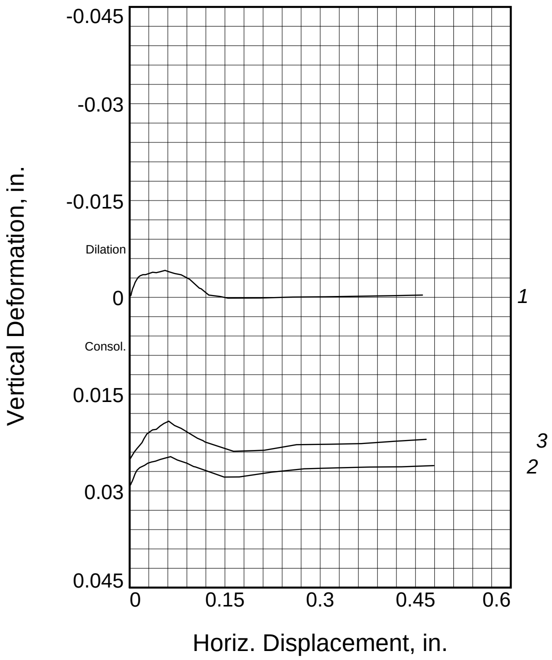
Strength calculations use strain adjusted areas

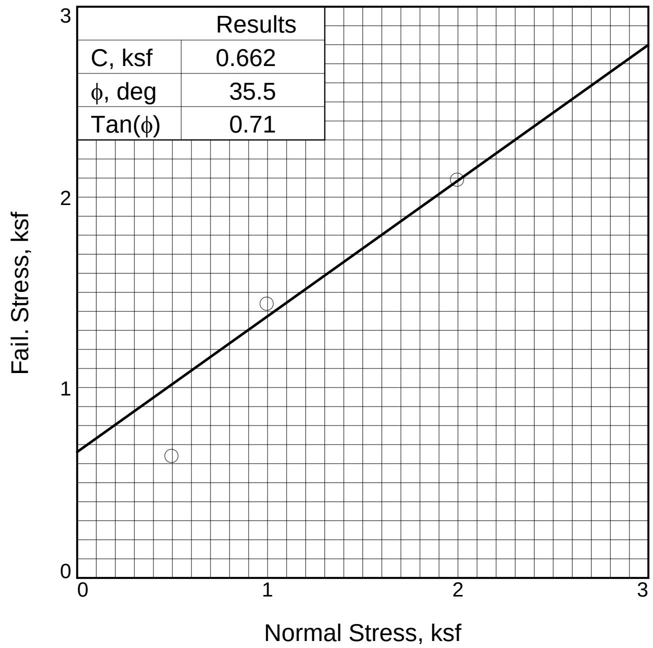
Fail. Stress = 2.868 ksf **at reading no.** 19

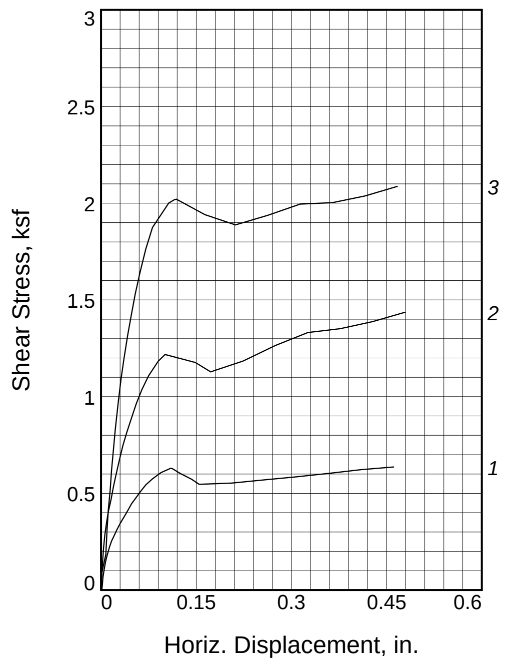
No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
0	0.0000	0.000	0.0	0.000	0.6370
1	0.0000	4.357	4.4	0.128	0.6343
2	0.0025	4.174	4.2	0.123	0.6343
3	0.0045	8.966	9.0	0.264	0.6342
4	0.0065	14.453	14.5	0.425	0.6341
5	0.0085	19.640	19.6	0.579	0.6339
6	0.0095	26.205	26.2	0.772	0.6338
7	0.0105	32.110	32.1	0.947	0.6337
8	0.0115	37.051	37.1	1.093	0.6337
9	0.0130	43.028	43.0	1.271	0.6342
10	0.0150	49.279	49.3	1.457	0.6346
11	0.0170	54.528	54.5	1.614	0.6346
12	0.0195	60.369	60.4	1.789	0.6350
13	0.0225	65.890	65.9	1.955	0.6357
14	0.0270	72.596	72.6	2.159	0.6358
15	0.0315	77.395	77.4	2.307	0.6360
16	0.0375	82.335	82.3	2.462	0.6364
17	0.0450	87.418	87.4	2.625	0.6372
18	0.0555	92.284	92.3	2.786	0.6377
19	0.0660	94.470	94.5	2.868	0.6378
20	0.0675	94.341	94.3	2.866	0.6379
21	0.0960	89.305	89.3	2.754	0.6379

Test Readings for Specimen No. 3

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
22	0.1245	84.280	84.3	2.640	0.6379
23	0.1755	79.628	79.6	2.565	0.6378
24	0.2265	76.760	76.8	2.545	0.6376
25	0.2775	74.086	74.1	2.530	0.6374
26	0.3285	72.025	72.0	2.536	0.6375







	Sample No.	1	2	3
Initial	Water Content, %	16.0	16.0	16.0
	Dry Density, pcf	100.9	100.7	100.1
	Saturation, %	64.6	64.3	63.2
	Void Ratio	0.6704	0.6736	0.6844
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.030	1.030	1.030
At Test	Water Content, %	22.9	21.2	22.1
	Dry Density, pcf	100.9	100.7	100.1
	Saturation, %	92.0	85.1	87.1
	Void Ratio	0.6704	0.6736	0.6844
	Diameter, in.	2.500	2.500	2.500
	Height, in.	1.030	1.030	1.030
Normal Stress, ksf		0.500	1.000	2.000
Fail. Stress, ksf		0.636	1.436	2.087
	Displacement, in.	0.461	0.479	0.467
Ult. Stress, ksf				
	Displacement, in.			
Strain rate, in./min.		0.001	0.001	0.001

Sample Type: Laboratory Molded

Description: Silt (ML)

Assumed Specific Gravity= 2.70

Remarks:

Client: Arcadis, Inc.

Project: Border Wall Geotechnical Services

Location: TP-11

Depth: 3 ft.

Proj. No.: 88215034 **Date Sampled:**

DIRECT SHEAR TEST REPORT

Terracon Consultants, Inc.

Houston, TX

DIRECT SHEAR TEST

5/25/2021

Date:
Client: Arcadis, Inc.
Project: Border Wall Geotechnical Services
Project No.: 88215034
Location: TP-11
Depth: 3 ft.
Description: Silt (ML)
Remarks:
Type of Sample:
Assumed Specific Gravity= 2.70 **LL= 31** **PL= 23** **PI= 8**

Parameters for Specimen No. 1

Specimen Parameter	Initial	Consolidated	Final
Moisture content: Moist soil+tare, gms.	219.330		221.500
Moisture content: Dry soil+tare, gms.	197.560		191.580
Moisture content: Tare, gms.	61.780		60.640
Moisture, %	16.0	22.9	22.9
Moist specimen weight, gms.	155.39		
Diameter, in.	2.500	2.500	
Area, in.^{2}	4.909	4.909	
Height, in.	1.030	1.030	
Net decrease in height, in.		0.000	
Wet density, pcf	117.1	124.0	
Dry density, pcf	100.9	100.9	
Void ratio	0.6704	0.6704	
Saturation, %	64.6	92.0	

Test Readings for Specimen No. 1

Normal stress = 0.50 ksf
Strain rate, in./min. = 0.001
Strength calculations use strain adjusted areas
Fail. Stress = 0.636 ksf **at reading no.** 32

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
0	0.0000	-2.502	0.0	0.000	0.7046
1	0.0005	-2.520	0.0	-0.001	0.7048
2	0.0010	-2.538	0.0	-0.001	0.7048
3	0.0020	-1.580	0.9	0.027	0.7049
4	0.0030	-0.335	2.2	0.064	0.7053
5	0.0045	0.915	3.4	0.100	0.7059
6	0.0065	2.113	4.6	0.136	0.7063
7	0.0085	3.201	5.7	0.168	0.7069
8	0.0110	4.177	6.7	0.197	0.7073
9	0.0135	5.082	7.6	0.224	0.7077
10	0.0165	6.052	8.6	0.253	0.7079
11	0.0210	7.071	9.6	0.284	0.7081
12	0.0250	7.987	10.5	0.312	0.7081
13	0.0300	9.044	11.5	0.344	0.7083

Test Readings for Specimen No. 1

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
14	0.0360	10.127	12.6	0.377	0.7085
15	0.0420	11.198	13.7	0.411	0.7084
16	0.0480	12.348	14.9	0.447	0.7085
17	0.0556	13.407	15.9	0.480	0.7088
18	0.0631	14.437	16.9	0.513	0.7085
19	0.0706	15.373	17.9	0.544	0.7083
20	0.0811	16.287	18.8	0.575	0.7081
21	0.0946	17.218	19.7	0.608	0.7074
22	0.1096	17.754	20.3	0.629	0.7060
23	0.1126	17.669	20.2	0.628	0.7059
24	0.1246	16.759	19.3	0.603	0.7049
25	0.1426	15.606	18.1	0.573	0.7047
26	0.1546	14.661	17.2	0.546	0.7045
27	0.2057	14.384	16.9	0.553	0.7045
28	0.2567	14.378	16.9	0.569	0.7046
29	0.3077	14.344	16.8	0.586	0.7047
30	0.3588	14.325	16.8	0.604	0.7048
31	0.4098	14.309	16.8	0.622	0.7048
32	0.4608	14.123	16.6	0.636	0.7049

Parameters for Specimen No. 2

Specimen Parameter	Initial	Consolidated	Final
Moisture content: Moist soil+tare, gms.	219.330		216.260
Moisture content: Dry soil+tare, gms.	197.560		189.000
Moisture content: Tare, gms.	61.780		60.640
Moisture, %	16.0	21.2	21.2
Moist specimen weight, gms.	155.10		
Diameter, in.	2.500	2.500	
Area, in.²	4.909	4.909	
Height, in.	1.030	1.030	
Net decrease in height, in.		0.000	
Wet density, pcf	116.9	122.1	
Dry density, pcf	100.7	100.7	
Void ratio	0.6736	0.6736	
Saturation, %	64.3	85.1	

Test Readings for Specimen No. 2

Normal stress = 1.00 ksf

Strain rate, in./min. = 0.001

Strength calculations use strain adjusted areas

Fail. Stress = 1.436 ksf **at reading no.** 28

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
0	0.0000	0.000	0.0	0.000	0.7046
1	0.0000	0.118	0.1	0.003	0.6755
2	0.0015	3.065	3.1	0.090	0.6755
3	0.0025	5.099	5.1	0.150	0.6758
4	0.0040	7.433	7.4	0.218	0.6761
5	0.0060	9.779	9.8	0.288	0.6766
6	0.0085	11.787	11.8	0.347	0.6772
7	0.0115	13.767	13.8	0.406	0.6778
8	0.0160	15.938	15.9	0.471	0.6782
9	0.0195	18.033	18.0	0.534	0.6784
10	0.0235	20.046	20.0	0.595	0.6785
11	0.0285	22.409	22.4	0.667	0.6789
12	0.0345	25.018	25.0	0.747	0.6790
13	0.0405	27.204	27.2	0.815	0.6792
14	0.0480	29.614	29.6	0.891	0.6794
15	0.0555	31.954	32.0	0.965	0.6796
16	0.0646	34.186	34.2	1.037	0.6799
17	0.0751	36.326	36.3	1.108	0.6794
18	0.0901	38.478	38.5	1.183	0.6789
19	0.1006	39.370	39.4	1.217	0.6784
20	0.1036	39.251	39.3	1.216	0.6783
21	0.1486	37.069	37.1	1.176	0.6767
22	0.1726	35.089	35.1	1.128	0.6767
23	0.2237	35.764	35.8	1.184	0.6775
24	0.2747	37.080	37.1	1.264	0.6780
25	0.3257	37.870	37.9	1.331	0.6782
26	0.3768	37.259	37.3	1.351	0.6783
27	0.4278	37.051	37.1	1.388	0.6783

Test Readings for Specimen No. 2

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
28	0.4788	37.086	37.1	1.436	0.6785

Parameters for Specimen No. 3

Specimen Parameter	Initial	Consolidated	Final
Moisture content: Moist soil+tare, gms.	219.330		219.370
Moisture content: Dry soil+tare, gms.	197.560		190.930
Moisture content: Tare, gms.	61.780		62.070
Moisture, %	16.0	22.1	22.1
Moist specimen weight, gms.	154.10		
Diameter, in.	2.500	2.500	
Area, in.^{2}	4.909	4.909	
Height, in.	1.030	1.030	
Net decrease in height, in.		0.000	
Wet density, pcf	116.1	122.2	
Dry density, pcf	100.1	100.1	
Void ratio	0.6844	0.6844	
Saturation, %	63.2	87.1	

Test Readings for Specimen No. 3

Normal stress = 2.0 ksf

Strain rate, in./min. = 0.001

Strength calculations use strain adjusted areas

Fail. Stress = 2.087 ksf **at reading no.** 28

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
0	0.0000	0.000	0.0	0.000	0.7046
1	0.0000	2.822	2.8	0.083	0.6786
2	0.0005	2.654	2.7	0.078	0.6795
3	0.0075	6.255	6.3	0.184	0.6807
4	0.0095	10.543	10.5	0.311	0.6809
5	0.0115	14.141	14.1	0.417	0.6811
6	0.0145	17.589	17.6	0.520	0.6815
7	0.0165	20.972	21.0	0.620	0.6817
8	0.0195	24.505	24.5	0.726	0.6821
9	0.0225	28.035	28.0	0.832	0.6826
10	0.0270	32.616	32.6	0.970	0.6834
11	0.0315	36.599	36.6	1.091	0.6837
12	0.0360	40.038	40.0	1.196	0.6840
13	0.0420	43.971	44.0	1.318	0.6841
14	0.0480	47.528	47.5	1.429	0.6846
15	0.0540	50.796	50.8	1.532	0.6850
16	0.0616	54.300	54.3	1.644	0.6854
17	0.0706	57.978	58.0	1.764	0.6847
18	0.0811	61.276	61.3	1.875	0.6843
19	0.1066	64.514	64.5	2.001	0.6827
20	0.1156	64.746	64.7	2.018	0.6824
21	0.1186	64.720	64.7	2.021	0.6822
22	0.1636	60.664	60.7	1.941	0.6807

Test Readings for Specimen No. 3

No.	Horizontal Def. Dial in.	Load Dial	Load lbs.	Shear Stress ksf	Vertical Def. Dial in.
23	0.2117	57.420	57.4	1.888	0.6809
24	0.2627	57.237	57.2	1.938	0.6817
25	0.3137	57.191	57.2	1.996	0.6818
26	0.3648	55.648	55.6	2.003	0.6819
27	0.4158	54.810	54.8	2.037	0.6823
28	0.4668	54.321	54.3	2.087	0.6826

CHEMICAL LABORATORY TEST REPORT

Project Number: 88215034
Service Date: 05/07/21
Report Date: 05/11/21

Terracon
10400 State Highway 191
Midland, Texas 79707
432-684-9600

Client

ARCADIS US, Inc.
3850 N Causeway Blvd, Suite 990
Metairie, LA 70002

Project

Border Wall Geotechnical Services
1.75 Miles SW of Madero
Mission, TX

Sample Location	TP-1	TP-5	TP-8	TP-12
Sample Depth (ft.)	3	3	3	3
pH Analysis, ASTM - G51-18	7.30	7.50	7.50	7.60
Water Soluble Sulfate (SO4), ASTM C 1580 (mg/kg)	524	11	349	197
Sulfides, ASTM - D4658-15, (mg/kg)	nil	nil	nil	nil
Chlorides, ASTM D 512 , (mg/kg)	119	14	75	19
RedOx, ASTM D-1498, (mV)	+338	+347	+335	+337
Total Salts, ASTM D1125-14, (mg/kg)	1,605	732	1,455	826
Resistivity, ASTM G187, (ohm-cm)	1,239	4,337	1,342	2,994

Analyzed By: Nohelia Monasterios

Nohelia Monasterios
Field Engineer

CRUMB TEST (ASTM D6572)

Project No.: 88215034 Project Name: Border Wall Geotechnical Services Location: Mission, Texas

Boring No.: TP-3 Sample No.: ______ Depth: 3 [X] ft [ ] m

Visual Classification: Silty Sandy (SM) Color: ______

Moisture Content of Sample:	[ ] as-received	[ ] in situ	[ ] air-dried	
Tare Number	Wet Mass + Tare (g)	Dry Mass + Tare (g)	Tare Mass (g)	Water Content (%)

	Specimen 1	Specimen 2	Specimen 3
Specimen Identification:	TP-3		
Spec. Container Identification:			
Method:	[ ] A (Natural) [X] B (Remolded)	[ ] A (Natural) [ ] B (Remolded)	[ ] A (Natural) [ ] B (Remolded)
Water Type:	[X] Distilled [ ] Type IV	[ ] Distilled [ ] Type IV	[ ] Distilled [ ] Type IV
Initial Water Temp. (°C):			
Start Time (hh:mm:ss):			

Target Reading	Time Taken	Grade	Temp. (°C)	Target Reading	Time Taken	Grade	Temp. (°C)	Target Reading	Time Taken	Grade	Temp. (°C)
2 min ± 15 s		4		2 min ± 15 s				2 min ± 15 s			
1 h ± 8 min		4		1 h ± 8 min				1 h ± 8 min			
6 h ± 45 min		4		6 h ± 45 min				6 h ± 45 min			

	Specimen 1	Specimen 2	Specimen 3
Dispersive Classification:	Highly Dispersive		
Additional water added to remold the specimen (Method B):	[ ] Y [ ] N	[ ] Y [ ] N	[ ] Y [ ] N

Remarks: ______

Prepared By: ______ Date: ______

Tested By: ______ Date: ______

Input By: ______ Date: ______

Reviewed By: ______ Date: ______

FIG. X1.1 Example of a Crumb Test Data Sheet

CRUMB TEST (ASTM D6572)

Project No.: 88215034 Project Name: Border Wall Geotechnical Services Location: ____

Boring No.: TP-5 Sample No.: ____ Depth: 3 [X] ft [ ] m

Visual Classification: Silty Sand (SM) Color: ____

Moisture Content of Sample:	[ ] as-received	[ ] in situ	[ ] air-dried	
Tare Number	Wet Mass + Tare (g)	Dry Mass + Tare (g)	Tare Mass (g)	Water Content (%)

Specimen Identification:	TP-5	Specimen Identification:		Specimen Identification:	
Spec. Container Identification:		Spec. Container Identification:		Spec. Container Identification:	
Method:	[ ] A (Natural) [X] B (Remolded)	Method:	[ ] A (Natural) [ ] B (Remolded)	Method:	[ ] A (Natural) [ ] B (Remolded)
Water Type:	[X] Distilled [ ] Type IV	Water Type:	[ ] Distilled [ ] Type IV	Water Type:	[ ] Distilled [ ] Type IV
Initial Water Temp. (°C): ____		Initial Water Temp. (°C): ____		Initial Water Temp. (°C): ____	
Start Time (hh:mm:ss): ____		Start Time (hh:mm:ss): ____		Start Time (hh:mm:ss): ____	

Target Reading	Time Taken	Grade	Temp. (°C)	Target Reading	Time Taken	Grade	Temp. (°C)	Target Reading	Time Taken	Grade	Temp. (°C)
2 min ± 15 s		3		2 min ± 15 s				2 min ± 15 s			
1 h ± 8 min		4		1 h ± 8 min				1 h ± 8 min			
6 h ± 45 min		4		6 h ± 45 min				6 h ± 45 min			

Dispersive Classification:	Highly Dispersive	Dispersive Classification:		Dispersive Classification:	
Additional water added to remold the specimen (Method B): [ ] Y [ ] N		Additional water added to remold the specimen (Method B): [ ] Y [ ] N		Additional water added to remold the specimen (Method B): [ ] Y [ ] N	

Remarks: ____

Prepared By: ____ Date: ____ Tested By: ____ Date: ____ Input By: ____ Date: ____ Reviewed By: ____ Date: ____

FIG. X1.1 Example of a Crumb Test Data Sheet

CRUMB TEST (ASTM D6572)

Project No.: 88215034 Project Name: Border Wall Geotechnical Services Location:

Boring No.: TP-8 Sample No.: Depth: 3 [X] ft [ ] m

Visual Classification: Lean Clay (CL) Color:

Moisture Content of Sample:	[ ] as-received	[ ] in situ	[ ] air-dried	
Tare Number	Wet Mass + Tare (g)	Dry Mass + Tare (g)	Tare Mass (g)	Water Content (%)

Specimen Identification: TP-8	Specimen Identification:	Specimen Identification:
Spec. Container Identification:	Spec. Container Identification:	Spec. Container Identification:
Method: [ ] A (Natural) [X] B (Remolded)	Method: [ ] A (Natural) [ ] B (Remolded)	Method: [ ] A (Natural) [ ] B (Remolded)
Water Type: [X] Distilled [ ] Type IV	Water Type: [ ] Distilled [ ] Type IV	Water Type: [ ] Distilled [ ] Type IV
Initial Water Temp. (°C): ________	Initial Water Temp. (°C): ________	Initial Water Temp. (°C): ________
Start Time (hh:mm:ss): ________	Start Time (hh:mm:ss): ________	Start Time (hh:mm:ss): ________

Target Reading	Time Taken	Grade	Temp. (°C)	Target Reading	Time Taken	Grade	Temp. (°C)	Target Reading	Time Taken	Grade	Temp. (°C)
2 min ± 15 s		2		2 min ± 15 s				2 min ± 15 s			
1 h ± 8 min		2		1 h ± 8 min				1 h ± 8 min			
6 h ± 45 min		2		6 h ± 45 min				6 h ± 45 min			

Dispersive Classification: Intermediate	Dispersive Classification:	Dispersive Classification:
Additional water added to remold the specimen (Method B): [ ] Y [ ] N	Additional water added to remold the specimen (Method B): [ ] Y [ ] N	Additional water added to remold the specimen (Method B): [ ] Y [ ] N

Remarks: ________

Prepared By: ________ Date: ________
Tested By: ________ Date: ________
Input By: ________ Date: ________
Reviewed By: ________ Date: ________

FIG. X1.1 Example of a Crumb Test Data Sheet

CRUMB TEST (ASTM D6572)

Project No.: 88215034 Project Name: Border Wall Geotechnical Services Location: ____________

Boring No.: TP-10 Sample No.: ____________ Depth: 3 [X] ft [ ] m

Visual Classification: Lean Clay (CL) Color: ____________

Moisture Content of Sample:	[ ] as-received	[ ] in situ	[ ] air-dried	
Tare Number	Wet Mass + Tare (g)	Dry Mass + Tare (g)	Tare Mass (g)	Water Content (%)

Specimen Identification:	TP-10	Specimen Identification:		Specimen Identification:	
Spec. Container Identification:		Spec. Container Identification:		Spec. Container Identification:	
Method:	[ ] A (Natural) [X] B (Remolded)	Method:	[ ] A (Natural) [ ] B (Remolded)	Method:	[ ] A (Natural) [ ] B (Remolded)
Water Type:	[X] Distilled [ ] Type IV	Water Type:	[ ] Distilled [ ] Type IV	Water Type:	[ ] Distilled [ ] Type IV
Initial Water Temp. (°C): ________		Initial Water Temp. (°C): ________		Initial Water Temp. (°C): ________	
Start Time (hh:mm:ss): ________		Start Time (hh:mm:ss): ________		Start Time (hh:mm:ss): ________	

Target Reading	Time Taken	Grade	Temp. (°C)	Target Reading	Time Taken	Grade	Temp. (°C)	Target Reading	Time Taken	Grade	Temp. (°C)
2 min ± 15 s		2		2 min ± 15 s				2 min ± 15 s			
1 h ± 8 min		2		1 h ± 8 min				1 h ± 8 min			
6 h ± 45 min		2		6 h ± 45 min				6 h ± 45 min			

Dispersive Classification:	Intermediate	Dispersive Classification:		Dispersive Classification:	
Additional water added to remold the specimen (Method B): [ ] Y [ ] N		Additional water added to remold the specimen (Method B): [ ] Y [ ] N		Additional water added to remold the specimen (Method B): [ ] Y [ ] N	

Remarks: ____________

Prepared By: ________ Tested By: ________ Input By: ________ Reviewed By: ________

Date: ________ Date: ________ Date: ________ Date: ________

FIG. X1.1 Example of a Crumb Test Data Sheet

Terracon GeoReport

Project Name: Border Wall Geotechnical Services
Project No.: 88215034
Sample ID: TP-2
Compaction Charact.: Remolded **Max Dry Density, pcf:** 114.0 **Dry Density (95%), pcf:** 108.5
Water Content, %: 14.1
Distilled Water Added: yes
Sample Description Sandy Lean Clay (CL)
Flow Started On **Trial** **Compaction, %:** 95.1

Time	Head	Flow		Rate	Turbidity From Side						Clear From Top	Remarks
min.	in.	ml	sec	ml/sec	Very Dark	Dark	Mod. Dark	Slight Dark	Barely Visible	Clear		
1	2	10	9	1.11		X						
2	2	10	9	1.11		X						
3	2											
4	2											
5	2	25	37	0.67		X						
6	2											
7	2											
8	2											
9	2											
10	2	25	40	0.62		X						ND4
2	7	25										
3	7	25										
4	7	25										
5	7	25										
2	15	50										
3	15	50										
4	15	50										
5	15	50										
2	40	50										
3	40	100										
4	40	100										
5	40	100										

Dispersive Clasification: ND4 - Moderately Dispersive

Date: 5/24/2021

By: SR

Review:

Terracon GeoReport

Project Name: Border Wall Geotechnical Services
Project No.: 88215034
Sample ID: TP-5
Compaction Charact.: Remolded **Max Dry Density, pcf:** 105.1 **Dry Density (95%), pcf:** 101.3
Water Content, %: 16.3
Distilled Water Added: yes
Sample Description: Silty Clayey Sand (SC-SM)
Flow Started On **Trial** **Compaction, %:** 96.2

Time	Head	Flow		Rate	Turbidity From Side						Clear From Top	Remarks
min.	in.	ml	sec	ml/sec	Very Dark	Dark	Mod. Dark	Slight Dark	Barely Visible	Clear		
1	2	10	8	1.25	X							
2	2	10	12	0.83		X						
3	2											
4	2											
5	2	25	50	0.5		X						
6	2											
7	2											
8	2											
9	2											
10	2	25	43	0.58		X						ND4
2	7	25										
3	7	25										
4	7	25										
5	7	25										
2	15	50										
3	15	50										
4	15	50										
5	15	50										
2	40	50										
3	40	100										
4	40	100										
5	40	100										

Dispersive Clasification: ND4 - Moderately Dispersive

Date: 5/24/2021

By: SR

Review:

Project Name: Border Wall Geotechnical Services
Project No.: 88215034
Sample ID: TP-9
Compaction Charact.: Remolded **Max Dry Density, pcf:** 99.3 **Dry Density (95%), pcf:** 95.1
Water Content, %:
Distilled Water Added: yes
Sample Description: Lean Clay (CL)
Flow Started On **Trial** **Compaction, %:** 95.8

Time	Head	Flow		Rate	Turbidity From Side						Clear From Top	Remarks
min.	in.	ml	sec	ml/sec	Very Dark	Dark	Mod. Dark	Slight Dark	Barely Visible	Clear		
1	2	10	43	0.23				X				
2	2	10	49	0.20					X			
3	2											
4	2											
5	2	25	128	0.19					X			
6	2											
7	2											
8	2											
9	2											
10	2	25	130	0.19						X		
2	7	25	102	0.24					X			
3	7	25	115	0.22						X		
4	7	25	116	0.21						X		
5	7	25										
2	15	50	116	0.43						X		
3	15	50	108	0.46						X		
4	15	50	109	0.46						X		
5	15	50										
2	40	50	52	0.96						X		
3	40	100	119	0.84						X		
4	40	100	120	0.83						X		
5	40	100										ND1

Dispersive Clasification: ND1 - Non-Dispersive

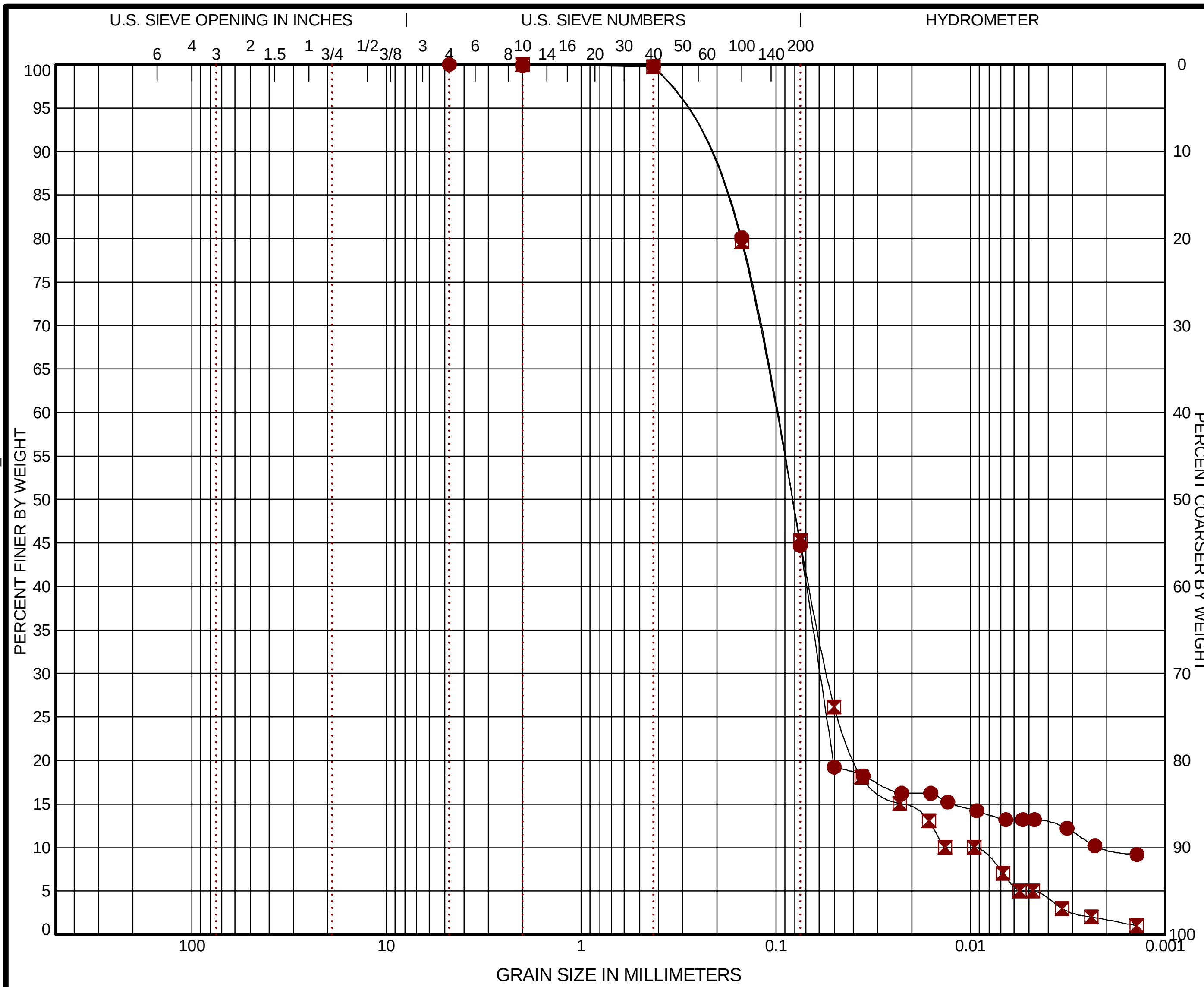
Date: 5/24/2021

By: SR

Review:

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**TP-1**	**1**	**0.0**	**0.1**	**55.2**	**34.8**		**9.9**	
⊠	**TP-1**	**3 - 5**	**0.0**	**0.0**	**54.7**	**43.6**		**1.7**	

	GRAIN SIZE		
	●	⊠	
D_{60}	**0.101**	**0.101**	
D_{30}	**0.059**	**0.054**	
D_{10}	**0.002**	**0.01**	
	COEFFICIENTS		
C_C	**16.77**	**3.07**	
C_U	**48.67**	**10.58**	

●		⊠			
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#4	**100.0**	**#10**	**100.0**		
#10	**99.92**	**#40**	**99.74**		
#40	**99.82**	**#100**	**79.63**		
#100	**80.08**	**#200**	**45.28**		
#200	**44.7**				

SOIL DESCRIPTION

●

⊠

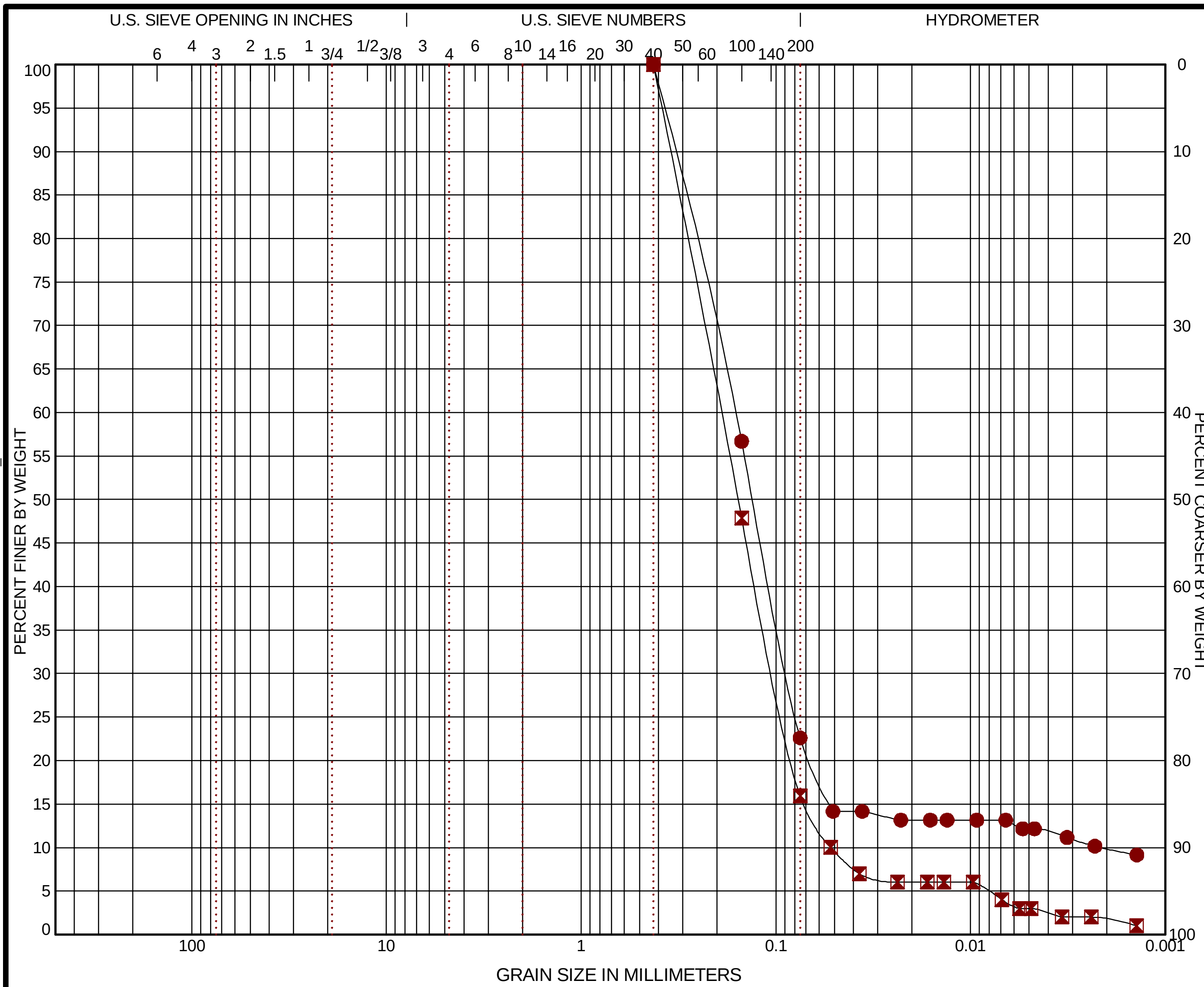
REMARKS

(5.026/13.219) x 100 = 38.02
Intermediate

PROJECT: Border Wall Geotechnical Services	Terracon 1506 Mid Cities Dr Pharr, TX	PROJECT NUMBER: 88215034
SITE: 1.75 Miles SW of Madero ,Texas Mission, TX		CLIENT: ARCADIS US, Inc. Metairie, LA
		EXHIBIT: B-1

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**TP-4**	**1**	**0.0**	**0.0**	**77.4**	**12.7**		**9.9**	
⊠	**TP-4**	**3 - 5**	**0.0**	**0.0**	**84.1**	**14.3**		**1.7**	

	GRAIN SIZE		
	●	⊠	
D_{60}	**0.162**	**0.191**	
D_{30}	**0.087**	**0.102**	
D_{10}	**0.002**	**0.052**	
	COEFFICIENTS		
C_C	**21.97**	**1.04**	
C_U	**76.27**	**3.65**	

●		⊠			
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#40	**100.0**	**#40**	**100.0**		
#100	**56.69**	**#100**	**47.87**		
#200	**22.6**	**#200**	**15.94**		

