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SBA Approved 8(a) Joint Venture

FXSA

SUNLAND PARK LEVEE REPAIR

DESIGN DOCUMENTATION REPORT

EAST LEVEE & WEST LEVEE SEGMENT DOWNSTREAM OF COUNTRY CLUB BRIDGE FINAL DESIGN

Contract No. IBM15D0001

Task Order No. IBM17T0010

February 04, 2022

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SUNLAND PARK LEVEE REPAIR -

EAST LEVEE & WEST LEVEE SEGMENT DOWNSTREAM OF C.C. BRIDGE

Design Documentation Report

Final Design

Prepared for:

U.S. International Boundary and Water Commission

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ACRONYMS AND ABBREVIATIONS

Arcadis	Arcadis U.S., Inc.
ASTM	ASTM International
AT&T	American Telephone & Telegraph
BFE	Base Flood Elevation
CADD	Computer Aided Design and Drafting
CFR	Code of Federal Regulations
CoEP	City of El Paso
CRRUA	Camino Real Regional Utility Authority
CW2	Canutillo West 2
DDR	Design Documentation Report
EPE	El Paso Electric
EPW	El Paso Water
FEMA	Federal Emergency Management Agency
fps	Foot per second
FXSA	Frank X. Spencer & Associates
Kenall	Kenall, Inc.
MDJV	Munoz and Dannenbaum Joint Venture
NAD 83	North American Datum of 1983
NAVD 88	North American Vertical Datum of 1988
NFIP	National Flood Insurance Program
%	percent
ROW	Right-of-Way
S&BI	S&B Infrastructure, Ltd.
SOW	Scope of Work
SUE	Subsurface Utility Exploration
SWCC	Soil-Water Characteristic Curves
TGS	Texas Gas Service
TxDOT	Texas Department of Transportation
URGFCP	Upper Rio Grande Flood Control Project

USACE	United States Army Corps of Engineers
USIBWC	United States International Boundary and Water Commission
3D	Three dimensional

1 PROJECT INFORMATION

1.1 General

The United States Section of the International Boundary and Water Commission (USIBWC) contracted with Arcadis U.S., Inc. (Arcadis) for the design of the Sunland Park Levee Repair within the Rio Grande Canalization Project under Task Order No. IBM17T0010 of Contract No. IBM15D0001 to provide engineering services for the preparation of plans and specifications. In addition, Arcadis will support USIBWC during the project procurement and contracting process. The Sunland Park West Levee is known as the Canutillo West 2 (CW2) Levee and the Sunland Park East Levee is known as the Canutillo East 2 (CE2) Levee.

On September 15, 2018, USIBWC issued Contract Modification P00002 seeking to separate the construction documents into two packages: East Levee and West Levee. The design for the West Levee, except for the portion downstream of Country Club Bridge was submitted to USIBWC on August 26, 2019. On September 25, 2020, USIBWC issued Contract Modification P00005 seeking to combine the East Levee and West Levee portion downstream of Country Club Bridge into one construction package. This report includes the improvements pertaining to Sunland Park Levee Repair for East Levee and West Levee downstream of Country Club Bridge.

The project team consists of Arcadis and the following subcontractors: Munoz and Dannenbaum Joint Venture (MDJV) and Frank X. Spencer & Associates (FXSA). Altogether, Arcadis and its subcontractors form the Arcadis Team.

1.2 Project History

In September 2010, the USIBWC issued a construction contract to rehabilitate the Sunland Park Levee segment of the Upper Rio Grande Flood Control Project (URGFCP). Construction was completed in November 2012. However, on completion, the levee experienced considerable erosion. The USIBWC contracted with Kenall, Inc. (Kenall) in April 2015 to perform a Forensic Geotechnical Investigation and provide the USIBWC a professional opinion on whether the newly constructed levee segment would perform satisfactorily during the 100-year flood event pursuant to 44 Code of Federal Regulations (CFR) §65.10 (Federal Emergency Management Agency [FEMA]). Kenall identified several construction-related problems, and its investigation found dispersive clay within the existing levee embankment along the entire length of this reach of the levee system. Kenall found that as a result of the erosion of the dispersive clay, the Sunland Park Levee Segment does not meet the National Flood Insurance Program (NFIP) Regulation 44 CFR §65.10(b).

Previous studies for the design of the Sunland Park Levees include:

- "*Sunland Park Levee Forensic Geotechnical Investigation*" by Kenall, dated July 29, 2016.
- *"Design Report for Upper Rio Grande Flood Control Project, Sunland Park Canutillo Levee Segment*" by S&B Infrastructure, Ltd. (S&BI), dated July 23, 2010.

- *"Final Technical Memorandum Volume I for Geotechnical Explorations of Levee System within the Rio Grande Canalization Project- Canutillo Levee Segment*" by Raba-Kistner Consultants, Inc., dated December 22, 2008.
- *"Final Technical Memorandum Volume II for Geotechnical Explorations of Levee System within the Rio Grande Canalization Project- Canutillo Levee Segment*" by Raba-Kistner Consultants, Inc., dated December 22, 2008.

1.3 Scope of Work

The scope of work (SOW) titled "Design Scope of Work, Rio Grande Canalization, Sunland Park Levee Repair in El Paso County, TX and Doña Ana County, NM," dated March 2, 2017, was provided for this task order. The main project goals are to:

- Address the deficiencies noted in Kenall's forensic report.
- Provide a design intended to produce levees that fully meet FEMA requirements for 44 CFR §65.10(b) (1) through (7). The final design is also intended to meet the requirements included in the United States Army Corps of Engineers' (USACE) Engineering Manual 1110-2-1913.
- Remove the existing floodgate gate at the Country Club Bridge (eastside) and design a new gate to withstand the 100-year flood.
- Raise or modify the levee closure structures along the levee alignment that are too low to be operated during flood flows.

The revised SOW, dated August 6, 2018, for the Contract Modification P00002 included the following work:

- Separate the construction documents into two packages:
    1. Sunland Park Levee Repair – East Levee
    2. Sunland Park Levee Repair – West Levee
- Provide catwalk design for access to the low structures that are currently inaccessible during flood flows.
- Provide boundary survey monuments within this project limit.

The final West Levee design upstream of Country Club Bridge was submitted in August 2019. The construction phase of this levee segment was awarded in September 2019. This Design Documentation Report (DDR) includes the design of the East Levee and West Levee portion downstream of Country Club Bridge per Contract Modification P00005 issued on September 25, 2020.

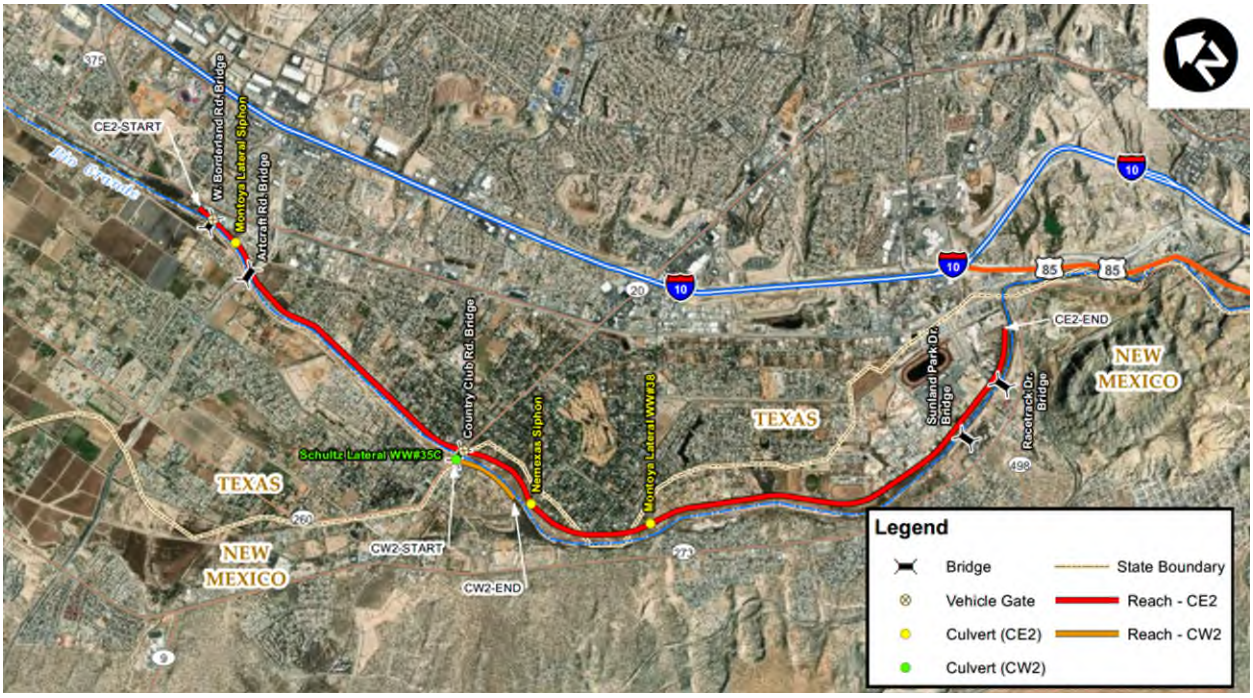
1.4 Purpose of Design Documentation Report

This Design Documentation Report (DDR) presents the basis for design for the levee segments and structural components for improvements pertaining to Sunland Park Levee Repair: East Levee and West Levee downstream of Country Club Bridge. DDR includes design assumptions and criteria, calculations, and design approach for all aspects of construction. The design provided in this DDR, which is detailed in the construction drawings and specifications, complies with industry standards and meets the applicable requirements of Subsections 1.2, Project Goal, and 1.5, Project Requirements, and Section 11, Regulatory Requirements, of the SOW.

1.5 Project Location

The project is located within the United States on land owned or controlled by the USIBWC within its Rio Grande Canalization Project, in El Paso County, Texas, and Doña Ana County, New Mexico. A layout of the project is included on Figure 1.

Figure 1. Sunland Park – East and West Levee Locations



The general location, length, coordinates, and stationing for the Sunland Park Levee segments included in this DDR are summarized in Table 1.

Table 1. Project Location Summary for Sunland Park Levees

Levee Segment	General Location	Length	Starting Station (Sta.)	Ending Station (Sta.)	Starting Coordinates	Ending Coordinates
East Levee	North of Borderland Bridge to Montoya Drain	8.4 miles	1510+19.16	1956+50[(1)]	31º 53' 16.7" N 106º 35' 53.4" W	31º 48' 7.1" N 106º 32' 54" W
West Levee	Upstream of Country Club Bridge to Nemexas Drain	0.6 miles	1666+00	1698+00[(1)]	31º 50' 47.7" N 106º 36' 24.4" W	31º 50' 17" N 106º 36' 23" W

Note: [(1)]Ending Sta. 1956+50 for East Levee (CE2) and 1698+00 for West Levee (CW2) was indicated in the SOW per Task Order IBM10C0018. Due to the change of the centerline alignment, our design dictates that the ending station for CE2 corresponds to Sta. 1956+25.86 and CW2 corresponds to Sta. 1697+74.24.

2 EXISTING CONDITIONS

2.1 Site Visit

The Arcadis team visited the entire East and West Levees on April 3, 2017 and October 11, 2017 to observe the levee and structure conditions. During the site visits, the Arcadis team observed the pertinent project features, the structures defined by USIBWC as deficient, and the ongoing maintenance activities at the crest of the levee. In general, at the time of the site visits, the existing top of levee consisted approximately 2-inch-thick gravel layer for vehicle access. Several access ramps are located throughout the alignment. In some cases, improvements on privately owned properties have encroached on the levee Right-of-Way (ROW) as described in Section 6.4. Several major and minor public roads intersect the levee segments. The major crossings along the East and West Levee alignments are included in Table 2. Photographic logs of the general observations are presented in Appendix A.1.

Table 2. Major Crossings along Sunland Park Levees

Approximate Sta.	Description	Ownership
EAST LEVEE		
1517+77	Borderland Bridge	City of El Paso
1546+13	Artcraft Bridge	Texas Department of Transportation
1667+08	Country Club Bridge	City of El Paso
1906+68	Sunland Park Bridge	New Mexico Department of Transportation
1933+03	Racetrack Bridge	New Mexico Department of Transportation
WEST LEVEE		
1666+59	Country Club Bridge	City of El Paso

The existing structures along the levee alignment are included in Table 3. Photographic logs and the locations of all the existing structures along the levee reach are included in Appendix A.2. The structure type and size along with survey data are included in the contract drawings.

Table 3. Existing Structures crossing Sunland Park Levees

Station	Structure Description	Structure Size	Ownership
EAST LEVEE			
1531+82	Montoya Lateral Siphon	3.5’ x 4’ Box Culvert	EPCWID1
1706+44	Nemexas Siphon	72” Diameter concrete culvert	EBID
1761+95	Montoya Lateral WW#38	48” RCP	EPCWID1
WEST LEVEE			
1667+04	Schultz Lateral WW#35C	30” RCP	EBID

The known existing utilities are provided in Table 4. The utilities in Table 4 have been field verified using historical documentation and by contacting known utility owners in the area to verify utility locations. We do not anticipate any conflicts with utilities for any of the proposed improvements for this project. The contract documents will require the construction contractor to verify utilities before start of construction.

Table 4. Existing Utilities along Sunland Park Levees

Utility Station	Description	Owner	Contact Information	Notes	Known Conflict?
EAST LEVEE					
1511+19	Railroad Tracks	BNSF Railway Co.	(915) 534-2364 805 S. Santa Fe St. El Paso, TX 79901	NA	No
1517+59	Overhead Electric	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171, El Paso, TX 79901	Runs parallel to Borderland Rd. along the north edge of the bridge.	No
1518+08	Natural Gas Line	Texas Gas Service (TGS)	Saul Magallanes saul.magallanes@onegas.com (o) 915-682-7356 (c) 915-727-6131 4700 Pollard St., El Paso, TX 79930	6” Steel gas line encased in 10” steel south of Borderland Rd. Bridge with 3’ minimum cover	No
1519+64	Water Line	El Paso Water (EPW)	Amy Castner acastner@epwater.org (915) 594-5647 OR Al Quijano pquijano@epwater.org 1154 Hawkins Blvd. El Paso, TX 79925	36” Concrete H.P. just south of Borderland Rd. El. 3756 TOP	No
1543+57	Overhead Electric	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171, El Paso, TX 79901	North of Artcraft Rd. bridge - Runs parallel to OE 1543+87	No
1543+87	Overhead Electric	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171, El Paso, TX 79901	North of Artcraft Rd. bridge - Runs parallel to OE 1543+57	No
1545+48	Electrical Conduit	TXDOT	Susan Ryde Susan.ryde@txdot.gov (915) 790-4207	Attached to northside of Artcraft Rd. bridge	No
1570+80	Overhead Electric	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com	Runs parallel with east levee - Behind residences on Sixta Dr. between	No

Utility Station	Description	Owner	Contact Information	Notes	Known Conflict?
			100 N. Stanton St. Suite 171, El Paso, TX 79901	Ontiveros St. and Jardin Bello Ave.	
1619+55	Sanitary Sewer	El Paso Water (EPW)	Amy Castner acastner@epwater.org (915) 594-5647 OR Al Quijano pquijano@epwater.org 1154 Hawkins Blvd. El Paso, TX 79925	3729 TOP; 24" I.D. with 40"steel casing - Along Maime Rd.	No
1666+63	Electrical Conduit	City of El Paso (CoEP)	Basilio Ontiveros Traffic Signals Maintenance Supervisor Communications Streets & Maintenance ontiverosbx@elpasotexas.gov Office: (915) 212-7075	3.5' at junction box, goes down about 6' as it approaches the levee. Attached to northside of Country Club Rd. Bridge	No
1666+79	Overhead Electric	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171, El Paso, TX 79901	Runs parallel to Country Club Rd. Bridge	No
1666+86	Water Line	El Paso Water (EPW)	Amy Castner acastner@epwater.org (915) 594-5647 OR Al Quijano pquijano@epwater.org 1154 Hawkins Blvd. El Paso, TX 79925	5 ft min cover. 12" A.C encased in steel pipe - North side of Country Club Bridge	No
1667+00	Fiber Optic Line	AT&T	Shawn Sather OSP Planning & Engineering ss1038@att.com Cell: 915-342-3172 Ofc: 915-595-5118	4" Line, Fiber optic line attached to southside of Country Club Rd. Bridge. 8.5’ from top of FO to MH cover. Contact AT&T to verify location	No
1667+00	Fiber Optic Line	AT&T	Shawn Sather OSP Planning & Engineering ss1038@att.com Cell: 915-342-3172 Ofc: 915-595-5118	4" Line, Fiber optic line attached to southside of Country Club Rd. Bridge. 8.5’ from top of FO to MH cover. Contact AT&T to verify location (South side of CC Bridge)	No
1667+06	Natural Gas Line	Texas Gas Service (TGS)	Saul Magallanes saul.magallanes@onegas.com (o) 915-682-7356	Line is attached to bridge. Elevation	No

Utility Station	Description	Owner	Contact Information	Notes	Known Conflict?
			(c) 915-727-6131 4700 Pollard St., El Paso, TX 79930	approaching the levee is unknown. 4" steel line, attached to southside of Country Club Rd. Bridge	
1667+42	Fiber Optic Line	AT&T	Shawn Sather OSP Planning & Engineering ss1038@att.com Cell: 915-342-3172 Ofc: 915-595-5118	4" line, Attached to southside of Country Club Rd. Bridge. Approximately 8' cover from top of conduit to MH cover. Contact AT&T to verify.	No
1667+48	Overhead Electric	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171, El Paso, TX 79901	Runs above southside of Country Club Rd. Bridge	No
1687+00	Natural Gas Line	New Mexico Gas Co.	Gabriel Ortiz Operations Supervisor - Southern Rio Grande Area Gabriel.Ortiz@nmgasco.com O: (505) 697-5408 c: (915) 497-3485 350 Acosta Road, Anthony, NM 88021	3.5 feet cover. Discovered during previous construction of East Levee toe drain	Yes
1701+23	Overhead Electric	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171, El Paso, TX 79901	Crosses Rio Grande and East levee from Santa Teresita Dr. towards Rinconada Ln.	No
1706+33	Natural Gas Line	New Mexico Gas Co.	Gabriel Ortiz Operations Supervisor - Southern Rio Grande Area Gabriel.Ortiz@nmgasco.com O: (505) 697-5408 c: (915) 497-3485 350 Acosta Road, Anthony, NM 88021	3.5 ft from surface. Parallel with east levee	No
1730+42	Natural Gas Line	New Mexico Gas Co.	Gabriel Ortiz Operations Supervisor - Southern Rio Grande Area Gabriel.Ortiz@nmgasco.com O: (505) 697-5408 c: (915) 497-3485 350 Acosta Road, Anthony, NM 88022	3.5 ft from surface. Crosses levee, approx. 1730+42	No