SOIL DESCRIPTION

●

⊠

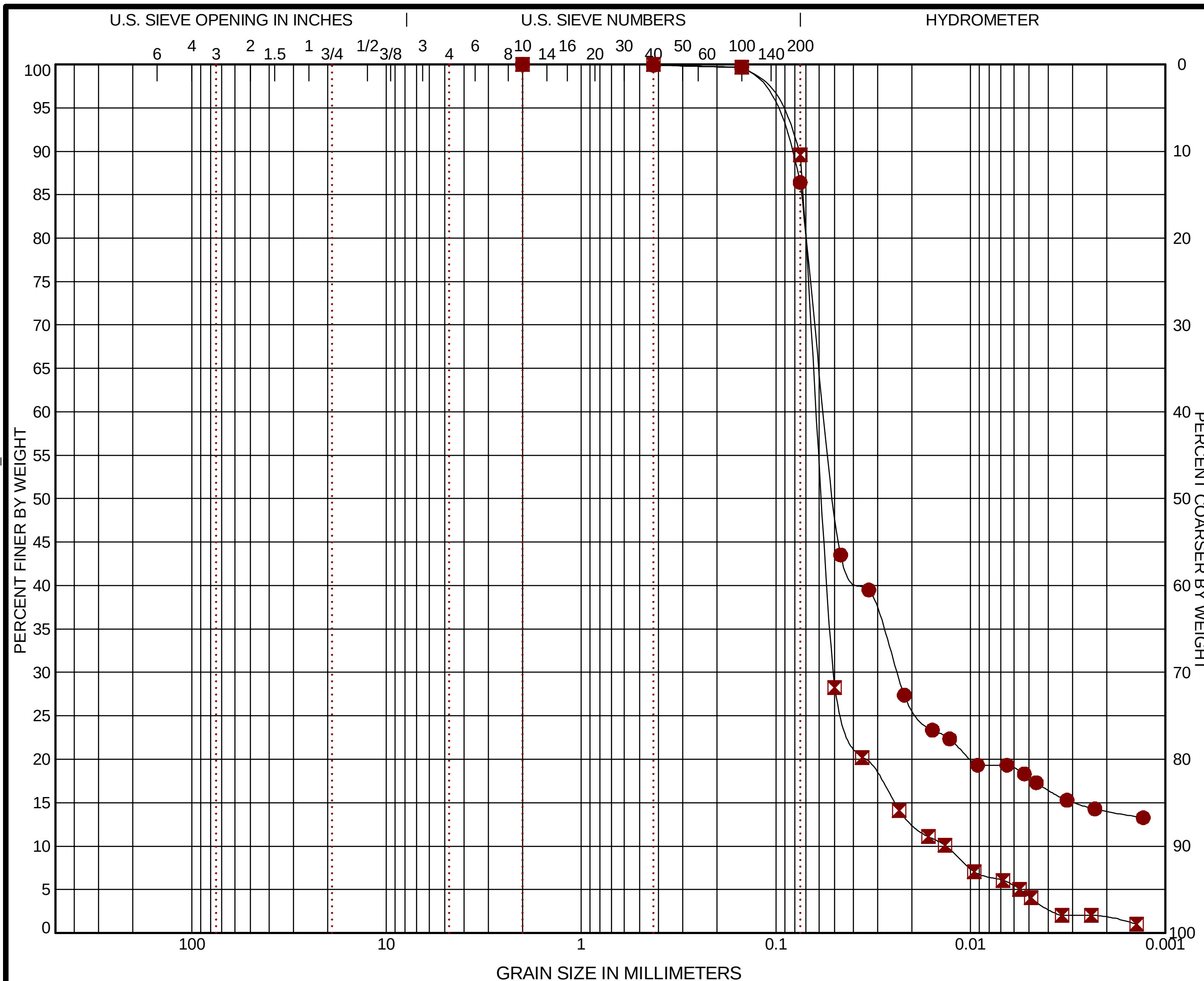
REMARKS

(3.00/12.156) x 100 = 25.68
Non-dispersive

PROJECT: Border Wall Geotechnical Services	Terracon 1506 Mid Cities Dr Pharr, TX	PROJECT NUMBER: 88215034
SITE: 1.75 Miles SW of Madero ,Texas Mission, TX		CLIENT: ARCADIS US, Inc. Metairie, LA
		EXHIBIT: B-1

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**TP-11**	**1**	**0.0**	**0.0**	**13.6**	**72.4**		**14.0**	
⊠	**TP-11**	**3 - 5**	**0.0**	**0.0**	**10.4**	**87.9**		**1.7**	

	GRAIN SIZE		
	●	⊠	
D_{60}	**0.056**	**0.062**	
D_{30}	**0.024**	**0.05**	
D_{10}		**0.013**	
	COEFFICIENTS		
C_C		**3.08**	
C_U		**4.60**	

●		⊠			
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#10	**100.0**	**#10**	**100.0**		
#40	**99.9**	**#40**	**99.98**		
#100	**99.66**	**#100**	**99.68**		
#200	**86.41**	**#200**	**89.58**		

SOIL DESCRIPTION

●

⊠

REMARKS

(4.03/17.8) x 100 = 22.64
Non-dispersive

PROJECT: Border Wall Geotechnical Services

SITE: 1.75 Miles SW of Madero ,Texas
Mission, TX

Terracon
1506 Mid Cities Dr
Pharr, TX

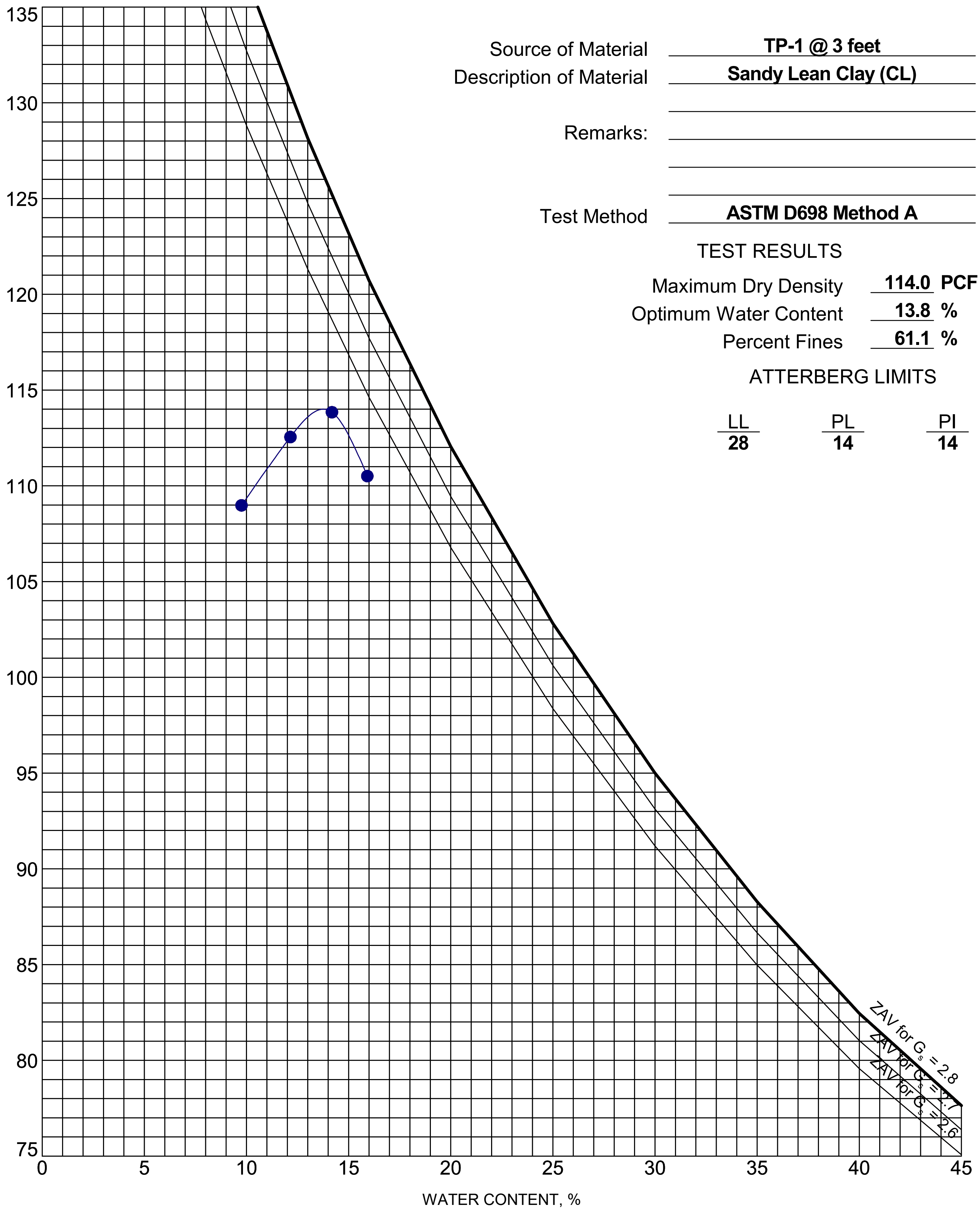
PROJECT NUMBER: 88215034

CLIENT: ARCADIS US, Inc.
Metairie, LA

EXHIBIT: B-1

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material **TP-1 @ 3 feet**

Description of Material **Sandy Lean Clay (CL)**

Remarks:

Test Method **ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density **114.0 PCF**

Optimum Water Content **13.8 %**

Percent Fines **61.1 %**

ATTERBERG LIMITS

LL	PL	PI
28	**14**	**14**

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. COMPACTION - V2 88215034 BORDER WALL GEOTE.GPJ TERRACON_DATATEMPLATE.GDT 6/25/21

PROJECT: Border Wall Geotechnical Services

SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

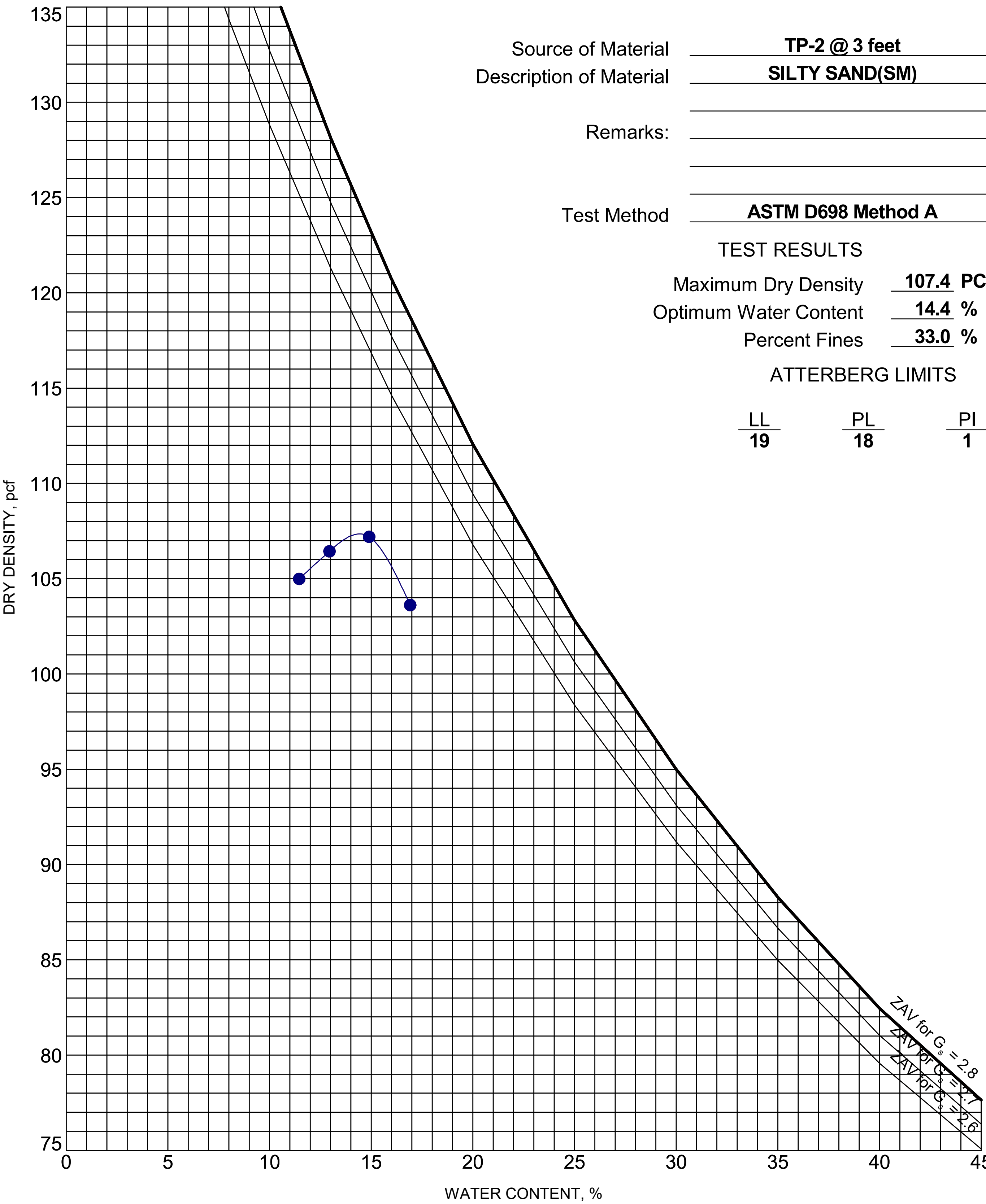
Terracon
1506 Mid Cities Dr
Pharr, TX

PROJECT NUMBER: 88215034

CLIENT: ARCADIS US, Inc.
Metairie, LA

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material **TP-2 @ 3 feet**

Description of Material **SILTY SAND(SM)**

Remarks:

Test Method **ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density **107.4** **PCF**

Optimum Water Content **14.4** **%**

Percent Fines **33.0** **%**

ATTERBERG LIMITS

LL	PL	PI
19	**18**	**1**

PROJECT: Border Wall Geotechnical Services

SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

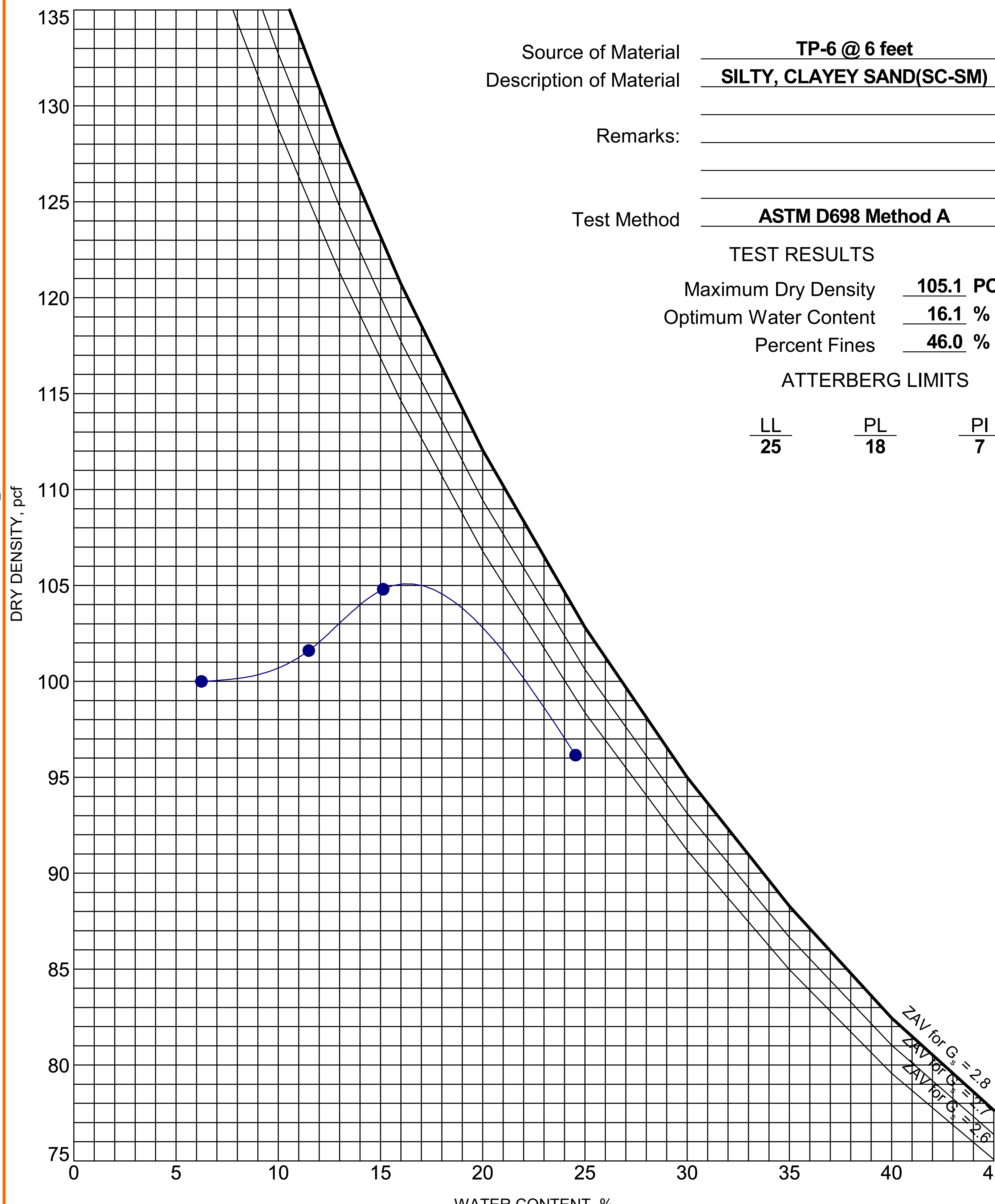
Terracon
1506 Mid Cities Dr
Pharr, TX

PROJECT NUMBER: 88215034

CLIENT: ARCADIS US, Inc.
Metairie, LA

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material **TP-6 @ 6 feet**

Description of Material **SILTY, CLAYEY SAND(SC-SM)**

Remarks:

Test Method **ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density **105.1 PCF**

Optimum Water Content **16.1 %**

Percent Fines **46.0 %**

ATTERBERG LIMITS

LL	PL	PI
25	**18**	**7**

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. COMPACTION - V2 88215034 BORDER WALL GEOTE.GPJ TERRACON_DATATEMPLATE.GDT 6/25/21

PROJECT: Border Wall Geotechnical Services

PROJECT NUMBER: 88215034

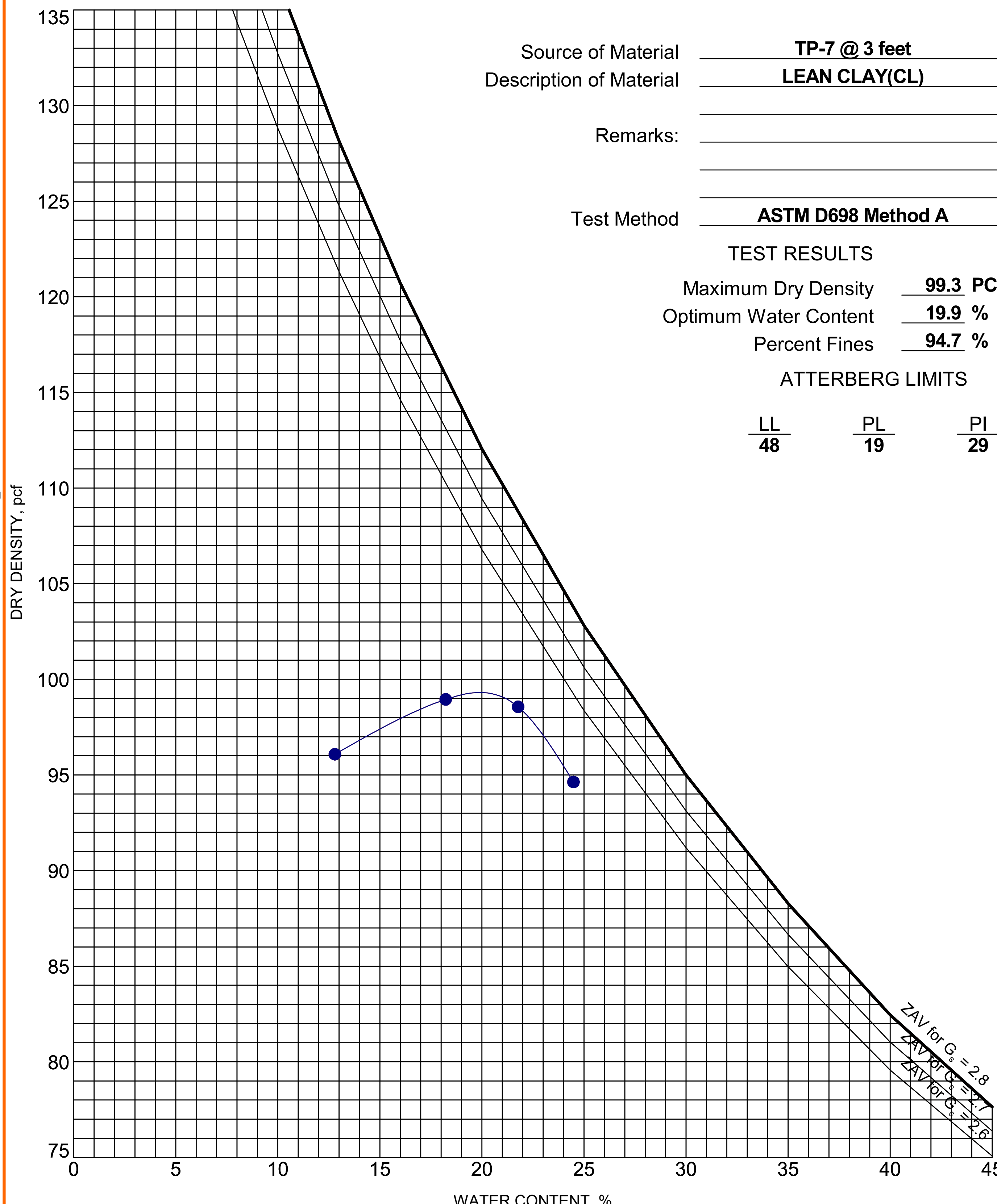
SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

Terracon
1506 Mid Cities Dr
Pharr, TX

CLIENT: ARCADIS US, Inc.
Metairie, LA

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material	**TP-7 @ 3 feet**
Description of Material	**LEAN CLAY(CL)**
Remarks:	
Test Method	**ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density	**99.3 PCF**
Optimum Water Content	**19.9 %**
Percent Fines	**94.7 %**

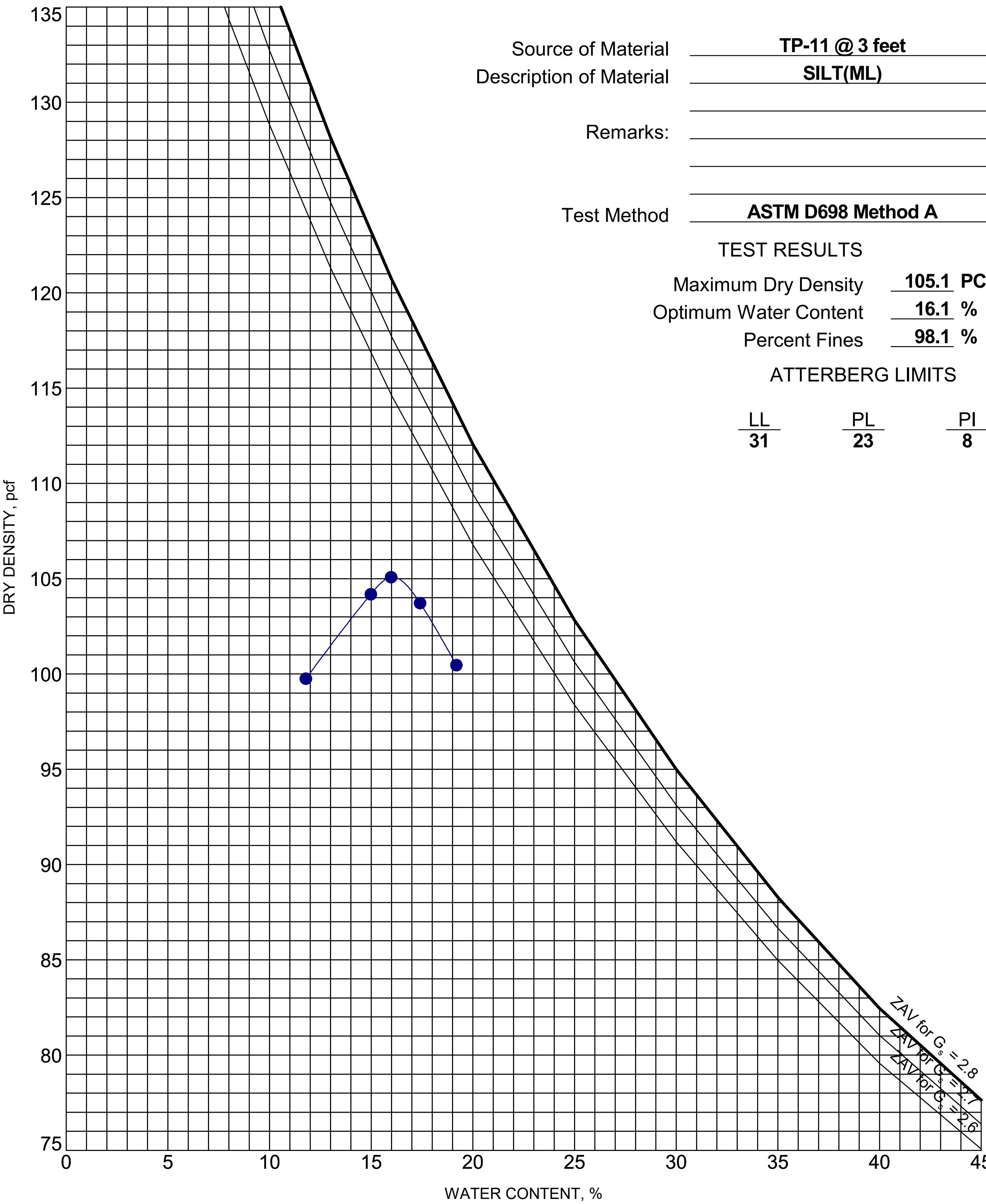
ATTERBERG LIMITS

LL	PL	PI
48	**19**	**29**

PROJECT: Border Wall Geotechnical Services	Terracon	PROJECT NUMBER: 88215034
SITE: 1.75 Miles SW of Madero,Texas Mission, Texas	1506 Mid Cities Dr Pharr,	CLIENT: ARCADIS US, Inc. Metairie, LA

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material **TP-11 @ 3 feet**

Description of Material **SILT(ML)**

Remarks:

Test Method **ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density **105.1 PCF**

Optimum Water Content **16.1 %**

Percent Fines **98.1 %**

ATTERBERG LIMITS

LL	PL	PI
31	**23**	**8**

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. COMPACTION - V2 88215034 BORDER WALL GEOTE.GPJ TERRACON_DATATEMPLATE.GDT 6/25/21

PROJECT: Border Wall Geotechnical Services

SITE: 1.75 Miles SW of Madero,Texas
Mission, Texas

Terracon
1506 Mid Cities Dr
Pharr, TX

PROJECT NUMBER: 88215034

CLIENT: ARCADIS US, Inc.
Metairie, LA

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557

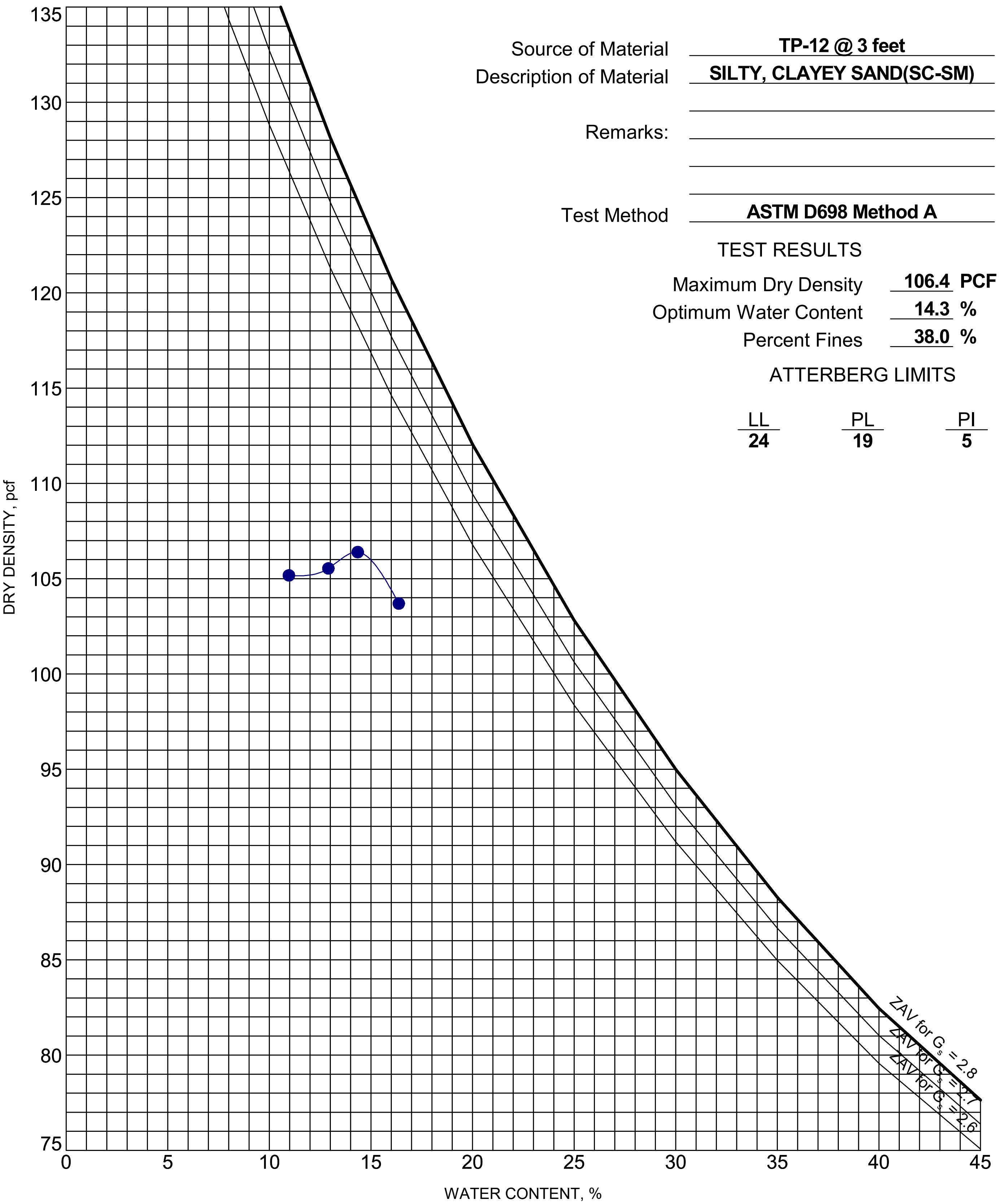
Source of Material	**TP-12 @ 3 feet**
Description of Material	**SILTY, CLAYEY SAND(SC-SM)**
Remarks:	
Test Method	**ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density	**106.4**	**PCF**
Optimum Water Content	**14.3**	**%**
Percent Fines	**38.0**	**%**

ATTERBERG LIMITS

LL	PL	PI
24	**19**	**5**



LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. COMPACTION - V2 88215034 BORDER WALL GEOTE.GPJ TERRACON_DATATEMPLATE.GDT 6/25/21

PROJECT: Border Wall Geotechnical Services	Terracon	PROJECT NUMBER: 88215034
SITE: 1.75 Miles SW of Madero,Texas Mission, Texas	1506 Mid Cities Dr Pharr, TX	CLIENT: ARCADIS US, Inc. Metairie, LA

Arcadis U.S., Inc.
1025 Westheimer Road, Suite 800
Houston
Texas 77042
Phone: 713.953.4800
www.arcadis.com

Appendix C

USDA NRCS Site-Specific Soils Report

United States Department of Agriculture

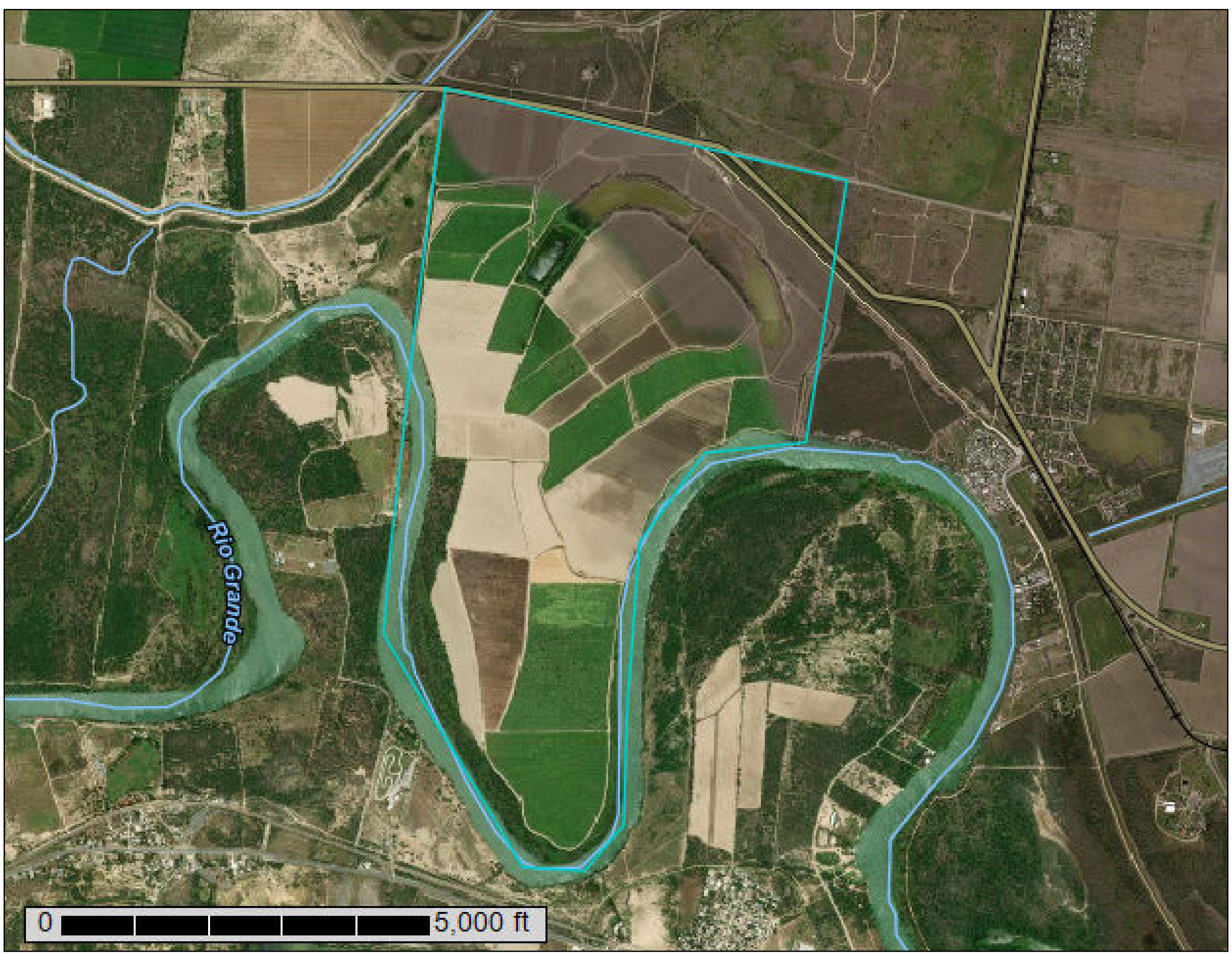
NRCS

Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for

Hidalgo County, Texas



Arcadis 000299

June 22, 2021

Preface

Soil surveys contain information that affects land use planning in survey areas. They highlight soil limitations that affect various land uses and provide information about the properties of the soils in the survey areas. Soil surveys are designed for many different users, including farmers, ranchers, foresters, agronomists, urban planners, community officials, engineers, developers, builders, and home buyers. Also, conservationists, teachers, students, and specialists in recreation, waste disposal, and pollution control can use the surveys to help them understand, protect, or enhance the environment.

Various land use regulations of Federal, State, and local governments may impose special restrictions on land use or land treatment. Soil surveys identify soil properties that are used in making various land use or land treatment decisions. The information is intended to help the land users identify and reduce the effects of soil limitations on various land uses. The landowner or user is responsible for identifying and complying with existing laws and regulations.

Although soil survey information can be used for general farm, local, and wider area planning, onsite investigation is needed to supplement this information in some cases. Examples include soil quality assessments (http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/health/) and certain conservation and engineering applications. For more detailed information, contact your local USDA Service Center (https://offices.sc.egov.usda.gov/locator/app?agency=nrcs) or your NRCS State Soil Scientist (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/contactus/?cid=nrcs142p2_053951).

Great differences in soil properties can occur within short distances. Some soils are seasonally wet or subject to flooding. Some are too unstable to be used as a foundation for buildings or roads. Clayey or wet soils are poorly suited to use as septic tank absorption fields. A high water table makes a soil poorly suited to basements or underground installations.

The National Cooperative Soil Survey is a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local agencies. The Natural Resources Conservation Service (NRCS) has leadership for the Federal part of the National Cooperative Soil Survey.

Information about soils is updated periodically. Updated information is available through the NRCS Web Soil Survey, the site for official soil survey information.

The U.S. Department of Agriculture (USDA) prohibits discrimination in all its programs and activities on the basis of race, color, national origin, age, disability, and where applicable, sex, marital status, familial status, parental status, religion, sexual orientation, genetic information, political beliefs, reprisal, or because all or a part of an individual's income is derived from any public assistance program. (Not all prohibited bases apply to all programs.) Persons with disabilities who require alternative means for communication of program information (Braille, large print, audiotape, etc.) should contact USDA's TARGET Center at (202) 720-2600 (voice and TDD). To file a complaint of discrimination, write to USDA, Director, Office of Civil Rights, 1400 Independence Avenue, S.W., Washington, D.C. 20250-9410 or call (800) 795-3272 (voice) or (202) 720-6382 (TDD). USDA is an equal opportunity provider and employer.

Contents

How Soil Surveys Are Made

Soil surveys are made to provide information about the soils and miscellaneous areas in a specific area. They include a description of the soils and miscellaneous areas and their location on the landscape and tables that show soil properties and limitations affecting various uses. Soil scientists observed the steepness, length, and shape of the slopes; the general pattern of drainage; the kinds of crops and native plants; and the kinds of bedrock. They observed and described many soil profiles. A soil profile is the sequence of natural layers, or horizons, in a soil. The profile extends from the surface down into the unconsolidated material in which the soil formed or from the surface down to bedrock. The unconsolidated material is devoid of roots and other living organisms and has not been changed by other biological activity.

Currently, soils are mapped according to the boundaries of major land resource areas (MLRAs). MLRAs are geographically associated land resource units that share common characteristics related to physiography, geology, climate, water resources, soils, biological resources, and land uses (USDA, 2006). Soil survey areas typically consist of parts of one or more MLRA.

The soils and miscellaneous areas in a survey area occur in an orderly pattern that is related to the geology, landforms, relief, climate, and natural vegetation of the area. Each kind of soil and miscellaneous area is associated with a particular kind of landform or with a segment of the landform. By observing the soils and miscellaneous areas in the survey area and relating their position to specific segments of the landform, a soil scientist develops a concept, or model, of how they were formed. Thus, during mapping, this model enables the soil scientist to predict with a considerable degree of accuracy the kind of soil or miscellaneous area at a specific location on the landscape.

Commonly, individual soils on the landscape merge into one another as their characteristics gradually change. To construct an accurate soil map, however, soil scientists must determine the boundaries between the soils. They can observe only a limited number of soil profiles. Nevertheless, these observations, supplemented by an understanding of the soil-vegetation-landscape relationship, are sufficient to verify predictions of the kinds of soil in an area and to determine the boundaries.

Soil scientists recorded the characteristics of the soil profiles that they studied. They noted soil color, texture, size and shape of soil aggregates, kind and amount of rock fragments, distribution of plant roots, reaction, and other features that enable them to identify soils. After describing the soils in the survey area and determining their properties, the soil scientists assigned the soils to taxonomic classes (units). Taxonomic classes are concepts. Each taxonomic class has a set of soil characteristics with precisely defined limits. The classes are used as a basis for comparison to classify soils systematically. Soil taxonomy, the system of taxonomic classification used in the United States, is based mainly on the kind and character of soil properties and the arrangement of horizons within the profile. After the soil scientists classified and named the soils in the survey area, they compared the individual soils with similar soils in the same taxonomic class in other areas so that they could confirm data and assemble additional data based on experience and research.

The objective of soil mapping is not to delineate pure map unit components; the objective is to separate the landscape into landforms or landform segments that have similar use and management requirements. Each map unit is defined by a unique combination of soil components and/or miscellaneous areas in predictable proportions. Some components may be highly contrasting to the other components of the map unit. The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The delineation of such landforms and landform segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, onsite investigation is needed to define and locate the soils and miscellaneous areas.

Soil scientists make many field observations in the process of producing a soil map. The frequency of observation is dependent upon several factors, including scale of mapping, intensity of mapping, design of map units, complexity of the landscape, and experience of the soil scientist. Observations are made to test and refine the soil-landscape model and predictions and to verify the classification of the soils at specific locations. Once the soil-landscape model is refined, a significantly smaller number of measurements of individual soil properties are made and recorded. These measurements may include field measurements, such as those for color, depth to bedrock, and texture, and laboratory measurements, such as those for content of sand, silt, clay, salt, and other components. Properties of each soil typically vary from one point to another across the landscape.

Observations for map unit components are aggregated to develop ranges of characteristics for the components. The aggregated values are presented. Direct measurements do not exist for every property presented for every map unit component. Values for some properties are estimated from combinations of other properties.