Utility Station	Description	Owner	Contact Information	Notes	Known Conflict?
1730+80	Natural Gas Line	New Mexico Gas Co.	Gabriel Ortiz Operations Supervisor - Southern Rio Grande Area Gabriel.Ortiz@nmgasco.com O: (505) 697-5408 c: (915) 497-3485 350 Acosta Road, Anthony, NM 88021	3.5 ft from surface. Discovered during previous construction of East Levee toe drain	Yes
1735+69	Overhead Electric	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171, El Paso, TX 79902	South of La Mancha Ct.	No
1761+65	Overhead Electric	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171, El Paso, TX 79903	Runs parallel along west side of 48" Montoya Lateral	No
1762+67	Water Line	CRRUA	Brent Westmoreland Executive Director brentw@donaanacounty.org (575) 589-1075 4950 McNutt Rd. P.O. Box 429 Sunland Park, NM 88063	Approximately 3 ft cover. Crosses east levee, adjacent to 48" culvert (Sta. 1761+95), west of Equestrian Ln.	No
1807+35	Water Line	CRRUA	Brent Westmoreland Executive Director brentw@donaanacounty.org (575) 589-1075 4950 McNutt Rd. P.O. Box 429 Sunland Park, NM 88063	4’ min cover. 10" WL - crosses east levee behind residences on Apaloosa Dr.	No
1811+95	Overhead Electric	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171, El Paso, TX 79903	Crosses east levee behind residences on Apaloosa Dr.	No
1812+62	Water Line	CRRUA	Brent Westmoreland Executive Director brentw@donaanacounty.org (575) 589-1075 4950 McNutt Rd. P.O. Box 429 Sunland Park, NM 88063	6" WL - parallel with east levee, behind residences on Apaloosa Dr. before crossing levee at approx. 1812+62. Approx. @ 3733	No
1812+80	Water Line	CRRUA	Brent Westmoreland Executive Director brentw@donaanacounty.org (575) 589-1075 4950 McNutt Rd. P.O. Box 429 Sunland Park, NM 88063	Approx. 3’ cover. 6" WL - parallel with east levee near to drain	No

Utility Station	Description	Owner	Contact Information	Notes	Known Conflict?
1830+50	Water Line	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171, El Paso, TX 79903	24" WL – turns downriver along the levee riverward toe and continues to the Rio Grande Power Plant.	No
1836+00	Water Line	CRRUA	Brent Westmoreland Executive Director brentw@donaanacounty.org (575) 589-1075 4950 McNutt Rd. P.O. Box 429 Sunland Park, NM 88063	Approx. 3' cover. 6" PVC line discovered during previous construction of East Levee toe drain, crosses levee at 1836+00	Yes
1839+61	Water Line	CRRUA	Brent Westmoreland Executive Director brentw@donaanacounty.org (575) 589-1075 4950 McNutt Rd. P.O. Box 429 Sunland Park, NM 88063	3' cover. 6" PVC line discovered during previous construction of East Levee toe drain	Yes
1840+83	Fiber Optic Line	AT&T	Shawn Sather OSP Planning & Engineering ss1038@att.com Cell: 915-342-3172 Ofc: 915-595-5118 4700 Pollard St., El Paso, TX 79930	Approx. @ 3735. Crosses East Levee at Frontera Rd.	No
1840+92	Overhead Electric	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171, El Paso, TX 79903	Crosses East levee at Frontera Rd	No
1932+38	Sanitary Sewer	CRRUA	Brent Westmoreland Executive Director brentw@donaanacounty.org (575) 589-1075 4950 McNutt Rd. P.O. Box 429 Sunland Park, NM 88063	Approx. 3' cover. Crosses east levee road on the west side of Racetrack Dr. bridge	No
1932+65	Natural Gas Line	New Mexico Gas Co.	Gabriel Ortiz Operations Supervisor - Southern Rio Grande Area Gabriel.Ortiz@nmgasco.com O: (505) 697-5408 c: (915) 497-3485 350 Acosta Road, Anthony, NM 88021	Approx. 2.5' from top of NW side of bridge. Runs along west side of Racetrack Dr.	No
1932+77	Electrical Conduit	Verizon	Bruce Turkiewicz; Engr IV Spec-Ntwk Eng & Ops O: (915) 519 - 3362 c: (254) 721 - 8977	Runs along west edge of Racetrack Dr. Bridge. Approx. @ 3742.7	No

Utility Station	Description	Owner	Contact Information	Notes	Known Conflict?
			4045 Doniphan Park Circle El Paso, TX 79922		
1933+31	Fiber Optic Line	Verizon	Manny Armendariz Technician manny.armendariz@one.verizon.com	Approx. 4 ft deep, shallower at MHs. 3" FOC - south edge of racetrack drive bridge	No
1933+39	Water Line	CRRUA	Brent Westmoreland Executive Director brentw@donaanacounty.org (575) 589-1075 4950 McNutt Rd. P.O. Box 429 Sunland Park, NM 88063	4’ min cover. 12" WL - runs along east side of Racetrack Dr. bridge	No
1956+95	Water Line	El Paso Electric (EPE)	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171, El Paso, TX 79903	24" WL - above grade vertical offset crossing the Montoya drain riverward of the levee culvert headwall	No
WEST LEVEE					
1666+38	Water Line	EPW	Amy Castner acastner@epwater.org (915) 594-5647 OR Al Quijano pquijano@epwater.org 1154 Hawkins Blvd. El Paso, TX 79925	12" A.C encased in steel pipe. 5’ min cover.	No
1666+77	Fiber Optic Line	City of El Paso	Victor Pereda PeredaVM@elpasotexas.gov (915) 472-3549 OR Basilio Ontiveros Ontiverosbx@elpasotexas.gov (915) 212-7075	3.5’ deep at JBE and drops 6’ as it approaches the levee. 4” Line attached to upstream side of Country Club Bridge and then parallel to the road	No
1666+83	Fiber Optic Line	AT&T	Shawn Sather OSP Planning & Engineering ss1038@att.com Cell: (915) 342-3172 Ofc: (915) 595-5118	8’ min cover at MH. 4" Line. Contact AT&T to verify location.	No
1666+82	Overhead Electric	EPE	Raul Guel (915) 543-4015 raul.guel@epelectric.com	Runs above downstream of Country Club Rd.	No

Utility Station	Description	Owner	Contact Information	Notes	Known Conflict?
			100 N. Stanton St. Suite 171 El Paso, TX 79901	and continues parallel to the road	
1667+17	Natural Gas Line	TGS	Saul Magallanes saul.magallanes@onegas.com (o) (915) 682-7356 (c) (915) 727-6131 4700 Pollard St., El Paso, TX 79930	4" aboveground steel line, attached to downstream of Country Club Bridge and then parallel to the road. Approx. at El 3752.	No
1667+36	Electrical Conduit	AT&T	Shawn Sather OSP Planning & Engineering ss1038@att.com Cell: (915) 342-3172 Ofc: (915) 595-5118	Attached to downstream of Country Club Rd. Bridge and then parallel to the road. About 8 ft deep at the manhole. El at 3750	No
1667+36	Electrical Conduit	AT&T	Shawn Sather OSP Planning & Engineering ss1038@att.com Cell: (915) 342-3172 Ofc: (915) 595-5118	Attached to downstream of Country Club Rd. Bridge and then parallel to the road. About 8 ft deep at the manhole. El at 3751	No
1696+82	Overhead Electric	EPE	Raul Guel (915) 543-4015 raul.guel@epelectric.com 100 N. Stanton St. Suite 171 El Paso, TX 79901	Runs parallel to the levee and crosses the levee at Sta. 1696+83.	No

2.2 Survey Data

FXSA performed surveying services including topographic and boundary surveys of the USIBWC's ROW. The topographic survey and the boundary survey for both east and west levees were performed from November 2017 through February 2018 and from November 2017 through April 2018, respectively. The coordinates, bearings, and distances on the topographic and boundary surveys are based on the New Mexico Coordinate System of 1983, North American Datum of 1983 (NAD 83) (2011), Central Zone (3002). These are shown as grid distances and may be converted to surface distances by dividing by the combined scale factor of 0.99973833. The elevations for this project are based on the North American Vertical Datum of 1988 (NAVD 88), Geoid 2012B. The field survey included performing cross-section surveys along the levee alignment at 100-foot intervals for the entire project reach. The survey data were processed to generate a topographic map with elevation contours using Autodesk Civil 3D Software.

3 GEOTECHNICAL

A geotechnical memorandum describing the geotechnical basis of design is included in Appendix B.1. It should be noted that this geotechnical memorandum includes evaluations of both east and west levees per the original SOW.

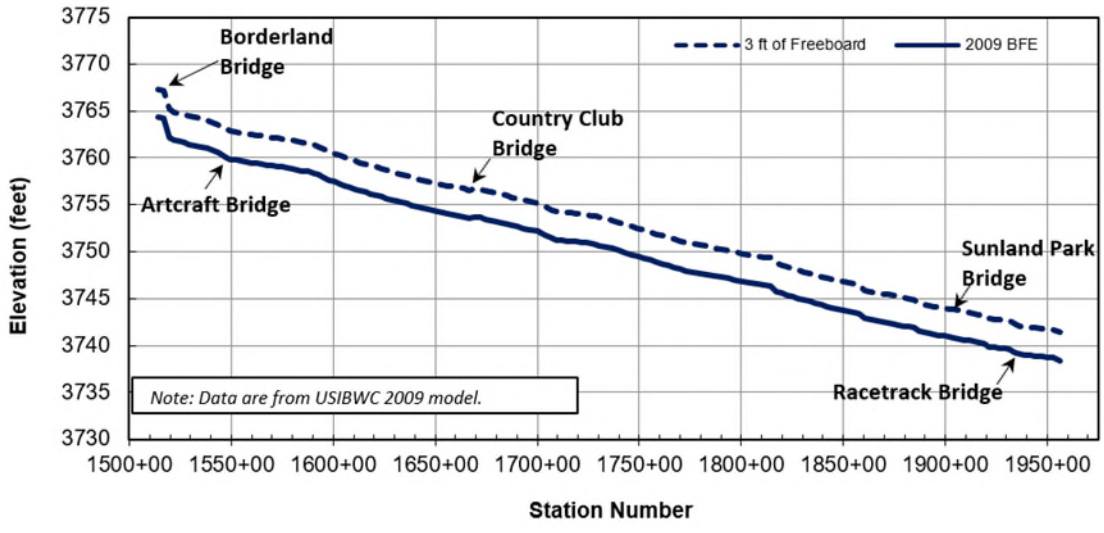
4 HYDROLOGY AND HYDRAULICS

4.1 Hydraulic Model

USIBWC provided the hydraulic model within the project limits. The SOW specified the use of the 2005 FLO-2D model developed by Tetra Tech. During the project, it was determined that the 2005 model provided was not running correctly; therefore, USIBWC provided us with a 2009 FLO-2D model. We used the 2009 USIBWC FLO-2D model to determine the effects of levee improvements on base flood elevations (BFE) and velocities. The BFE in this model indicates the 100-year flood condition. The BFEs extracted from the 2009 model were used as the basis for determining the required freeboard elevations under the 100-year flood condition.

The model was used to check that the project improvements to the existing levee would not have any impact on the hydraulic conditions. The current model and the required +3 feet freeboard requirements are shown on Figure 2. The required project levee elevations are based on the top of the new embankment fill and below the aggregate roadway as presented in the contract drawings. At the bridge locations, FEMA's minimum freeboard required is 4 feet within 100 feet of the bridge. However, based on a prior agreement between USIBWC and FEMA, the minimum freeboard required is 3 feet within 100 feet of the bridge. A table showing the station number, BFE and freeboard elevations is presented in Appendix B.5.

Figure 2. Base Flood Elevations (BFE) along River Channel

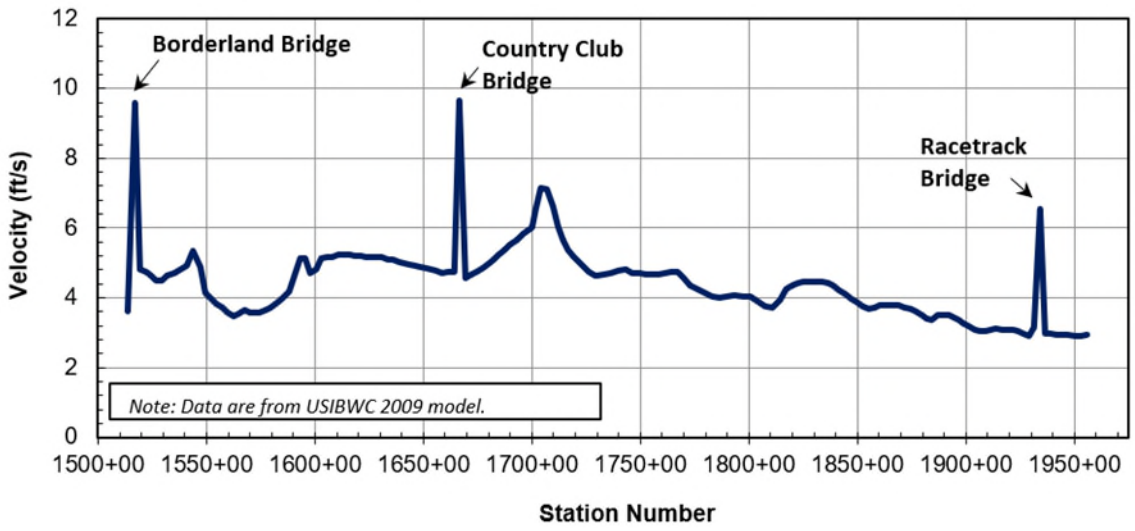


4.2 Channel Velocities

The estimated channel velocities for the 100-year flood extracted from the 2009 FLO-2D model are shown on Figure 3. Estimated velocities in the Rio Grande channel along the Sunland Park East Levee range from 2.89 feet per second (fps) to 9.64 fps. High channel velocities over 9 fps were encountered at the Borderland Bridge and at the Country Club Bridge locations. Channel velocities higher than 6 fps were encountered between Sta. 1695+00 and Sta. 1713+00 on East Levee and downstream of Sta. 1696+00 on the West Levee. It appears that the increase in the velocities at this portion of the levee reflects the narrower channel width. Riprap will be placed on the riverside levee slopes at these locations as shown on the contract drawings. The Racetrack Bridge also indicated velocities slightly higher than 6 fps. No remediation is planned at this location since bridge abutments are covered with wire encased stone riprap.

The difference between the velocities presented on Figure 3 and the velocities presented in Kenall's 2016 report are likely due to the different hydraulic models used in the previous design and our design. The previous design was based on the USIBWC 2003 Hydraulic Model, and we used the USIBWC 2009 FLO-2D Model for our design.

Figure 3. Simulated Maximum Channel Velocities along River Channel



5 PROJECT AND FEMA REQUIREMENTS

5.1 General

The design objectives for this project are to meet FEMA requirements in 44 CFR §65.10(b) (1) through (7), which include: 1) freeboard, 2) closures, 3) embankment protection, 4) embankment and foundation stability, 5) settlement, 6) interior drainage, and 7) other design criteria for unique situations. The new levee should also meet the requirements of the USACE's Engineering Manual 1110-2-1913 titled "Design

and Construction of Levees" (USACE 2000). This section describes the project requirements and basic design criteria for the project.

The SOW for this project required addressing the deficiencies noted in Kenall's forensic report (Kenall 2016). After a review and evaluation of Kenall's forensic study, we found other issues that are not in conformance with the FEMA requirements based on the most recent survey and the 2009 FLO-2D model: 1) there is inadequate freeboard up to 10 inches at some locations along the alignment as described in Section 6; 2) there is inadequate freeboard in some locations near structures as described in more detail in Section 6.1; and 3) there are higher channel velocities than those reported in Kenall's report as described in Section 4.2.

5.2 Levee and Geotechnical Design

The levee design criteria are based on addressing the dispersive clay that exists throughout the levee reach as identified by Kenall. To establish the earthwork design criteria, we evaluated 11 alternatives for treatment of dispersive clay as described more fully in Appendix B.1. Following the design scoping meeting held on December 13, 2017, the USIBWC selected the clay blanket option; thus, dispersive clay would be replaced with imported, non-dispersive compacted clay. This alternative will address the deficiencies regarding dispersive clay noted in the forensic report by Kenall.

This alternative, as originally presented by Kenall, involves excavating at least 2 feet of the existing dispersive clay soil and replacing it with imported clay that is not dispersive. The imported clay will be moisture-conditioned and compacted to achieve a well-constructed envelope of non-dispersive soil around the core of the existing levee, which may contain dispersive clay. The dispersive clay at the levee core will be isolated, reducing the threat of internal or external erosion.

The imported non-dispersive clay borrow should classify as CL in accordance with ASTM International (ASTM) D2487, Unified Soil Classification System, with a plasticity index between 15 to 30 and a maximum liquid limit of 45 percent (%). Fat clay should not be used due to the higher susceptibility to desiccation cracking.

The clay borrow must be non-dispersive and meet the testing requirements listed below:

- Crumb Test (ASTM D6572) - Grade 1 (non-dispersive)
- Pinhole Test (ASTM D4647) - ND1 or ND2
- Double Hydrometer (ASTM D4221) – Percent dispersive less than 30
- At least 65% passing a U.S. Standard No. 200 sieve
- One hundred percent (100%) of the dry material passing the 1-inch sieve
- Contain less than 35% sand content by weight.

The embankment fill will be compacted to at least 95% of maximum density at a moisture content above optimum moisture content as determined by ASTM D698, Standard Proctor. Other levee and geotechnical design criteria include:

- Freeboard will be at least 3 feet above the BFE. The gravel road on the levee will not be included as part of the 3-foot freeboard requirement.

- The levee footprint will not be moved from its currently existing alignment.
- The levee slopes horizontal:vertical (H:V) will be 2.5H:1V or flatter (USIBWC prefers 3H:1V slopes where possible).
- The finished gravel road will be at least 16 feet wide and a minimum of 6 inches thick. The maintenance roadway will slope toward the riverside at 2% along the alignment.
- Ramps along the levee will be designed as shown in the SOW.
- Dispersive clays will not be used in construction.
- Results of seepage analysis will be incorporated into stability analysis.
- Soil properties are based on the previous studies provided by USIBWC.
- Factors of safety will meet the USACE requirements.

Sections 5.3 through 5.9 were considered in the design to satisfy the FEMA and USACE requirements.

5.3 Freeboard Requirements

We designed the final elevation of the non-dispersive clay cap to be at least 3 feet above the BFE (100-year flood). The BFE is extracted from the 2009 FLO-2D model provided by USIBWC. The landside will be slightly higher due to the 2% cross-slope requirements. The gravel roadway is not included in the freeboard requirements. At the bridge locations, FEMA requires the minimum freeboard to be at least 4 feet within 100 feet of the bridge. However, based on a prior agreement of USIBWC with FEMA, the minimum freeboard required is 3 feet within 100 feet of the bridge. There are three bridge crossing locations (East Levee - South Borderland Bridge, East Levee - North Country Club, and West Levee - South Country Club Bridge) along the levee reaches where the levee height needs to be increased to 3 feet at/or near the bridge to meet minimum freeboard requirements.

5.4 Closures

Access to the levee closures (sluice gates) that are too low to be operated in floods will be raised to be above the 100-year flood elevations. The levee closures are explained in more detail in Section 8.

5.5 Embankment Protection

The proposed levee is designed to be protected with at least 2 feet of non-dispersive clay cap. The installation of the new non-dispersive clay, topsoil, and vegetative cover is anticipated to protect the levee from erosion due to the floodwaters acting against the levee for short periods of time.

In addition, to evaluate the erosion, the estimated channel velocities for the 100-year flood were compared with published values of permissible velocities. Based on the USACE Engineer Manual (EM) 1110-2-1601 (USACE 1994), the maximum mean channel velocities on clay are 6.0 fps and on grass-covered clay are 8 fps. Channel velocities in the Rio Grande channel along the levees range from 2.89 fps to 9.64 fps.

Channel velocities over 9 fps are expected at the bridge locations. Borderland Bridge and Country Club Bridge abutments are covered with concrete slope paving. Racetrack Bridge and Sunland Park Bridge abutments are covered with wire encased stone riprap. As a result, no significant hydraulic impact should exist at the bridge location.