While a soil survey is in progress, samples of some of the soils in the area generally are collected for laboratory analyses and for engineering tests. Soil scientists interpret the data from these analyses and tests as well as the field-observed characteristics and the soil properties to determine the expected behavior of the soils under different uses. Interpretations for all of the soils are field tested through observation of the soils in different uses and under different levels of management. Some interpretations are modified to fit local conditions, and some new interpretations are developed to meet local needs. Data are assembled from other sources, such as research information, production records, and field experience of specialists. For example, data on crop yields under defined levels of management are assembled from farm records and from field or plot experiments on the same kinds of soil.

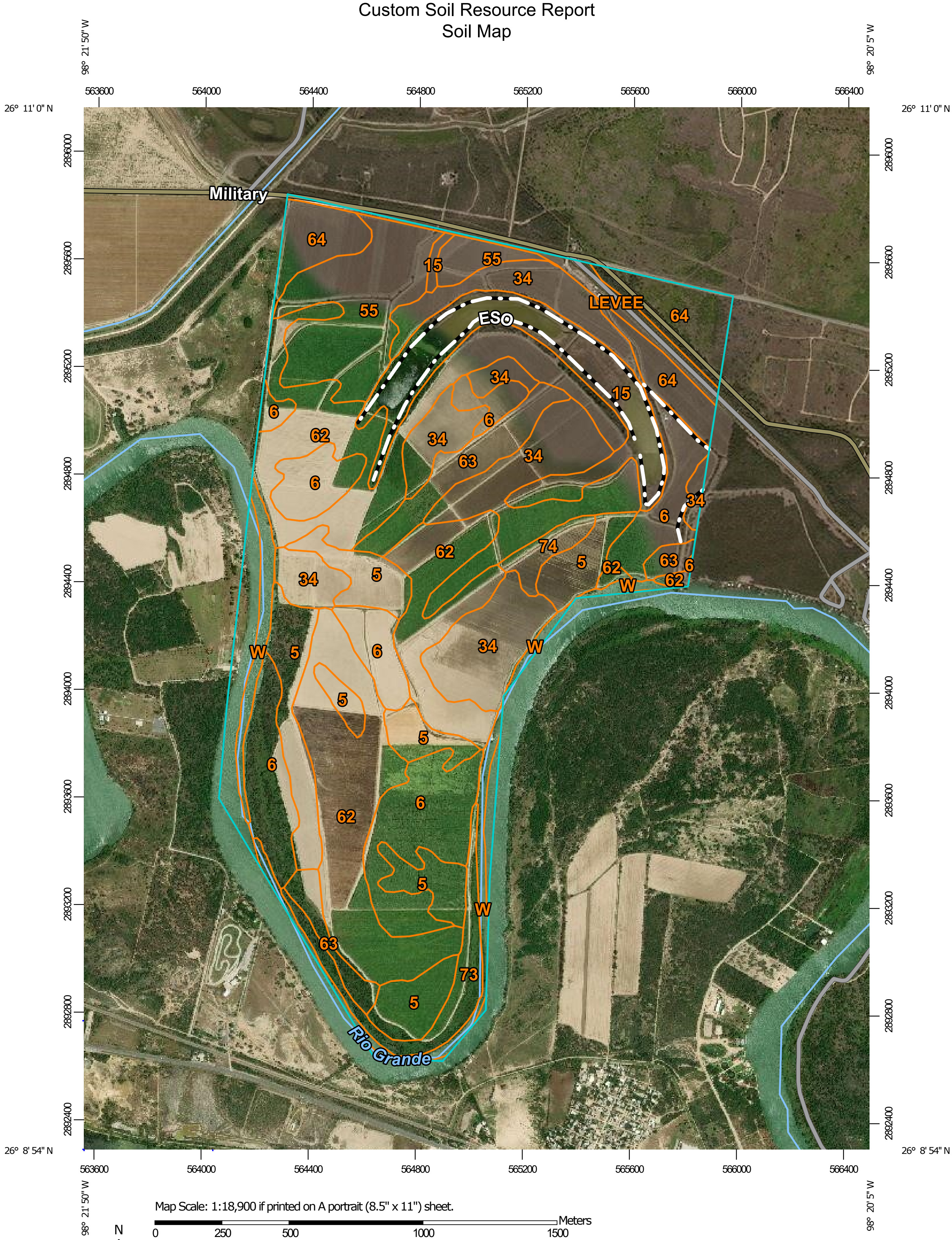
Predictions about soil behavior are based not only on soil properties but also on such variables as climate and biological activity. Soil conditions are predictable over long periods of time, but they are not predictable from year to year. For example, soil scientists can predict with a fairly high degree of accuracy that a given soil will have a high water table within certain depths in most years, but they cannot predict that a high water table will always be at a specific level in the soil on a specific date.

After soil scientists located and identified the significant natural bodies of soil in the survey area, they drew the boundaries of these bodies on aerial photographs and identified each as a specific map unit. Aerial photographs show trees, buildings, fields, roads, and rivers, all of which help in locating boundaries accurately.

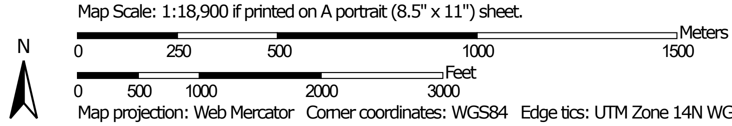
Soil Map

The soil map section includes the soil map for the defined area of interest, a list of soil map units on the map and extent of each map unit, and cartographic symbols displayed on the map. Also presented are various metadata about data used to produce the map, and a description of each soil map unit.

Soil Map



Map Scale: 1:18,900 if printed on A portrait (8.5" x 11") sheet.



Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 14N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Unit Polygons

Soil Map Unit Lines

Soil Map Unit Points

Special Point Features

Blowout

Borrow Pit

Clay Spot

Closed Depression

Gravel Pit

Gravelly Spot

Landfill

Lava Flow

Marsh or swamp

Mine or Quarry

Miscellaneous Water

Perennial Water

Rock Outcrop

Saline Spot

Sandy Spot

Severely Eroded Spot

Sinkhole

Slide or Slip

Sodic Spot

Spoil Area

Stony Spot

Very Stony Spot

Wet Spot

Other

Special Line Features

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Hidalgo County, Texas
Survey Area Data: Version 19, Jun 11, 2020

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Dec 10, 2010—Nov 5, 2017

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
5	Camargo silt loam, 0 to 1 percent slopes, rarely flooded	100.4	10.9%
6	Camargo silty clay loam, 0 to 1 percent slopes, rarely flooded	133.7	14.5%
15	Grulla clay, frequently flooded and ponded	68.1	7.4%
34	Matamoros silty clay	146.7	16.0%
55	Reynosa silty clay loam, 0 to 1 percent slopes	67.7	7.4%
62	Rio Grande silt loam	205.0	22.3%
63	Rio Grande silty clay loam	38.7	4.2%
64	Runn silty clay	55.8	6.1%
73	Zalla loamy fine sand, undulating	23.4	2.5%
74	Zalla silt loam	9.1	1.0%
LEVEE	Levee	14.2	1.5%
W	Water	32.4	3.5%
Totals for Area of Interest		**919.5**	**100.0%**

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

Hidalgo County, Texas

5—Camargo silt loam, 0 to 1 percent slopes, rarely flooded

Map Unit Setting

National map unit symbol: 2sxv7
Elevation: 0 to 300 feet
Mean annual precipitation: 20 to 27 inches
Mean annual air temperature: 73 to 75 degrees F
Frost-free period: 300 to 365 days
Farmland classification: All areas are prime farmland

Map Unit Composition

Camargo and similar soils: 90 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Camargo

Setting

Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Calcareous alluvium

Typical profile

Ap - 0 to 8 inches: silt loam
C - 8 to 80 inches: silty clay loam

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 1.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: RareNone
Frequency of ponding: None
Calcium carbonate, maximum content: 30 percent
Maximum salinity: Nonsaline to slightly saline (0.0 to 4.0 mmhos/cm)
Sodium adsorption ratio, maximum: 4.0
Available water capacity: High (about 9.9 inches)

Interpretive groups

Land capability classification (irrigated): 2e
Land capability classification (nonirrigated): 2e
Hydrologic Soil Group: B
Ecological site: R083DY013TX - Loamy Bottomland
Hydric soil rating: No

Minor Components

Rio grande

Percent of map unit: 7 percent

Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Ecological site: R083DY013TX - Loamy Bottomland
Hydric soil rating: No

Matamoros

Percent of map unit: 3 percent
Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Concave
Across-slope shape: Concave
Hydric soil rating: No

6—Camargo silty clay loam, 0 to 1 percent slopes, rarely flooded

Map Unit Setting

National map unit symbol: 2sxv5
Elevation: 0 to 300 feet
Mean annual precipitation: 20 to 27 inches
Mean annual air temperature: 73 to 75 degrees F
Frost-free period: 300 to 365 days
Farmland classification: All areas are prime farmland

Map Unit Composition

Camargo and similar soils: 90 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Camargo

Setting

Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Calcareous silty alluvium

Typical profile

Ap - 0 to 9 inches: silty clay loam
C - 9 to 80 inches: silty clay loam

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 1.98 in/hr)
Depth to water table: More than 80 inches

Frequency of flooding: RareNone
Frequency of ponding: None
Calcium carbonate, maximum content: 30 percent
Maximum salinity: Nonsaline to slightly saline (0.0 to 4.0 mmhos/cm)
Sodium adsorption ratio, maximum: 4.0
Available water capacity: High (about 9.6 inches)

Interpretive groups
Land capability classification (irrigated): 2s
Land capability classification (nonirrigated): 2e
Hydrologic Soil Group: B
Ecological site: R083DY013TX - Loamy Bottomland
Hydric soil rating: No

Minor Components

Rio grande
Percent of map unit: 5 percent
Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Ecological site: R083DY013TX - Loamy Bottomland
Hydric soil rating: No

Matamoros
Percent of map unit: 3 percent
Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Concave
Across-slope shape: Concave
Hydric soil rating: No

Raymondville
Percent of map unit: 2 percent
Landform: Terraces
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Ecological site: R083DY025TX - Clay Loam
Hydric soil rating: No

15—Grulla clay, frequently flooded and ponded

Map Unit Setting
National map unit symbol: dbkq
Elevation: 50 to 550 feet
Mean annual precipitation: 19 to 25 inches
Mean annual air temperature: 73 degrees F
Frost-free period: 314 to 341 days
Farmland classification: Not prime farmland

Map Unit Composition

Grulla and similar soils: 90 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Grulla

Setting

Landform: Sloughs, oxbows
Down-slope shape: Concave
Across-slope shape: Concave
Parent material: Calcareous clayey alluvium

Typical profile

H1 - 0 to 7 inches: clay
H2 - 7 to 65 inches: clay

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Somewhat poorly drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): Very low to moderately low (0.00 to 0.06 in/hr)
Depth to water table: About 0 inches
Frequency of flooding: NoneFrequent
Frequency of ponding: Frequent
Calcium carbonate, maximum content: 5 percent
Maximum salinity: Nonsaline to slightly saline (0.0 to 4.0 mmhos/cm)
Available water capacity: High (about 9.0 inches)

Interpretive groups

Land capability classification (irrigated): 4w
Land capability classification (nonirrigated): 4w
Hydrologic Soil Group: D
Ecological site: R083DY009TX - Clayey Bottomland
Hydric soil rating: Yes

Minor Components

Unnamed

Percent of map unit: 10 percent
Hydric soil rating: No

34—Matamoros silty clay

Map Unit Setting

National map unit symbol: dbld
Elevation: 30 to 200 feet
Mean annual precipitation: 17 to 27 inches

Mean annual air temperature: 72 to 73 degrees F
Frost-free period: 320 to 340 days
Farmland classification: Not prime farmland

Map Unit Composition

Matamoros and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Matamoros

Setting

Landform: Flood plains
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Calcareous clayey alluvium

Typical profile

H1 - 0 to 7 inches: silty clay
H2 - 7 to 65 inches: stratified very fine sandy loam to silty clay to clay

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Moderately well drained
Runoff class: Medium
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: OccasionalNone
Frequency of ponding: None
Calcium carbonate, maximum content: 5 percent
Maximum salinity: Nonsaline to slightly saline (0.0 to 4.0 mmhos/cm)
Sodium adsorption ratio, maximum: 2.0
Available water capacity: High (about 10.0 inches)

Interpretive groups

Land capability classification (irrigated): 2s
Land capability classification (nonirrigated): 2s
Hydrologic Soil Group: C
Ecological site: R083DY009TX - Clayey Bottomland
Hydric soil rating: No

Minor Components

Unnamed

Percent of map unit: 15 percent
Hydric soil rating: No

55—Reynosa silty clay loam, 0 to 1 percent slopes

Map Unit Setting

National map unit symbol: dbm4
Elevation: 700 to 1,200 feet
Mean annual precipitation: 17 to 27 inches
Mean annual air temperature: 70 to 73 degrees F
Frost-free period: 250 to 270 days
Farmland classification: All areas are prime farmland

Map Unit Composition

Reynosa and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Reynosa

Setting

Landform: Stream terraces
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Calcareous loamy alluvium

Typical profile

H1 - 0 to 15 inches: silty clay loam
H2 - 15 to 48 inches: silty clay loam
H3 - 48 to 80 inches: silty clay loam

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 1.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum content: 30 percent
Maximum salinity: Nonsaline to slightly saline (0.0 to 4.0 mmhos/cm)
Available water capacity: High (about 11.8 inches)

Interpretive groups

Land capability classification (irrigated): 1
Land capability classification (nonirrigated): 3c
Hydrologic Soil Group: B
Ecological site: R083DY013TX - Loamy Bottomland
Hydric soil rating: No

Minor Components

Unnamed

Percent of map unit: 15 percent
Hydric soil rating: No

62—Rio Grande silt loam

Map Unit Setting

National map unit symbol: dbmd
Elevation: 100 to 1,400 feet
Mean annual precipitation: 1 to 28 inches
Mean annual air temperature: 70 to 73 degrees F
Frost-free period: 280 to 340 days
Farmland classification: Not prime farmland

Map Unit Composition

Rio grande and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Rio Grande

Setting

Landform: Stream terraces
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Calcareous silty alluvium

Typical profile

H1 - 0 to 8 inches: silt loam
H2 - 8 to 65 inches: stratified loamy very fine sand to silt loam

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): High (1.98 to 5.95 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: OccasionalNone
Frequency of ponding: None
Calcium carbonate, maximum content: 20 percent
Maximum salinity: Nonsaline to slightly saline (0.0 to 4.0 mmhos/cm)
Available water capacity: High (about 11.4 inches)

Interpretive groups

Land capability classification (irrigated): 1

Land capability classification (nonirrigated): 3c
Hydrologic Soil Group: A
Ecological site: R083DY013TX - Loamy Bottomland
Hydric soil rating: No

Minor Components

Unnamed

Percent of map unit: 15 percent
Hydric soil rating: No

63—Rio Grande silty clay loam

Map Unit Setting

National map unit symbol: dbmf
Elevation: 100 to 1,400 feet
Mean annual precipitation: 1 to 28 inches
Mean annual air temperature: 70 to 73 degrees F
Frost-free period: 280 to 340 days
Farmland classification: Not prime farmland

Map Unit Composition

Rio grande and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Rio Grande

Setting

Landform: Stream terraces
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Calcareous silty alluvium

Typical profile

H1 - 0 to 8 inches: silty clay loam
H2 - 8 to 65 inches: stratified loamy very fine sand to silt loam

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 1.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: OccasionalNone
Frequency of ponding: None
Calcium carbonate, maximum content: 20 percent
Maximum salinity: Nonsaline to slightly saline (0.0 to 4.0 mmhos/cm)

Available water capacity: High (about 11.5 inches)

Interpretive groups

Land capability classification (irrigated): 1
Land capability classification (nonirrigated): 3c
Hydrologic Soil Group: B
Ecological site: R083DY013TX - Loamy Bottomland
Hydric soil rating: No

Minor Components

Unnamed

Percent of map unit: 15 percent
Hydric soil rating: No

64—Runn silty clay

Map Unit Setting

National map unit symbol: dbmg
Elevation: 100 to 200 feet
Mean annual precipitation: 20 to 27 inches
Mean annual air temperature: 72 to 75 degrees F
Frost-free period: 260 to 320 days
Farmland classification: All areas are prime farmland

Map Unit Composition

Runn and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Runn

Setting

Landform: Delta plains
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Calcareous silty alluvium

Typical profile

H1 - 0 to 55 inches: silty clay
H2 - 55 to 65 inches: silty clay

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Moderately well drained
Runoff class: Medium
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None

Frequency of ponding: None
Calcium carbonate, maximum content: 5 percent
Maximum salinity: Slightly saline to strongly saline (4.0 to 16.0 mmhos/cm)
Sodium adsorption ratio, maximum: 4.0
Available water capacity: High (about 10.1 inches)

Interpretive groups

Land capability classification (irrigated): 2s
Land capability classification (nonirrigated): 2s
Hydrologic Soil Group: C
Ecological site: R083DY009TX - Clayey Bottomland
Hydric soil rating: No

Minor Components

Unnamed

Percent of map unit: 15 percent
Hydric soil rating: No

73—Zalla loamy fine sand, undulating

Map Unit Setting

National map unit symbol: dbms
Elevation: 30 to 820 feet
Mean annual precipitation: 18 to 30 inches
Mean annual air temperature: 72 to 73 degrees F
Frost-free period: 270 to 345 days
Farmland classification: Not prime farmland

Map Unit Composition

Zalla and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Zalla

Setting

Landform: Flood plains
Down-slope shape: Convex
Across-slope shape: Convex
Parent material: Calcareous sandy alluvium

Typical profile

H1 - 0 to 9 inches: loamy fine sand
H2 - 9 to 65 inches: fine sand

Properties and qualities

Slope: 1 to 5 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Somewhat excessively drained
Runoff class: Negligible

Capacity of the most limiting layer to transmit water (Ksat): High to very high (5.95 to 19.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: OccasionalNone
Frequency of ponding: None
Calcium carbonate, maximum content: 25 percent
Maximum salinity: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Available water capacity: Low (about 4.2 inches)

Interpretive groups
Land capability classification (irrigated): 4w
Land capability classification (nonirrigated): 4w
Hydrologic Soil Group: A
Ecological site: R083DY013TX - Loamy Bottomland
Hydric soil rating: No

Minor Components

Unnamed
Percent of map unit: 15 percent
Hydric soil rating: No

74—Zalla silt loam

Map Unit Setting
National map unit symbol: dbmt
Elevation: 30 to 820 feet
Mean annual precipitation: 18 to 30 inches
Mean annual air temperature: 72 to 73 degrees F
Frost-free period: 270 to 345 days
Farmland classification: Not prime farmland

Map Unit Composition
Zalla and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Zalla

Setting
Landform: Flood plains
Down-slope shape: Linear
Across-slope shape: Concave
Parent material: Calcareous sandy alluvium

Typical profile
H1 - 0 to 9 inches: silt loam
H2 - 9 to 65 inches: fine sand

Properties and qualities
Slope: 0 to 1 percent

Depth to restrictive feature: More than 80 inches
Drainage class: Somewhat excessively drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): High (1.98 to 5.95 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: Rare
Frequency of ponding: None
Calcium carbonate, maximum content: 25 percent
Maximum salinity: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Available water capacity: Low (about 5.1 inches)

Interpretive groups

Land capability classification (irrigated): 3s
Land capability classification (nonirrigated): 4e
Hydrologic Soil Group: A
Ecological site: R083DY013TX - Loamy Bottomland
Hydric soil rating: No

Minor Components

Unnamed

Percent of map unit: 15 percent
Hydric soil rating: No

LEVEE—Levee

Map Unit Composition

Levees: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

W—Water

Map Unit Composition

Water: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

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Appendix D

Structural Assessment Calculations

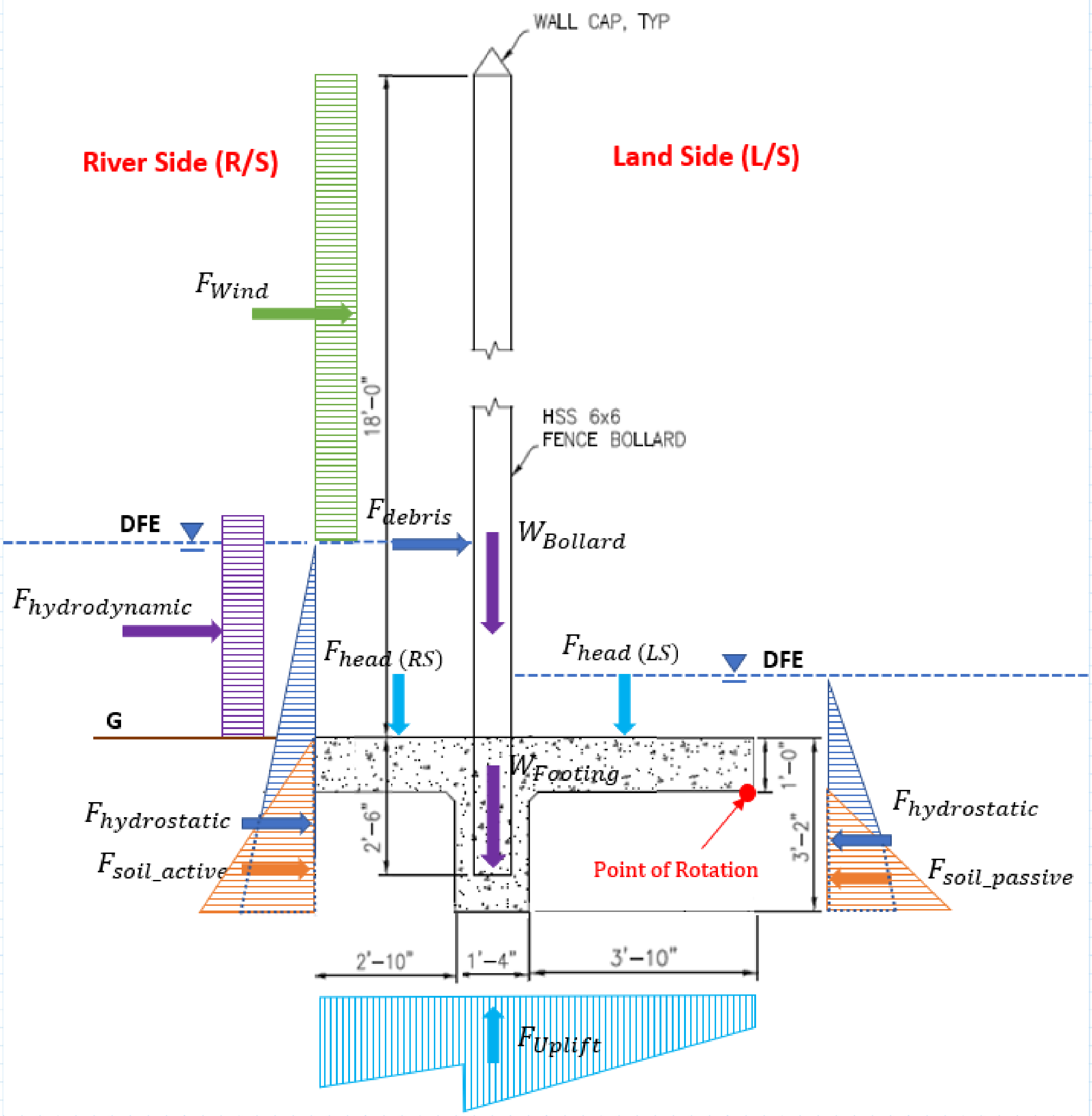
The following computation is aimed at investigating the external and the internal stabilities during raising waters for the 300-yr flood event of the existing bollard fence along the Rio Grande River near McAllen, Texas.

The following stability criteria is followed:

Loading Condition: Unusual Event (300-yr flood)	
Location of Resultant: 75% of Base in Compression	
Minimum Sliding F.S.: 1.2	EM-1110-2-2100 (Table 3-3)
Minimum Floatation F.S.: 1.2	EM-1110-2-2100 (Table 3-4)

References:

USACE, EM-1110-2-2100 Stability Analysis of Concrete Structure.
ASCE/SEI 7-10, Minimum Design Loads for Buildings and Other Structures.
FEMA P-55, Coastal Construction Manual.
ACI 318-14, Building Code Requirements for Structural Concrete and Commentary.



Loading Diagram of Typical Bollard Fence for Case A and B (Rising Waters from River Side)

General Inputs

Material Properties

Water unit weight:	$\gamma_w := 62.4 \cdot pcf$	
Soil Unit Weight	$\gamma_s := 115.0 \cdot pcf$	*(Ref. Expert Report, Section 5)*
Gravel Unit Weight (assumed for bollard fill) Remark: TGR plans show grout but gravel was used instead.	$\gamma_g := 105.0 \cdot pcf$	
Concrete Unit Weight (assumed for a concrete slightly reinforced)	$\gamma_c := 145.0 \cdot pcf$	
Steel Unit Weight	$\gamma_{steel} := 490 \cdot pcf$	
Unit Weight of Buoyant Soil	$\gamma_{s.buoy} := \gamma_s - \gamma_w = 52.6\ pcf$	
Angle of Internal Friction	$\phi := 35\ °$	*(Ref. Expert Report, Section 5)*
Soil Cohesion	$C := 0$	
Allowable Bearing Capacity of Soil	$\sigma_{bearing} := 1500\ psf$	*(Ref. Expert Report, Section 5)*
Coefficient of Friction with Concrete	$f := 0.25$	*(Ref. Expert Report, Section 5)*

Active Earth Pressure Coefficient $K_a := \dfrac{(1 - \sin(\phi))}{(1 + \sin(\phi))} = 0.27$ $\quad K_a = 0.27$

Passive Earth Pressure Coefficient $K_p := \dfrac{(1 + \sin(\phi))}{(1 - \sin(\phi))} = 3.69$ $\quad K_p = 3.69$

Arcadis 000327

Case A: Rising Waters Coming from River Side

This loading condition accounts for maximum flow velocity during rising waters coming from the river side.

Elevations & Geometry (Ref. Expert Report, Section 4)

Flood Elevation (River Side): $EL_{DFE.RS} := 113.7\ ft$

Flood Elevation (Land Side): $EL_{DFE.LS} := 112.9\ ft$

Grade Elevation (River Side): $EL_{grade.RS} := 112.0\ ft$

Grade Elevation (Land Side): $EL_{grade.LS} := 112.0\ ft$

Soil Elevation (River Side): $EL_{soil.RS} := 112.0\ ft$

Soil Elevation (Land Side): $EL_{soil.LS} := 111.0\ ft$

Base of Footing Elevation (River Side): $EL_{base.bott.RS} := 111.0\ ft$

Base of Footing Elevation (Land Side): $EL_{base.bott.LS} := 111.0\ ft$

Shear Key Bottom Elevation: $EL_{key.bott} := 108.834\ ft$

Water Velocity $V_{water} := 7.9\ \frac{ft}{s}$

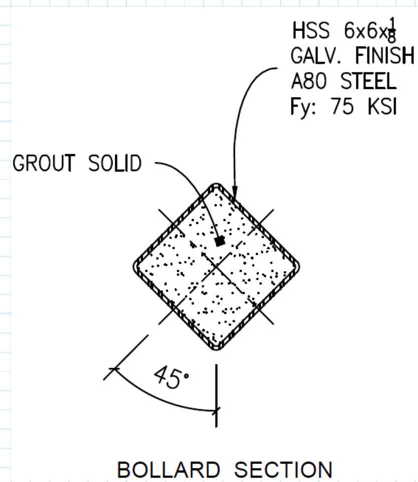
Elevations & Geometry (Ref. TGR Drawings)

Bollard Height (Above Base): $H_B := 18 \cdot ft$

Bollard Height (Embedded): $H_{B.Embedded} := 2 \cdot ft + 6\ in$

Bollard Thickness: $T_B := 0.125\ in$

Bollard Width (HSS6x6x1/8): $L_B := 6\ in$



TGR Drawing Sheet 2 of 2

Length of Heel: $L_{heel} := 2\ ft + 10\ in$

Length of Toe: $L_{toe} := 3\ ft + 10\ in$

Length of Shear Key: $L_{s\_key} := 1\ ft + 4\ in$

Shear Key Depth: $D_{s\_key} := 2\ ft + 2\ in$

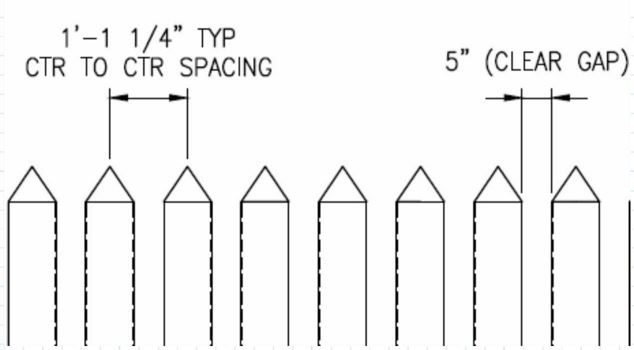
Length of Stem: $L_{stem} := 1\ ft + 4\ in$

Length of Base: $L_{base} := L_{toe} + L_{heel} + L_{stem} = 8\ ft$

Base Thickness: $T_{base} := 12\ in$

Bollard Fence Assessment

McAllen, TX Bollard Fence



Bollard Fence Elevation
Ref. TGR Drawing Sheet 2 of 2

Fence Imperviousness Factor below DFE (with a 30% debris blockage)

$$I_{imp.below.DFE} := \frac{\left(1\ ft + 1.25\ in - \frac{(5\ in)}{(1+30\%)}\right)}{1\ ft} = 0.78$$

Fence Imperviousness Factor above DFE (No blockage)

$$I_{imp.above.DFE} := \frac{\left(1\ ft + 1.25\ in - \frac{(5\ in)}{(1+0\%)}\right)}{1\ ft} = 0.69$$

Load Calculation

Dead Load

Bollard Cross-Section Area (HSS6x6x1/8): $A_B := 2.7\ in^2$

Bollard Weight: $W_B := A_B \cdot \left(H_B + H_{B.Embedded}\right) \cdot \gamma_{steel} = 188.34\ lbf$

Bollard Fill (Above Base): $W_{B\_Fill} := \left({L_B}^2 - A_B\right) \cdot H_B \cdot \gamma_g = 437.06\ lbf$

Bollard Total Weight (Cap weight ignored): $W_{B\_total} := W_B + W_{B\_Fill} = 625.41\ lbf$

Base Cross-Sectional Area: $A_{base} := L_{base} \cdot T_{base} = 8\ ft^2$

Toe Weight: $W_{toe} := L_{toe} \cdot T_{base} \cdot \gamma_c \cdot 1\ ft = 0.56\ kip$

Heel Weight: $W_{heel} := L_{heel} \cdot T_{base} \cdot \gamma_c \cdot 1\ ft = 0.41\ kip$

Stem Base Weight: $W_{stem} := L_{stem} \cdot T_{base} \cdot \gamma_c \cdot 1\ ft = 0.19\ kip$

Base Weight: $W_{Base} := A_{base} \cdot T_{base} \cdot \gamma_c = \left(1.16 \cdot 10^3\right)\ lbf$

Shear Key Area: $A_{s\_key} := L_{s\_key} \cdot D_{s\_key} = 2.89\ ft^2$

Shear Key Weight: $W_{s\_key} := D_{s\_key} \cdot L_{s\_key} \cdot \gamma_c \cdot 1\ ft = 0.42\ kip$

Total Weight: $W_{total} := W_{B\_total} + W_{Base} + W_{s\_key} = 2.2\ kip$

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Moments about Toe End

Resisting Moment (Heel) $M_{r.heel} := W_{heel} \cdot \left(L_{toe} + 0.5 \cdot L_{heel} + L_{s\_key}\right) = 2.7\ kip \cdot ft$

Resisting Moment (Toe) $M_{r.toe} := W_{toe} \cdot \left(0.5 \cdot L_{toe}\right) = 1.07\ kip \cdot ft$

Resisting Moment (Bollard) $M_{r.Bollard} := W_{B\_total} \cdot \left(0.5\ L_{s\_key} + L_{toe}\right) = 2.81\ kip \cdot ft$

Resisting Moment (Stem) $M_{r.stem} := W_{stem} \cdot \left(0.5\ L_{stem} + L_{toe}\right) = 0.87\ kip \cdot ft$

Resisting Moment from Shear Key $M_{r.key} := W_{s\_key} \cdot \left(\left(0.5 \cdot L_{s\_key}\right) + L_{toe}\right) = 1.89\ kip \cdot ft$

Wind Load

Risk Category based on Use or Occupancy of Building and Other Structures: Risk Category I

Wind Speed (ASCE 7-10 Online Hazard Tool for the Project Location https://asce7hazardtool.online/): $V_{wind} := 121\ mph$

Velocity Pressure Exposure Coefficient: $K_z := 1.16$

Topographic Factor: $K_{zt} := 1.0$

Wind Direction Factor: $K_d := 0.85$

Design Wind Pressure ASCE 7-10 Eq. 27.3-1: $q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot \left(\frac{V_{wind}}{mph}\right)^2 \cdot psf = 36.96\ psf$

Wind Force from River Side $F_{wind.RS} := q_z \cdot \left(H_B - EL_{DFE.RS} + EL_{grade.RS}\right) \cdot I_{imp.above.DFE} = 0.41\ \frac{kip}{ft}$

Moment Arm for Wind Force from River Side $L_{wind.RS} := \frac{\left(H_B - EL_{DFE.RS} + EL_{grade.RS}\right)}{2} + \left(EL_{DFE.RS} - EL_{base.bott.RS}\right) = 10.85\ ft$

Moment due to Wind from River Side $M_{o.wind.RS} := F_{wind.RS} \cdot L_{wind.RS} \cdot 1\ ft = 4.49\ kip \cdot ft$

Remark: Wind acting from the land side has been ignored since it will not be concurrent with river side wind.

Debris Impact Load

The debris object is assumed to be at or near the water surface level when it strikes (e.g. Stillwater elevation)

Water Velocity	$V_{water} = 7.9\ \frac{ft}{s}$	Ref. Expert Report, Section 4
Weight of Object:	$W_o := 1000\ lbf$	Ref. FEMA P-55 Section 8.5.10
Depth Coefficient (for a Floodway or Zone V):	$C_D := 1.0$	Ref. FEMA P-55 Section 8.5.10 Table 8-3
Blockage Coefficient (Assumed 30% Blockage):	$C_B := 1.0$	Ref. FEMA P-55 Section 8.5.10 Table 8-4
Building Structure Coefficient:	$C_{Str} := 0.8$	
Impact Force:	$F_i := W_o \cdot V_{water} \cdot \frac{sec}{ft} \cdot C_D \cdot C_B \cdot C_{Str} = 6.32\ kip$	FEMA P-55, Section 8.5.10 Eq. 8.9

For internal stability (e.g., flexural and shear strengths) of the bollard fence the above concentrated load calculated using Eq. 8.9 from FEMA P-55 Section 8.5.10 will be used later in these calculation.

For external stability, a minimum Debris Impact load 0.1 k/ft of wall is considered, as recommended by USACE (per ASCE 7-10, Chapter C5, Special Impact Loads.)

Distributed Debris Impact Load: $P_{debris} := I_{imp.below.DFE} \cdot 0.1\ \frac{kip}{ft} = 0.08\ \frac{kip}{ft}$

Moment Arm to Debris Impact Load: $L_{debris} := EL_{DFE.RS} - EL_{grade.RS} = 1.7\ ft$

Overturning Moment due to Debris Load: $M_{o.debris} := P_{debris} \cdot L_{debris} \cdot 1\ ft = 0.13\ kip \cdot ft$

Hydrodynamic Load

Since the velocity of water is less than 10 ft/sec, the dynamic effect of current is converted to equivalent surcharge depth dh, as per ASCE 7-10, Section 5.4.3

Coefficient for Drag or Shape Factor: $\alpha := 1.25$

Gravity: $g := 32.2\ \frac{ft}{s^2}$

Equivalent Surcharge Depth: $d_h := \frac{\alpha \cdot V_{water}^2}{2 \cdot g} = 1.21\ ft$

Design Stillwater Depth 300 Years Flood: $d_{300yr} := EL_{DFE.RS} - EL_{grade.RS} = 1.7\ ft$

Water Height due to Hydrodynamic Current: $H_{hydrodyn.} := d_{300yr} + d_h = 2.91\ ft$

Hydrodynamic Force: $F_{hydrodyn.} := d_h \cdot \gamma_w \cdot H_{hydrodyn.} \cdot I_{imp.below.DFE} = 0.17\ \frac{kip}{ft}$

Moment Arm for Hydrodynamic Load: $L_{hydrodyn.} := \frac{H_{hydrodyn.}}{2} = 1.46\ ft$

Hydrodynamic Moment due to Flood: $M_{o.hydn} := F_{hydrodyn.} \cdot L_{hydrodyn.} \cdot 1\ ft = 0.25\ kip \cdot ft$

Hydrostatic Load

For water to DFE (300-yr flood), Unusual Condition

Hydrostatic Force DFE Flood Acting on Footing, R/S:
$$F_{hyd.RS} := 0.5 \cdot \gamma_w \cdot \left(EL_{DFE.RS} - EL_{base.bott.RS}\right)^2 \cdot I_{imp.below.DFE} = 0.18\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Acting on Footing, R/S:
$$L_{hyd.RS} := \frac{EL_{DFE.RS} - EL_{base.bott.RS}}{3} = 0.9\ ft$$

Overturning Moment Due to DFE Flood Acting on Footing, R/S:
$$M_{o.hyd.RS} := F_{hyd.RS} \cdot L_{hyd.RS} \cdot 1\ ft = 0.16\ kip \cdot ft$$

Hydrostatic Force DFE Flood Acting on Footing, L/S:
$$F_{hyd.LS} := 0.5 \cdot \gamma_w \cdot \left(EL_{DFE.LS} - EL_{base.bott.LS}\right)^2 \cdot I_{imp.below.DFE} = 0.09\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Acting on Footing, L/S:
$$L_{hyd.LS} := \frac{EL_{DFE.LS} - EL_{base.bott.LS}}{3} = 0.63\ ft$$

Resisting Moment Due to DFE Flood Acting on Footing, L/S:
$$M_{r.hyd.LS} := F_{hyd.LS} \cdot L_{hyd.LS} \cdot 1\ ft = 0.06\ kip \cdot ft$$

Hydrostatic Force DFE Flood acting on Shear Key, R/S:
$$F_{hyd.key.RS} := \gamma_w \cdot \left(\frac{D_{s\_key}}{2}\right) \cdot \left(\left(EL_{DFE.RS} - EL_{base.bott.RS}\right) + \left(\left(EL_{DFE.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right)\right) = 0.51\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Acting on Shear Key, R/S: (AISC Table 17-27)
$$L_{hyd.key.RS} := \frac{D_{s\_key}}{3}\ \frac{\left(2 \cdot \left(\left(EL_{DFE.RS} - EL_{base.bott.RS}\right)\right) + D_{s\_key}\right) + \left(EL_{DFE.RS} - EL_{base.bott.RS}\right)}{\left(\left(\left(EL_{DFE.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) + \left(EL_{DFE.RS} - EL_{base.bott.RS}\right)\right)} = 0.98\ ft$$

Resisting Moment Due to DFE Flood Acting on Shear Key, R/S:
$$M_{r.hyd.key.RS} := F_{hyd.key.RS} \cdot L_{hyd.key.RS} \cdot 1\ ft = 0.5\ kip \cdot ft$$

Hydrostatic Force DFE Flood Acting on Shear Key, L/S:
$$F_{hyd.key.LS} := \gamma_w \cdot \left(\frac{D_{s\_key}}{2}\right) \cdot \left(\left(EL_{DFE.LS} - EL_{base.bott.LS}\right) + \left(\left(EL_{DFE.LS} - EL_{base.bott.LS}\right) + D_{s\_key}\right)\right) = 0.4\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Acting on Shear Key, L/S: (AISC Table 17-27)
$$L_{hyd.key.LS} := \frac{D_{s\_key}}{3}\ \frac{\left(2 \cdot \left(EL_{DFE.LS} - EL_{base.bott.LS}\right) + D_{s\_key}\right) + \left(EL_{DFE.LS} - EL_{base.bott.LS}\right)}{\left(\left(\left(EL_{DFE.LS} - EL_{base.bott.LS}\right) + D_{s\_key}\right) + \left(EL_{DFE.LS} - EL_{base.bott.LS}\right)\right)} = 0.95\ ft$$

Overturning Moment Due to DFE Flood Acting on Shear Key, L/S:
$$M_{o.hyd.key.LS} := F_{hyd.key.LS} \cdot L_{hyd.key.LS} \cdot 1\ ft = 0.38\ kip \cdot ft$$

Weight of Flood Water Sitting on Heel:
$$W_{water.base.heel} := \gamma_w \cdot \left(EL_{DFE.RS} - EL_{grade.RS}\right) \cdot \left(L_{heel} + \frac{L_{stem}}{2}\right) = 0.37\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Water Sitting on Heel
$$L_{w.hyd.heel} := \left(\frac{L_{heel} + \left(L_{stem} \div 2\right)}{2} + \left(L_{toe} + \left(L_{stem} \div 2\right)\right)\right) = 6.25\ ft$$

Resisting Moment Due to Weight of Flood Water on Heel:
$$M_{r.hyd.heel} := W_{water.base.heel} \cdot L_{w.hyd.heel} \cdot 1\ ft = 2.32\ kip \cdot ft$$

Weight of Flood Water Sitting on Toe:
$$W_{water.base.toe} := \gamma_w \cdot \left(EL_{DFE.LS} - EL_{grade.LS}\right) \cdot \left(L_{toe} + \frac{L_{stem}}{2}\right) = 0.25\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Water Sitting on Toe:
$$L_{w.hyd.toe} := \left(\frac{L_{toe} + \left(L_{stem} \div 2\right)}{2}\right) = 2.25\ ft$$

Resisting Moment Due to Weight of Flood Water on Toe:
$$M_{r.hyd.toe} := W_{water.base.toe} \cdot L_{w.hyd.toe} \cdot 1\ ft = 0.57\ kip \cdot ft$$

Weight of Flood Water Sitting on Footing: $W_{water.base} := W_{water.base.toe} + W_{water.base.heel} = 0.62 \ \frac{kip}{ft}$

Resisting Moment Due to Weight of Flood Water on Footing: $M_{r.hyd} := M_{r.hyd.toe} + M_{r.hyd.heel} = 2.89 \ kip \cdot ft$

Earth Pressure Load

Lateral Earth Pressure from River Side (DFE - 300 yr. flood))

Horizontal Earth Force Acting on Footing, R/S: $F_{soil.RS} := 0.5 \cdot K_a \cdot \gamma_{s.buoy} \cdot \left(EL_{grade.RS} - EL_{base.bott.RS}\right)^2 = \left(7.13 \cdot 10^{-3}\right) \frac{kip}{ft}$

Lever Arm for Horizontal Earth Force, R/S: $L_{soil.RS} := \frac{EL_{grade.RS} - EL_{base.bott.RS}}{3} = 0.33 \ ft$

Earth Force Acting on Shear Key, R/S:

$$F_{soil.key.RS} := 0.5 \cdot K_a \cdot \gamma_{s.buoy} \cdot \left(\left(EL_{grade.RS} - EL_{base.bott.RS}\right) + \left(EL_{grade.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) \cdot D_{s\_key} = 0.064 \ \frac{kip}{ft}$$

Lever Arm for Horizontal Earth Force Acting on Shear Key, R/S:

$$L_{key.RS} := \frac{D_{s\_key} \cdot \left(\left(2 \ \left(EL_{grade.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) + \left(\left(EL_{grade.RS} - EL_{base.bott.RS}\right)\right)\right)}{3 \cdot \left(\left(\left(EL_{grade.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) + \left(EL_{grade.RS} - EL_{base.bott.RS}\right)\right)} = 0.9 \ ft$$

Moments from R/S Lateral Earth Pressure: $M_{o.soil.RS} := F_{soil.RS} \cdot L_{soil.RS} \cdot 1 \ ft = \left(2.38 \cdot 10^{-3}\right) \ kip \cdot ft$

$M_{r.key.RS} := F_{soil.key.RS} \cdot L_{key.RS} \cdot 1 \ ft = 0.06 \ kip \cdot ft$

Lateral Earth Pressure from Land Side (DFE - 300 yr. flood)

Horizontal Earth Force acting on Footing, L/S: $F_{soil.LS} := 0.5 \cdot K_p \cdot \gamma_{s.buoy} \cdot \left(EL_{base.bott.LS} - EL_{base.bott.LS}\right)^2 = 0 \ \frac{kip}{ft}$

Remark: Soil replaced by roadway which does not contribute to passive resistance.