Channel velocities of slightly higher than 6 fps were encountered downstream of Sta. 1696+00 on the West Levee and corresponding levee segment across the river on the East Levee from Sta. 1695+00 to Sta. 1713+00, most likely due to the narrower channel width at that cross section. Riprap will be required on the riverside levee slopes on these levee segments for both West and East Levee.

During rain and flood events, the water may accumulate at the toe. Toe drains were designed at the East Levee toe with riprap every 50 feet to reduce erosion due to water accumulation. Specific details are shown in the contract drawings. The distance of 50 feet is a practical number with the intent to get the water out of the toe so that it does not erode.

For West Levee, due to the limited clearance for construction, the toe drains cannot be extended similarly to East Levee. Therefore, we recommend that the riprap be grouted with flowable fill at the toe to create a less permeable material so that the water daylights. Contract drawings provide more details of this design.

For the remainder of the alignment, results of the H&H analysis indicate that there is no significant hydraulic impact to the levee slopes as the anticipated velocities on the levee slopes are lower than the mean permissible velocities in the channel.

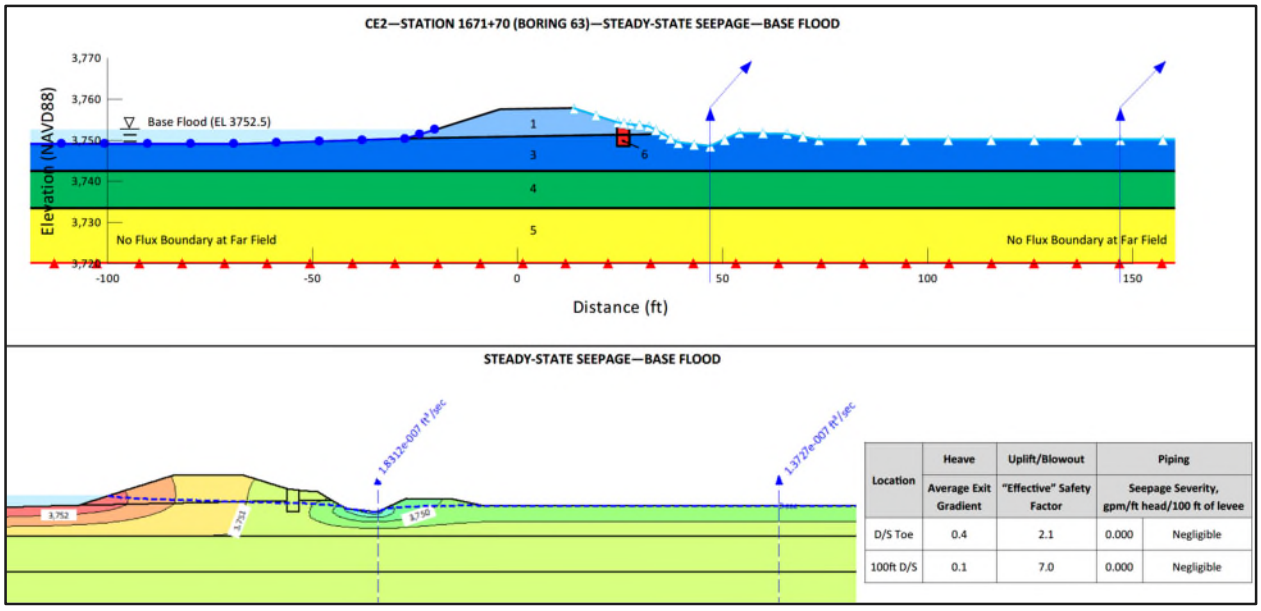
5.6 Embankment and Foundation Stability

5.6.1 Seepage Analysis

The previous design (S&BI 2010) included toe drains on the landside to reduce the seepage exit gradients at the landside toe. The seepage analyses in the previous design assumed a steady-state condition, which implies that the water will stand against the levee long enough to enable development of steady-state seepage conditions. Given the relatively short duration of a flood event, Arcadis is in the opinion that a transient seepage condition should be modeled for the seepage analysis in lieu of the steady-state seepage as there is not sufficient time for steady-state seepage to develop.

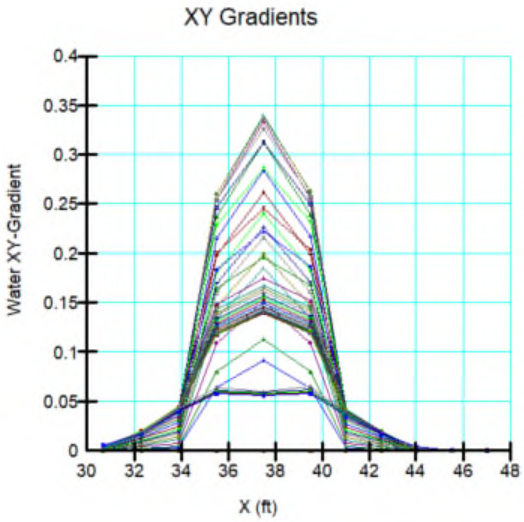
The forensic study from Kenall (2016) included steady-state seepage analyses for three cross sections with toe drains on the landside. The exit gradients at CE2 Sta. 1671+70 from Kenall are shown on Figure 4. Arcadis analyzed transient seepage without toe drains at CE2 Sta. 1672+00 using Kenall's soil properties without toe drains. The resulting exit seepage gradients were less than 0.5, which is the maximum permissible value per USACE requirements. Based on these findings, Arcadis believes that toe drains are not needed on the landside. The results of the seepage analysis without toe drains are shown on Figure 5.

Figure 4. Steady-State Seepage Results East Levee with Toe Drains (from Kenall 2016)



Location	Heave	Uplift/Blowout	Piping	
	Average Exit Gradient	"Effective" Safety Factor	Seepage Severity, gpm/ft head/100 ft of levee	
D/S Toe	0.4	2.1	0.000	Negligible
100ft D/S	0.1	7.0	0.000	Negligible

Figure 5. Exit Gradients at Levee Toe without Toe Drains – Transient Seepage (CE2 Sta. 1672+00)



Additional details for seepage analyses for Sta. 1672+00, 1722+00 and 1826+00 on the East Levee and Sta. 1675+00 on the West Levee are presented in Appendix B.2. For these analyses, we have assumed the subsurface profile and soil properties provided by USIBWC based on historic studies.

Transient Seepage Model: For the long-term condition, we performed transient seepage analysis to determine the pore pressures developed during the 100-year flood. We evaluated the seepage conditions using Geo-Slope software SEEP/W, a finite element program. Seepage evaluations were performed using transient seepage analysis based on the hydrograph extracted from the FLO-2D model provided by USIBWC as shown on Figures 6 and 7. No toe drains were assumed in these seepage models.

Figure 6. Elevation Hydrograph at CE2 Sta. 1672+00

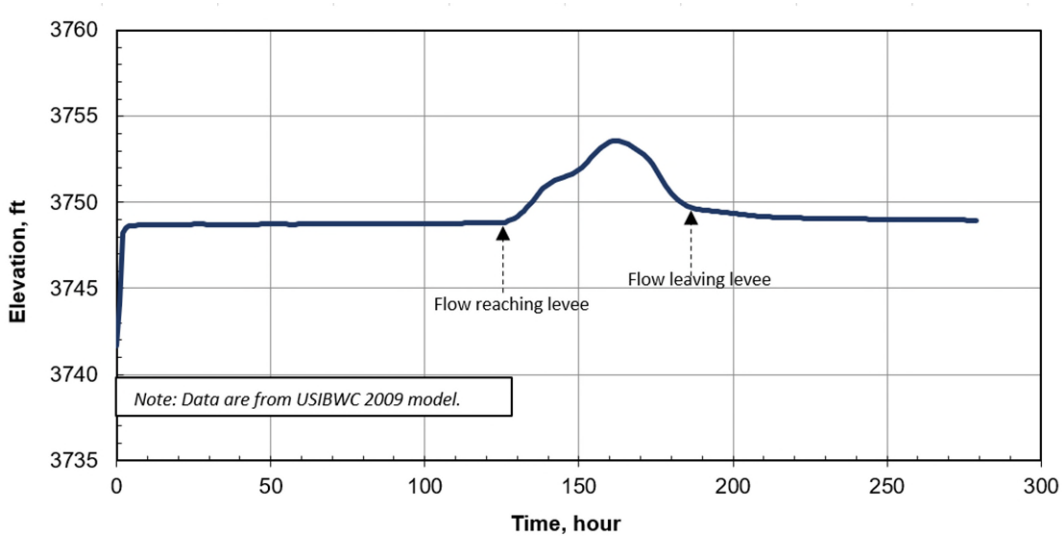
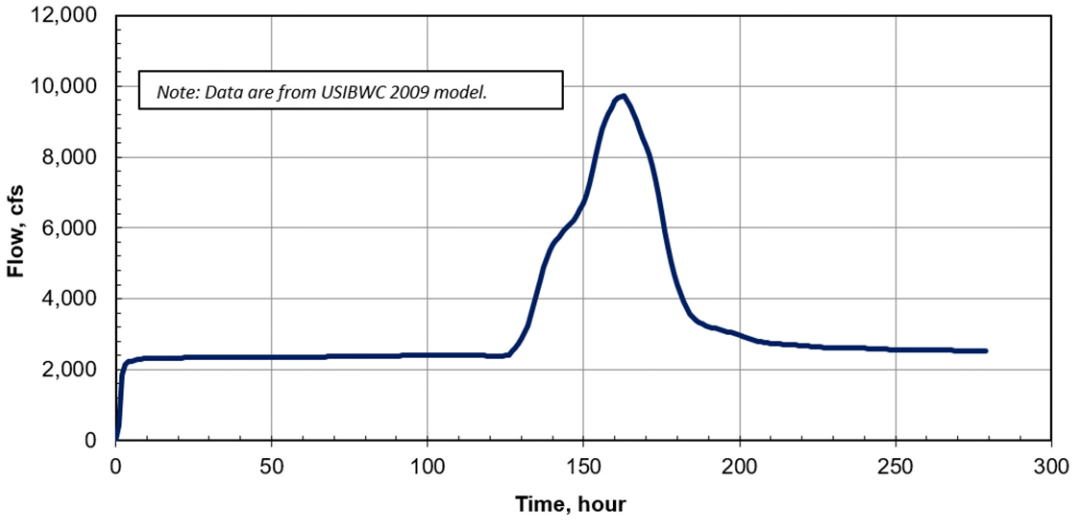


Figure 7. Flow Hydrograph at CE2 Sta. 1672+00



Unsaturated Characteristics: The saturated permeability values (Table 5) used in the analysis were provided by USIBWC based on historic studies. We selected unsaturated soil characteristics for the materials above the initial groundwater. For the transient unsaturated analysis, the Soil-Water Characteristic Curves (SWCC) (i.e., matric suction versus volumetric water content) of the materials were estimated from the sample functions available in GeoStudio. The values of zero-pressure saturated volumetric content used for the soil layers ranged from 0.38 for sand to 0.5 for clay materials. These values were based on the typical volumetric water contents presented in SEEP/W manual. Given the SWCC and saturated permeability, the unsaturated permeability versus matric suction functions of the

materials were predicted based on the equation of Van Genuchten method included in the SEEP/W software.

Table 5. Soil Properties used for Seepage Analysis

Material	Hydraulic Conductivity (fps)	Vertical to Horizontal Hydraulic Conductivity Ratio
Clay Blanket	4.9E-08	0.33
Sandy Lean Clay (levee) (1)	4.9E-08	0.33
Sandy Lean Clay (1)	3.0E-08	0.10
Fat Clay (1)	3.0E-08	0.10
Silty Sand (1)	5.0E-05	0.33
Poorly graded Sand (w. silt) (1)	3.0E-04	0.33
Note: (1) Soil properties provided by USIBWC		

Boundary Conditions: We applied two types of boundary conditions for the seepage analysis. For transient seepage analysis, the floodside boundary condition consisted of the 100-year flood event. On the protected side, we applied the seepage face review option for the boundary condition. Seepage face boundary conditions are used when the phreatic surface intersection with the ground surface needs to be determined.

Seepage Analysis Results and Conclusions: In general, a seepage exit gradient of 0.5 or less near the toe of the levee slope (where the exit gradient is defined as the ratio of entry pore pressure divided by the soil weight of an element at the discharge point) is considered acceptable. The allowable exit gradient increases linearly from 0.5 at the landside levee toe.

The transient seepage analyses indicate that significant infiltration of water from flood flows into the river side of the levee embankment is unlikely to result in a steady-state condition. Both cross sections that we analyzed indicated exit gradients of less than 0.5, which is the permissible value.

5.6.2 Slope Stability Analysis

Previous geotechnical studies from Kenall (2016) and S&BI (2008, 2010) indicated that cross sections along the levee were evaluated for seepage and slope stability. Based on our review of these previous geotechnical studies, the slope stability factors of safety were adequate for the proposed work. In this project, minor fills of less than one foot will be added to the levee portions that are being widened to satisfy the 16-foot-wide gravel roadway criteria. Typically, a wider levee means a longer seepage path and an increased stability. Therefore, seepage and slope stability will not be adversely affected by the proposed levee improvements and may be improved where new levees will be wider than the existing levees.

Arcadis acknowledges that we have evaluated the slope stability runs included in Kenall's forensic report based on the soil properties included in Kenall's report provided by USIBWC. The slope stability factors of safety appear to satisfy the minimum criteria. In addition, Arcadis selected three cross sections on the East Levee and one cross section on the West Levee to be analyzed independently from the previous design. The cross sections selected correspond to Sta. 1672+00, 1722+00 and 1826+00 on the East Levee and Sta. 1675+00 on the West Levee.

Slope stability analyses were evaluated for four loading conditions, or critical cases, per USACE EM 1110-2-1913 "Design and Construction of Levees," which include:

- **End of Construction**: This analysis is appropriate for levee raises as proposed in our design. Short-term (Undrained) stability analysis applies to the stability evaluation at the end-of-construction. Total stress analysis with undrained shear strength properties is typically used for low-permeability soils, and effective stress analysis with drained shear strength properties is typically used for free-draining soils. Slope stability was checked for both landside and riverside.
- **Rapid Drawdown:** This case assumes that a prolonged flood level drops faster than the soil can drain. The floodside slope may become unstable due to development of excess pore water pressures.
- **Long-Term Stability (100-year Flood)**: This condition is critical for the landside of the embankment and applies to stability evaluation for a transient seepage condition. The transient seepage conditions represent a period when the excess pore pressure generated during the design hydrograph has peaked within the levee embankment and foundation. Effective stress analysis with drained shear strength properties is used for free-draining soils, while total stress analysis with undrained shear strength properties is used for low-permeability soils. For the 100-year flood, the pore pressures needed in the slope stability analysis were imported directly from the seepage analysis model.
- **Seismic (Earthquake) Stability Analysis**: This analysis was not performed as historical seismic events have not produced ground motions sufficient to require this analysis.

Slope stability analyses coupled with seepage analyses were performed using the Spencer's Method (a force and equilibrium method) as programmed into SLOPE/W. Stability of the levee slopes depends on several factors including but not limited to: maximum flood elevation and duration, levee geometry and berms, external loads, seepage paths, levee and foundation materials, soil compaction during construction, penetrations in the levee, and drainage layers. These factors are described in more detail as follows:

- **Flood**: Maximum flood elevation and duration is based on the hydrograph for the 100-year flood extracted from the FLO-2D model as presented on Figure 6. From Figure 6, it can be concluded that, during the 100-year event, the base flood reaches the peak for about 1.5 days (time between peak and when flow reaches the levee) and falls back to the levee toe at about 1.1 days (time between peak and when flow leaves the levee). Due to this short duration, the design hydrograph has not been sufficiently long to establish steady-state seepage.
- **Geometry:** Levee geometry is described in more detail in Section 6.2. We have used levee slopes of 3H:1V for our analysis. Steeper slopes of 2.5H:1V were also analyzed for the cross section at CW2 Sta. 1675+00 as presented in Appendix B.2.

- **Geotechnical Loads:** A vertical surcharge loading of 250 pounds per square foot was assumed to act on the levee crest to reflect potential area loads from operation and maintenance equipment.
- **Seepage:** We evaluated seepage using transient seepage analysis using the hydrograph presented on Figures 6 and 7. Before the flood, the soils are in a partially saturated condition and the degree of saturation increases as the flood seeps through the levee and the levee foundation layers. Due to the short duration of the flood, the seepage path or the phreatic surface will be lower than a steady-state seepage condition as the soil layers will not have sufficient time to become fully saturated. A lower phreatic surface leads to a higher factor of safety for the levee.
- **Materials:** The subsurface stratigraphy and soil characteristics for the cross sections selected for analysis were obtained from Kenall (2016). The embankment fill material consists of sandy lean clay. The levee foundation consists in general of sandy lean clay and occasionally fat clay. Silty sand and sand underlie the clay soils. The specific stratigraphy included in our analyses is presented in Appendix B.2. For our analyses, we have assumed the soil properties based on historic studies provided from USIBWC (Table 6).

Table 6. Soil Properties used for Sunland Levees Slope Stability Analysis

Material	Moist Unit Weight (pcf)	Φ' (Effective / Drained) (degree)	C' (Effective / Drained) (psf)	Φ (Total / Undrained) (degree)	C (Total / Undrained) (psf)
Clay Blanket	127	29	50	0	1,000
Sandy Lean Clay (levee) (1)	127	29	50	18	100
Sandy Lean Clay (1)	127	28	50	11	175
Fat Clay (1)	118	28	100	13	325
Silty Sand (1)	131	32	0	--	--
Poorly graded Sand (w. silt) (1)	124	34	0	--	--

Notes: (1) Soil properties based on historic studies provided by USIBWC.

- **Compaction:** Proper soil compaction is required during construction for the clay blanket to achieve adequate strength. For our design, a 2-foot-thick clay blanket will encapsulate the existing levee. The new fill will be compacted to at least 95% of maximum density at a moisture content between 0 and +3% of optimum moisture.
- **Penetrations:** Several penetrations (structures and utilities) exist along the levee alignment. No seepage issues were noticed from our field observations from these levee penetrations. Penetrations and no seepage issues have been reported by USIBWC personnel. Therefore, we believe that the existing penetrations will not present any potential seepage path.

- **Drainage:** Toe drains were installed during the previous construction. The toe drains are about 3 feet wide by 4 feet deep gravel-filled trenches at the landside toe. Our analyses of transient seepage conditions indicate that toe drains are not needed along the levee. The existing toe drains will be left in place. Some of the existing toe drains may be damaged during construction. If damaged, the existing toe drains do not need to be repaired or replaced. However, a new geotextile layer extending 1 foot on each side will be placed above the toe drains and below the new embankment fill in locations where the toe drains are damaged during construction.

Slope Stability Results: The slope stability results are summarized in Table 7 and shown in Appendix B.2.