Horizontal Earth Force Acting on Shear Key, L/S: $F_{soil.key.LS} := 0.5 \cdot \left(K_p \cdot \gamma_{s.buoy} \cdot \left(D_{s\_key}\right)^2\right) = 0.46 \ \frac{kip}{ft}$

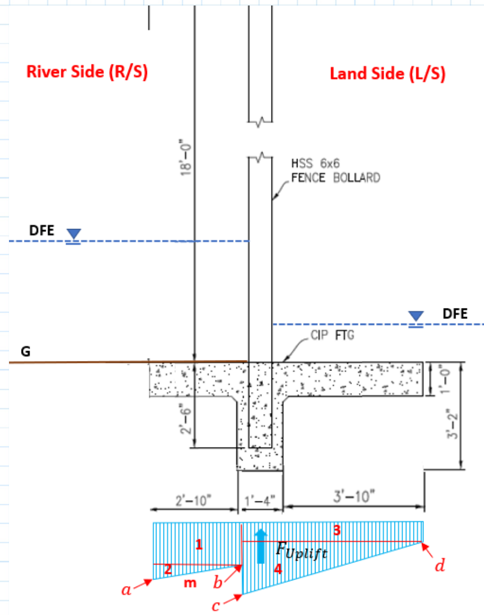
Lever Arm for Horizontal Earth Force Acting on Footing , L/S: $L_{soil.LS} := \frac{EL_{base.bott.LS} - EL_{base.bott.LS}}{3} = 0 \ ft$

Lever Arm for Horizontal Earth Force Acting on Shear Key, L/S: $L_{soil.key.LS} := \frac{2 \ D_{s\_key}}{3} = 1.44 \ ft$

Resisting Moment from L/S Lateral Earth Pressure: $M_{r.soil.LS} := F_{soil.LS} \cdot L_{soil.LS} \cdot 1 \ ft = 0 \ kip \cdot ft$

Overturning Moment due to Lateral Earth Pressure on Shear Key, L/S: $M_{o.soil.key.LS} := F_{soil.key.LS} \cdot L_{soil.key.LS} \cdot 1 \ ft = 0.66 \ kip \cdot ft$

Uplift Load



Design Flood Elevation (DFE): $EL_{DFE.RS} = 113.7\ ft$

Depth of Water to DFE on R/S: $d_{s.RS} := EL_{DFE.RS} - EL_{base.bott.RS} = 2.7\ ft$

Depth of Water to DFE on L/S: $d_{s.LS} := EL_{DFE.LS} - EL_{base.bott.LS} = 1.9\ ft$

Slope $m := \dfrac{\left(d_{s.RS} - d_{s.LS}\right)}{L_{base}} = 0.1$

Uplift Pressure below Heel:

$$P_{uplift.a} := \gamma_w \cdot \left(d_{s.RS}\right) \cdot ft = 168.48\ plf$$

$$P_{uplift.b} := P_{uplift.a} - \left(m \cdot L_{heel} \cdot \frac{P_{uplift.a}}{ft}\right) = 120.74\ plf$$

Uplift Pressure Below Shear Key:

$$P_{uplift.c} := P_{uplift.b} + \gamma_w \cdot \left(d_{s.RS} - D_{s\_key} + T_{base}\right) \cdot ft = 216.42\ plf$$

$$P_{uplift.d} := \gamma_w \cdot \left(d_{s.LS}\right) \cdot ft = 118.56\ plf$$

Uplift below Heel (Area 1+2): $V_{uplift.area.1.2} := \left(P_{uplift.a} + P_{uplift.b}\right) \cdot \dfrac{L_{heel}}{2} = 0.41\ kip$

Lever Arm for Uplift under the Heel: $L_{arm.area.1.2} := \dfrac{L_{heel} \cdot \left(2 \cdot P_{uplift.a} + P_{uplift.b}\right)}{3 \cdot \left(P_{uplift.a} + P_{uplift.b}\right)} + L_{s\_key} + L_{toe} = 6.66\ ft$

Overturning Moment due to Uplift below Heel: $M_{o.1.2} := V_{uplift.area.1.2} \cdot L_{arm.area.1.2} = 2.73\ kip \cdot ft$

Uplift below Shear Key and Toe (Area 3+4): $V_{uplift.area.3.4} := \left(P_{uplift.c} + P_{uplift.d}\right) \cdot \dfrac{L_{toe} + L_{s\_key}}{2} = 0.87\ kip$

Lever Arm for Uplift under the Shear Key and Toe : $L_{arm.area.3.4} := \dfrac{\left(L_{toe} + L_{s\_key}\right) \cdot \left(2 \cdot P_{uplift.c} + P_{uplift.d}\right)}{3 \cdot \left(P_{uplift.c} + P_{uplift.d}\right)} = 2.83\ ft$

Overturning Moment due to Uplift below Shear Key and Toe : $M_{o.3.4} := V_{uplift.area.3.4} \cdot L_{arm.area.3.4} = 2.45\ kip \cdot ft$

Overturning Moment due to Uplift: $M_{o.uplift} := M_{o.1.2} + M_{o.3.4} = 5.18\ kip \cdot ft$

Sum of Uplift: $V_{uplift} := V_{uplift.area.1.2} + V_{uplift.area.3.4} = 1.28\ kip$

Vertical Resultant Force: $V_{net} := \left(W_{water.base} \cdot ft + W_{total} - V_{uplift}\right) = 1.55\ kip$

Sum of Lateral Loads from River Side (DFE Water on R/S)

$$F_{lateral.RS} := F_{wind.RS} + F_{hyd.RS} + P_{debris} + F_{hydrodyn.} + F_{soil.RS} + F_{soil.key.RS} = 0.91\ \frac{kip}{ft}$$

Sum of Lateral Loads from Land Side (DFE Water on L/S)

$$F_{lateral.LS} := F_{soil.LS} + F_{hyd.LS} + F_{soil.key.LS} = 0.54\ \frac{kip}{ft}$$

Net Lateral Force:

$$F_{lateral.net} := F_{lateral.RS} - F_{lateral.LS} = 0.37\ \frac{kip}{ft}$$ *(acting in the flow direction)*

Sum of Moments from Flood

$$M_{o.flood} := M_{o.debris} + M_{o.hyd.RS} + M_{o.hydn} + M_{o.uplift} + M_{o.hyd.key.LS} = 6.11\ kip \cdot ft$$

$$M_{r.flood} := M_{r.hyd.LS} + M_{r.hyd.key.RS} + M_{r.hyd} = 3.45\ kip \cdot ft$$

Moment from Wind

$$M_{o.wind.RS} = 4.49\ kip \cdot ft$$

Sum of Moments from Soil

$$M_{o.soil} := M_{o.soil.RS} + M_{o.soil.key.LS} = 0.66\ kip \cdot ft$$

$$M_{r.soil} := M_{r.key.RS} = 0.06\ kip \cdot ft$$

Sum of Resisting Moments from Structure

$$M_{r.struct} := M_{r.Bollard} + M_{r.toe} + M_{r.key} + M_{r.heel} + M_{r.stem} = 9.34\ kip \cdot ft$$

Sum of Overturning and Resisting Moments on Flood Wall

$$M_{o.sum} := M_{o.flood} + M_{o.soil} + M_{o.wind.RS} = 11.27\ kip \cdot ft$$

$$M_{r.sum} := M_{r.flood} + M_{r.struct} + M_{r.soil} = 12.84\ kip \cdot ft$$

Overturning Stability Check (With Debris Impact Load) (Not a Criteria but for informational purposes)

Overturning Factor of Safety

$$FS_{overturning} := \frac{M_{r.sum}}{M_{o.sum}} = 1.14$$

Location of Resultant Force Check (With Debris Impact Load)

Table 3-5 Requirements for Location of the Resultant – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	100% of Base in Compression	75% of Base in Compression	Resultant Within Base

EM 1110-2-2100, Table 3-5

Kern Length $Kern := \frac{L_{base}}{3} = 2.67\ ft$

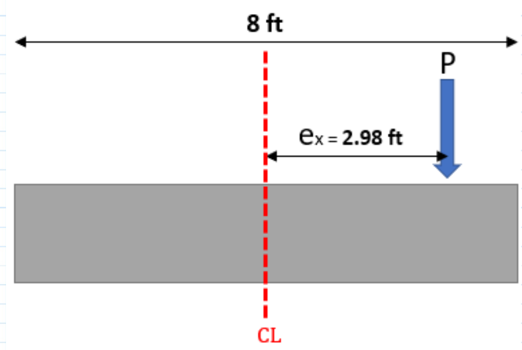
Balance Moment $M_{balance} := M_{r.sum} - M_{o.sum} = 1.58\ kip \cdot ft$

Resultant Location $x_R := \frac{M_{balance}}{V_{net}} = 1.02\ ft$

Eccentricity $e_x := \frac{L_{base}}{2} - x_R = 2.98\ ft$

Check_Resultant_Location_with_Debris_Impact :=
- if $|e_x| \le \frac{Kern}{2}$
  - "Resultant within the Kern"
- if $\frac{Kern}{2} < |e_x| < \frac{L_{base}}{2}$
  - "Resultant Outside the Kern but within the base"
- else
  - "Failed"

Check_Resultant_Location_with_Debris_Impact = "Resultant Outside the Kern but within the base"



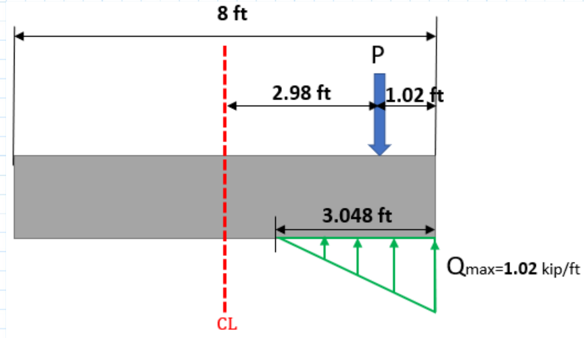
When Eccentricity exceeds the Kern, the pressure distribution under the footing takes a triangular shape as shown in the figure below. For this type of situation, the pressure Qmax becomes equal to 2P/3C. Here, P is total vertical force, and C is resultant location.

$P_{down} := V_{net} = 1.55\ kip$

$C_1 := x_R = 1.02\ ft$

$Q_{max} := \frac{2 \cdot V_{net}}{3 \cdot x_R} = 1.02\ \frac{kip}{ft}$

However, from equilibrium of forces, the area of the pressure triangle has to be equal to the resultant vertical force. Therefore, P = 0.5*B*Qmax. Here, B is the length of the pressure triangle.



Length of the Pressure Triangle $B := \frac{P_{down}}{0.5 \cdot Q_{max}} = 3.048\ ft$

75% of Base $0.75 \cdot L_{base} = 6\ ft$

$Base\_Compression\_Ratio\_Check\_with\_Debris\_Impact :=$ if $B \geq 0.75 \cdot L_{base}$
  "OK, More than 75% of the Base is in Compression"
else
  "FAILED"

$Base\_Compression\_Ratio\_Check\_with\_Debris\_Impact =$ "FAILED"

Sliding Safety Factor Check (With Debris Impact Load)

Sum of Horizontal Load on the River Side

$F_{RS} := F_{lateral.RS} \cdot 1\ ft = 0.91\ kip$

Sum of Horizontal Load on the Land Side

$F_{LS} := F_{lateral.LS} \cdot 1\ ft = 0.54\ kip$

Cohesion

$C_{Cohesion} := C \cdot L_{base} \cdot 1 \div ft = 0$

Friction Resistance Force

$F_R := V_{net} \cdot f = 0.39\ kip$

$FS_{Sliding} := \frac{F_R + F_{LS}}{F_{RS}} = 1.02$

$Sliding\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact :=$ if $FS_{Sliding} \geq 1.2$
  "OK, adequate safety factor"
else
  "FAILED"
= "FAILED"

$Sliding\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact =$ "FAILED"

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Sum of Overturning and Resisting Moments on Flood Wall (Without Debris Impact Load)

$$M_{o.sum.wo.debris.impact} := M_{o.hyd.RS} + M_{o.hydn} + M_{o.uplift} + M_{o.hyd.key.LS} + M_{o.soil.RS} + M_{o.soil.key.LS} + M_{o.wind.RS} = 11.13\ kip \cdot ft$$

$$M_{r.sum.wo.debris.impact} := M_{r.hyd.LS} + M_{r.hyd.key.RS} + M_{r.hyd} + M_{r.struct} + M_{r.soil} = 12.84\ kip \cdot ft$$

Overturning Stability Check (Not a Criteria but for informational purposes)

Overturning Factor of Safety **(Without Debris Impact Load)**

$$FS_{overturning.wo.debris.impact} := \frac{M_{r.sum.wo.debris.impact}}{M_{o.sum.wo.debris.impact}} = 1.15$$

Location of Resultant Force Check (Without Debris Impact Load)

Kern Length $Kern := \frac{L_{base}}{3} = 2.67\ ft$

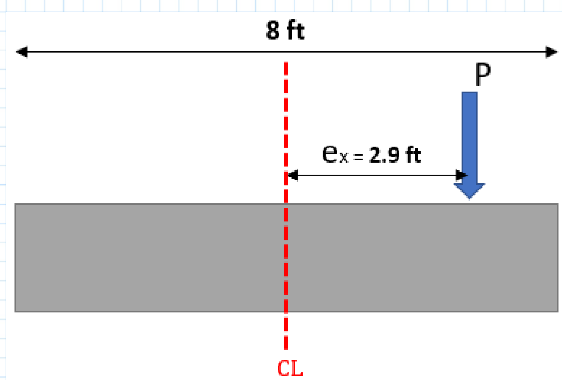
Balance Moment $M_{balance.wo.debris.impact} := M_{r.sum.wo.debris.impact} - M_{o.sum.wo.debris.impact} = 1.71\ kip \cdot ft$

Resultant Location $x_{R.wo.debris.impact} := \frac{M_{balance.wo.debris.impact}}{V_{net}} = 1.1\ ft$

Eccentricity $e_{x.wo.debris.impact} := \frac{L_{base}}{2} - x_{R.wo.debris.impact} = 2.9\ ft$

$Check\_Resultant\_Location\_wo\_Debris\_Impact :=$ if $\left|e_{x.wo.debris.impact}\right| \le \frac{Kern}{2}$
    "Resultant within the Kern"
if $\frac{Kern}{2} < \left|e_{x.wo.debris.impact}\right| < \frac{L_{base}}{2}$
    "Resultant Outside the Kern but within the base"
else
    "Failed"

$Check\_Resultant\_Location\_wo\_Debris\_Impact =$ "Resultant Outside the Kern but within the base"

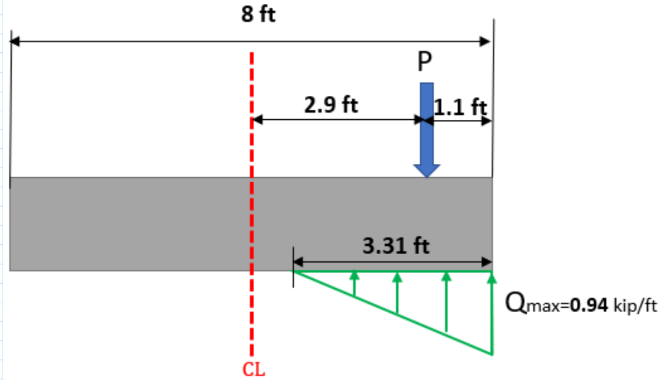


When Eccentricity exceeds the Kern, the pressure distribution under the footing takes a triangular shape as shown in the figure below. For this type of situation, the pressure Qmax becomes equal to 2P/3C.

$P_{down} := V_{net} = 1.55\ \mathbf{kip}$

$C_1 := x_{R.wo.debris.impact} = 1.1\ \mathbf{ft}$

$$Q_{max.wo.debris.impact} := \frac{2 \cdot V_{net}}{3 \cdot x_{R.wo.debris.impact}} = 0.94\ \frac{\mathbf{kip}}{\mathbf{ft}}$$



However, from equilibrium of forces, the area of the pressure triangle has to be equal to the resultant vertical force. Therefore, P = 0.5*B*Qmax.

Length of the Pressure Triangle $B_{wo.debris.impact} := \dfrac{P_{down}}{0.5 \cdot Q_{max.wo.debris.impact}} = 3.31\ \mathbf{ft}$ $\quad 0.75 \cdot L_{base} = 6\ \mathbf{ft}$

$Base\_Compression\_Ratio\_Check\_wo\_Debris\_Impact :=$ if $B_{wo.debris.impact} \ge 0.75 \cdot L_{base}$
"OK, More than 75% of the Base is in Compression"
else
"FAILED"

$Base\_Compression\_Ratio\_Check\_wo\_Debris\_Impact =$ "FAILED"

Sliding Safety Factor Check (Without Debris Impact Load)

Sum of Horizontal Load on the River Side **(Without Debris Impact Load)**

$$F_{RS.wo.debris.impact} := \left(F_{wind.RS} + F_{hyd.RS} + F_{hydrodyn.} + F_{soil.RS} + F_{soil.key.RS}\right) \cdot 1\ \mathbf{ft} = 0.84\ \mathbf{kip}$$

Sum of Horizontal Load on the Land Side

$$F_{LS} = 0.54\ \mathbf{kip}$$

Cohesion

$$C_{Cohesion} := C \cdot L_{base} \cdot 1 \div \mathbf{ft} = 0$$

Friction Resistance Force

$$F_R := V_{net} \cdot f = 0.39\ \mathbf{kip}$$

$$FS_{Sliding.wo.debris.impact} := \frac{F_R + F_{LS}}{F_{RS.wo.debris.impact}} = 1.11$$

$$Sliding\_Factor\_of\_Safety\_Check\_wo\_Debris\_Impact := \begin{cases} \text{if } FS_{Sliding.wo.debris.impact} \geq 1.2 \\ \quad \text{"OK, adequate safety factor"} \\ \text{else} \\ \quad \text{"FAILED"} \end{cases} = \text{"FAILED"}$$

$Sliding\_Factor\_of\_Safety\_Check\_wo\_Debris\_Impact = \text{"FAILED"}$

Floatation Stability Check (With Debris Impact Load)

Downward Vertical Force

$$V_{downward} := W_{water.base} \cdot ft + W_{total} = 2.83 \ kip$$

Upward Vertical Force

$$V_{upward} := V_{uplift} = 1.28 \ kip$$

$$FS_{floatation} := \frac{V_{downward}}{V_{upward}} = 2.22$$

$$Floatation\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact := \begin{cases} \text{if } FS_{floatation} \geq 1.2 \\ \quad \text{"OK, adequate safety factor"} \\ \text{else} \\ \quad \text{"FAILED"} \end{cases} = \text{"OK, adequate safety factor"}$$

$Floatation\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact = \text{"OK, adequate safety factor"}$

Table 3-4 Required Factors of Safety for Flotation – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	1.3	1.2	1.1

EM 1110-2-2100, Table 3-4

Bearing Pressure Check (With Debris Impact Load)

Length of the Pressure Triangle $B := \frac{P_{down}}{0.5 \cdot Q_{max}} = 3.05\ ft$

Effective width of the base for Bearing Pressure $L_{effective} := B = 3.05\ ft$

Bearing Pressure per 1 Foot Section $Bearing_{Pressure} := \frac{V_{net}}{L_{effective} \cdot 1\ ft} = 0.51\ ksf$

Allowable Bearing Pressure $\sigma_{bearing} = 1.5\ ksf$ *(Ref. Expert Report, Section 5)*

$Bearing\_Pressure\_Check\_with\_Debris\_Impact :=$
if $Bearing_{Pressure} \le \sigma_{bearing}$
  "OK, Bearing Pressure is within Allowable"
else
  "FAILED"

$Bearing\_Pressure\_Check\_with\_Debris\_Impact =$ "OK, Bearing Pressure is within Allowable"

Flexural Design Assessment (With Debris Impact Load)

Load Combination (ASCE 7-10, Section 2.3)

Sum of Lateral Loads from River Side

$F_{lateral.RS.Factored} := 1.0 \left(F_{hyd.RS} + P_{debris} + F_{hydrodyn.}\right) + 1.6 \cdot \left(F_{soil.RS}\right) + 0.5 \cdot F_{wind.RS} = 0.65 \ \frac{kip}{ft}$ *(ASCE 7-10, Section 2.3.2, Eq. 4)*

Sum of Lateral Loads from Land Side

$F_{lateral.LS.Factored} := 1.0 \left(F_{hyd.LS}\right) + 0.9 \cdot \left(F_{soil.LS} + F_{soil.key.LS}\right) = 0.5 \ \frac{kip}{ft}$ *(ASCE 7-10, Section 2.3.2, Eq. 6)*

Net Lateral Force

$F_{net.lateral..Factored} := F_{lateral.RS.Factored} - F_{lateral.LS.Factored} = 0.15 \ \frac{kip}{ft}$ *(acting in the flow direction)*

Flood Factored Moment

$M_{o.flood.factored} := 1.0 \cdot M_{o.flood} = 6.11 \ kip \cdot ft$

$M_{r.flood.factored} := 1.0 \cdot M_{r.flood} = 3.45 \ kip \cdot ft$

$M_{o.wind.RS.factored} := 0.5 \cdot M_{o.wind.RS} = 2.25 \ kip \cdot ft$

Soil Factored Moment

$M_{o.soil.factored} := 1.6 \cdot M_{o.soil} = 1.06 \ kip \cdot ft$

$M_{r.soil.factored} := 0.9 \cdot M_{r.soil} = 0.05 \ kip \cdot ft$

Structural Dead Weight Factored Moment

$M_{r.struct.factored.case1} := 1.2 \cdot M_{r.struct} = 11.21 \ kip \cdot ft$ *(ASCE 7-10, Section 2.3.2, Eq. 2)*

$M_{r.struct.factored.case2} := 0.9 \cdot M_{r.struct} = 8.41 \ kip \cdot ft$ *(ASCE 7-10, Section 2.3.2, Eq. 6)*

Sum of Overturning and Resisting Moment of the Wall (Factored)

$M_{o.sum.factored.case1} := M_{o.flood.factored} + M_{o.wind.RS.factored} + M_{o.soil.factored} = 9.41 \ kip \cdot ft$

$M_{r.sum.factored.case1} := M_{r.flood.factored} + M_{r.struct.factored.case1} + M_{r.soil.factored} = 14.71 \ kip \cdot ft$

$M_{o.sum.factored.case2} := M_{o.flood.factored} + M_{o.wind.RS.factored} + M_{o.soil.factored} = 9.41 \ kip \cdot ft$

$M_{r.sum.factored.case2} := M_{r.flood.factored} + M_{r.struct.factored.case2} + M_{r.soil.factored} = 11.9 \ kip \cdot ft$

$M_{balance.case1} := M_{r.sum.factored.case1} - M_{o.sum.factored.case1} = 5.29 \ kip \cdot ft$

$M_{balance.case2} := M_{r.sum.factored.case2} - M_{o.sum.factored.case2} = 2.49 \ kip \cdot ft$

Flood Factored Forces

$$V_{flood.factored} := 1.0 \cdot \left(W_{water.base} \cdot ft - V_{uplift}\right) = -0.65\ kip$$

Structural Dead Weight Factored Forces

$$V_{struct.factored.case1} := 1.2 \cdot W_{total} = 2.65\ kip$$

$$V_{struct.factored.case2} := 0.9 \cdot W_{total} = 1.98\ kip$$

$$V_{net.factored.case1} := V_{flood.factored} + V_{struct.factored.case1} = 1.99\ kip$$

$$V_{net.factored.case2} := V_{flood.factored} + V_{struct.factored.case2} = 1.33\ kip$$

When Eccentricity falls outside of the Kern, the pressure distribution under the footing takes a triangular shape as shown in the figure below. For this type of situation, the pressure Q max becomes equal to 2P/3C.

$$M_{balance} := M_{balance.case1} = 5.29\ kip \cdot ft$$

$$x_R := \frac{M_{balance}}{V_{net.factored.case1}} = 2.65\ ft$$

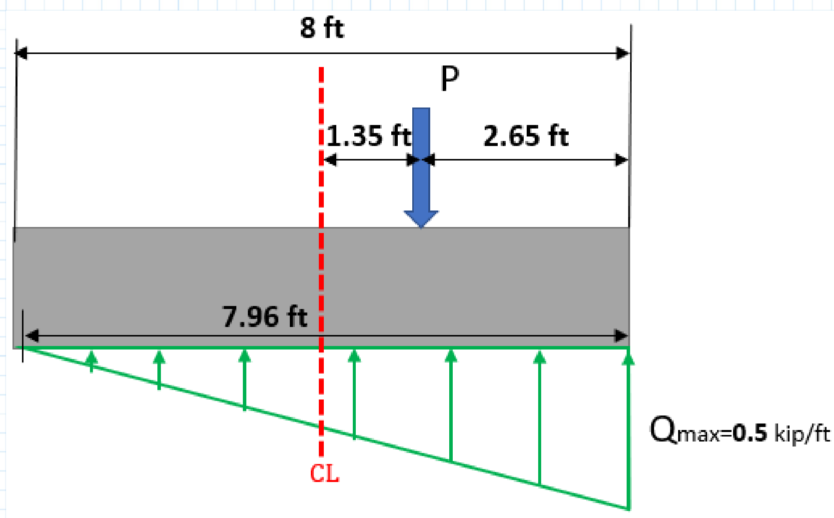
$$e_x := \frac{L_{base}}{2} - x_R = 1.35\ ft$$

$$C_1 := x_R = 2.65\ ft$$

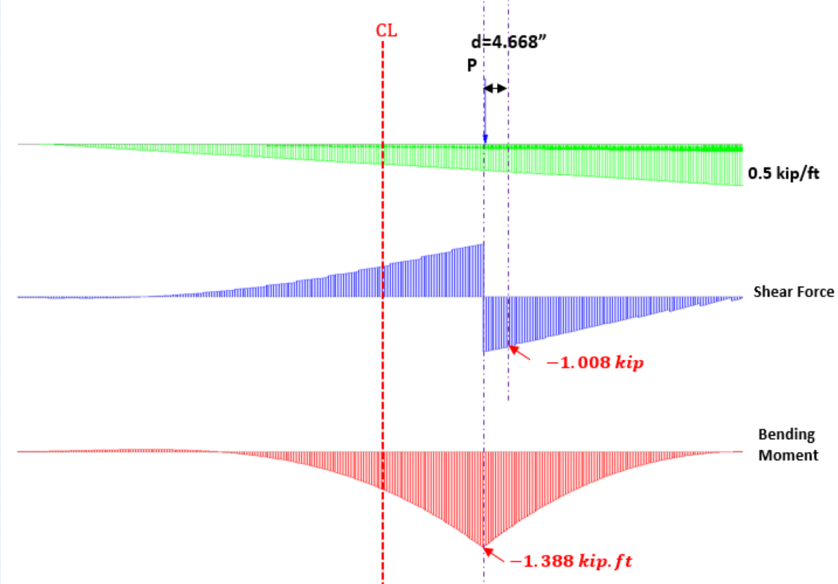
$$Q_{max} := \frac{2 \cdot V_{net.factored.case1}}{3 \cdot x_R} = 0.5\ \frac{kip}{ft}$$

$$P_{down} := V_{net.factored.case1} = 1.99\ kip$$

$$B := \frac{P_{down}}{0.5 \cdot Q_{max}} = 7.96\ ft$$



Remark: See attached STAAD.Pro Report.



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From the above analysis moment and shear values for design are as follows:

Bending Moment $M_u := -1.388\ kip \cdot ft$

Shear Force $V_u := -1.008\ kip$

Footing Section Width $B := 12\ in$

Footing Section Depth $h := T_{base} = 12\ in$

Yield Strength of Steel $f_y := 60\ ksi$

Compressive Strength of Concrete $f'_c := 4\ ksi$

Flexure (Tension-controlled Section) Strength Reduction Factor $\phi_f := 0.9$

Shear Strength Reduction Factor $\phi_s := 0.75$

$\beta_1 := 0.85$

ACI 318-14 Section 21.2.1, Table 21.2.1 and 21.2.2: Strength Reduction factor for Flexure and Shear

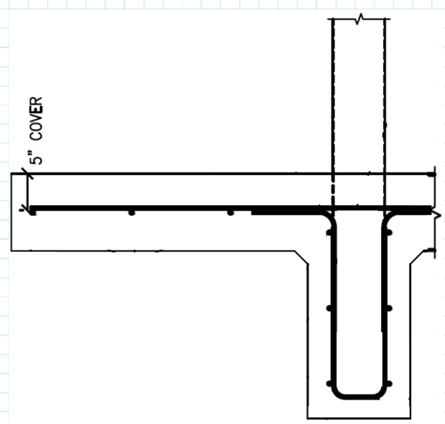
Reinforcement Spacing $S_b := 1\ ft + 1.25\ in = 13.25\ in$ From TGR Drawing

Reinforcement Diameter (#5 bar) $d_b := 0.625\ in$

Concrete Cover $Cover := 5\ in - \left(\dfrac{d_b}{2}\right) = 4.69\ in$ For the Controlling Flexural Moment (Negative)

Depth to Reinforcement

Effective Depth $d_e := h - Cover = 7.31\ in$ For the Controlling Flexural Moment (Negative)



TGR Drawing Sheet 2 of 2

Area of Reinforcing Steel $A_b := 0.31\ in^2$

Reinforcing Steel Area per Foot Width $A_{s.prov} := \left(\dfrac{B}{S_b}\right) \cdot A_b = 0.28\ in^2$

$$c_v := \left(\frac{A_{s.prov} \cdot f_y}{\beta_1 \cdot B \cdot (0.85 \cdot f'_c)}\right) = 0.49\ in$$

$$R_u := \frac{|M_u|}{\phi_f \cdot B \cdot (d_e)^2} = 28.84\ psi$$

$$\rho_{9.6.1.1} := \left(0.85 \cdot \frac{f'_c}{f_y}\right) \cdot \left(1 - \sqrt{1 - \left(2 \cdot \frac{R_u}{0.85 \cdot f'_c}\right)}\right) = 0.0005$$

According to ACI 318-14 section 9.6.1.2: At every section of a Flexural member where tensile reinforcement is required, the minimum ratio of reinforcement is:

$$\rho_{9.6.1.2} := \max\left(\left(3 \cdot \sqrt{\frac{f'_c}{f_y} \cdot \frac{psi}{f_y}}\right), \left(\frac{200}{\frac{f_y}{psi}}\right)\right) = 0.0033$$

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$$\rho_{9.6.1.3} := min\left(\left(1.333 \cdot \rho_{9.6.1.1}\right), \left(\rho_{9.6.1.2}\right)\right) = 0.0006$$

$$A_{s.reqd} := \rho_{9.6.1.3} \cdot B \cdot d_e = 0.06\ \boldsymbol{in}^2 \qquad A_{s.prov} = 0.28\ \boldsymbol{in}^2$$

$$Minimum\_Reinforcement\_Check := \left\| \begin{array}{l} \text{if } A_{s.prov} \geq A_{s.reqd} \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“Recalculate”} \end{array} \right| = \text{“OK”}$$

$$Minimum\_Reinforcement\_Check = \text{“OK”}$$

Reinforcement Shrinkage and Temperature Check

Main Reinforcement Shrinkage and Temperature Check

ACI 318-14 - 24.4.3.2: The ratio of deformed shrinkage and temperature reinforcement area to gross concrete area shall be greater than or equal to 0.0018.

Concrete Gross Area $A_{g.1} := T_{base} \cdot \boldsymbol{ft} = 1\ \boldsymbol{ft}^2$

$$0.0018 \cdot A_{g.1} = 0.26\ \boldsymbol{in}^2 \qquad A_{s.prov} = 0.28\ \boldsymbol{in}^2$$

$$Reinforcement\_Shrinkage\_and\_Temperature\_Check\_in\_Longitudinal\_Direction := \left\| \begin{array}{l} \text{if } 0.0018 \cdot A_{g.1} \geq A_{s.prov} \\ \quad \| \text{“S\&T requirement is not met”} \\ \text{else} \\ \quad \| \text{“S\&T requirement is met”} \end{array} \right|$$

$$Reinforcement\_Shrinkage\_and\_Temperature\_Check\_in\_Longitudinal\_Direction = \text{“S\&T requirement is met”}$$

Transverse Reinforcement Shrinkage and Temperature Check

Concrete Gross Area (transversely) $A_{g.2} := L_{base} \cdot T_{base} = \left(1.15 \cdot 10^3\right)\ \boldsymbol{in}^2$

Area of Transverse Reinforcing Steel $A_{b.t} := 0.31\ \boldsymbol{in}^2$ TGR Drawing Sheet 2 of 2

Number of bars $N_{bars} := 7$ TGR Drawing Sheet 2 of 2

Reinforcing Steel Area per Foot Width $A_{s.t.prov} := N_{bars}\ A_{b.t} = 2.17\ \boldsymbol{in}^2$ $\quad 0.0018 \cdot A_{g.2} = 2.07\ \boldsymbol{in}^2$

$$Reinforcement\_Shrinkage\_and\_Temperature\_Check\_in\_Transverse\_Direction := \left\| \begin{array}{l} \text{if } 0.0018 \cdot A_{g.2} \geq A_{s.t.prov} \\ \quad \| \text{“S\&T requirement is not met”} \\ \text{else} \\ \quad \| \text{“S\&T requirement is met”} \end{array} \right|$$

$$Reinforcement\_Shrinkage\_and\_Temperature\_Check\_in\_Transverse\_Direction = \text{“S\&T requirement is met”}$$

Nominal Moment $M_n := A_{s.prov} \cdot f_y \cdot \left(d_e - \frac{\beta_1 \cdot c_v}{2}\right) = 9.98\ \mathbf{kip} \cdot \mathbf{ft}$

Factored Resisting Moment $M_r := \phi_f \cdot M_n = 8.98\ \mathbf{kip} \cdot \mathbf{ft}$

$|M_u| = 1.39\ \mathbf{kip} \cdot \mathbf{ft}$

$$Moment\_Capacity\_Check := \begin{Vmatrix} \text{if } M_r \geq |M_u| \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{Vmatrix} = \text{"OK"}$$

$\frac{M_r}{|M_u|} = 6.47$

$Moment\_Capacity\_Check = \text{"OK"}$

Shear Capacity Assessment

Nominal Shear Capacity of Concrete $V_{Capacity} := 2 \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}} \cdot \mathbf{psi} \cdot B \cdot d_e = 11.1\ \mathbf{kip}$

Factored Shear Capacity $\phi V_{n.psw} := \phi_s \cdot V_{Capacity} = 8.32\ \mathbf{kip}$

$|V_u| = 1.01\ \mathbf{kip}$

$$Shear\_Capacity\_Check := \begin{Vmatrix} \text{if } \phi V_{n.psw} \geq |V_u| \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{Vmatrix} = \text{"OK"}$$

$Shear\_Capacity\_Check = \text{"OK"}$

Internal Stability of the Bollard Fence

For internal stability of the bollard fence the concentrated load calculated using Eq. 8.9 from FEMA P-55 Section 8.5.10 is used at service level.

HSS Steel Yield Strength $F_y := 75\ \mathbf{ksi}$ TGR Drawing Sheet 2 of 2

Wind Force from R/S (No Blockage): $F_{Wind} := F_{wind.RS} \cdot \mathbf{ft} = 0.41\ \mathbf{kip}$ From Page 5

Hydrostatic Force from R/S (With a 30% Debris Blockage): $F_{hyd.RS} := 0.5 \cdot \gamma_w \cdot \left(EL_{DFE.RS} - EL_{grade.RS}\right)^2 \cdot I_{imp.below.DFE} \cdot \mathbf{ft} = 0.07\ \mathbf{kip}$

Hydrostatic Force from L/S (With a 30% Debris Blockage): $F_{hyd.LS} := 0.5 \cdot \gamma_w \cdot \left(EL_{DFE.LS} - EL_{grade.LS}\right)^2 \cdot I_{imp.below.DFE} \cdot \mathbf{ft} = 0.02\ \mathbf{kip}$

Impact Force: $F_{impact} := F_i = 6.32\ \mathbf{kip}$ From Page 5 FEMA P-55, Section 8.5.10 Eq. 8.9

Hydrodynamic Force from R/S (With a 30% Debris Blockage): $F_{hydrodyn} := F_{hydrodyn.} \cdot ft = 0.17\ kip$ From Page 6

Sum of Moment on a Bollard Fence

$$M_{1.bollard} := 0.6 \cdot F_{Wind} \cdot \left(\frac{(H_B - EL_{DFE.RS})}{2} + (EL_{DFE.RS} - EL_{grade.RS}) \right) + 0.75 \cdot F_{hyd.RS} \cdot \left(\frac{EL_{DFE.RS} - EL_{grade.RS}}{3} \right) - 0.75 \cdot F_{hyd.LS} \cdot \left(\frac{EL_{DFE.LS} - EL_{grade.LS}}{3} \right) + 0.75 \cdot F_i \cdot (EL_{DFE.RS} - EL_{grade.RS}) + 0.75 \cdot F_{hydrodyn} \cdot \left(\frac{H_{hydrodyn.}}{2} \right) = -3.2\ kip \cdot ft$$

Required Section Modulus $S_{x.req.1.bollard} := \frac{|M_{1.bollard}|}{0.6 \cdot F_y} = 0.85\ in^3$

Bollard Section Width $a := 6\ in$

Bollard Section Thickness $t := 0.116\ in$

Bollard Hollow Width $b := a - (2 \cdot t) = 5.77\ in$

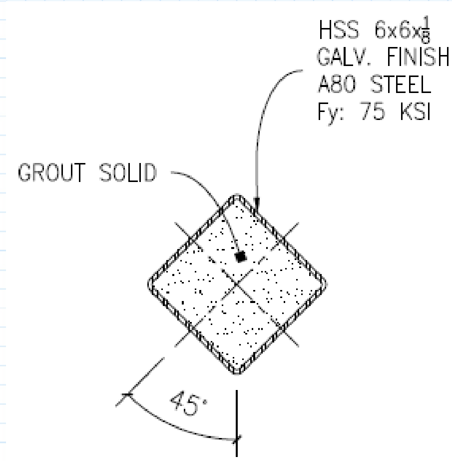
Section Modulus

$$S_{x.1} := \frac{a^3}{6 \cdot \sqrt{2}} = 25.46\ in^3$$

$$S_{x.2} := \frac{b^3}{6 \cdot \sqrt{2}} = 22.62\ in^3$$

AISC Construction Manual Table 17-27

Provided Section Modulus $S_{x.prov} := S_{x.1} - S_{x.2} = 2.84\ in^3$



BOLLARD SECTION

TGR Drawing Sheet 2 of 2

$$One\_Bollard\_Flexural\_Capacity\_Check := \begin{array}{l} \text{if } S_{x.prov} \geq S_{x.req.1.bollard} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{array} = \text{"OK"}$$

$One\_Bollard\_Flexural\_Capacity\_Check = \text{"OK"}$

Remark: If a 1,000 lbm floating debris impacts one bollard the HHS will not experience permanent deformations.

Case B: Rising Waters Coming from River Side

This loading condition accounts for the maximum water surface during rising water coming from the river side in the western segment of the bollard fence.

Elevations & Geometry (Ref. Expert Report, Section 4)

Flood Elevation (River Side): $EL_{DFE.RS} := 129.03\ ft$

Flood Elevation (Land Side): $EL_{DFE.LS} := 128.7\ ft$

Grade Elevation (River Side): $EL_{grade.RS} := 112.74\ ft$

Grade Elevation (Land Side): $EL_{grade.LS} := 112.74\ ft$

Soil Elevation (River Side): $EL_{soil.RS} := 112.74\ ft$

Soil Elevation (Land Side): $EL_{soil.LS} := 111.74\ ft$

Base of Footing Elevation (River Side): $EL_{base.bott.RS} := 111.74\ ft$

Base of Footing Elevation (Land Side): $EL_{base.bott.LS} := 111.74\ ft$

Shear Key Bottom Elevation: $EL_{key.bott} := 109.57\ ft$

Water Velocity $V_{water} := 7.0\ \frac{ft}{s}$

Elevations & Geometry (Ref. TGR Drawings)

Bollard Height (Above Base): $H_B = 18\ ft$

Bollard Height (Embedded): $H_{B.Embedded} = 2.5\ ft$

Bollard Thickness: $T_B = 0.13\ in$

Bollard Width (HSS6x6x1/8): $L_B = 6\ in$

Length of Heel: $L_{heel} = 2.83\ ft$

Length of Toe: $L_{toe} = 3.83\ ft$

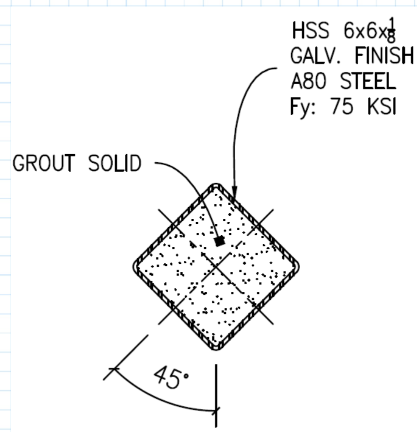
Length of Shear Key: $L_{s\_key} = 1.33\ ft$

Shear Key Depth: $D_{s\_key} = 2.17\ ft$

Length of Stem: $L_{stem} = 1.33\ ft$

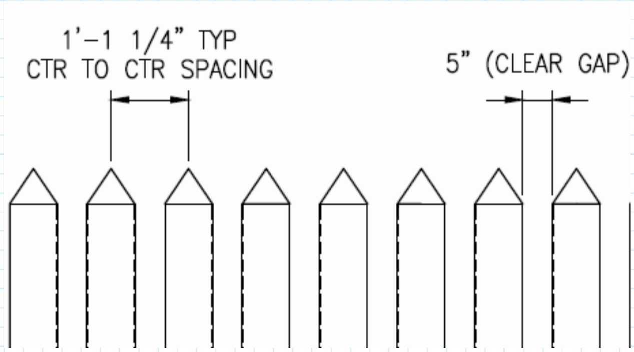
Length of Base: $L_{base} = 8\ ft$

Base Thickness: $T_{base} = 1\ ft$



BOLLARD SECTION

TGR Drawing Sheet 2 of 2



Bollard Fence Elevation
Ref. TGR Drawing Sheet 2 of 2

Fence Imperviousness Factor below DFE (with a 30% debris blockage) $I_{imp.below.DFE} = 0.78$

Fence Imperviousness Factor above DFE (No blockage) $I_{imp.above.DFE} = 0.69$

Load Calculation

Dead Load

Bollard Cross-Section Area (HSS6x6x1/8): $A_B = 2.7\ in^2$

Bollard Weight: $W_B = 188.34\ lbf$

Bollard Fill (Above Base): $W_{B\_Fill} = 437.06\ lbf$

Bollard Total Weight (Cap weight ignored): $W_{B\_total} = 625.41\ lbf$

Base Cross-Sectional Area: $A_{base} = 8\ ft^2$

Toe Weight: $W_{toe} = 0.56\ kip$

Heel Weight: $W_{heel} = 0.41\ kip$

Stem Base Weight: $W_{stem} = 0.19\ kip$

Base Weight: $W_{Base} = 1.16\ kip$

Shear Key Area: $A_{s\_key} = 2.89\ ft^2$

Shear Key Weight: $W_{s\_key} = 0.42\ kip$

Total Weight: $W_{total} = 2.2\ kip$

Moments about Toe End

Resisting Moment (Heel) $M_{r.heel} := W_{heel} \cdot (L_{toe} + 0.5 \cdot L_{heel} + L_{s\_key}) = 2.7\ \boldsymbol{kip \cdot ft}$

Resisting Moment (Toe) $M_{r.toe} := W_{toe} \cdot (0.5 \cdot L_{toe}) = 1.07\ \boldsymbol{kip \cdot ft}$

Resisting Moment (Bollard) $M_{r.Bollard} := W_{B\_total} \cdot (0.5\ L_{s\_key} + L_{toe}) = 2.81\ \boldsymbol{kip \cdot ft}$

Resisting Moment (Stem) $M_{r.stem} := W_{stem} \cdot (0.5\ L_{stem} + L_{toe}) = 0.87\ \boldsymbol{kip \cdot ft}$

Resisting Moment from Shear Key $M_{r.key} := W_{s\_key} \cdot ((0.5 \cdot L_{s\_key}) + L_{toe}) = 1.89\ \boldsymbol{kip \cdot ft}$

Wind Load

Risk Category based on Use or Occupancy of Building and Other Structures: Risk Category I

Wind Speed (ASCE 7-10 Online Hazard Tool for the Project Location https://asce7hazardtool.online/): $V_{wind} = 121\ \boldsymbol{mph}$

Velocity Pressure Exposure Coefficient: $K_z := 1.16$

Topographic Factor: $K_{zt} := 1.0$

Wind Direction Factor: $K_d := 0.85$

Design Wind Pressure ASCE 7-10 Eq. 27.3-1: $q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot \left(\frac{V_{wind}}{\boldsymbol{mph}}\right)^2 \cdot \boldsymbol{psf} = 36.96\ \boldsymbol{psf}$

Wind Force from River Side $F_{wind.RS} := q_z \cdot (H_B - EL_{DFE.RS} + EL_{grade.RS}) \cdot I_{imp.above.DFE} = 0.04\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$

Moment Arm for Wind Force from River Side $L_{wind.RS} := \frac{(H_B - EL_{DFE.RS} + EL_{grade.RS})}{2} + (EL_{DFE.RS} - EL_{base.bott.RS}) = 18.15\ \boldsymbol{ft}$

Moment due to Wind from River Side $M_{o.wind.RS} := F_{wind.RS} \cdot L_{wind.RS} \cdot 1\ \boldsymbol{ft} = 0.79\ \boldsymbol{kip \cdot ft}$

Remark: Wind acting from the land side has been ignored since it will not be concurrent with river side wind.

Debris Impact Load

The debris object is assumed to be at or near the water surface level when it strikes (e.g. Stillwater elevation)

Water Velocity $V_{water} = 7\ \frac{ft}{s}$ Ref. Expert Report, Section 4

Weight of Object: $W_o := 1000\ lbf$ Ref. FEMA P-55 Section 8.5.10

Depth Coefficient (for a Floodway or Zone V): $C_D := 1.0$ Ref. FEMA P-55 Section 8.5.10 Table 8-3

Blockage Coefficient (Assumed 30% Blockage): $C_B := 1.0$ Ref. FEMA P-55 Section 8.5.10 Table 8-4

Building Structure Coefficient: $C_{Str} := 0.8$

Impact Force: $F_i := W_o \cdot V_{water} \cdot \frac{sec}{ft} \cdot C_D \cdot C_B \cdot C_{Str} = 5.6\ kip$ FEMA P-55, Section 8.5.10 Eq. 8.9

For internal stability (e.g., flexural and shear strengths) of the bollard fence the above concentrated load calculated using Eq. 8.9 from FEMA P-55 Section 8.5.10 will be used later in these calculation.

For external stability, a minimum Debris Impact load 0.1 k/ft of wall is considered, as recommended by USACE (per ASCE 7-10, Chapter C5, Special Impact Loads.)

Distributed Debris Impact Load: $P_{debris} := I_{imp.below.DFE} \cdot 0.1\ \frac{kip}{ft} = 0.08\ \frac{kip}{ft}$

Moment Arm to Debris Impact Load: $L_{debris} := EL_{DFE.RS} - EL_{grade.RS} = 16.29\ ft$

Overturning Moment due to Debris Load: $M_{o.debris} := P_{debris} \cdot L_{debris} \cdot 1\ ft = 1.28\ kip \cdot ft$

Hydrodynamic Load

Since the velocity of water is less than 10 ft/sec, the dynamic effect of current is converted to equivalent surcharge depth dh, as per ASCE 7-10, Section 5.4.3.

Coefficient for Drag or Shape Factor: $\alpha := 1.25$

Gravity: $g := 32.2\ \frac{ft}{s^2}$

Equivalent Surcharge Depth: $d_h := \frac{\alpha \cdot V_{water}^{\ 2}}{2 \cdot g} = 0.95\ ft$

Design Stillwater Depth 300 Years Flood: $d_{300yr} := EL_{DFE.RS} - EL_{grade.RS} = 16.29\ ft$

Water Height due to Hydrodynamic Current: $H_{hydrodyn.} := d_{300yr} + d_h = 17.24\ ft$

Hydrodynamic Force: $F_{hydrodyn.} := d_h \cdot \gamma_w \cdot H_{hydrodyn.} \cdot I_{imp.below.DFE} = 0.8\ \frac{kip}{ft}$

Moment Arm for Hydrodynamic Load: $L_{hydrodyn.} := \frac{H_{hydrodyn.}}{2} = 8.62\ ft$

Hydrodynamic Moment due to Flood: $M_{o.hydn} := F_{hydrodyn.} \cdot L_{hydrodyn.} \cdot 1\ ft = 6.91\ kip \cdot ft$

Hydrostatic Load

For water to DFE (300-yr flood), Unusual Condition

Hydrostatic Force DFE Flood Acting on Footing, R/S:
$$F_{hyd.RS} := 0.5 \cdot \gamma_w \cdot \left(EL_{DFE.RS} - EL_{base.bott.RS}\right)^2 \cdot I_{imp.below.DFE} = 7.31\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Acting on Footing, R/S:
$$L_{hyd.RS} := \frac{EL_{DFE.RS} - EL_{base.bott.RS}}{3} = 5.76\ ft$$

Overturning Moment Due to DFE Flood Acting on Footing, R/S:
$$M_{o.hyd.RS} := F_{hyd.RS} \cdot L_{hyd.RS} \cdot 1\ ft = 42.13\ kip \cdot ft$$

Hydrostatic Force DFE Flood Acting on Footing, L/S:
$$F_{hyd.LS} := 0.5 \cdot \gamma_w \cdot \left(EL_{DFE.LS} - EL_{base.bott.LS}\right)^2 \cdot I_{imp.below.DFE} = 7.03\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Acting on Footing, L/S:
$$L_{hyd.LS} := \frac{EL_{DFE.LS} - EL_{base.bott.LS}}{3} = 5.65\ ft$$

Resisting Moment Due to DFE Flood Acting on Footing, L/S:
$$M_{r.hyd.LS} := F_{hyd.LS} \cdot L_{hyd.LS} \cdot 1\ ft = 39.76\ kip \cdot ft$$

Hydrostatic Force DFE Flood acting on Shear Key, R/S:
$$F_{hyd.key.RS} := \gamma_w \cdot \left(\frac{D_{s\_key}}{2}\right) \cdot \left(\left(EL_{DFE.RS} - EL_{base.bott.RS}\right) + \left(\left(EL_{DFE.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right)\right) = 2.48\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Acting on Shear Key, R/S: (AISC Table 17-27)
$$L_{hyd.key.RS} := \frac{D_{s\_key}}{3} \frac{\left(\left(2 \cdot \left(\left(EL_{DFE.RS} - EL_{base.bott.RS}\right)\right) + D_{s\_key}\right) + \left(EL_{DFE.RS} - EL_{base.bott.RS}\right)\right)}{\left(\left(\left(EL_{DFE.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) + \left(EL_{DFE.RS} - EL_{base.bott.RS}\right)\right)} = 1.06\ ft$$

Resisting Moment Due to DFE Flood Acting on Shear Key, R/S:
$$M_{r.hyd.key.RS} := F_{hyd.key.RS} \cdot L_{hyd.key.RS} \cdot 1\ ft = 2.64\ kip \cdot ft$$

Hydrostatic Force DFE Flood Acting on Shear Key, L/S:
$$F_{hyd.key.LS} := \gamma_w \cdot \left(\frac{D_{s\_key}}{2}\right) \cdot \left(\left(EL_{DFE.LS} - EL_{base.bott.LS}\right) + \left(\left(EL_{DFE.LS} - EL_{base.bott.LS}\right) + D_{s\_key}\right)\right) = 2.44\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Acting on Shear Key, L/S: (AISC Table 17-27)
$$L_{hyd.key.LS} := \frac{D_{s\_key}}{3} \frac{\left(\left(2 \cdot \left(EL_{DFE.LS} - EL_{base.bott.LS}\right) + D_{s\_key}\right) + \left(EL_{DFE.LS} - EL_{base.bott.LS}\right)\right)}{\left(\left(\left(EL_{DFE.LS} - EL_{base.bott.LS}\right) + D_{s\_key}\right) + \left(EL_{DFE.LS} - EL_{base.bott.LS}\right)\right)} = 1.06\ ft$$

Overturning Moment Due to DFE Flood Acting on Shear Key, L/S:
$$M_{o.hyd.key.LS} := F_{hyd.key.LS} \cdot L_{hyd.key.LS} \cdot 1\ ft = 2.59\ kip \cdot ft$$

Weight of Flood Water Sitting on Heel:
$$W_{water.base.heel} := \gamma_w \cdot \left(EL_{DFE.RS} - EL_{grade.RS}\right) \cdot \left(L_{heel} + \frac{L_{stem}}{2}\right) = 3.56\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Water Sitting on Heel
$$L_{w.hyd.heel} := \left(\frac{L_{heel} + \left(L_{stem} \div 2\right)}{2} + \left(L_{toe} + \left(L_{stem} \div 2\right)\right)\right) = 6.25\ ft$$

Resisting Moment Due to Weight of Flood Water on Heel:
$$M_{r.hyd.heel} := W_{water.base.heel} \cdot L_{w.hyd.heel} \cdot 1\ ft = 22.24\ kip \cdot ft$$

Weight of Flood Water Sitting on Toe:
$$W_{water.base.toe} := \gamma_w \cdot \left(EL_{DFE.LS} - EL_{grade.LS}\right) \cdot \left(L_{toe} + \frac{L_{stem}}{2}\right) = 4.48\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Water Sitting on Toe:
$$L_{w.hyd.toe} := \left(\frac{L_{toe} + \left(L_{stem} \div 2\right)}{2}\right) = 2.25\ ft$$

Resisting Moment Due to Weight of Flood Water on Toe:
$$M_{r.hyd.toe} := W_{water.base.toe} \cdot L_{w.hyd.toe} \cdot 1\ ft = 10.08\ kip \cdot ft$$

Weight of Flood Water Sitting on Footing: $W_{water.base} := W_{water.base.toe} + W_{water.base.heel} = 8.04\ \frac{kip}{ft}$

Resisting Moment Due to Weight of Flood Water on Footing: $M_{r.hyd} := M_{r.hyd.toe} + M_{r.hyd.heel} = 32.32\ kip \cdot ft$

Earth Pressure Load

Lateral Earth Pressure from River Side (DFE - 300 yr. flood))

Horizontal Earth Force Acting on Footing, R/S: $F_{soil.RS} := 0.5 \cdot K_a \cdot \gamma_{s.buoy} \cdot \left(EL_{grade.RS} - EL_{base.bott.RS}\right)^2 = \left(7.13 \cdot 10^{-3}\right)\ \frac{kip}{ft}$

Lever Arm for Horizontal Earth Force, R/S: $L_{soil.RS} := \frac{EL_{grade.RS} - EL_{base.bott.RS}}{3} = 0.33\ ft$

Earth Force Acting on Shear Key, R/S:

$$F_{soil.key.RS} := 0.5 \cdot K_a \cdot \gamma_{s.buoy} \cdot \left(\left(EL_{grade.RS} - EL_{base.bott.RS}\right) + \left(EL_{grade.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) \cdot D_{s\_key} = 0.064\ \frac{kip}{ft}$$

Lever Arm for Horizontal Earth Force Acting on Shear Key, R/S:

$$L_{key.RS} := \frac{D_{s\_key} \cdot \left(\left(2\ \left(EL_{grade.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) + \left(\left(EL_{grade.RS} - EL_{base.bott.RS}\right)\right)\right)}{3 \cdot \left(\left(\left(EL_{grade.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) + \left(EL_{grade.RS} - EL_{base.bott.RS}\right)\right)} = 0.9\ ft$$

Moments from R/S Lateral Earth Pressure: $M_{o.soil.RS} := F_{soil.RS} \cdot L_{soil.RS} \cdot 1\ ft = \left(2.38 \cdot 10^{-3}\right)\ kip \cdot ft$

$M_{r.key.RS} := F_{soil.key.RS} \cdot L_{key.RS} \cdot 1\ ft = 0.06\ kip \cdot ft$

Lateral Earth Pressure from Land Side (DFE - 300 yr. flood)

Horizontal Earth Force acting on Footing, L/S: $F_{soil.LS} := 0.5 \cdot K_p \cdot \gamma_{s.buoy} \cdot \left(EL_{base.bott.LS} - EL_{base.bott.LS}\right)^2 = 0\ \frac{kip}{ft}$

Remark: Soil replaced by roadway which does not contribute to passive resistance.

Horizontal Earth Force Acting on Shear Key, L/S: $F_{soil.key.LS} := 0.5 \cdot \left(K_p \cdot \gamma_{s.buoy} \cdot \left(D_{s\_key}\right)^2\right) = 0.46\ \frac{kip}{ft}$

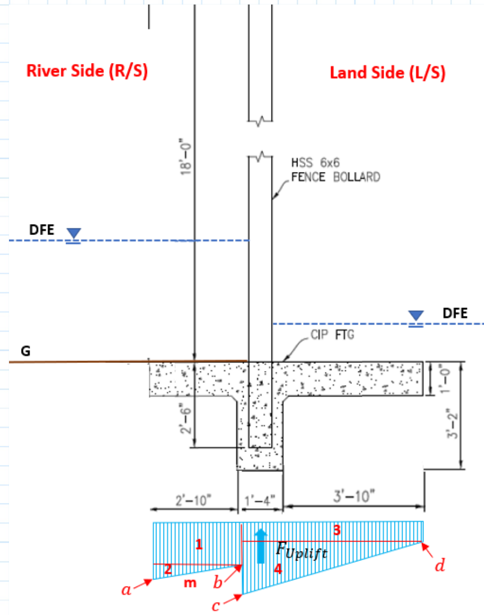
Lever Arm for Horizontal Earth Force Acting on Footing , L/S: $L_{soil.LS} := \frac{EL_{base.bott.LS} - EL_{base.bott.LS}}{3} = 0\ ft$

Lever Arm for Horizontal Earth Force Acting on Shear Key, L/S: $L_{soil.key.LS} := \frac{2\ D_{s\_key}}{3} = 1.44\ ft$

Resisting Moment from L/S Lateral Earth Pressure: $M_{r.soil.LS} := F_{soil.LS} \cdot L_{soil.LS} \cdot 1\ ft = 0\ kip \cdot ft$

Overturning Moment due to Lateral Earth Pressure on Shear Key, L/S: $M_{o.soil.key.LS} := F_{soil.key.LS} \cdot L_{soil.key.LS} \cdot 1\ ft = 0.66\ kip \cdot ft$

Uplift Load



Design Flood Elevation (DFE): $EL_{DFE.RS} = 129.03\ ft$

Depth of Water to DFE on R/S: $d_{s.RS} := EL_{DFE.RS} - EL_{base.bott.RS} = 17.29\ ft$

Depth of Water to DFE on L/S: $d_{s.LS} := EL_{DFE.LS} - EL_{base.bott.LS} = 16.96\ ft$

Slope $m := \frac{(d_{s.RS} - d_{s.LS})}{L_{base}} = 0.04$

Uplift Pressure below Heel:

$$P_{uplift.a} := \gamma_w \cdot (d_{s.RS}) \cdot ft = (1.08 \cdot 10^3)\ plf$$

$$P_{uplift.b} := P_{uplift.a} - \left(m \cdot L_{heel} \cdot \frac{P_{uplift.a}}{ft}\right) = 952.8\ plf$$

Uplift Pressure Below Shear Key:

$$P_{uplift.c} := P_{uplift.b} + \gamma_w \cdot (d_{s.RS} - D_{s\_key} + T_{base}) \cdot ft = (1.96 \cdot 10^3)\ plf$$

$$P_{uplift.d} := \gamma_w \cdot (d_{s.LS}) \cdot ft = (1.06 \cdot 10^3)\ plf$$

Uplift below Heel (Area 1+2): $V_{uplift.area.1.2} := (P_{uplift.a} + P_{uplift.b}) \cdot \frac{L_{heel}}{2} = 2.88\ kip$

Lever Arm for Uplift under the Heel: $L_{arm.area.1.2} := \frac{L_{heel} \cdot (2 \cdot P_{uplift.a} + P_{uplift.b})}{3 \cdot (P_{uplift.a} + P_{uplift.b})} + L_{s\_key} + L_{toe} = 6.61\ ft$

Overturning Moment due to Uplift below Heel: $M_{o.1.2} := V_{uplift.area.1.2} \cdot L_{arm.area.1.2} = 19.03\ kip \cdot ft$

Uplift below Shear Key and Toe (Area 3+4): $V_{uplift.area.3.4} := (P_{uplift.c} + P_{uplift.d}) \cdot \frac{L_{toe} + L_{s\_key}}{2} = 7.79\ kip$

Lever Arm for Uplift under the Shear Key and Toe : $L_{arm.area.3.4} := \frac{(L_{toe} + L_{s\_key}) \cdot (2 \cdot P_{uplift.c} + P_{uplift.d})}{3 \cdot (P_{uplift.c} + P_{uplift.d})} = 2.84\ ft$

Overturning Moment due to Uplift below Shear Key and Toe : $M_{o.3.4} := V_{uplift.area.3.4} \cdot L_{arm.area.3.4} = 22.14\ kip \cdot ft$

Overturning Moment due to Uplift: $M_{o.uplift} := M_{o.1.2} + M_{o.3.4} = 41.17\ kip \cdot ft$

Sum of Uplift: $V_{uplift} := V_{uplift.area.1.2} + V_{uplift.area.3.4} = 10.67\ kip$

Vertical Resultant Force: $V_{net} := (W_{water.base} \cdot ft + W_{total} - V_{uplift}) = -0.43\ kip$

Sum of Lateral Loads from River Side (DFE Water on R/S)

$$F_{lateral.RS} := F_{wind.RS} + F_{hyd.RS} + P_{debris} + F_{hydrodyn.} + F_{soil.RS} + F_{soil.key.RS} = 8.3 \ \frac{kip}{ft}$$

Sum of Lateral Loads from Land Side (DFE Water on L/S)

$$F_{lateral.LS} := F_{soil.LS} + F_{hyd.LS} + F_{soil.key.LS} = 7.49 \ \frac{kip}{ft}$$

Net Lateral Force:

$$F_{lateral.net} := F_{lateral.RS} - F_{lateral.LS} = 0.82 \ \frac{kip}{ft}$$

(acting in the flow direction)

Sum of Moments from Flood

$$M_{o.flood} := M_{o.debris} + M_{o.hyd.RS} + M_{o.hydn} + M_{o.uplift} + M_{o.hyd.key.LS} = 94.08 \ kip \cdot ft$$

$$M_{r.flood} := M_{r.hyd.LS} + M_{r.hyd.key.RS} + M_{r.hyd} = 74.72 \ kip \cdot ft$$

Moment from Wind

$$M_{o.wind.RS} = 0.79 \ kip \cdot ft$$

Sum of Moments from Soil

$$M_{o.soil} := M_{o.soil.RS} + M_{o.soil.key.LS} = 0.66 \ kip \cdot ft$$

$$M_{r.soil} := M_{r.key.RS} = 0.06 \ kip \cdot ft$$

Sum of Resisting Moments from Structure

$$M_{r.struct} := M_{r.Bollard} + M_{r.toe} + M_{r.key} + M_{r.heel} + M_{r.stem} = 9.34 \ kip \cdot ft$$

Sum of Overturning and Resisting Moments on Flood Wall

$$M_{o.sum} := M_{o.flood} + M_{o.soil} + M_{o.wind.RS} = 95.52 \ kip \cdot ft$$

$$M_{r.sum} := M_{r.flood} + M_{r.struct} + M_{r.soil} = 84.11 \ kip \cdot ft$$

Overturning Stability Check (With Debris Impact Load) (Not a Criteria but for informational purposes)

Overturning Factor of Safety

$$FS_{overturning} := \frac{M_{r.sum}}{M_{o.sum}} = 0.88$$

Location of Resultant Force Check (With Debris Impact Load)

Table 3-5 Requirements for Location of the Resultant – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	100% of Base in Compression	75% of Base in Compression	Resultant Within Base

EM 1110-2-2100, Table 3-5

Kern Length $Kern := \frac{L_{base}}{3} = 2.67\ ft$

Balance Moment $M_{balance} := M_{r.sum} - M_{o.sum} = -11.41\ kip \cdot ft$

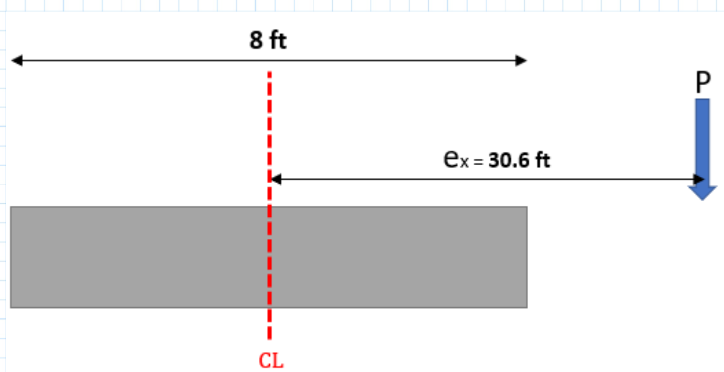
Resultant Location $x_R := \frac{M_{balance}}{V_{net}} = 26.6\ ft$

Eccentricity $e_x := \frac{L_{base}}{2} + x_R = 30.6\ ft$

Check_Resultant_Location_with_Debris_Impact :=
if $|e_x| \le \frac{Kern}{2}$
  "Resultant within the Kern"
if $\frac{Kern}{2} < |e_x| < \frac{L_{base}}{2}$
  "Resultant Outside the Kern but within the base"
else
  "Failed"

Check_Resultant_Location_with_Debris_Impact = "Failed"

Remark: The location of the resultant is outside of the base, suggesting the bollard fence base will go progressively in tension and eventually the system will overturn for the event been analyzed, unless it fails due to sliding first.



Arcadis 000356

Sliding Safety Factor Check (With Debris Impact Load)

Sum of Horizontal Load on the River Side

$F_{RS} := F_{lateral.RS} \cdot 1\ ft = 8.3\ kip$

Sum of Horizontal Load on the Land Side

$F_{LS} := F_{lateral.LS} \cdot 1\ ft = 7.49\ kip$

Cohesion

$C_{Cohesion} := C \cdot L_{base} \cdot 1 \div ft = 0$

Friction Resistance Force

$F_R := V_{net} \cdot f = -0.11\ kip$

$$FS_{Sliding} := \frac{F_R + F_{LS}}{F_{RS}} = 0.89$$

$$Sliding\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact := \left\| \begin{array}{l} \text{if } FS_{Sliding} \geq 1.2 \\ \quad \| \text{“OK, adequate safety factor”} \\ \text{else} \\ \quad \| \text{“FAILED”} \end{array} \right\| = \text{“FAILED”}$$

$Sliding\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact = \text{“FAILED”}$

Remark: The factor of safety for this event does not meet EM 1110-2-2100 Stability Criteria and being less than the unity suggest that the bollard fence system will fail for the flood event being analyzed.

Sum of Overturning and Resisting Moments on Flood Wall (Without Debris Impact Load)

$M_{o.sum.wo.debris.impact} := M_{o.sum} - M_{o.debris} = 94.25\ kip \cdot ft$

$M_{r.sum.wo.debris.impact} := M_{r.sum} = 84.11\ kip \cdot ft$

Overturning Stability Check (Not a Criteria but for informational purposes)

Overturning Factor of Safety **(Without Debris Impact Load)**

$$FS_{overturning.wo.debris.impact} := \frac{M_{r.sum.wo.debris.impact}}{M_{o.sum.wo.debris.impact}} = 0.89$$

Arcadis 000357

Location of Resultant Force Check (Without Debris Impact Load)

Kern Length $Kern := \frac{L_{base}}{3} = 2.67\ ft$

Balance Moment $M_{balance.wo.debris.impact} := M_{o.sum.wo.debris.impact} - M_{r.sum.wo.debris.impact} = 10.13\ kip \cdot ft$

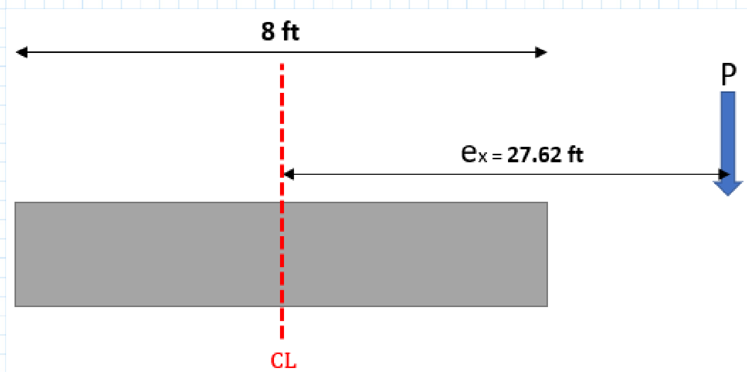
Resultant Location $x_{R.wo.debris.impact} := \frac{M_{balance.wo.debris.impact}}{V_{net}} = -23.62\ ft$

Eccentricity $e_{x.wo.debris.impact} := \frac{L_{base}}{2} - x_{R.wo.debris.impact} = 27.62\ ft$

$$Check\_Resultant\_Location\_wo\_Debris\_Impact := \begin{cases} \text{"Resultant within the Kern"} & \text{if } \left|e_{x.wo.debris.impact}\right| \le \frac{Kern}{2} \\ \text{"Resultant Outside the Kern but within the base"} & \text{if } \frac{Kern}{2} < \left|e_{x.wo.debris.impact}\right| < \frac{L_{base}}{2} \\ \text{"Failed"} & \text{else} \end{cases}$$

$Check\_Resultant\_Location\_wo\_Debris\_Impact = \text{"Failed"}$

Remark: The location of the resultant is outside of the base, suggesting the bollard fence base will go progressively in tension and eventually the system will overturn for the event been analyzed, unless it fails due to sliding first.



Sliding Safety Factor Check (Without Debris Impact Load)

Sum of Horizontal Load on the River Side **(Without Debris Impact Load)**

$$F_{RS.wo.debris.impact} := \left(F_{wind.RS} + F_{hyd.RS} + F_{hydrodyn.} + F_{soil.RS} + F_{soil.key.RS}\right) \cdot 1\ ft = 8.23\ kip$$

Sum of Horizontal Load on the Land Side

$$F_{LS} = 7.49\ kip$$

Cohesion

$$C_{Cohesion} := C \cdot L_{base} \cdot 1 \div ft = 0$$

Friction Resistance Force

$$F_R := V_{net} \cdot f = -0.11 \ \mathbf{kip}$$

$$FS_{Sliding.wo.debris.impact} := \frac{F_R + F_{LS}}{F_{RS.wo.debris.impact}} = 0.9$$

$$Sliding\_Factor\_of\_Safety\_Check\_wo\_Debris\_Impact := \begin{array}{l} \text{if } FS_{Sliding.wo.debris.impact} \geq 1.2 \\ \quad \text{"OK, adequate safety factor"} \\ \text{else} \\ \quad \text{"FAILED"} \end{array} = \text{"FAILED"}$$

$Sliding\_Factor\_of\_Safety\_Check\_wo\_Debris\_Impact = \text{"FAILED"}$

Remark: The factor of safety for this event does not meet EM 1110-2-2100 Stability Criteria and being less than the unity suggest that the bollard fence system will fail for the flood event being analyzed.

Floatation Stability Check (With Debris Impact Load)

Downward Vertical Force

$$V_{downward} := W_{water.base} \cdot \mathbf{ft} + W_{total} = 10.24 \ \mathbf{kip}$$

Upward Vertical Force

$$V_{upward} := V_{uplift} = 10.67 \ \mathbf{kip}$$

$$FS_{floatation} := \frac{V_{downward}}{V_{upward}} = 0.96$$

$$Floatation\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact := \begin{array}{l} \text{if } FS_{floatation} \geq 1.2 \\ \quad \text{"OK, adequate safety factor"} \\ \text{else} \\ \quad \text{"FAILED"} \end{array} = \text{"FAILED"}$$

$Floatation\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact = \text{"FAILED"}$

Table 3-4 Required Factors of Safety for Flotation – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	1.3	1.2	1.1

EM 1110-2-2100, Table 3-4

Bearing Pressure Check (With Debris Impact Load)

Length of the Pressure Triangle $B := 0.0001\ ft$

Effective width of the base for Bearing Pressure $L_{effective} := B = \left(1 \cdot 10^{-4}\right)\ ft$

Bearing Pressure per 1 Foot Section $Bearing_{Pressure} := \dfrac{V_{net}}{L_{effective} \cdot 1\ ft} = -4.291 \cdot 10^{3}\ ksf$

Allowable Bearing Pressure $\sigma_{bearing} = 1.5\ ksf$ *(Ref. Expert Report, Section 5)*

$Bearing\_Pressure\_Check\_with\_Debris\_Impact :=$
- if $Bearing_{Pressure} \le \sigma_{bearing}$
  - "OK, Bearing Pressure is within Allowable"
- also if $Bearing_{Pressure} < 0$
  - "Fails Due to Buoyancy"
- else
  - "FAILED"

= "Fails Due to Buoyancy"

$Bearing\_Pressure\_Check\_with\_Debris\_Impact =$ "Fails Due to Buoyancy"

Sum of Moment on a Bollard Fence

$$M_{1.bollard} := 0.6 \cdot F_{Wind} \cdot \left(\frac{(H_B - EL_{DFE.RS})}{2} + (EL_{DFE.RS} - EL_{grade.RS}) \right) + 0.75 \cdot F_{hyd.RS} \cdot 1\ ft \cdot \left(\frac{EL_{DFE.RS} - EL_{grade.RS}}{3} \right) - 0.75 \cdot F_{hyd.LS} \cdot 1\ ft \cdot \left(\frac{EL_{DFE.LS} - EL_{grade.LS}}{3} \right) + 0.75 \cdot F_i \cdot (EL_{DFE.RS} - EL_{grade.RS}) + 0.75 \cdot F_{hydrodyn} \cdot \left(\frac{H_{hydrodyn.}}{2} \right) = 61.49\ kip \cdot ft$$

Required Section Modulus $S_{x.req.1.bollard} := \frac{|M_{1.bollard}|}{0.6 \cdot F_y} = 16.4\ in^3$

Bollard Section Width $a := 6\ in$

Bollard Section Thickness $t := 0.116\ in$

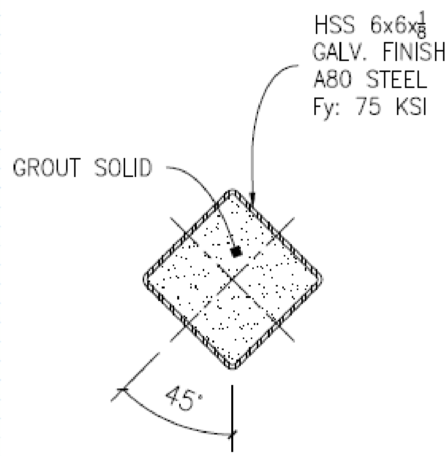
Bollard Hollow Width $b := a - (2 \cdot t) = 5.77\ in$

Section Modulus

$$S_{x.1} := \frac{a^3}{6 \cdot \sqrt{2}} = 25.46\ in^3$$

$$S_{x.2} := \frac{b^3}{6 \cdot \sqrt{2}} = 22.62\ in^3$$

AISC Construction Manual Table 17-27



BOLLARD SECTION

TGR Drawing Sheet 2 of 2

Provided Section Modulus $S_{x.prov} := S_{x.1} - S_{x.2} = 2.84\ in^3$

$One\_Bollard\_Flexural\_Capacity\_Check :=$ if $S_{x.prov} \geq S_{x.req.1.bollard}$ "OK" else "NOT OK" = "NOT OK"

$One\_Bollard\_Flexural\_Capacity\_Check =$ "NOT OK"

Assumption: If the 1,000 lbm floating debris impacts six bollards: $6 \cdot S_{x.prov} = 17.04\ in^3$

$Six\_Bollards\_Flexural\_Capacity\_Check :=$ if $6 \cdot S_{x.prov} \geq S_{x.req.1.bollard}$ "OK" else "NOT OK" = "OK"

$Six\_Bollards\_Flexural\_Capacity\_Check =$ "OK"

Remark: In order to avoid permanent deformations, the 1,000 lbm floating debris impacts would have to be distributed over 6 or more bollards.