Table 7. Slope Stability Results

Cross Section	Case	Minimum Required Factor of Safety (1)	Analysis Type	Calculated Factor of Safety
EAST LEVEE				
Sta. 1672+00(2)	End of Construction - Landside	1.3	Total Stress Analysis	2.3
	End of Construction - Riverside	1.3	Total Stress Analysis	3.0
	100-Year Flood	1.4	Effective Stress Analysis	2.0
	Rapid Drawdown	1.2	Total Stress Analysis	1.9
Sta. 1722+00	End of Construction - Landside	1.3	Total Stress Analysis	3.1
	End of Construction - Riverside	1.3	Total Stress Analysis	3.0
	100-Year Flood	1.4	Effective Stress Analysis	2.3
	Rapid Drawdown	1.2	Total Stress Analysis	1.9
Sta. 1826+00	End of Construction - Landside	1.3	Total Stress Analysis	2.4
	End of Construction - Riverside	1.3	Total Stress Analysis	2.7
	100-Year Flood	1.4	Effective Stress Analysis	1.8
	Rapid Drawdown	1.2	Total Stress Analysis	1.7
WEST LEVEE				
	End of Construction - Landside	1.3	Total Stress Analysis	3.0

Cross Section	Case	Minimum Required Factor of Safety (1)	Analysis Type	Calculated Factor of Safety
Sta. 1675+00	End of Construction - Riverside	1.3	Total Stress Analysis	3.0
	100-Year Flood	1.4	Effective Stress Analysis	2.4
	Rapid Drawdown	1.2	Total Stress Analysis	1.9
Sta. 1675+00 (2.5H:1V Slope on Landside)	End of Construction - Landside	1.3	Total Stress Analysis	2.6
	End of Construction - Riverside	1.3	Total Stress Analysis	3.0
	100-Year Flood	1.4	Effective Stress Analysis	2.0
	Rapid Drawdown	1.2	Total Stress Analysis	1.9

Note: (1) These factors of safety are recommended by USACE EM 1110-2-1913 for steady-state conditions.

(2) The drain was assumed empty in the analysis.

5.7 Settlement

Detailed settlement analyses performed in previous studies were evaluated for the proposed work. Due to the age of the levee and the presence of sandy soils, no further consolidation settlement associated with the original levee construction is expected to occur. Settlement analyses presented in Kenall (2016) indicate that a maximum total settlement of 4 inches is anticipated at any location along the levee. As there have been several years since the last construction, we anticipate that most of the settlement has already occurred, and some minor settlement is anticipated for the self-weight of the new fill. A minor increase in height is needed for levee portions that are being widened.

We evaluated potential future settlement for the construction of 2 feet of clay blanket as presented in Appendix B.3. The settlement analysis was based on the USACE EM 1110-1-1904 “Soil Mechanics Design – Settlement Analysis” per FEMA criteria. To accommodate the settlement of the levee during placement of the clay cap, 3 inches of overbuild will be included in the proposed levee designs.

5.8 Interior Drainage

The interior drainage systems associated with the levees usually include storage areas, gravity outlets, pumping stations, or a combination thereof to control interior drainage. In the event that water ponds against the landside of the levee during heavy rainfall, we evaluated the effect of 1 foot of standing water on the landside on embankment and foundation stability. A conservative steady-state seepage condition was analyzed assuming 1 foot of standing water on the landside for as shown in Appendix B.2. The slope stability analyses indicate factors of safety above 2 for these conditions, which are adequate based on the minimum factors of safety in Table 7. Based on our review of surface topographic conditions on the landside of the levee, we do not expect water to pond against the levees for long periods of time, and we do not expect surface runoff on the landside to cause erosion of the levees.

5.9 Other Design Criteria for Unique Solutions

Other design criteria for unique solutions are not applicable in this project.

6 CIVIL DESIGN

6.1 Levee Crossings

Based on the most recent topographic survey completed by FXSA in 2017 and 2018, the existing levee elevations are typically greater than 6 inches above the minimum freeboard requirement especially from South of the Country Club Bridge to the end of the project on the East and West levees. More variation between the existing levee and the design elevations is occurs within 100 feet of the bridges, where the FEMA freeboard requirement is typically 4 feet. However, due to an agreement of USIBWC with FEMA, the required freeboard within 100 feet of the bridges is 3 feet. Based on that agreement and at the direction of USIBWC, 3 feet of freeboard has been maintained for the entire levee segment including the bridge crossings.

6.1.1 East Levee Borderland Bridge Transition

The northern (upstream) end of the Borderland Bridge currently meets the minimum freeboard requirement with the existing levee height and concrete floodwall. The existing floodwall is approximately 55 feet long and starts at Sta 1516+99. The new levee design has a positive cross slope of 2% away from the floodwall. The centerline alignment will slope down at 6% from Sta. 1516+78 to a low point at Sta. 1517+48 to the edge of the road at Sta. 1517+53. This will ensure positive drainage of the levee road from east to west. The levee crest widens to approximately 29 feet at the edge of the road to allow access to the levee from the W. Borderland Rd. A portion of the levee at the edge of the road cannot be used for access due to the existing impact attenuator.

The southern (downstream) end of the Borderland Bridge has an existing floodwall that extends south of the bridge and meets the minimum freeboard requirements. The existing floodwall is approximately 31 feet long and starts at Sta. 1518+08. However, the existing levee does not meet the minimum freeboard requirements at the end of the floodwall at Sta. 1518+44. The existing ground elevation (El.) at the end of the floodwall is 3764.77 and the minimum freeboard requirement is El. 3765.97. At the centerline alignment, the proposed ground elevation will be raised to El. 3766.22. The new levee design has a positive cross slope of 2% away from the floodwall. This will ensure positive drainage of the levee road from east to west. From the edge of the road at Sta. 1518+04 to Sta. 1518+12 the elevation will remain constant at El. 3764.85. The centerline alignment will slope up at 6% from Sta. 1518+12 to 1518+35 which at this point the centerline alignment will start sloping down at 0.5% to Sta. 1519+20. Due to ROW constraints, the landside hinge follows the ROW from the edge of the road to Sta. 1518+42 at this point, and the landside hinge transitions to a typical 8.5 feet east of the centerline alignment at Sta. 1518+94.

6.1.2 East Levee Country Club Bridge Transition

The northern (upstream) end of the Country Club Bridge currently does not meet the minimum freeboard requirement with the existing levee height and concrete floodwall. From the edge of the road to the end of the floodwall at Sta. 1666+29, the floodwall is sloping down to El. 3756.11 and the existing ground is El. 3755.55. At the centerline alignment, the proposed ground elevation at the north end of the floodwall is El. 3757.03. The floodwall will be raised to a constant elevation of El. 3758.08 which is elevation of the floodwall at the edge of the road. The new levee design has a positive cross slope of 2% away from the floodwall. The centerline alignment will slope down at 6% from Sta. 1666+34 to a low point at Sta. 1666+71 to the edge of the road at Sta. 1666+73. This will ensure positive drainage of the levee road from east to west and upstream to downstream. On the landside side of the crest it starts to widen at Sta. 1666+26 to the edge of the road at Sta. 1666+87. The levee crest width at edge of the road is approximately 28 feet to allow access to the levee from the Country Club Bridge Rd. A portion of the levee at the edge of the road cannot be used for access due to the existing impact attenuator. Due to the proximity of a drainage canal and structure at the landside toe the slope in this area is 2.5H:1V.

The southern (downstream) end of the Country Club Bridge has an existing concrete floodwall that extends south of the bridge and meets the minimum freeboard requirements. The existing floodwall is approximately 170 feet long and ends at Sta. 1668+66. The new levee design has a positive cross slope of 2% away from the floodwall. This will ensure positive drainage of the levee road from east to west. From the edge of the road at Sta. 1667+41 to Sta. 1667+65 the elevation will remain constant at El. 3754.59. The centerline alignment will slope up at 3% from Sta. 1667+65 to 1668+62 which at this point the centerline alignment will stay constant at El. 3757.50 to Sta. 1671+02.

6.1.3 East Levee Sunland Park Bridge Transition

The western (upstream) end of the Sunland Park Bridge currently meets the minimum freeboard requirement with the existing levee height. The new levee design has a positive cross slope of 2% away from the riverside edge of crest. Existing rock riprap is on the riverside slope and the crest will meet the existing rock riprap. The levee crest transitions from a typical 17-foot-wide crest at Sta. 1905+43 to 34.5 feet wide at the edge of the road. The centerline alignment will slope up at 6% from Sta. 1905+03 to Sta. 1906+09. At Sta. 1906+09 the centerline alignment will remain constant at El. 3750.60 to the edge of the road at Sta. 1906+29.

The eastern (downstream) end of the Sunland Park Bridge currently meets the minimum freeboard requirements with the existing levee height. The new levee design has a positive cross slope of 2% away from the riverside edge of crest. Existing rock riprap is on the riverside slope and the crest will meet the existing rock riprap. The levee crest is wider at the edge of road and transitions to a typical 17-foot-wide crest from edge of road to Sta. 1907+75. From Sta 1907+75 to 1909+78, the landside crest will meet the existing concrete paving `on the landside slope. The existing concrete paving is noted to be in stable condition and does not need to be replaced. From the edge of the road at Sta. 1907+05 to Sta. 1907+30 the elevation will remain constant at El. 3752.04. The centerline alignment will slope down at 5.00% from Sta. 1907+30 to 1907+75 which at this point the centerline alignment will start sloping down at a more gradual slope of 4.10% to Sta. 1908+77. At Sta. 1908+77 the centerline alignment starts sloping at 2.50%

and starts a more gradual slope of 1.67% from Sta. 1909+01 to 1909+61. At this point the centerline alignment will remain constant at El. 3744.00 until Sta 1911+01.

6.1.4 East Levee Racetrack Bridge Transition

The northern (upstream) end of the Racetrack Dr. Bridge meets the current freeboard requirements with the existing levee height. The new levee design has a positive cross slope of 2% away from the riverside edge of crest. The riverside edge of crest from the end of levee ramp ER-18 at Sta. 1929+98 to the edge of pavement follows the existing riverside edge of crest. The landside edge of crest could not follow the existing edge of crest due to distance from the Montoya Intercepting Drain. The landside edge of crest maintains 8.5 feet from the centerline alignment until Sta. 1932+13 where the crest widens with a 50-foot radius curve which ends at Sta. 1932+51 and approximately 25 feet from the centerline alignment. The landside levee crest meets the edge of road at Sta 1932+52. The landside slope is 2.5H:1V to not encroach into the Montoya Intercepting Drain or headwall structure. The centerline alignment will maintain El. 3743.00 from Sta. 1926+01 to Sta. 1930+69 at this point the centerline alignment will slope up at 2.5% to Sta. 1932+01 and maintain El. 3746.30 until it meets the edge of road at 1932+56.