Case C: Rising Waters Coming from Land Side

This loading condition accounts for the maximum water surface during rising water coming from the land side in the eastern segment of the bollard fence.

Elevations & Geometry (Ref. Expert Report, Section 4)

Flood Elevation (River Side): $EL_{DFE.RS} := 128.3\ ft$

Flood Elevation (Land Side): $EL_{DFE.LS} := 128.8\ ft$

Grade Elevation (River Side): $EL_{grade.RS} := 111.83\ ft$

Grade Elevation (Land Side): $EL_{grade.LS} := 111.83\ ft$

Soil Elevation (River Side): $EL_{soil.RS} := 111.83\ ft$

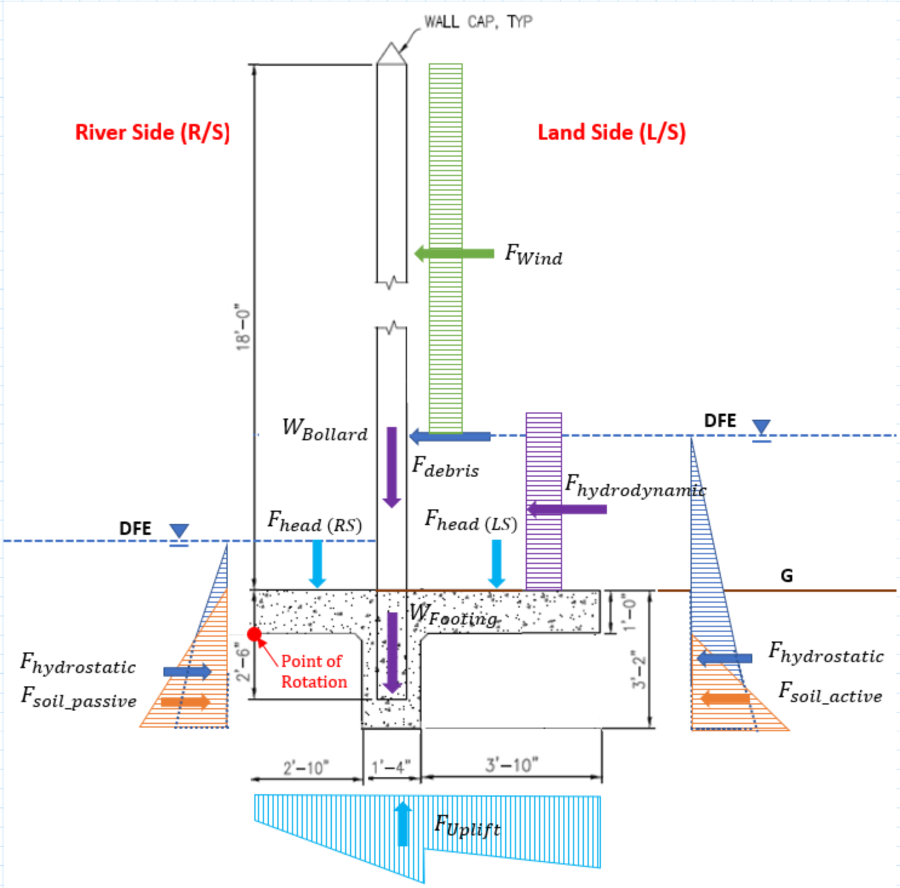
Soil Elevation (Land Side): $EL_{soil.LS} := 110.83\ ft$

Base of Footing Elevation (River Side): $EL_{base.bott.RS} := 110.83\ ft$

Base of Footing Elevation (Land Side): $EL_{base.bott.LS} := 110.83\ ft$

Shear Key Bottom Elevation: $EL_{key.bott} := 108.66\ ft$

Water Velocity $V_{water} := 6.0\ \frac{ft}{s}$



Loading Diagram of Typical Bollard Fence for Case C (Rising Waters from Land Side)

Elevations & Geometry (Ref. TGR Drawings)

Bollard Height (Above Base): $H_B = 18\ ft$

Bollard Height (Embedded): $H_{B.Embedded} = 2.5\ ft$

Bollard Thickness: $T_B = 0.13\ in$

Bollard Width (HSS6x6x1/8): $L_B = 6\ in$

Length of Toe: $L_{toe} := 2\ ft + 10\ in$

Length of Heel: $L_{heel} := 3\ ft + 10\ in$

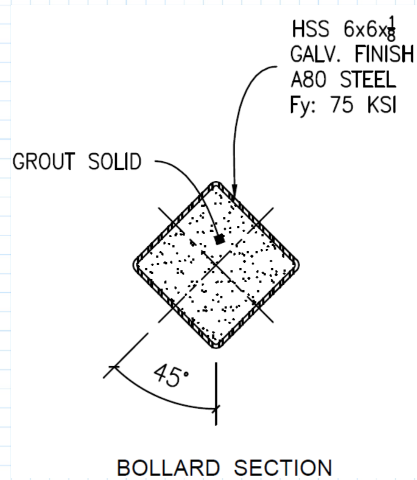
Length of Shear Key: $L_{s\_key} = 1.33\ ft$

Shear Key Depth: $D_{s\_key} = 2.17\ ft$

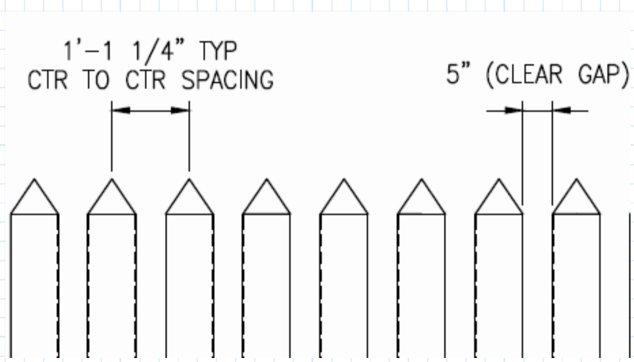
Length of Stem: $L_{stem} = 1.33\ ft$

Length of Base: $L_{base} = 8\ ft$

Base Thickness: $T_{base} = 12\ in$



TGR Drawing Sheet 2 of 2



Bollard Fence Elevation
Ref. TGR Drawing Sheet 2 of 2

Fence Imperviousness Factor below DFE (with a 30% debris blockage): $I_{imp.below.DFE} = 0.78$

Fence Imperviousness Factor above DFE (No blockage): $I_{imp.above.DFE} = 0.69$

Load Calculation

Dead Load

Bollard Cross-Section Area (HSS6x6x1/8): $A_B = 2.7\ in^2$

Bollard Weight: $W_B = 188.34\ lbf$

Bollard Fill (Above Base): $W_{B\_Fill} = 437.06\ lbf$

Bollard Total Weight (Cap weight ignored): $W_{B\_total} = 625.41\ lbf$

Base Cross-Sectional Area: $A_{base} = 8\ ft^2$

Toe Weight: $W_{toe} := L_{toe} \cdot T_{base} \cdot \gamma_c \cdot 1\ ft = 0.41\ kip$

Heel Weight: $W_{heel} := L_{heel} \cdot T_{base} \cdot \gamma_c \cdot 1\ ft = 0.56\ kip$

Stem Base Weight: $W_{stem} = 0.19\ kip$

Base Weight: $W_{Base} = \left(1.16 \cdot 10^3\right)\ lbf$

Shear Key Area: $A_{s\_key} = 2.89\ ft^2$

Shear Key Weight: $W_{s\_key} = 0.42\ kip$

Total Weight: $W_{total} = 2.2\ kip$

Moments about Toe End

Resisting Moment (Heel) $M_{r.heel} := W_{heel} \cdot \left(L_{toe} + 0.5 \cdot L_{heel} + L_{s\_key}\right) = 3.38\ kip \cdot ft$

Resisting Moment (Toe) $M_{r.toe} := W_{toe} \cdot \left(0.5 \cdot L_{toe}\right) = 0.58\ kip \cdot ft$

Resisting Moment (Bollard) $M_{r.Bollard} := W_{B\_total} \cdot \left(0.5\ L_{s\_key} + L_{toe}\right) = 2.19\ kip \cdot ft$

Resisting Moment (Stem) $M_{r.stem} := W_{stem} \cdot \left(0.5\ L_{stem} + L_{toe}\right) = 0.68\ kip \cdot ft$

Resisting Moment from Shear Key $M_{r.key} := W_{s\_key} \cdot \left(\left(0.5 \cdot L_{s\_key}\right) + L_{toe}\right) = 1.47\ kip \cdot ft$

Wind Load

Risk Category based on Use or Occupancy of Building and Other Structures: Risk Category I

Wind Speed (ASCE 7-10 Online Hazard Tool for the Project Location https://asce7hazardtool.online/): $V_{wind} := 121\ mph$

Velocity Pressure Exposure Coefficient: $K_z := 1.16$

Topographic Factor: $K_{zt} := 1.0$

Wind Direction Factor: $K_d := 0.85$

Design Wind Pressure ASCE 7-10 Eq. 27.3-1: $q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot \left(\frac{V_{wind}}{mph}\right)^2 \cdot psf = 36.96\ psf$

Wind Force from Land Side $F_{wind.LS} := q_z \cdot \left(H_B - EL_{DFE.LS} + EL_{grade.LS}\right) \cdot I_{imp.above.DFE} = 0.03\ \frac{kip}{ft}$

Moment Arm for Wind Force from Land Side $L_{wind.LS} := \frac{\left(H_B - EL_{DFE.LS} + EL_{grade.LS}\right)}{2} + \left(EL_{DFE.LS} - EL_{base.bott.LS}\right) = 18.49\ ft$

Moment due to Wind from Land Side $M_{o.wind.LS} := F_{wind.LS} \cdot L_{wind.LS} \cdot 1\ ft = 0.48\ kip \cdot ft$

Remark: Wind acting from the river side has been ignored since it will not be concurrent with land side wind.

Debris Impact Load

The debris object is assumed to be at or near the water surface level when it strikes (e.g. Stillwater elevation)

Water Velocity $V_{water} = 6\ \frac{ft}{s}$ Ref. Expert Report, Section 4

Weight of Object: $W_o := 1000\ lbf$ Ref. FEMA P-55 Section 8.5.10

Depth Coefficient (for a Floodway or Zone V): $C_D := 1.0$ Ref. FEMA P-55 Section 8.5.10 Table 8-3

Blockage Coefficient (Assumed 30% Blockage): $C_B := 1.0$ Ref. FEMA P-55 Section 8.5.10 Table 8-4

Building Structure Coefficient: $C_{Str} := 0.8$

Impact Force: $F_i := W_o \cdot V_{water} \cdot \frac{sec}{ft} \cdot C_D \cdot C_B \cdot C_{Str} = 4.8\ kip$ FEMA P-55, Section 8.5.10 Eq. 8.9

For internal stability (e.g., flexural and shear strengths) of the bollard fence the above concentrated load calculated using Eq. 8.9 from FEMA P-55 Section 8.5.10 will be used later in these calculation.

For external stability, a minimum Debris Impact load 0.1 k/ft of wall is considered, as recommended by USACE (per ASCE 7-10, Chapter C5, Special Impact Loads.)

Distributed Debris Impact Load: $P_{debris} := I_{imp.below.DFE} \cdot 0.1 \, \frac{kip}{ft} = 0.08 \, \frac{kip}{ft}$

Moment Arm to Debris Impact Load: $L_{debris} := EL_{DFE.LS} - EL_{grade.LS} = 16.97 \, ft$

Overturning Moment due to Debris Load: $M_{o.debris} := P_{debris} \cdot L_{debris} \cdot 1 \, ft = 1.33 \, kip \cdot ft$

Hydrodynamic Load

Since the velocity of water is less than 10 ft/sec, the dynamic effect of current is converted to equivalent surcharge depth dh, as per ASCE 7-10, Section 5.4.3

Coefficient for Drag or Shape Factor: $\alpha := 1.25$

Gravity $g := 32.2 \, \frac{ft}{s^2}$

Equivalent Surcharge Depth $d_h := \frac{\alpha \cdot V_{water}^{\ 2}}{2 \cdot g} = 0.7 \, ft$

Design Stillwater Depth 300 Years Flood: $d_{300yr} := EL_{DFE.LS} - EL_{grade.LS} = 16.97 \, ft$

Water Height due to Hydrodynamic Current: $H_{hydrodyn.} := d_{300yr} + d_h = 17.67 \, ft$

Hydrodynamic Force: $F_{hydrodyn.} := d_h \cdot \gamma_w \cdot H_{hydrodyn.} \cdot I_{imp.below.DFE} = 0.6 \, \frac{kip}{ft}$

Moment Arm for Hydrodynamic Load: $L_{hydrodyn.} := \frac{H_{hydrodyn.}}{2} = 8.83 \, ft$

Hydrodynamic Moment due to Flood: $M_{o.hydn} := F_{hydrodyn.} \cdot L_{hydrodyn.} \cdot 1 \, ft = 5.33 \, kip \cdot ft$

Hydrostatic Load

For water to DFE (300-yr flood), Unusual Condition

Hydrostatic Force DFE Flood Acting on Footing, L/S: $F_{hyd.LS} := 0.5 \cdot \gamma_w \cdot \left(EL_{DFE.LS} - EL_{base.bott.LS}\right)^2 \cdot I_{imp.below.DFE} = 7.9 \, \frac{kip}{ft}$

Lever Arm for DFE Flood Acting on Footing, L/S: $L_{hyd.LS} := \frac{EL_{DFE.LS} - EL_{base.bott.LS}}{3} = 5.99 \, ft$

Overturning Moment Due to DFE Flood Acting on Footing, L/S: $M_{o.hyd.LS} := F_{hyd.LS} \cdot L_{hyd.LS} \cdot 1 \, ft = 47.29 \, kip \cdot ft$

Hydrostatic Force DFE Flood Acting on Footing, R/S: $F_{hyd.RS} := 0.5 \cdot \gamma_w \cdot \left(EL_{DFE.RS} - EL_{base.bott.RS}\right)^2 \cdot I_{imp.below.DFE} = 7.46 \, \frac{kip}{ft}$

Lever Arm for DFE Flood Acting on Footing, R/S: $L_{hyd.RS} := \frac{EL_{DFE.RS} - EL_{base.bott.RS}}{3} = 5.82 \, ft$

Resisting Moment Due to DFE Flood Acting on Footing, R/S: $M_{r.hyd.RS} := F_{hyd.RS} \cdot L_{hyd.RS} \cdot 1 \, ft = 43.45 \, kip \cdot ft$

Hydrostatic Force DFE Flood acting on Shear Key, L/S: $F_{hyd.key.LS} := \gamma_w \cdot \left(\frac{D_{s\_key}}{2}\right) \cdot \left(\left(EL_{DFE.LS} - EL_{base.bott.LS}\right) + \left(\left(EL_{DFE.LS} - EL_{base.bott.LS}\right) + D_{s\_key}\right)\right) = 2.58 \, \frac{kip}{ft}$

Lever Arm for DFE Flood Acting on Shear Key, L/S: (AISC Table 17-27)

$$L_{hyd.key.LS} := \frac{D_{s\_key}}{3} \frac{\left(\left(2 \cdot \left(\left(EL_{DFE.LS} - EL_{base.bott.LS}\right)\right) + D_{s\_key}\right) + \left(EL_{DFE.LS} - EL_{base.bott.LS}\right)\right)}{\left(\left(\left(EL_{DFE.LS} - EL_{base.bott.LS}\right) + D_{s\_key}\right) + \left(EL_{DFE.LS} - EL_{base.bott.LS}\right)\right)} = 1.06\ ft$$

Resisting Moment Due to DFE Flood Acting on Shear Key, L/S:

$$M_{r.hyd.key.LS} := F_{hyd.key.LS} \cdot L_{hyd.key.LS} \cdot 1\ ft = 2.74\ kip \cdot ft$$

Hydrostatic Force DFE Flood Acting on Shear Key, R/S:

$$F_{hyd.key.LS} := \gamma_w \cdot \left(\frac{D_{s\_key}}{2}\right) \cdot \left(\left(EL_{DFE.RS} - EL_{base.bott.RS}\right) + \left(\left(EL_{DFE.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right)\right) = 2.51\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Acting on Shear Key, R/S: (AISC Table 17-27)

$$L_{hyd.key.RS} := \frac{D_{s\_key}}{3} \frac{\left(\left(2 \cdot \left(EL_{DFE.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) + \left(EL_{DFE.RS} - EL_{base.bott.RS}\right)\right)}{\left(\left(\left(EL_{DFE.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) + \left(EL_{DFE.RS} - EL_{base.bott.RS}\right)\right)} = 1.06\ ft$$

Overturning Moment Due to DFE Flood Acting on Shear Key, R/S:

$$M_{o.hyd.key.RS} := F_{hyd.key.RS} \cdot L_{hyd.key.RS} \cdot 1\ ft = 2.64\ kip \cdot ft$$

Weight of Flood Water Sitting on Heel:

$$W_{water.base.heel} := \gamma_w \cdot \left(EL_{DFE.LS} - EL_{grade.LS}\right) \cdot \left(L_{heel} + \frac{L_{stem}}{2}\right) = 4.77\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Water Sitting on Heel

$$L_{w.hyd.heel} := \left(\frac{L_{heel} + \left(L_{stem} \div 2\right)}{2} + \left(L_{toe} + \left(L_{stem} \div 2\right)\right)\right) = 5.75\ ft$$

Resisting Moment Due to Weight of Flood Water on Heel:

$$M_{r.hyd.heel} := W_{water.base.heel} \cdot L_{w.hyd.heel} \cdot 1\ ft = 27.4\ kip \cdot ft$$

Weight of Flood Water Sitting on Toe:

$$W_{water.base.toe} := \gamma_w \cdot \left(EL_{DFE.RS} - EL_{grade.RS}\right) \cdot \left(L_{toe} + \frac{L_{stem}}{2}\right) = 3.6\ \frac{kip}{ft}$$

Lever Arm for DFE Flood Water Sitting on Toe:

$$L_{w.hyd.toe} := \left(\frac{L_{toe} + \left(L_{stem} \div 2\right)}{2}\right) = 1.75\ ft$$

Resisting Moment Due to Weight of Flood Water on Toe:

$$M_{r.hyd.toe} := W_{water.base.toe} \cdot L_{w.hyd.toe} \cdot 1\ ft = 6.29\ kip \cdot ft$$

Weight of Flood Water Sitting on Footing:

$$W_{water.base} := W_{water.base.toe} + W_{water.base.heel} = 8.36\ \frac{kip}{ft}$$

Resisting Moment Due to Weight of Flood Water on Footing:

$$M_{r.hyd} := M_{r.hyd.toe} + M_{r.hyd.heel} = 33.69\ kip \cdot ft$$

Earth Pressure Load

Lateral Earth Pressure from River Side (DFE - 300 yr. flood))

Horizontal Earth Force Acting on Footing, R/S:

$$F_{soil.RS} := 0.5 \cdot K_a \cdot \gamma_{s.buoy} \cdot \left(EL_{grade.RS} - EL_{base.bott.RS}\right)^2 = \left(7.13 \cdot 10^{-3}\right)\ \frac{kip}{ft}$$

Lever Arm for Horizontal Earth Force, R/S:

$$L_{soil.RS} := \frac{EL_{grade.RS} - EL_{base.bott.RS}}{3} = 0.33\ ft$$

Earth Force Acting on Shear Key, R/S:

$$F_{soil.key.RS} := 0.5 \cdot K_a \cdot \gamma_{s.buoy} \cdot \left(\left(EL_{grade.RS} - EL_{base.bott.RS}\right) + \left(EL_{grade.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) \cdot D_{s\_key} = 0.064\ \frac{kip}{ft}$$

Lever Arm for Horizontal Earth Force Acting on Shear Key, R/S:

$$L_{key.RS} := \frac{D_{s\_key} \cdot \left(\left(2\ \left(EL_{grade.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) + \left(\left(EL_{grade.RS} - EL_{base.bott.RS}\right)\right)\right)}{3 \cdot \left(\left(\left(EL_{grade.RS} - EL_{base.bott.RS}\right) + D_{s\_key}\right) + \left(EL_{grade.RS} - EL_{base.bott.RS}\right)\right)} = 0.9\ ft$$

Moments from R/S Lateral Earth Pressure:

$$M_{o.soil.RS} := F_{soil.RS} \cdot L_{soil.RS} \cdot 1\ ft = \left(2.38 \cdot 10^{-3}\right)\ kip \cdot ft$$

$$M_{o.soil.key.RS} := F_{soil.key.RS} \cdot L_{key.RS} \cdot 1\ ft = 0.06\ kip \cdot ft$$

Lateral Earth Pressure from Land Side (DFE - 300 yr. flood)

Horizontal Earth Force acting on Footing, L/S:

$$F_{soil.LS} := 0.5 \cdot K_p \cdot \gamma_{s.buoy} \cdot \left(EL_{base.bott.LS} - EL_{base.bott.LS}\right)^2 = 0\ \frac{kip}{ft}$$

Remark: Soil replaced by roadway which does not contribute to passive resistance.

Horizontal Earth Force Acting on Shear Key, L/S:

$$F_{soil.key.LS} := 0.5 \cdot \left(K_p \cdot \gamma_{s.buoy} \cdot \left(D_{s\_key}\right)^2\right) = 0.46\ \frac{kip}{ft}$$

Lever Arm for Horizontal Earth Force Acting on Footing , L/S:

$$L_{soil.LS} := \frac{EL_{base.bott.LS} - EL_{base.bott.LS}}{3} = 0\ ft$$

Lever Arm for Horizontal Earth Force Acting on Shear Key, L/S:

$$L_{soil.key.LS} := \frac{2\ D_{s\_key}}{3} = 1.44\ ft$$

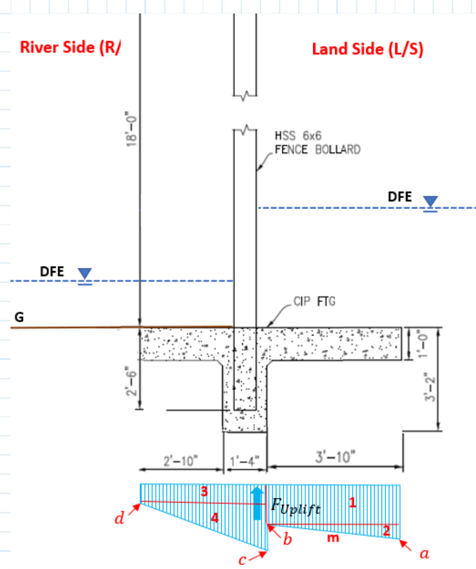
Resisting Moment from L/S Lateral Earth Pressure:

$$M_{r.soil.LS} := F_{soil.LS} \cdot L_{soil.LS} \cdot 1\ ft = 0\ kip \cdot ft$$

Resisting Moment due to Lateral Earth Pressure on Shear Key, L/S:

$$M_{r.soil.key.LS} := F_{soil.key.LS} \cdot L_{soil.key.LS} \cdot 1\ ft = 0.66\ kip \cdot ft$$

Uplift Load



Design Flood Elevation (DFE): $EL_{DFE.LS} = 128.8\ ft$

Depth of Water to DFE on L/S: $d_{s.LS} := EL_{DFE.LS} - EL_{base.bott.LS} = 17.97\ ft$

Depth of Water to DFE on R/S: $d_{s.RS} := EL_{DFE.RS} - EL_{base.bott.RS} = 17.47\ ft$

Slope

$$m := \frac{\left(d_{s.LS} - d_{s.RS}\right)}{L_{base}} = 0.06$$

Uplift Pressure below Heel:

$$P_{uplift.a} := \gamma_w \cdot \left(d_{s.LS}\right) \cdot ft = \left(1.12 \cdot 10^3\right)\ plf$$

$$P_{uplift.b} := P_{uplift.a} - \left(m \cdot L_{heel} \cdot \frac{P_{uplift.a}}{ft}\right) = 852.68\ plf$$

Uplift Pressure Below Shear Key:

$$P_{uplift.c} := P_{uplift.b} + \gamma_w \cdot (d_{s.LS} - D_{s\_key} + T_{base}) \cdot ft = (1.9 \cdot 10^3)\ plf$$

$$P_{uplift.d} := \gamma_w \cdot (d_{s.RS}) \cdot ft = (1.09 \cdot 10^3)\ plf$$

Uplift below Heel (Area 1+2): $V_{uplift.area.1.2} := (P_{uplift.a} + P_{uplift.b}) \cdot \frac{L_{heel}}{2} = 3.78\ kip$

Lever Arm for Uplift under the Heel: $L_{arm.area.1.2} := \frac{L_{heel} \cdot (2 \cdot P_{uplift.a} + P_{uplift.b})}{3 \cdot (P_{uplift.a} + P_{uplift.b})} + L_{s\_key} + L_{toe} = 6.17\ ft$

Overturning Moment due to Uplift below Heel: $M_{o.1.2} := V_{uplift.area.1.2} \cdot L_{arm.area.1.2} = 23.35\ kip \cdot ft$

Uplift below Shear Key and Toe (Area 3+4): $V_{uplift.area.3.4} := (P_{uplift.c} + P_{uplift.d}) \cdot \frac{L_{toe} + L_{s\_key}}{2} = 6.23\ kip$

Lever Arm for Uplift under the Shear Key and Toe : $L_{arm.area.3.4} := \frac{(L_{toe} + L_{s\_key}) \cdot (2 \cdot P_{uplift.c} + P_{uplift.d})}{3 \cdot (P_{uplift.c} + P_{uplift.d})} = 2.27\ ft$

Overturning Moment due to Uplift below Shear Key and Toe : $M_{o.3.4} := V_{uplift.area.3.4} \cdot L_{arm.area.3.4} = 14.16\ kip \cdot ft$

Overturning Moment due to Uplift: $M_{o.uplift} := M_{o.1.2} + M_{o.3.4} = 37.5\ kip \cdot ft$

Sum of Uplift: $V_{uplift} := V_{uplift.area.1.2} + V_{uplift.area.3.4} = 10.02\ kip$

Vertical Resultant Force: $V_{net} := (W_{water.base} \cdot ft + W_{total} - V_{uplift}) = 0.55\ kip$

Sum of Lateral Loads from Land Side (DFE Water on L/S)

$$F_{lateral.LS} := F_{wind.LS} + F_{hyd.LS} + P_{debris} + F_{hydrodyn.} + F_{soil.LS} + F_{soil.key.LS} = 9.06\ \frac{kip}{ft}$$

Sum of Lateral Loads from River Side (DFE Water on R/S)

$$F_{lateral.RS} := F_{soil.RS} + F_{hyd.RS} + F_{soil.key.RS} = 7.53\ \frac{kip}{ft}$$

Net Lateral Force:

$$F_{lateral.net} := F_{lateral.LS} - F_{lateral.RS} = 1.53\ \frac{kip}{ft}$$

(acting Land Side to River Side)

Sum of Moments from Flood

$$M_{o.flood} := M_{o.debris} + M_{o.hydn} + M_{o.uplift} + M_{o.hyd.LS} + M_{o.hyd.key.RS} = 94.1\ kip \cdot ft$$

$$M_{r.flood} := M_{r.hyd.RS} + M_{r.hyd.key.LS} + M_{r.hyd} = 79.89\ kip \cdot ft$$

Moment from Wind $M_{o.wind.LS} = 0.48\ kip \cdot ft$

Sum of Moments from Soil

$$M_{o.soil} := M_{o.soil.RS} + M_{o.soil.key.RS} = 0.06\ \boldsymbol{kip \cdot ft}$$

$$M_{r.soil} := M_{r.soil.key.LS} + M_{r.soil.LS} = 0.66\ \boldsymbol{kip \cdot ft}$$

Sum of Resisting Moments from Structure

$$M_{r.struct} := M_{r.Bollard} + M_{r.toe} + M_{r.key} + M_{r.heel} + M_{r.stem} = 8.3\ \boldsymbol{kip \cdot ft}$$

Sum of Overturning and Resisting Moments on Flood Wall

$$M_{o.sum} := M_{o.flood} + M_{o.soil} + M_{o.wind.LS} = 94.64\ \boldsymbol{kip \cdot ft}$$

$$M_{r.sum} := M_{r.flood} + M_{r.struct} + M_{r.soil} = 88.84\ \boldsymbol{kip \cdot ft}$$

Overturning Stability Check (With Debris Impact Load) (Not a Criteria but for informational purposes)

Overturning Factor of Safety

$$FS_{overturning} := \frac{M_{r.sum}}{M_{o.sum}} = 0.94$$

Location of Resultant Force Check (With Debris Impact Load)

Table 3-5 Requirements for Location of the Resultant – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	100% of Base in Compression	75% of Base in Compression	Resultant Within Base

EM 1110-2-2100, Table 3-5

Kern Length $Kern := \frac{L_{base}}{3} = 2.67\ ft$

Balance Moment $M_{balance} := M_{r.sum} - M_{o.sum} = -5.8\ kip \cdot ft$

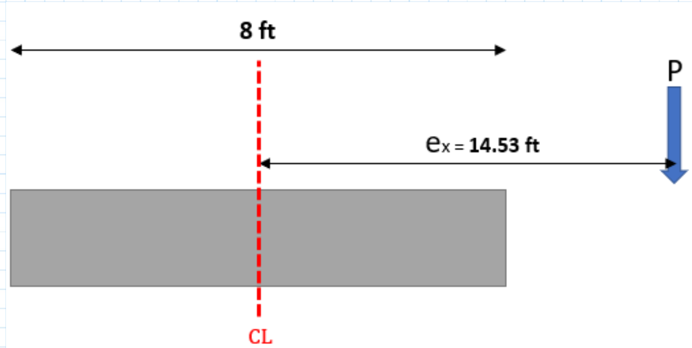
Resultant Location $x_R := \frac{M_{balance}}{V_{net}} = -10.53\ ft$

Eccentricity $e_x := \frac{L_{base}}{2} - x_R = 14.53\ ft$

$$Check\_Resultant\_Location\_with\_Debris\_Impact := \begin{array}{|l} \text{if } |e_x| \le \frac{Kern}{2} \\ \quad \| \text{"Resultant within the Kern"} \\ \text{if } \frac{Kern}{2} < |e_x| < \frac{L_{base}}{2} \\ \quad \| \text{"Resultant Outside the Kern but within the base"} \\ \text{else} \\ \quad \| \text{"Failed"} \end{array}$$

$Check\_Resultant\_Location\_with\_Debris\_Impact = \text{"Failed"}$

Remark: The location of the resultant is outside of the base, suggesting the bollard fence base will go progressively in tension and eventually the system will overturn for the event been analyzed, unless it fails due to sliding first.



Sliding Safety Factor Check (With Debris Impact Load)

Sum of Horizontal Load on the River Side

$F_{RS} := F_{lateral.RS} \cdot 1\ ft = 7.53\ kip$

Sum of Horizontal Load on the Land Side

$F_{LS} := F_{lateral.LS} \cdot 1\ ft = 9.06\ kip$

Cohesion

$C_{Cohesion} := C \cdot L_{base} \cdot 1 \div ft = 0$

Friction Resistance Force

$F_R := V_{net} \cdot f = 0.14\ kip$

$$FS_{Sliding} := \frac{F_R + F_{RS}}{F_{LS}} = 0.85$$

Sliding_Factor_of_Safety_Check_with_Debris_Impact := if $FS_{Sliding} \geq 1.2$ "OK, adequate safety factor" else "FAILED" = "FAILED"

Sliding_Factor_of_Safety_Check_with_Debris_Impact = "FAILED"

Sum of Overturning and Resisting Moments on Flood Wall (Without Debris Impact Load)

$M_{o.sum.wo.debris.impact} := M_{o.sum} - M_{o.debris} = 93.31\ kip \cdot ft$

$M_{r.sum.wo.debris.impact} := M_{r.sum} = 88.84\ kip \cdot ft$

Overturning Stability Check (Not a Criteria but for informational purposes)

Overturning Factor of Safety **(Without Debris Impact Load)**

$$FS_{overturning.wo.debris.impact} := \frac{M_{r.sum.wo.debris.impact}}{M_{o.sum.wo.debris.impact}} = 0.95$$

Location of Resultant Force Check (Without Debris Impact Load)

Kern Length $\quad Kern := \frac{L_{base}}{3} = 2.67\ ft$

Balance Moment $\quad M_{balance.wo.debris.impact} := M_{r.sum.wo.debris.impact} - M_{o.sum.wo.debris.impact} = -4.47\ kip \cdot ft$

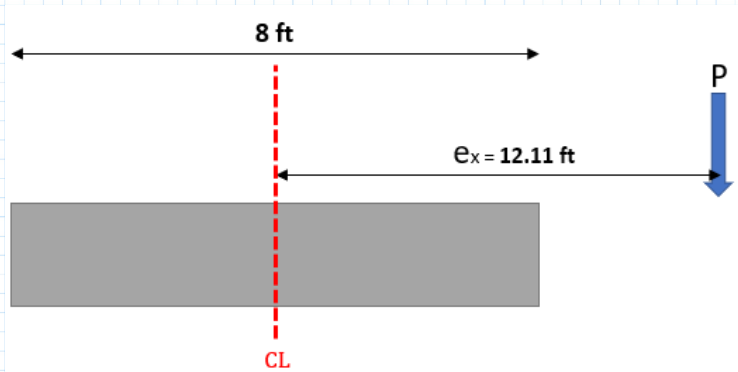
Resultant Location $\quad x_{R.wo.debris.impact} := \frac{M_{balance.wo.debris.impact}}{V_{net}} = -8.11\ ft$

Eccentricity $e_{x.wo.debris.impact} := \frac{L_{base}}{2} - x_{R.wo.debris.impact} = 12.11\ ft$

$$Check\_Resultant\_Location\_wo\_Debris\_Impact := \begin{array}{l} \text{if } \left|e_{x.wo.debris.impact}\right| \le \frac{Kern}{2} \\ \quad \text{“Resultant within the Kern”} \\ \text{if } \frac{Kern}{2} < \left|e_{x.wo.debris.impact}\right| < \frac{L_{base}}{2} \\ \quad \text{“Resultant Outside the Kern but within the base”} \\ \text{else} \\ \quad \text{“Failed”} \end{array}$$

$Check\_Resultant\_Location\_wo\_Debris\_Impact = \text{“Failed”}$

Remark: The location of the resultant is outside of the base, suggesting the bollard fence base will go progressively in tension and eventually the system will overturn for the event been analyzed, unless it fails due to sliding first.



Sliding Safety Factor Check (Without Debris Impact Load)

Sum of Horizontal Load on the Land Side **(Without Debris Impact Load)**

$$F_{LS.wo.debris.impact} := \left(F_{lateral.LS} - P_{debris}\right) \cdot 1\ ft = 8.98\ kip$$

Sum of Horizontal Load on the River Side

$$F_{RS} = 7.53\ kip$$

Cohesion

$$C_{Cohesion} := C \cdot L_{base} \cdot 1 \div ft = 0$$

Friction Resistance Force

$$F_R := V_{net} \cdot f = 0.14\ kip$$

$$FS_{Sliding.wo.debris.impact} := \frac{F_R + F_{RS}}{F_{LS.wo.debris.impact}} = 0.85$$

$$Sliding\_Factor\_of\_Safety\_Check\_wo\_Debris\_Impact := \begin{array}{l} \text{if } FS_{Sliding.wo.debris.impact} \geq 1.2 \\ \quad \text{"OK, adequate safety factor"} \\ \text{else} \\ \quad \text{"FAILED"} \end{array} = \text{"FAILED"}$$

$Sliding\_Factor\_of\_Safety\_Check\_wo\_Debris\_Impact = \text{"FAILED"}$

Remark: The factor of safety for this event does not meet EM 1110-2-2100 Stability Criteria and being less than the unity suggest that the bollard fence system will fail for the flood event being analyzed.

Floatation Stability Check (With Debris Impact Load)

Downward Vertical Force

$$V_{downward} := W_{water.base} \cdot ft + W_{total} = 10.57 \; kip$$

Upward Vertical Force

$$V_{upward} := V_{uplift} = 10.02 \; kip$$

$$FS_{floatation} := \frac{V_{downward}}{V_{upward}} = 1.06$$

$$Floatation\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact := \begin{array}{l} \text{if } FS_{floatation} \geq 1.2 \\ \quad \text{"OK, adequate safety factor"} \\ \text{else} \\ \quad \text{"FAILED"} \end{array} = \text{"FAILED"}$$

$Floatation\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact = \text{"FAILED"}$

Table 3-4 Required Factors of Safety for Flotation – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	1.3	1.2	1.1

EM 1110-2-2100, Table 3-4

Bearing Pressure Check (With Debris Impact Load)

Length of the Pressure Triangle $B := -0.0001\ ft$

Effective width of the base for Bearing Pressure $L_{effective} := B = -1 \cdot 10^{-4}\ ft$

Bearing Pressure per 1 Foot Section $Bearing_{Pressure} := \frac{V_{net}}{L_{effective} \cdot 1\ ft} = -5.511 \cdot 10^{3}\ ksf$

Allowable Bearing Pressure $\sigma_{bearing} = 1.5\ ksf$ *(Ref. Expert Report, Section 5)*

$Bearing\_Pressure\_Check\_with\_Debris\_Impact :=$
```
if Bearing_Pressure ≤ σ_bearing
    "OK, Bearing Pressure is within Allowable"
also if Bearing_Pressure < 0
    "Fails Due to Buoyancy"
else
    "FAILED"
```
= "Fails Due to Buoyancy"

$Bearing\_Pressure\_Check\_with\_Debris\_Impact =$ "Fails Due to Buoyancy"

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Job Information

	Engineer	Checked	Approved
Name:	MM		
Date:	22-Jul-21		

Project ID	
Project Name	

Structure Type	SPACE FRAME

Number of Nodes	17	Highest Node	17
Number of Elements	16	Highest Beam	16

Number of Basic Load Cases	1
Number of Combination Load Cases	0

Included in this printout are data for:

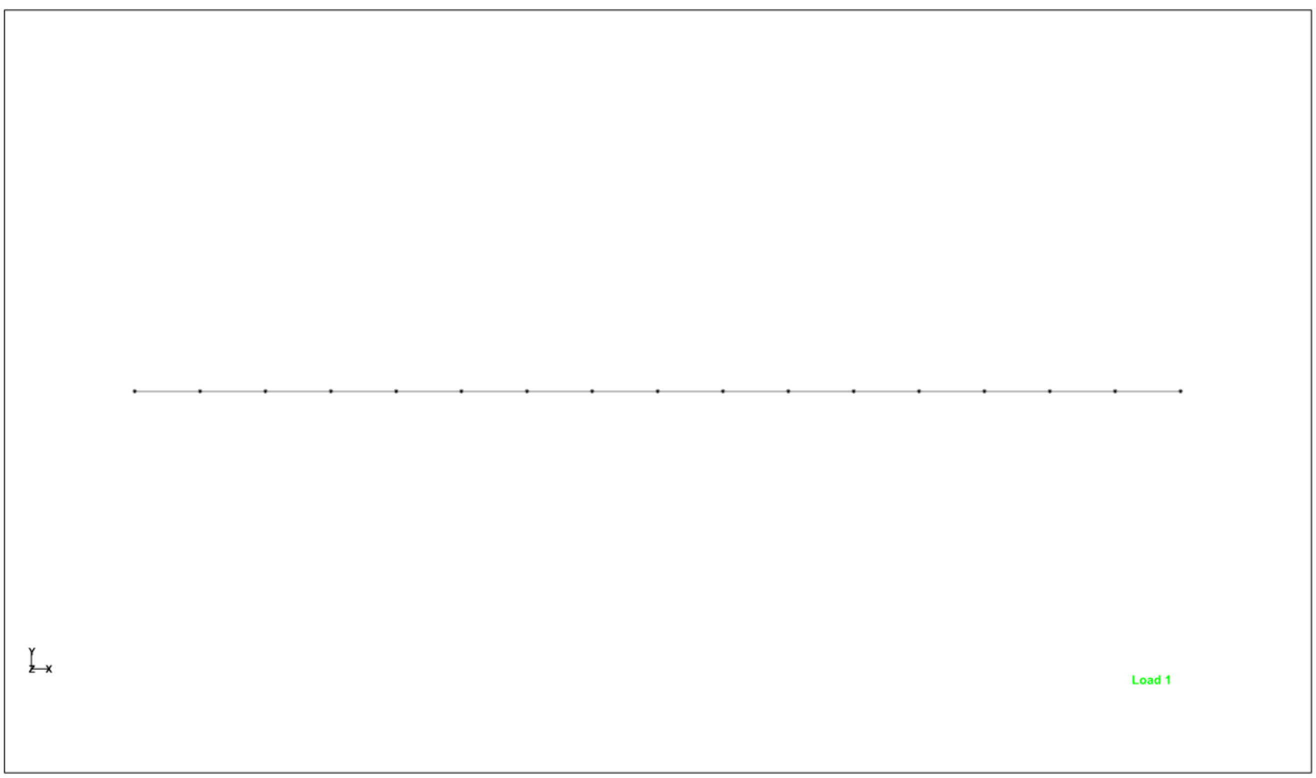
All	The Whole Structure

Nodes

Node	X (ft)	Y (ft)	Z (ft)
1	0	0	0
2	8.000	0	0
3	0.500	0	0
4	1.000	0	0
5	1.500	0	0
6	2.000	0	0
7	2.500	0	0
8	3.000	0	0
9	3.500	0	0
10	4.000	0	0
11	4.500	0	0
12	5.000	0	0
13	5.500	0	0
14	6.000	0	0
15	6.500	0	0
16	7.000	0	0
17	7.500	0	0
18	5.350	0	0

Beams

Beam	Node A	Node B	Length (ft)	Property	β (degrees)
1	1	3	0.500	1	0
2	3	4	0.500	1	0
3	4	5	0.500	1	0
4	5	6	0.500	1	0
5	6	7	0.500	1	0
6	7	8	0.500	1	0
7	8	9	0.500	1	0
8	9	10	0.500	1	0
9	10	11	0.500	1	0
10	11	12	0.500	1	0
11	12	13	0.500	1	0
12	13	14	0.500	1	0
13	14	15	0.500	1	0
14	15	16	0.500	1	0
15	16	17	0.500	1	0
16	17	2	0.500	1	0



Whole Structure

Materials

Mat	Name	E (kip/in^2)	ν	Density (kip/in^3)	α (/°F)
1	CONCRETE	3.15E+3	0.170	8.68e-05	5.5E -6

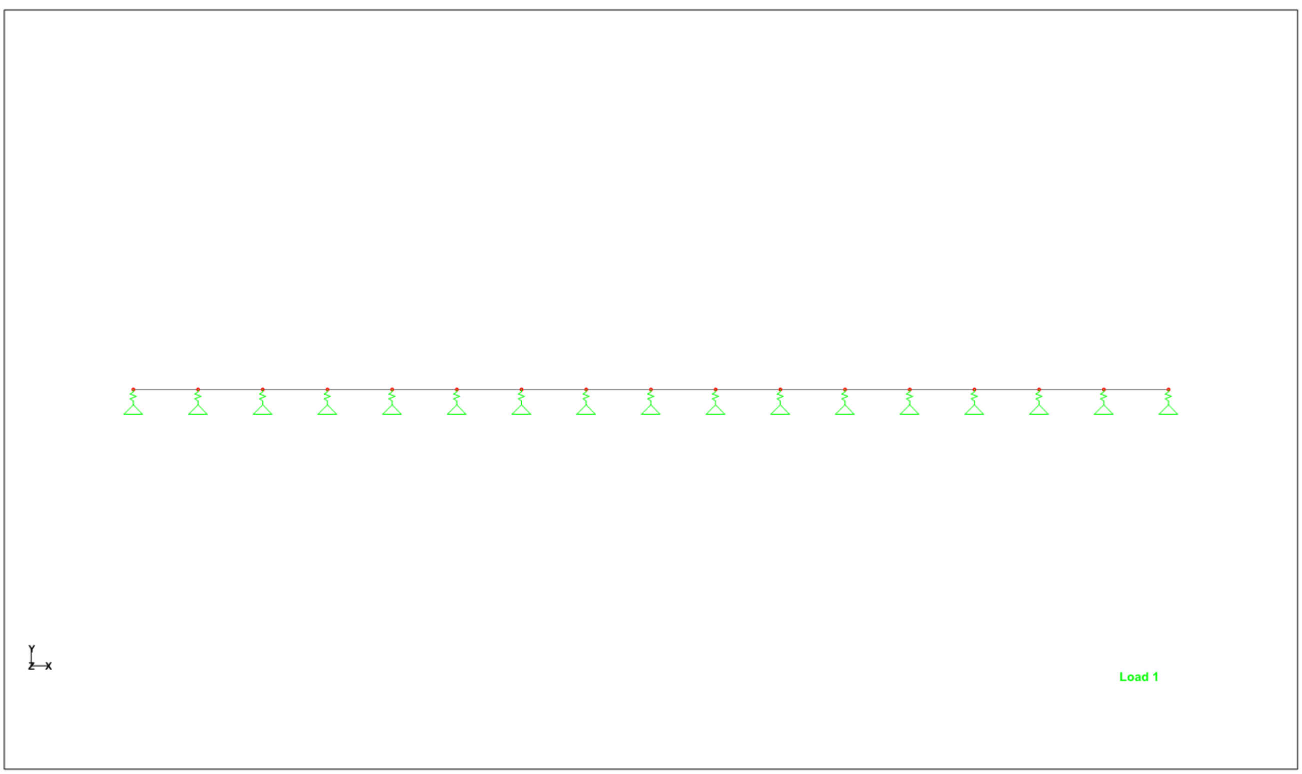
Material

Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip˜ft/deg)	rY (kip˜ft/deg)	rZ (kip˜ft/deg)
1	-	7.188	-	-	-	-
2	-	7.188	-	-	-	-
3	-	7.188	-	-	-	-
4	-	7.188	-	-	-	-
5	-	7.188	-	-	-	-
6	-	7.188	-	-	-	-
7	-	7.188	-	-	-	-
8	-	7.188	-	-	-	-
9	-	7.188	-	-	-	-
10	-	7.188	-	-	-	-
11	-	7.188	-	-	-	-
12	-	7.188	-	-	-	-
13	-	7.188	-	-	-	-
14	-	7.188	-	-	-	-
15	-	7.188	-	-	-	-
16	-	7.188	-	-	-	-
17	-	7.188	-	-	-	-

Coefficient of subgrade modulus k=100 pci was recommended by Geotechnical Discipline Expert.

Spring value = coefficient of subgrade modulus(k) x spring spacing x 12in/ft_ foundation = kx6"x12" = 7.188 kip/in.



Springs

Primary Load Cases

Number	Name	Type
1	LOAD CASE 1	None

1 LOAD CASE 1 : Node Loads

Node	FX (kip)	FY (kip)	FZ (kip)	MX (kip`ft)	MY (kip`ft)	MZ (kip`ft)
18	-	-1.990	-	-	-	-

Remark: P=1.99 kip at ultimate level (factored). See p. 18 of 50 of Bollard Fence Assessment calculations.

1 LOAD CASE 1 : Beam Loads

Beam	Type		Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
1	TRAP	lbf/ft	GY	0	0.04	20.000	0.333	-
	TRAP	lbf/ft	GY	20.000	0.333	25.000	0.500	-
2	TRAP	lbf/ft	GY	25.000	0	30.000	0.167	-
	TRAP	lbf/ft	GY	30.000	0.167	40.000	0.500	-
3	TRAP	lbf/ft	GY	49.550	0	71.000	0.333	-
	TRAP	lbf/ft	GY	71.000	0.333	81.725	0.500	-
4	TRAP	lbf/ft	GY	81.725	0	92.450	0.167	-
	TRAP	lbf/ft	GY	92.450	0.167	113.900	0.500	-
5	TRAP	lbf/ft	GY	113.900	0	113.900	0.000	-
	TRAP	lbf/ft	GY	113.900	0	135.350	0.333	-
	TRAP	lbf/ft	GY	135.350	0.333	146.075	0.500	-
6	TRAP	lbf/ft	GY	146.075	0	156.800	0.167	-
	TRAP	lbf/ft	GY	156.800	0.167	178.250	0.500	-
7	TRAP	lbf/ft	GY	178.250	0	178.250	0.000	-
	TRAP	lbf/ft	GY	178.250	0	199.700	0.333	-
	TRAP	lbf/ft	GY	199.700	0.333	210.425	0.500	-
8	TRAP	lbf/ft	GY	210.425	0	221.150	0.167	-
	TRAP	lbf/ft	GY	221.150	0.167	242.600	0.500	-
9	TRAP	lbf/ft	GY	242.600	0	242.600	0.000	-
	TRAP	lbf/ft	GY	242.600	0	264.050	0.333	-
	TRAP	lbf/ft	GY	264.050	0.333	274.775	0.500	-
10	TRAP	lbf/ft	GY	274.775	0	285.500	0.167	-
	TRAP	lbf/ft	GY	285.500	0.167	306.950	0.500	-
11	TRAP	lbf/ft	GY	306.950	0	306.950	0.000	-
	TRAP	lbf/ft	GY	306.950	0	328.400	0.333	-
	TRAP	lbf/ft	GY	328.400	0.333	339.125	0.500	-
12	TRAP	lbf/ft	GY	339.125	0	349.850	0.167	-
	TRAP	lbf/ft	GY	349.850	0.167	371.300	0.500	-
13	TRAP	lbf/ft	GY	371.300	0	371.300	0.000	-
	TRAP	lbf/ft	GY	371.300	0	392.750	0.333	-
	TRAP	lbf/ft	GY	392.750	0.333	403.475	0.500	-
14	TRAP	lbf/ft	GY	403.475	0	414.200	0.167	-
	TRAP	lbf/ft	GY	414.200	0.167	435.650	0.500	-
15	TRAP	lbf/ft	GY	435.650	0	435.650	0.000	-
	TRAP	lbf/ft	GY	435.650	0	457.100	0.333	-
	TRAP	lbf/ft	GY	457.100	0.333	467.825	0.500	-
16	TRAP	lbf/ft	GY	467.825	0	478.550	0.167	-
	TRAP	lbf/ft	GY	478.550	0.167	500.000	0.500	-

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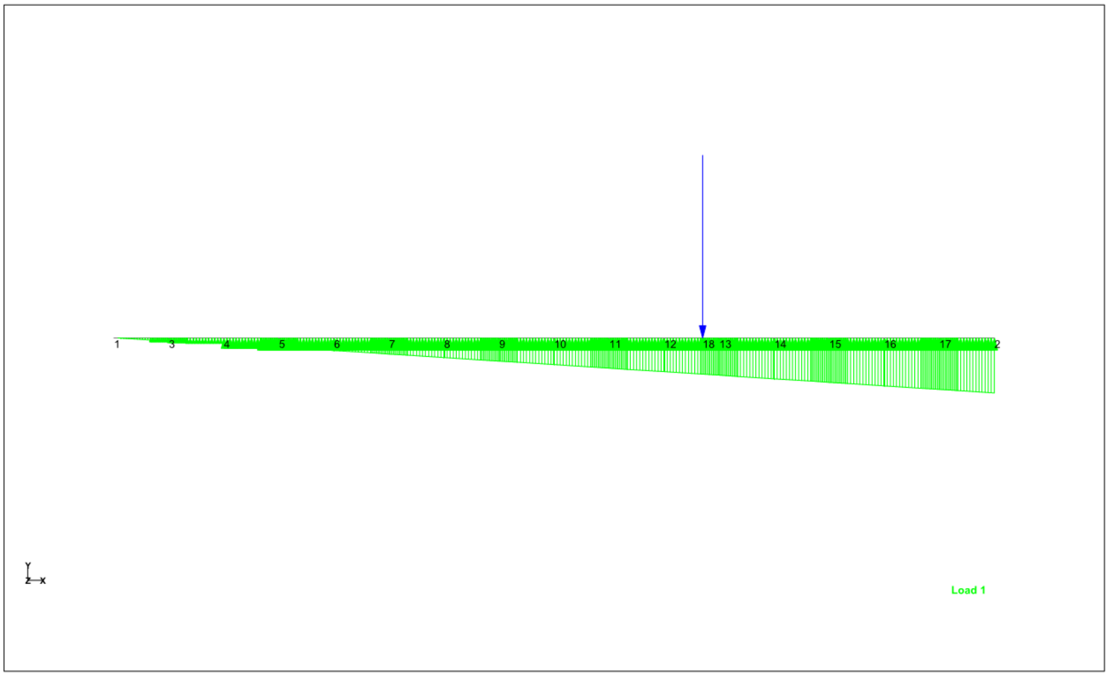
	Part		
Job Title Bollard Fence Foundation - Shear Forces and Bending Moments	Ref		
	By MM	Date 22-Jul-21	Chd
Client	File Structure.STD	Date/Time 17-Aug-2021 11:38	



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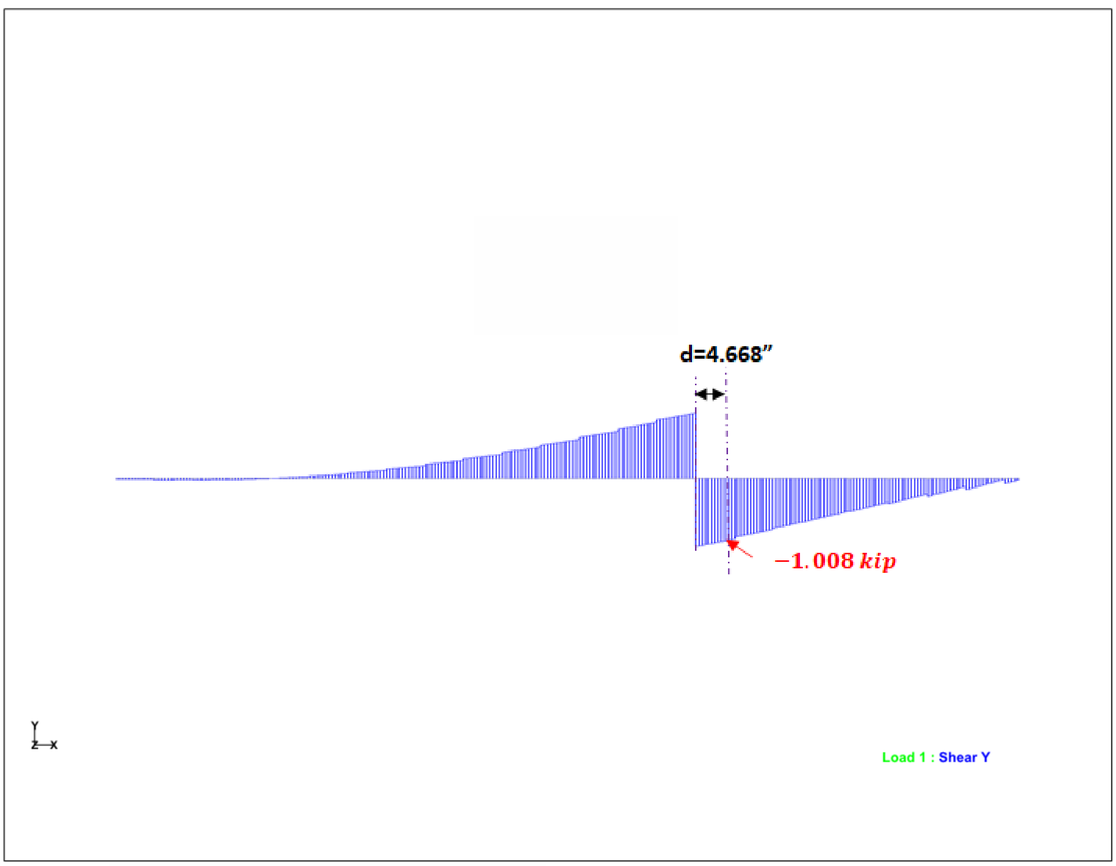

Shear Force

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	Part		
Job Title Bollard Fence Foundation - Shear Forces and Bending Moments	Ref		
	By MM	Date 22-Jul-21	Chd
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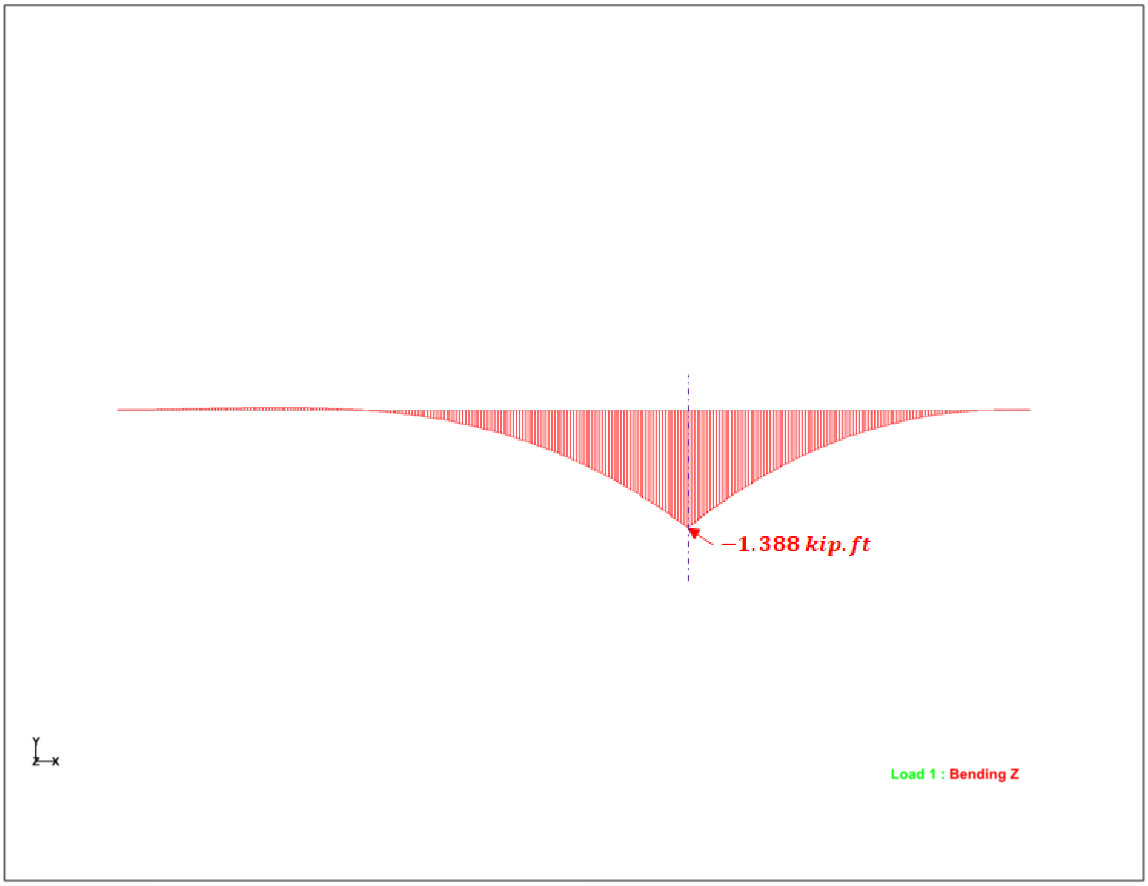


Bending Moment

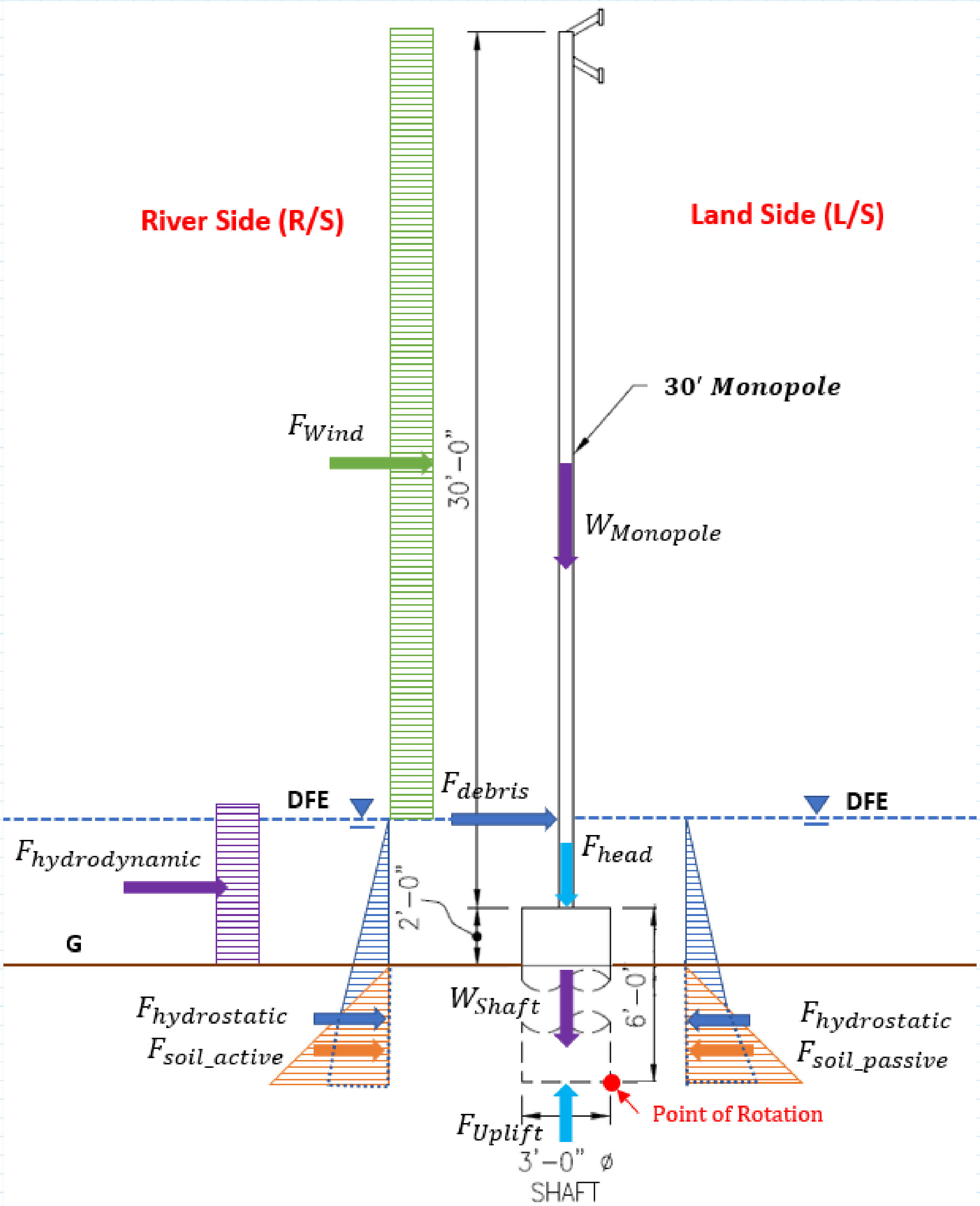
The following computation is aimed at investigating the external stability of the existing monopole during rising waters for the 300-yr flood event along the Rio Grande River near McAllen, Texas.

The following stability criteria is followed:

Loading Condition: Unusual Event (300-yr flood)
Location of Resultant: 75% of Base in Compression
Minimum Sliding F.S.: 1.2 — EM-1110-2-2100 (Table 3-3)
Minimum Floatation F.S.: 1.2 — EM-1110-2-2100 (Table 3-4)

References:

USACE, EM-1110-2-2100 Stability Analysis of Concrete Structure.
ASCE/SEI 7-10, Minimum Design Loads for Buildings and Other Structures.
FEMA P-55, Coastal Construction Manual.



Loading Diagram of Typical Monopole for Case A (Rising Waters from River Side)

General Inputs

Material Properties

Water Unit Weight: $\gamma_w := 62.4 \cdot pcf$

Concrete Unit Weight (assumed for a concrete slightly reinforced) $\gamma_c := 145.0 \cdot pcf$

Soil Unit Weight $\gamma_s := 115.0 \cdot pcf$ *(Ref. Expert Report, Section 5)*

Unit Weight of Buoyant Soil $\gamma_{s.buoy} := \gamma_s - \gamma_w = 52.6\ pcf$

Angle of Internal Friction $\phi := 35\ °$ *(Ref. Expert Report, Section 5)*

Soil Cohesion $C := 0$

Allowable Bearing Capacity of Soil $\sigma_{bearing} := 1500\ psf$ *(Ref. Expert Report, Section 5)*

Coefficient of Friction with Concrete $f := 0.25$ *(Ref. Expert Report, Section 5)*

Active Earth Pressure Coefficient $K_a := \dfrac{(1 - \sin(\phi))}{(1 + \sin(\phi))} = 0.271$ $K_a = 0.271$

Passive Earth Pressure Coefficient $K_p := \dfrac{(1 + \sin(\phi))}{(1 - \sin(\phi))} = 3.69$ $K_p = 3.69$

Case A: Rising Waters Coming from River Side

Elevations & Geometry (Ref. Expert Report, Section 4)

Flood Elevation (River Side): $EL_{DFE.RS} := 113.7\ ft$

Flood Elevation (Land Side): $EL_{DFE.LS} := 113.7\ ft$

Grade Elevation at Monopole: $EL_{grade.monopole} := 110.0\ ft$

Base of Shaft Elevation: $EL_{base.shaft.bott} := EL_{grade.monopole} - 4\ ft = 106\ ft$

Water Velocity $V_{water} := 7.9\ \dfrac{ft}{s}$

Elevations & Geometry (Ref. TGR Drawings)

Monopole Height (above shaft): $H_{mono} := 30 \cdot ft$

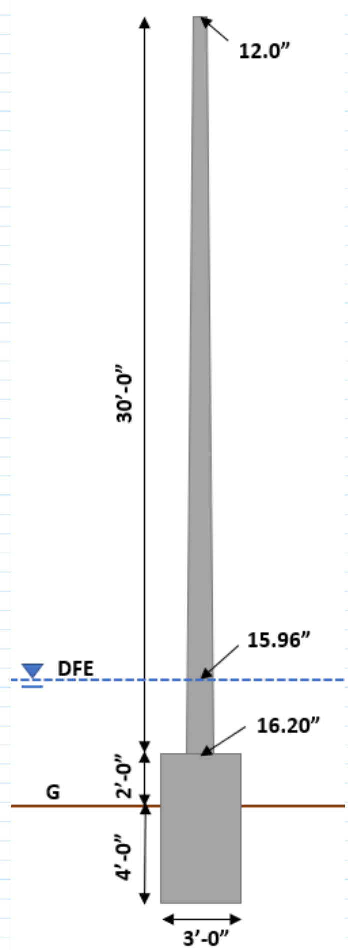
Outer Diameter at Tip: $D_{o.tip} := 12.0\ in$

Outer Diameter at Base: $D_o := 16.20\ in$

Outer Diameter at DFE (R/S): $D_{o.DFE.RS} := 15.96\ in$

Foundation Shaft Depth: $H_{shaft} := 6\ ft$

Foundation Shaft Diameter: $D_{shaft} := 3\ ft$



Physical Properties for 30-ft Direct Embed Steel Poles

	Light	Medium	Heavy
TESSCO SKU	331020	351929	341210
Design Number	T30LA	T30MA	T30HA
Tip OD, in (cm)	6.50 (16.51)	9.00 (22.86)	12.00 (30.48)
OD @ Grade, in (cm)	10.70 (27.18)	13.20 (33.53)	16.20 (41.15)
Butt OD, in (cm)	12.10 (30.73)	14.74 (37.45)	18.02 (45.77)
Number of Sides	12	16	18
Δ Dia, in/ft (cm/m)	0.14 (1.17)	0.14 (1.17)	0.14 (1.17)
Side Taper, in/ft (cm/m)	0.07 (0.58)	0.07 (0.58)	0.07 (0.58)
Embedment, ft (m)	10.00 (3.05)	16.00 (4.88)	13.00 (3.96)
Auger Dia, ft (m)	2.50 (0.76)	2.50 (0.76)	3.00 (0.91)
Backfill Type	Aggregate	Aggregate	Aggregate
Total Length, ft (m)	40 (12.19)	41 (12.49)	43 (13.10)
Bare Pole Wt, lbs (kg)	793 (360)	1,031 (468)	1,368 (621)
No. of Sections	1	1	1

TESSCO Monopole Geometry and Properties
Remark: Similar to existing monopole installed by TGR

Load Calculation

Dead Load

Monopole Weight: $W_{mono} := 1368\ lbf$

Shaft Cross-Sectional Area: $A_{shaft} := \frac{\pi \cdot D_{shaft}^2}{4} = 7.069\ ft^2$

Monopole Cross-Sectional Area at Base: $A_{mono.base} := \frac{\pi \cdot D_o^2}{4} = 1.431\ ft^2$

Monopole Cross-Sectional Area at DFE: $A_{mono.DFE} := \frac{\pi \cdot D_{o.DFE.RS}^2}{4} = 1.389\ ft^2$

Weight of Concrete Shaft: $W_{shaft} := A_{shaft} \cdot H_{shaft} \cdot \gamma_c = 6.15\ kip$

Total Weight: $W_{total} := W_{mono} + W_{shaft} = 7.518\ kip$

Moments about Middle of the Shaft

Resisting Moment (Monopole) $M_{r.mono} := W_{mono} \cdot \left(\frac{D_{shaft}}{2}\right) = 2.052\ kip \cdot ft$

Resisting Moment (Shaft) $M_{r.shaft} := W_{shaft} \cdot \left(\frac{D_{shaft}}{2}\right) = 9.225\ kip \cdot ft$

Wind Load

Risk Category based on Use or Occupancy of Building and Other Structures: Risk Category I

Wind Speed (ASCE 7-10 Online Hazard Tool for the Project Location https://asce7hazardtool.online/): $V_{wind} := 121\ mph$

Velocity Pressure Exposure Coefficient: $K_z := 1.16$

Topographic Factor: $K_{zt} := 1.0$

Wind Direction Factor: $K_d := 0.85$

Design Wind Pressure ASCE 7-10 Eq. 27.3-1: $q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot \left(\frac{V_{wind}}{mph}\right)^2 \cdot psf = 36.96\ psf$

Wind Force from River Side $F_{wind.RS} := q_z \cdot \left(H_{mono} - EL_{DFE.RS} + EL_{grade.monopole} + 2\ ft\right) \cdot \left(\frac{D_{o.tip} + D_{o.DFE.RS}}{2}\right) = 1.218\ kip$

Moment Arm for Wind Force from River Side

$$L_{wind.RS} := \frac{\left(H_{mono} - EL_{DFE.RS} + EL_{grade.monopole} + 2\ ft\right)}{2} + \left(EL_{DFE.RS} - EL_{base.shaft.bott}\right) = 21.85\ ft$$

Moment due to Wind from River Side $M_{o.wind.RS} := F_{wind.RS} \cdot L_{wind.RS} = 26.623\ kip \cdot ft$

Remark: Wind acting from the land side has been ignored since it will not be concurrent with river side wind.

Debris Impact Load

The debris object is assumed to be at or near the water surface level when it strikes (e.g. Stillwater elevation)

Water Velocity $V_{water} = 7.9\ \frac{ft}{s}$ Ref. Expert Report, Section 4

Weight of Object: $W_o := 1000\ lbf$ Ref. FEMA P-55 Section 8.5.10

Depth Coefficient (for a Floodway or Zone V): $C_D := 1.0$ Ref. FEMA P-55 Section 8.5.10 Table 8-3

Blockage Coefficient (Assumed 30% Blockage): $C_B := 1.0$ Ref. FEMA P-55 Section 8.5.10 Table 8-4

Building Structure Coefficient: $C_{Str} := 0.8$

Impact Force: $F_i := W_o \cdot V_{water} \cdot \frac{sec}{ft} \cdot C_D \cdot C_B \cdot C_{Str} = 6.32\ kip$ FEMA P-55, Section 8.5.10 Eq. 8.9

For external stability, a minimum Debris Impact load 0.1 k/ft of wall is considered, as recommended by USACE (per ASCE 7-10, Chapter C5, Special Impact Loads.)

Distributed Debris Impact Load: $P_{debris} := 0.1\ \frac{kip}{ft} \cdot D_o = 0.135\ kip$

Moment Arm to Debris Impact Load: $L_{debris} := EL_{DFE.RS} - EL_{base.shaft.bott} = 7.7\ ft$

Overturning Moment due to Debris Load: $M_{o.debris} := P_{debris} \cdot L_{debris} = 1.04\ kip \cdot ft$

Hydrodynamic Load

Since the velocity of water is less than 10 ft/sec, the dynamic effect of current is converted to equivalent surcharge depth dh, as per ASCE 7-16, Cl. 5.4.3.4

Coefficient for Drag or Shape Factor: $\alpha := 1.25$

Gravity $g := 32.2\ \frac{ft}{s^2}$

Equivalent Surcharge Depth $d_h := \frac{\alpha \cdot V_{water}{}^2}{2 \cdot g} = 1.211\ ft$

Design Stillwater Depth 300 Years Flood: $d_{300yr} := EL_{DFE.RS} - EL_{grade.monopole} = 3.7\ ft$

Water Height due to Hydrodynamic Current: $H_{hydrodyn.} := d_{300yr} + d_h = 4.911\ ft$

Hydrodynamic Force Acting on Monopole: $F_{hydrodyn.mono} := \gamma_w \cdot (H_{hydrodyn.} - 2\ ft) \cdot \left(\frac{D_{o.DFE.RS} + D_o}{2}\right) \cdot ft = 0.243\ kip$

Moment Arm for Hydrodynamic Load: $L_{hydrodyn.mono} := \frac{H_{hydrodyn.} - 2\ ft}{2} + (2\ ft + EL_{grade.monopole} - EL_{base.shaft.bott}) = 7.456\ ft$

Hydrodynamic Moment due to Flood Acting on Monopole: $M_{o.hydn.mono} := F_{hydrodyn.mono} \cdot L_{hydrodyn.mono} = 1.815\ kip \cdot ft$

Hydrodynamic Force Acting on Shaft: $F_{hydrodyn.shaft} := \gamma_w \cdot 2\ ft \cdot (D_{shaft}) \cdot ft = 0.374\ kip$

Moment Arm for Hydrodynamic Load: $L_{hydrodyn.shaft} := \frac{2\ ft}{2} + (EL_{grade.monopole} - EL_{base.shaft.bott}) = 5\ ft$

Hydrodynamic Moment due to Flood Acting on Shaft: $M_{o.hydn.shaft} := F_{hydrodyn.shaft} \cdot L_{hydrodyn.shaft} = 1.872\ kip \cdot ft$

Hydrodynamic Force Acting on Monopole and Shaft: $F_{hydrodyn.} := F_{hydrodyn.mono} + F_{hydrodyn.shaft} = 0.618\ kip$

Hydrodynamic Moment due to Flood Acting on Monopole and Shaft: $M_{o.hydn.} := M_{o.hydn.mono} + M_{o.hydn.shaft} = 3.687\ kip \cdot ft$

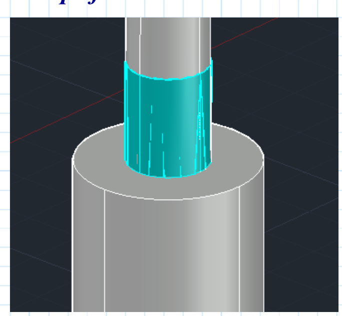
Hydrostatic Load

For water to DFE (300-yr flood), Unusual Condition

Weight of Flood Water Sitting on Shaft: $W_{water.shaft} := \gamma_w \cdot (EL_{DFE.RS} - EL_{grade.monopole} - 2\ ft) \cdot \frac{(A_{shaft} - A_{mono.base})}{ft} = 0.598\ \frac{kip}{ft}$

Lever Arm for DFE Flood Water Sitting on Shaft: $L_{w.hyd.shaft} := \left(\frac{D_{shaft}}{2}\right) = 1.5\ ft$

Resisting Moment Due to Weight of Flood Water on Shaft: $M_{r.hyd.shaft} := W_{water.shaft} \cdot L_{w.hyd.shaft} \cdot 1\ ft = 0.897\ kip \cdot ft$



Volume of Water on Monopole: $V_{water.mono} := 61.5\ in^3$

Weight of Flood Water Sitting on Monopole: $W_{water.mono} := \gamma_w \cdot V_{water.mono} = (2.22 \cdot 10^{-3})\ kip$

Lever Arm for DFE Flood Water Sitting on Monopole: $L_{w.hyd.mono} := \left(\frac{D_{shaft}}{2}\right) = 1.5\ ft$

Resisting Moment Due to Weight of Flood Water on Monopole: $M_{r.hyd.mono} := W_{water.mono} \cdot L_{w.hyd.mono} = 0.003\ kip \cdot ft$

Weight of Flood Water Sitting on Shaft and Monopole: $W_{water.base} := W_{water.shaft} + \dfrac{W_{water.mono}}{ft} = 0.6\ \dfrac{kip}{ft}$

Resisting Moment Due to Weight of Flood Water on Shaft and Monopole: $M_{r.hyd} := M_{r.hyd.shaft} + M_{r.hyd.mono} = 0.9\ kip \cdot ft$

Earth Pressure Load

Lateral Earth Pressure from River Side (DFE - 300 yr. flood))

Horizontal Earth Force Acting on Shaft, R/S: $F_{soil.RS} := 0.5 \cdot K_a \cdot \gamma_{s.buoy} \cdot \left(EL_{grade.monopole} - EL_{base.shaft.bott}\right)^2 \cdot D_{shaft} = 0.342\ kip$

Lever Arm for Horizontal Earth Force, R/S: $L_{soil.RS} := \dfrac{EL_{grade.monopole} - EL_{base.shaft.bott}}{3} = 1.333\ ft$

Moments from R/S Lateral Earth Pressure: $M_{o.soil.RS} := F_{soil.RS} \cdot L_{soil.RS} = 0.456\ kip \cdot ft$

Lateral Earth Pressure from Land Side (DFE - 300 yr. flood)

Horizontal Earth Force acting on Shaft, L/S: $F_{soil.LS} := 0.5 \cdot K_p \cdot \gamma_{s.buoy} \cdot \left(EL_{grade.monopole} - EL_{base.shaft.bott}\right)^2 \cdot D_{shaft} = 4.658\ kip$

Lever Arm for Horizontal Earth Force Acting on Shaft, L/S: $L_{soil.LS} := \dfrac{EL_{grade.monopole} - EL_{base.shaft.bott}}{3} = 1.333\ ft$

Resisting Moment from L/S Lateral Earth Pressure: $M_{r.soil.LS} := F_{soil.LS} \cdot L_{soil.LS} = 6.211\ kip \cdot ft$

Uplift Load

Volume of Water Displaced:

$$V_{displaced} := \left(A_{shaft} \cdot \left(EL_{grade.monopole} + 2\ ft - EL_{base.shaft.bott}\right)\right) + \left(\left(\frac{A_{mono.DFE} + A_{mono.base}}{2}\right) \cdot \left(EL_{DFE.RS} - EL_{grade.monopole} - 2\ ft\right)\right) = 44.809\ ft^3$$

Uplift Force below Shaft: $P_{uplift} := \gamma_w \cdot V_{displaced} = \left(2.796 \cdot 10^3\right)\ lbf$

Lever Arm for Uplift under the Shaft: $L_{arm.area} := \dfrac{D_{shaft}}{2} = 1.5\ ft$

Overturning Moment due to Uplift: $M_{o.uplift} := P_{uplift} \cdot L_{arm.area} = 4.194\ kip \cdot ft$

Sum of Uplift: $V_{uplift} := P_{uplift} = 2.796\ kip$

Vertical Resultant Force: $V_{net} := \left(W_{water.base} \cdot ft + W_{total} - V_{uplift}\right) = 5.322\ kip$

Sum of Lateral Loads from River Side (DFE Water on R/S)

$$F_{lateral.RS} := F_{wind.RS} + P_{debris} + F_{hydrodyn.} + F_{soil.RS} = 2.313\ kip$$

Sum of Lateral Loads from Land Side (DFE Water on L/S)

$$F_{lateral.LS} := F_{soil.LS} = 4.658\ kip$$

Net Lateral Force:

$$F_{lateral.net} := F_{lateral.RS} - F_{lateral.LS} = -2.345\ kip$$

(acting opposite the flow direction)

Sum of Moments from Flood

$$M_{o.flood} := M_{o.debris} + M_{o.hydn.} + M_{o.uplift} = 8.921\ kip \cdot ft$$

$$M_{r.flood} := M_{r.hyd} = 0.9\ kip \cdot ft$$

Moment from Wind

$$M_{o.wind.RS} = 26.623\ kip \cdot ft$$

Sum of Moments from Soil

$$M_{o.soil} := M_{o.soil.RS} = 0.456\ kip \cdot ft$$

$$M_{r.soil} := M_{r.soil.LS} = 6.211\ kip \cdot ft$$

Sum of Resisting Moments from Structure

$$M_{r.struct} := M_{r.mono} + M_{r.shaft} = 11.277\ kip \cdot ft$$

Sum of Overturning and Resisting Moments on Monopole

$$M_{o.sum} := M_{o.flood} + M_{o.soil} + M_{o.wind.RS} = 35.999\ kip \cdot ft$$

$$M_{r.sum} := M_{r.flood} + M_{r.struct} + M_{r.soil} = 18.388\ kip \cdot ft$$

Overturning Stability Check (With Debris Impact Load) (Not a Criteria but for informational purposes)

Overturning Factor of Safety

$$FS_{overturning} := \frac{M_{r.sum}}{M_{o.sum}} = 0.511$$

Location of Resultant Force Check (With Debris Impact Load)

Table 3-5 Requirements for Location of the Resultant – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	100% of Base in Compression	75% of Base in Compression	Resultant Within Base

EM 1110-2-2100, Table 3-5

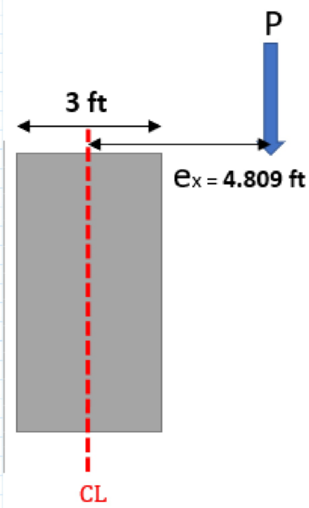
Kern Length $Kern := \frac{D_{shaft}}{3} = 1\ ft$

Balance Moment $M_{balance} := M_{r.sum} - M_{o.sum} = -17.611\ kip \cdot ft$

Resultant Location $x_R := \frac{M_{balance}}{V_{net}} = -3.309\ ft$

Eccentricity $e_x := \frac{D_{shaft}}{2} - x_R = 4.809\ ft$

$Check\_Resultant\_Location\_with\_Debris\_Impact :=$
- if $|e_x| \le \frac{Kern}{2}$
  - "Resultant within the Kern"
- if $\frac{Kern}{2} < |e_x| < \frac{D_{shaft}}{2}$
  - "Resultant Outside the Kern but within the base"
- else
  - "Failed"



$Check\_Resultant\_Location\_with\_Debris\_Impact =$ "Failed"

Sliding Safety Factor Check (With Debris Impact Load)

Sum of Horizontal Load on the River Side

$F_{RS} := F_{lateral.RS} = 2.313\ kip$

Sum of Horizontal Load on the Land Side

$F_{LS} := F_{lateral.LS} = 4.658\ kip$

Cohesion

$C_{Cohesion} := C \cdot D_{shaft} \cdot 1 \div ft = 0$

Friction Resistance Force

$F_R := V_{net} \cdot f = 1.33\ kip$

$$FS_{Sliding} := \frac{F_R + F_{LS}}{F_{RS}} = 2.589$$

$Sliding\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact :=$
- if $FS_{Sliding} \ge 1.2$
  - "OK, adequate safety factor"
- else
  - "FAILED"

= "OK, adequate safety factor"

$Sliding\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact =$ "OK, adequate safety factor"

Sum of Overturning and Resisting Moments on Flood Wall (Without Debris Impact Load)

$M_{o.sum.wo.debris.impact} := M_{o.sum} - M_{o.debris} = 34.96\ kip \cdot ft$

$M_{r.sum.wo.debris.impact} := M_{r.sum} = 18.388\ kip \cdot ft$

Overturning Stability Check (Not a Criteria but for informational purposes)

Overturning Factor of Safety **(Without Debris Impact Load)**

$$FS_{overturning.wo.debris.impact} := \frac{M_{r.sum.wo.debris.impact}}{M_{o.sum.wo.debris.impact}} = 0.526$$

Location of Resultant Force Check (Without Debris Impact Load)

Kern Length $\quad Kern := \frac{D_{shaft}}{3} = 1\ ft$

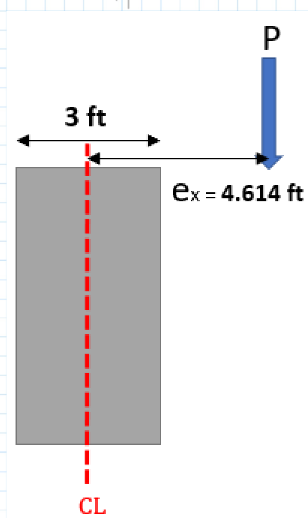
Balance Moment $\quad M_{balance.wo.debris.impact} := M_{r.sum.wo.debris.impact} - M_{o.sum.wo.debris.impact} = -16.572\ kip \cdot ft$

Resultant Location $\quad x_{R.wo.debris.impact} := \frac{M_{balance.wo.debris.impact}}{V_{net}} = -3.114\ ft$

Eccentricity $\quad e_{x.wo.debris.impact} := \frac{D_{shaft}}{2} - x_{R.wo.debris.impact} = 4.614\ ft$

$$Check\_Resultant\_Location\_wo\_Debris\_Impact := \begin{array}{l} \text{if } \left|e_{x.wo.debris.impact}\right| \le \frac{Kern}{2} \\ \quad \text{“Resultant within the Kern”} \\ \text{if } \frac{Kern}{2} < \left|e_{x.wo.debris.impact}\right| < \frac{D_{shaft}}{2} \\ \quad \text{“Resultant Outside the Kern but within the base”} \\ \text{else} \\ \quad \text{“Failed”} \end{array}$$

$Check\_Resultant\_Location\_wo\_Debris\_Impact = \text{“Failed”}$



Sliding Safety Factor Check (Without Debris Impact Load)

Sum of Horizontal Load on the River Side **(Without Debris Impact Load)**

$$F_{RS.wo.debris.impact} := \left(F_{wind.RS} + F_{hydrodyn.} + F_{soil.RS}\right) = 2.178\ \textbf{kip}$$

Sum of Horizontal Load on the Land Side

$$F_{LS} = 4.658\ \textbf{kip}$$

Cohesion

$$C_{Cohesion} := C \cdot D_{shaft} \cdot 1 \div \textbf{ft} = 0$$

Friction Resistance Force

$$F_R := V_{net} \cdot f = 1.33\ \textbf{kip}$$

$$FS_{Sliding.wo.debris.impact} := \frac{F_R + F_{LS}}{F_{RS.wo.debris.impact}} = 2.749$$

Sliding_Factor_of_Safety_Check_wo_Debris_Impact := if $FS_{Sliding.wo.debris.impact} \geq 1.2$
  "OK, adequate safety factor"
else
  "FAILED"
= "OK, adequate safety factor"

Sliding_Factor_of_Safety_Check_wo_Debris_Impact = "OK, adequate safety factor"

Floatation Stability Check (With Debris Impact Load)

Downward Vertical Force

$$V_{downward} := W_{water.base} \cdot \textbf{ft} + W_{total} = 8.118\ \textbf{kip}$$

Upward Vertical Force

$$V_{upward} := V_{uplift} = 2.796\ \textbf{kip}$$

$$FS_{floatation} := \frac{V_{downward}}{V_{upward}} = 2.903$$

Floatation_Factor_of_Safety_Check_with_Debris_Impact := if $FS_{floatation} \geq 1.2$
  "OK, adequate safety factor"
else
  "FAILED"
= "OK, adequate safety factor"

Floatation_Factor_of_Safety_Check_with_Debris_Impact = "OK, adequate safety factor"

Table 3-4 Required Factors of Safety for Flotation – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	1.3	1.2	1.1

EM 1110-2-2100, Table 3-4

Bearing Pressure Check (With Debris Impact Load)

Length of the Pressure Triangle $B := 0.0001\ ft$

Effective width of the base for Bearing Pressure $L_{effective} := B = \left(1 \cdot 10^{-4}\right)\ ft$

Bearing Pressure per 1 Foot Section $Bearing_{Pressure} := \dfrac{V_{net}}{L_{effective} \cdot 1\ ft} = \left(5.322 \cdot 10^{4}\right)\ ksf$

Allowable Bearing Pressure $\sigma_{bearing} = 1.5\ ksf$ *(Ref. Expert Report, Section 5)*

$Bearing\_Pressure\_Check\_with\_Debris\_Impact :=$
- if $Bearing_{Pressure} \le \sigma_{bearing}$
  - "OK, Bearing Pressure is within Allowable"
- also if $Bearing_{Pressure} < 0$
  - "Fails Due to Buoyancy"
- else
  - "FAILED"

$=$ "FAILED"

$Bearing\_Pressure\_Check\_with\_Debris\_Impact =$ "FAILED"

Case C: Rising Waters Coming from River Side

This loading condition accounts for rising waters coming from the land side along the easter segment of the fence. Albeit having slightly lower water surface elevation and flow velocity than Case B, the debris impact is not shielded by the bollard fence (hence controlling).

Elevations & Geometry (Ref. Expert Report, Section 4)

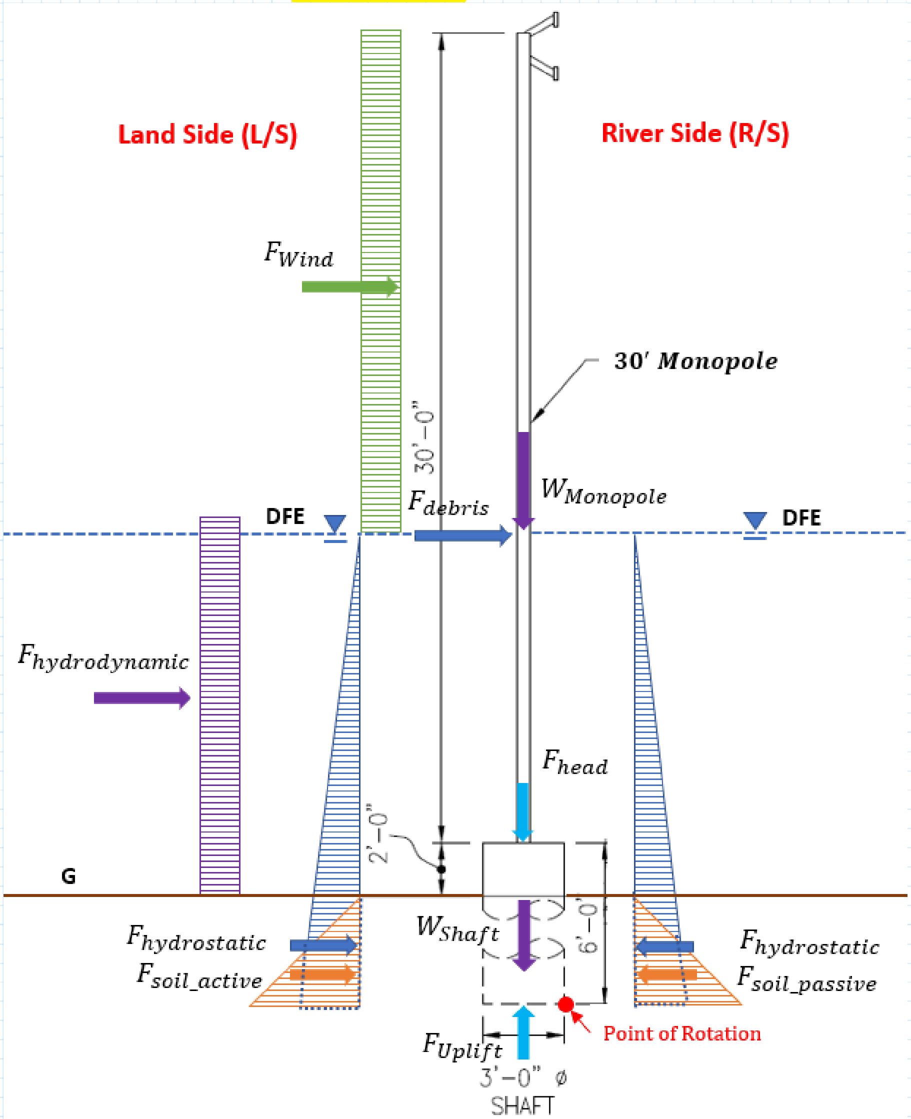
Flood Elevation (River Side): $EL_{DFE.RS} := 128.3\ ft$

Flood Elevation (Land Side): $EL_{DFE.LS} := 128.3\ ft$

Grade Elevation at Monopole: $EL_{grade.monopole} := 111.83\ ft$

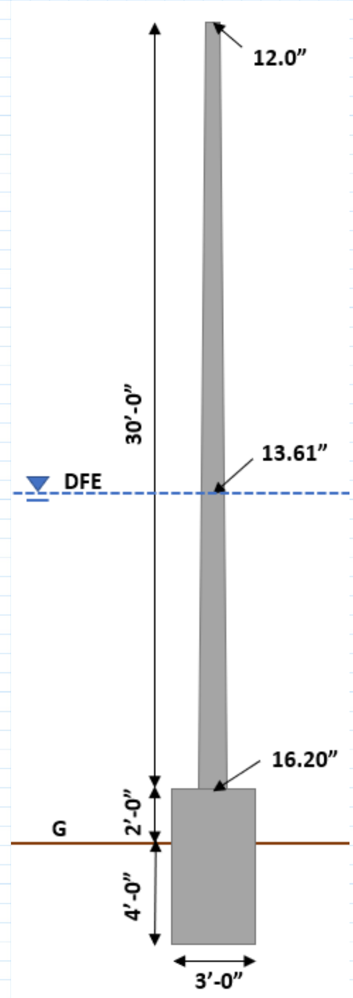
Base of Shaft Elevation: $EL_{base.shaft.bott} := EL_{grade.monopole} - 4\ ft = 107.83\ ft$

Water Velocity $V_{water} := 6.0\ \frac{ft}{s}$



Loading Diagram of Typical Monopole for Case C
(Rising waters coming from the land side along the eastern segment of the fence)

Elevations & Geometry (Ref. TGR Drawings)



Monopole Height (above shaft): $H_{mono} := 30 \cdot ft$

Outer Diameter at Tip: $D_{o.tip} := 12.0\ in$

Outer Diameter at Base: $D_o := 16.20\ in$

Outer Diameter at DFE (R/S): $D_{o.DFE.RS} := 13.61\ in$

Foundation Shaft Depth: $H_{shaft} := 6\ ft$

Foundation Shaft Diameter: $D_{shaft} := 3\ ft$

Physical Properties for 30-ft Direct Embed Steel Poles

	Light	Medium	Heavy
TESSCO SKU	331020	351929	341210
Design Number	T30LA	T30MA	T30HA
Tip OD, in (cm)	6.50 (16.51)	9.00 (22.86)	12.00 (30.48)
OD @ Grade, in (cm)	10.70 (27.18)	13.20 (33.53)	16.20 (41.15)
Butt OD, in (cm)	12.10 (30.73)	14.74 (37.45)	18.02 (45.77)
Number of Sides	12	16	18
Δ Dia, in/ft (cm/m)	0.14 (1.17)	0.14 (1.17)	0.14 (1.17)
Side Taper, in/ft (cm/m)	0.07 (0.58)	0.07 (0.58)	0.07 (0.58)
Embedment, ft (m)	10.00 (3.05)	16.00 (4.88)	13.00 (3.96)
Auger Dia, ft (m)	2.50 (0.76)	2.50 (0.76)	3.00 (0.91)
Backfill Type	Aggregate	Aggregate	Aggregate
Total Length, ft (m)	40 (12.19)	41 (12.49)	43 (13.10)
Bare Pole Wt, lbs (kg)	793 (360)	1,031 (468)	1,368 (621)
No. of Sections	1	1	1

TESSCO Monopole Geometry and Properties
Remark: Similar to existing monopole installed by TGR

Load Calculation

Dead Load

Monopole Weight: $W_{mono} := 1368\ lbf$

Shaft Cross-Sectional Area: $A_{shaft} := \frac{\pi \cdot D_{shaft}^2}{4} = 7.069\ ft^2$

Monopole Cross-Sectional Area at Base: $A_{mono.base} := \frac{\pi \cdot D_o^2}{4} = 1.431\ ft^2$

Monopole Cross-Sectional Area at DFE: $A_{mono.DFE} := \frac{\pi \cdot D_{o.DFE.RS}^2}{4} = 1.01\ ft^2$

Weight of Concrete Shaft: $W_{shaft} := A_{shaft} \cdot H_{shaft} \cdot \gamma_c = 6.15\ kip$

Total Weight: $W_{total} := W_{mono} + W_{shaft} = 7.518\ kip$

Moments about Middle of the Shaft

Resisting Moment (Monopole) $M_{r.Mono} := W_{mono} \cdot \left(\frac{D_{shaft}}{2}\right) = 2.052\ kip \cdot ft$

Resisting Moment (Shaft) $M_{r.shaft} := W_{shaft} \cdot \left(\frac{D_{shaft}}{2}\right) = 9.225\ kip \cdot ft$

Wind Load

Risk Category based on Use or Occupancy of Building and Other Structures: Risk Category I

Wind Speed (ASCE 7-10 Online Hazard Tool for the Project Location https://asce7hazardtool.online/): $V_{wind} := 121\ mph$

Velocity Pressure Exposure Coefficient: $K_z := 1.16$

Topographic Factor: $K_{zt} := 1.0$

Wind Direction Factor: $K_d := 0.85$

Design Wind Pressure ASCE 7-10 Eq. 27.3-1: $q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot \left(\frac{V_{wind}}{mph}\right)^2 \cdot psf = 36.96\ psf$

Wind Force from River Side $F_{wind.RS} := q_z \cdot \left(H_{mono} - EL_{DFE.RS} + EL_{grade.monopole} + 2\ ft\right) \cdot \left(\frac{D_{o.tip} + D_{o.DFE.RS}}{2}\right) = 0.612\ kip$

Moment Arm for Wind Force from River Side

$$L_{wind.RS} := \frac{\left(H_{mono} - EL_{DFE.RS} + EL_{grade.monopole} + 2\ ft\right)}{2} + \left(EL_{DFE.RS} - EL_{base.shaft.bott}\right) = 28.235\ ft$$

Moment due to Wind from River Side $M_{o.wind.RS} := F_{wind.RS} \cdot L_{wind.RS} = 17.292\ kip \cdot ft$

Remark: Wind acting from the land side has been ignored since it will not be concurrent with river side wind.

Debris Impact Load

The debris object is assumed to be at or near the water surface level when it strikes (e.g. Stillwater elevation)

Water Velocity $V_{water} = 6\ \frac{ft}{s}$ Ref. Expert Report, Section 4

Weight of Object: $W_o := 1000\ lbf$ Ref. FEMA P-55 Section 8.5.10

Depth Coefficient (for a Floodway or Zone V): $C_D := 1.0$ Ref. FEMA P-55 Section 8.5.10 Table 8-3

Blockage Coefficient (Assumed 30% Blockage): $C_B := 1.0$ Ref. FEMA P-55 Section 8.5.10 Table 8-4

Building Structure Coefficient: $C_{Str} := 0.8$

Impact Force: $F_i := W_o \cdot V_{water} \cdot \frac{sec}{ft} \cdot C_D \cdot C_B \cdot C_{Str} = 4.8\ kip$ FEMA P-55, Section 8.5.10 Eq. 8.9

For external stability, a minimum Debris Impact load 0.1 k/ft of wall is considered, as recommended by USACE (per ASCE 7-10, Chapter C5, Special Impact Loads.)

Distributed Debris Impact Load: $P_{debris} := 0.1 \frac{kip}{ft} \cdot D_o = 0.135\ kip$

Moment Arm to Debris Impact Load: $L_{debris} := EL_{DFE.RS} - EL_{base.shaft.bott} = 20.47\ ft$

Overturning Moment due to Debris Load: $M_{o.debris} := P_{debris} \cdot L_{debris} = 2.763\ kip \cdot ft$

Hydrodynamic Load

Since the velocity of water is less than 10 ft/sec, the dynamic effect of current is converted to equivalent surcharge depth dh, as per ASCE 7-16, Cl. 5.4.3.4

Coefficient for Drag or Shape Factor: $\alpha := 1.25$

Gravity $g := 32.2 \frac{ft}{s^2}$

Equivalent Surcharge Depth $d_h := \frac{\alpha \cdot V_{water}^{2}}{2 \cdot g} = 0.699\ ft$

Design Stillwater Depth 300 Years Flood: $d_{300yr} := EL_{DFE.RS} - EL_{grade.monopole} = 16.47\ ft$

Water Height due to Hydrodynamic Current: $H_{hydrodyn.} := d_{300yr} + d_h = 17.169\ ft$

Hydrodynamic Force Acting on Monopole: $F_{hydrodyn.mono} := \gamma_w \cdot (H_{hydrodyn.} - 2\ ft) \cdot \left(\frac{D_{o.DFE.RS} + D_o}{2}\right) \cdot ft = 1.176\ kip$

Moment Arm for Hydrodynamic Load: $L_{hydrodyn.mono} := \frac{H_{hydrodyn.} - 2\ ft}{2} + (2\ ft + EL_{grade.monopole} - EL_{base.shaft.bott}) = 13.584\ ft$

Hydrodynamic Moment due to Flood Acting on Monopole: $M_{o.hydn.mono} := F_{hydrodyn.mono} \cdot L_{hydrodyn.mono} = 15.971\ kip \cdot ft$

Hydrodynamic Force Acting on Shaft: $F_{hydrodyn.shaft} := \gamma_w \cdot 2\ ft \cdot (D_{shaft}) \cdot ft = 0.374\ kip$

Moment Arm for Hydrodynamic Load: $L_{hydrodyn.shaft} := \frac{2\ ft}{2} + (EL_{grade.monopole} - EL_{base.shaft.bott}) = 5\ ft$

Hydrodynamic Moment due to Flood Acting on Shaft: $M_{o.hydn.shaft} := F_{hydrodyn.shaft} \cdot L_{hydrodyn.shaft} = 1.872\ kip \cdot ft$

Hydrodynamic Force Acting on Monopole and Shaft: $F_{hydrodyn.} := F_{hydrodyn.mono} + F_{hydrodyn.shaft} = 1.55\ kip$

Hydrodynamic Moment due to Flood Acting on Monopole and Shaft: $M_{o.hydn.} := M_{o.hydn.mono} + M_{o.hydn.shaft} = 17.843\ kip \cdot ft$

Hydrostatic Load

For water to DFE (300-yr flood), Unusual Condition

Weight of Flood Water Sitting on Shaft: $W_{water.base} := \gamma_w \cdot \left(EL_{DFE.RS} - EL_{grade.monopole}\right) \cdot \frac{\left(A_{shaft} - A_{mono.base}\right)}{ft} = 5.794\ \frac{kip}{ft}$

Lever Arm for DFE Flood Water Sitting on Shaft: $L_{w.hyd} := \left(\frac{D_{shaft}}{2}\right) = 1.5\ ft$

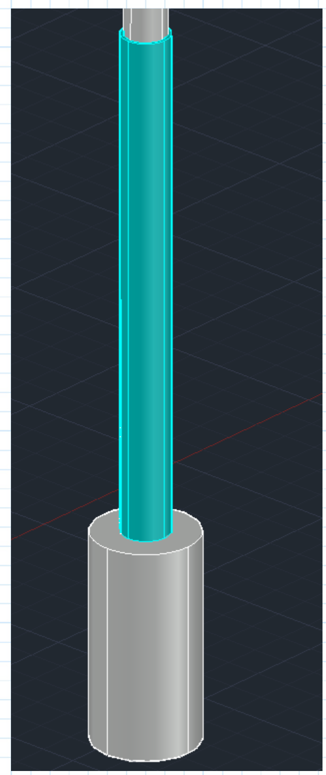
Resisting Moment Due to Weight of Flood Water on Shaft: $M_{r.hyd} := W_{water.base} \cdot L_{w.hyd} \cdot 1\ ft = 8.69\ kip \cdot ft$

Weight of Flood Water Sitting on Shaft: $W_{water.base} = 5.794\ \frac{kip}{ft}$

Resisting Moment Due to Weight of Flood Water on Shaft: $M_{r.hyd} = 8.69\ kip \cdot ft$

Weight of Flood Water Sitting on Shaft: $W_{water.shaft} := \gamma_w \cdot \left(EL_{DFE.RS} - EL_{grade.monopole} - 2\ ft\right) \cdot \frac{\left(A_{shaft} - A_{mono.base}\right)}{ft} = 5.09\ \frac{kip}{ft}$

Lever Arm for DFE Flood Water Sitting on Shaft: $L_{w.hyd.shaft} := \left(\frac{D_{shaft}}{2}\right) = 1.5\ ft$



Resisting Moment Due to Weight of Flood Water on Shaft: $M_{r.hyd.shaft} := W_{water.shaft} \cdot L_{w.hyd.shaft} \cdot 1\ ft = 7.635\ kip \cdot ft$

Volume of Water on Monopole: $V_{water.mono} := 4289.04\ in^3$

Weight of Flood Water Sitting on Monopole: $W_{water.mono} := \gamma_w \cdot V_{water.mono} = 0.15\ kip$

Lever Arm for DFE Flood Water Sitting on Monopole: $L_{w.hyd.mono} := \left(\frac{D_{shaft}}{2}\right) = 1.5\ ft$

Resisting Moment Due to Weight of Flood Water on Monopole: $M_{r.hyd.mono} := W_{water.mono} \cdot L_{w.hyd.mono} = 0.232\ kip \cdot ft$

Weight of Flood Water Sitting on Shaft and Monopole: $W_{water.base} := W_{water.shaft} + \frac{W_{water.mono}}{ft} = 5.245\ \frac{kip}{ft}$

Resisting Moment Due to Weight of Flood Water on Shaft and Monopole: $M_{r.hyd} := M_{r.hyd.shaft} + M_{r.hyd.mono} = 7.867\ kip \cdot ft$

Earth Pressure Load

Lateral Earth Pressure from River Side (DFE - 300 yr. flood))

Horizontal Earth Force Acting on Shaft, R/S: $F_{soil.RS} := 0.5 \cdot K_a \cdot \gamma_{s.buoy} \cdot \left(EL_{grade.monopole} - EL_{base.shaft.bott}\right)^2 \cdot D_{shaft} = 0.342\ kip$

Lever Arm for Horizontal Earth Force, R/S: $L_{soil.RS} := \frac{EL_{grade.monopole} - EL_{base.shaft.bott}}{3} = 1.333\ ft$

Moments from R/S Lateral Earth Pressure: $M_{o.soil.RS} := F_{soil.RS} \cdot L_{soil.RS} = 0.456\ kip \cdot ft$

Lateral Earth Pressure from Land Side (DFE - 300 yr. flood)

Horizontal Earth Force acting on Shaft, L/S: $F_{soil.LS} := 0.5 \cdot K_p \cdot \gamma_{s.buoy} \cdot \left(EL_{grade.monopole} - EL_{base.shaft.bott}\right)^2 \cdot D_{shaft} = 4.658 \ kip$

Lever Arm for Horizontal Earth Force Acting on Shaft, L/S: $L_{soil.LS} := \dfrac{EL_{grade.monopole} - EL_{base.shaft.bott}}{3} = 1.333 \ ft$

Resisting Moment from L/S Lateral Earth Pressure: $M_{r.soil.LS} := F_{soil.LS} \cdot L_{soil.LS} = 6.211 \ kip \cdot ft$

Uplift Load

Volume of Water Displaced:

$$V_{displaced} := \left(A_{shaft} \cdot \left(EL_{grade.monopole} + 2 \ ft - EL_{base.shaft.bott}\right)\right) + \left(\left(\frac{A_{mono.DFE} + A_{mono.base}}{2}\right) \cdot \left(EL_{DFE.RS} - EL_{grade.monopole} - 2 \ ft\right)\right) = 60.077 \ ft^3$$

Uplift Force below Shaft: $P_{uplift} := \gamma_w \cdot V_{displaced} = \left(3.749 \cdot 10^3\right) \ lbf$

Lever Arm for Uplift under the Shaft: $L_{arm.area} := \dfrac{D_{shaft}}{2} = 1.5 \ ft$

Overturning Moment due to Uplift: $M_{o.uplift} := P_{uplift} \cdot L_{arm.area} = 5.623 \ kip \cdot ft$

Sum of Uplift: $V_{uplift} := P_{uplift} = 3.749 \ kip$

Vertical Resultant Force: $V_{net} := \left(W_{water.base} \cdot ft + W_{total} - V_{uplift}\right) = 9.014 \ kip$

Sum of Lateral Loads from River Side (DFE Water on R/S)

$$F_{lateral.RS} := F_{wind.RS} + P_{debris} + F_{hydrodyn.} + F_{soil.RS} = 2.64 \ kip$$

Sum of Lateral Loads from Land Side (DFE Water on L/S)

$$F_{lateral.LS} := F_{soil.LS} = 4.658 \ kip$$

Net Lateral Force:

$$F_{lateral.net} := F_{lateral.RS} - F_{lateral.LS} = -2.019 \ kip$$

(acting opposite the flow direction)

Sum of Moments from Flood

$$M_{o.flood} := M_{o.debris} + M_{o.hydn.} + M_{o.uplift} = 26.229 \ kip \cdot ft$$

$$M_{r.flood} := M_{r.hyd} = 7.867 \ kip \cdot ft$$

Moment from Wind $M_{o.wind.RS} = 17.292 \ kip \cdot ft$

Sum of Moments from Soil

$$M_{o.soil} := M_{o.soil.RS} = 0.456 \ kip \cdot ft$$

$$M_{r.soil} := M_{r.soil.LS} = 6.211 \ kip \cdot ft$$

Sum of Resisting Moments from Structure

$$M_{r.struct} := M_{r.Mono} + M_{r.shaft} = 11.277\ \textbf{kip} \cdot \textbf{ft}$$

Sum of Overturning and Resisting Moments on Monopole

$$M_{o.sum} := M_{o.flood} + M_{o.soil} + M_{o.wind.RS} = 43.978\ \textbf{kip} \cdot \textbf{ft}$$

$$M_{r.sum} := M_{r.flood} + M_{r.struct} + M_{r.soil} = 25.355\ \textbf{kip} \cdot \textbf{ft}$$

Overturning Stability Check (With Debris Impact Load) (Not a Criteria but for informational purposes)

Overturning Factor of Safety

$$FS_{overturning} := \frac{M_{r.sum}}{M_{o.sum}} = 0.577$$

Location of Resultant Force Check (With Debris Impact Load)

Table 3-5 Requirements for Location of the Resultant – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	100% of Base in Compression	75% of Base in Compression	Resultant Within Base

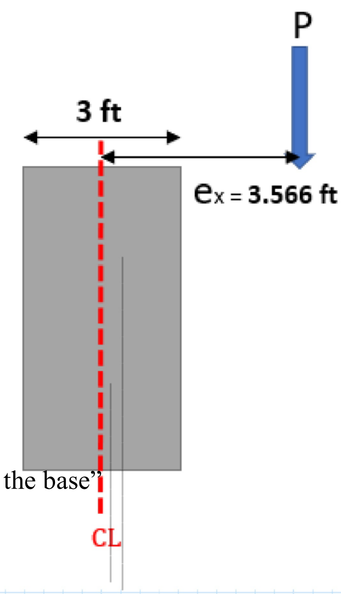
EM 1110-2-2100, Table 3-5

Kern Length $$Kern := \frac{D_{shaft}}{3} = 1\ \textbf{ft}$$

Balance Moment $$M_{balance} := M_{r.sum} - M_{o.sum} = -18.622\ \textbf{kip} \cdot \textbf{ft}$$

Resultant Location $$x_R := \frac{M_{balance}}{V_{net}} = -2.066\ \textbf{ft}$$

Eccentricity $$e_x := \frac{D_{shaft}}{2} - x_R = 3.566\ \textbf{ft}$$



$$Check\_Resultant\_Location\_with\_Debris\_Impact := \begin{array}{l} \text{if } |e_x| \le \frac{Kern}{2} \\ \quad \text{“Resultant within the Kern”} \\ \text{if } \frac{Kern}{2} < |e_x| < \frac{D_{shaft}}{2} \\ \quad \text{“Resultant Outside the Kern but within the base”} \\ \text{else} \\ \quad \text{“Failed”} \end{array}$$

$$Check\_Resultant\_Location\_with\_Debris\_Impact = \text{“Failed”}$$

Sliding Safety Factor Check (With Debris Impact Load)

Sum of Horizontal Load on the River Side

$F_{RS} := F_{lateral.RS} = 2.64 \; \boldsymbol{kip}$

Sum of Horizontal Load on the Land Side

$F_{LS} := F_{lateral.LS} = 4.658 \; \boldsymbol{kip}$

Cohesion

$C_{Cohesion} := C \cdot D_{shaft} \cdot 1 \div \boldsymbol{ft} = 0$

Friction Resistance Force

$F_R := V_{net} \cdot f = 2.253 \; \boldsymbol{kip}$

$$FS_{Sliding} := \frac{F_R + F_{LS}}{F_{RS}} = 2.619$$

$Sliding\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact :=$ if $FS_{Sliding} \ge 1.2$ "OK, adequate safety factor" else "FAILED" = "OK, adequate safety factor"

$Sliding\_Factor\_of\_Safety\_Check\_with\_Debris\_Impact =$ "OK, adequate safety factor"

Sum of Overturning and Resisting Moments on Flood Wall (Without Debris Impact Load)

$M_{o.sum.wo.debris.impact} := M_{o.sum} - M_{o.debris} = 41.214 \; \boldsymbol{kip \cdot ft}$

$M_{r.sum.wo.debris.impact} := M_{r.sum} = 25.355 \; \boldsymbol{kip \cdot ft}$

Overturning Stability Check (Not a Criteria but for informational purposes)

Overturning Factor of Safety **(Without Debris Impact Load)**

$$FS_{overturning.wo.debris.impact} := \frac{M_{r.sum.wo.debris.impact}}{M_{o.sum.wo.debris.impact}} = 0.615$$

Location of Resultant Force Check (Without Debris Impact Load)

Kern Length $Kern := \dfrac{D_{shaft}}{3} = 1 \; \boldsymbol{ft}$

Balance Moment $M_{balance.wo.debris.impact} := M_{r.sum.wo.debris.impact} - M_{o.sum.wo.debris.impact} = -15.859 \; \boldsymbol{kip \cdot ft}$

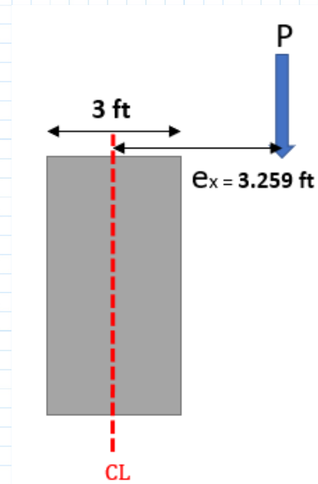
Resultant Location

$$x_{R.wo.debris.impact} := \frac{M_{balance.wo.debris.impact}}{V_{net}} = -1.759\ ft$$

Eccentricity

$$e_{x.wo.debris.impact} := \frac{D_{shaft}}{2} - x_{R.wo.debris.impact} = 3.259\ ft$$

Check_Resultant_Location_wo_Debris_Impact :=
- if $\left|e_{x.wo.debris.impact}\right| \le \frac{Kern}{2}$
  - "Resultant within the Kern"
- if $\frac{Kern}{2} < \left|e_{x.wo.debris.impact}\right| < \frac{D_{shaft}}{2}$
  - "Resultant Outside the Kern but within the base"
- else
  - "Failed"



Check_Resultant_Location_wo_Debris_Impact = "Failed"

Sliding Safety Factor Check (Without Debris Impact Load)

Sum of Horizontal Load on the River Side **(Without Debris Impact Load)**

$$F_{RS.wo.debris.impact} := \left(F_{wind.RS} + F_{hydrodyn.} + F_{soil.RS}\right) = 2.505\ kip$$

Sum of Horizontal Load on the Land Side

$$F_{LS} = 4.658\ kip$$

Cohesion

$$C_{Cohesion} := C \cdot D_{shaft} \cdot 1 \div ft = 0$$

Friction Resistance Force

$$F_R := V_{net} \cdot f = 2.253\ kip$$

$$FS_{Sliding.wo.debris.impact} := \frac{F_R + F_{LS}}{F_{RS.wo.debris.impact}} = 2.76$$

Sliding_Factor_of_Safety_Check_wo_Debris_Impact :=
- if $FS_{Sliding.wo.debris.impact} \ge 1.2$
  - "OK, adequate safety factor"
- else
  - "FAILED"

= "OK, adequate safety factor"

Sliding_Factor_of_Safety_Check_wo_Debris_Impact = "OK, adequate safety factor"

Floatation Stability Check (With Debris Impact Load)

Downward Vertical Force

$$V_{downward} := W_{water.base} \cdot ft + W_{total} = 12.763 \; kip$$

Upward Vertical Force

$$V_{upward} := V_{uplift} = 3.749 \; kip$$

$$FS_{floatation} := \frac{V_{downward}}{V_{upward}} = 3.404$$

Floatation_Factor_of_Safety_Check_with_Debris_Impact := if $FS_{floatation} \geq 1.2$ "OK, adequate safety factor" else "FAILED" = "OK, adequate safety factor"

Floatation_Factor_of_Safety_Check_with_Debris_Impact = "OK, adequate safety factor"

Table 3-4 Required Factors of Safety for Flotation – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	1.3	1.2	1.1

EM 1110-2-2100, Table 3-4

Bearing Pressure Check (With Debris Impact Load)

Length of the Pressure Triangle $B := 0.0001 \; ft$

Effective width of the base for Bearing Pressure $L_{effective} := B = \left(1 \cdot 10^{-4}\right) \; ft$

Bearing Pressure per 1 Foot Section $Bearing_{Pressure} := \dfrac{V_{net}}{L_{effective} \cdot 1 \; ft} = \left(9.014 \cdot 10^{4}\right) \; ksf$

Allowable Bearing Pressure $\sigma_{bearing} = 1.5 \; ksf$ *(Ref. Expert Report, Section 5)*

Bearing_Pressure_Check_with_Debris_Impact := if $Bearing_{Pressure} \leq \sigma_{bearing}$ "OK, Bearing Pressure is within Allowable" also if $Bearing_{Pressure} < 0$ "Fails Due to Buoyancy" else "FAILED" = "FAILED"

Bearing_Pressure_Check_with_Debris_Impact = "FAILED"

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