The southern (downstream) end of the Racetrack Dr. Bridge meets the minimum freeboard requirements with the existing levee height. The new levee design starts at the end of the existing pavement with a typical 2% cross slope and 17-foot-wide crest. The centerline alignment maintains an El. 3746.03 from the edge of pavement at Sta. 1934+14 to Sta. 1934+39. From Sta. 1934+39 the centerline alignment slopes down at 2.5% to Sta. 1935+53 at which point the centerline alignment will slope down at a more gradual 0.5% slope to Sta. 1936+91.

6.1.5 West Levee Country Club Bridge Transition

The northern (upstream) end of the Country Club Bridge was addressed in a previous submittal and that portion of the levee is currently under construction. The bridge abutments are covered with existing concrete paving. However, there is an open area on the upstream side of Country Club Bridge where no concrete slope paving exists. We recommend that this open area is filled with concrete to match with existing concrete paving and reduce any seepage during flood events.

The southern (downstream) end of the Country Club Bridge has an existing floodwall that extends south of the bridge and meets the minimum freeboard requirements. The existing floodwall is approximately 136 feet long and ends at Sta. 1668+32. At the end of the existing floodwall, the existing levee must be raised to meet the minimum freeboard requirements, the freeboard requirement is El. 3756.60, and the existing ground is El. 3755.85. At the centerline alignment, the proposed ground elevation at the end of the floodwall will match the existing floodwall El. 3756.81. The new levee design has a positive cross slope of 2% away from the floodwall. This will ensure positive drainage of the levee road from west to east. From the edge of the road at Sta. 1666+69 to Sta. 1667+10, a 17-foot-wide concrete driveway has been designed to provide better access from the Country Club Rd. to the levee. The 50-foot radius curb returns to meet existing sidewalk will help make the turn from the east easier for vehicles. The design of the concrete sidewalk follows the TxDOT Standard Drawings PED-18, see reference drawings for information. The levee crest maintains a 17-foot-wide crest from the edge of the concrete floodwall to the west. The centerline alignment from Sta. 1667+10 to Sta. 1667+39 maintains a constant El. 3754.94. At

Sta. 1667+39 the centerline alignment slopes up at 2% to Sta. 1668+43 at El. 3757.01 where it begins to slope up at 0.05% to Sta. 3757.50.

It was noticed that some of the concrete paving on the downstream side of Country Club Bridge is severely cracked. We understand that the purpose of this concrete cover is for erosion protection of the bridge abutment. While the concrete is cracked, it should still serve its purpose of erosion control. We recommend that this area be evaluated on a regular basis and severity of the cracks be recorded during the operation and maintenance process.

It was brought up by USIBWC during the 90% design review that existing upstream wall is cracked. Per USIBWC, the transverse crack was seen in March of 2019 and does not seem to have changed, but there was no longitudinal crack evident at the same time. The longitudinal crack has recently appeared. The crack on the floodwall was not noticed during our site visit in 2017. The existing floodwall meets the minimum freeboard requirements. Therefore, no modifications of the floodwall are planned in our design. The floodwalls are relatively new and built in 2010. We recommend that the crack be properly inspected/evaluated and followed up with corrective measures including but not limited to, epoxy injections, carbon fiber wrap or a combination of both.

6.2 Proposed Levee

The key factors associated with the civil design included: 1) maintaining the footprint of the new levee within the USIBWC ROW; 2) maintaining a smooth transition near the bridges; 3) minimizing the amount of fill volume required to build the levee; and 4) maintaining sufficient distance from the existing river edge. The proposed levees will generally have 3H:1V slopes on the landside and the riverside. Maintaining this slope throughout the entire alignment, however, will lead to construction in some areas outside of the ROW along the landside slope. In those areas where the ROW is a constraint, the proposed levee slope has been reduced from the proposed 3H:1V slope to a maximum steepness of 2.5H:1V slope as shown on the contract drawings. On the West Levee, the levee centerline was shifted slightly towards the landside to maintain sufficient clear area for construction. The areas where the 2.5H:1V slope occurs for the East and West levees are listed in Table 8.

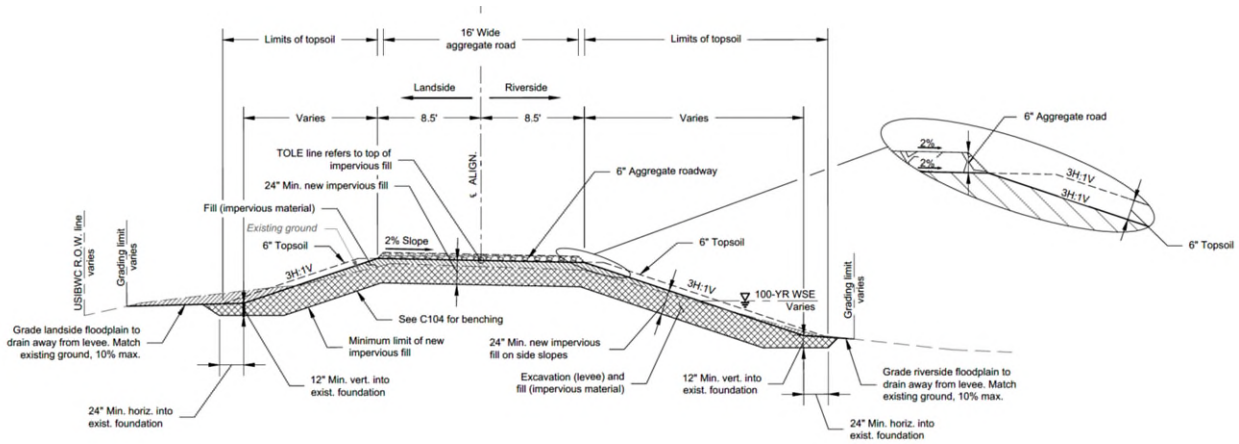
Table 8. Proposed Levee 2.5H:1V Slope Sections

Begin Station	End Station	Reason
EAST LEVEE		
1516+00	1517+24	Proximity of landside toe to existing rockwall.
1518+94	1520+00	Proximity of landside toe to ROW.
1536+00	1556+50	Proximity of landside toe to Montoya Main Lateral.
1844+71	1852+00	Proximity of landside toe to Montoya Intercepting Drain.
1927+00	1932+52	Proximity of landside toe to Montoya Intercepting Drain and concrete headwall.

Begin Station	End Station	Reason
1934+14	1956+26	Proximity of landside toe to Montoya Intercepting Drain.
WEST LEVEE		
1684+00	1692+00	Maintain minimum 10-foot distance from riverside toe to existing river edge.
1692+00	1697+74	Maintain minimum 10-foot distance from riverside toe to existing river edge. Proximity of landside toe to ROW.

The levee crest will have a 2% cross-slope toward the riverside to provide positive drainage for the levee crest. The top of the underlying new embankment material will also have a 2% cross-slope toward the riverside. The surface gravel roadway will be 16 feet wide and 6 inches thick. A typical levee sections for the East and West levees are included in Figure 8.

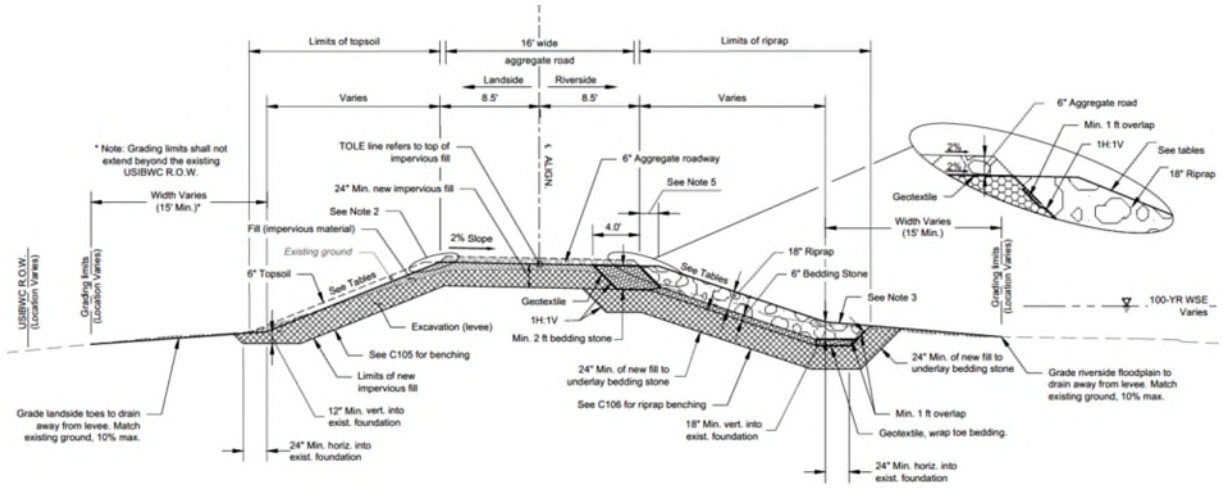
Figure 8. Typical Levee Section



The outer portion of the existing levee material will be excavated and replaced with new non-dispersive clay fill that will slightly increase the levee height and width of the crest in some portions of the alignment. This slight change in the levee geometry will not change the river flow characteristics. The existing levee material will be removed to at least 24 inches below final new embankment fill grade (bottom of gravel roadway). The existing levee will be excavated in benches cut into the slope face to enable creating a better bond between the existing and the newly placed material during placement and compaction. At the landside and riverside toe, the new levee fill is keyed in at least 12 inches into the foundation. To ensure proper compaction of the final lift, the compacted fill for the levee embankment is to be overbuilt at least 2 feet past the final slopes and grades and then trimmed to the finished grade. The overbuild will not be placed within 4 feet of any structure. A 6-inch layer of topsoil will be placed on the levee slopes for vegetation.

At locations with higher channel velocities as identified in Section 5.5, riprap will be added on the riverside levee slopes. Levee segments needing riprap include downstream of Sta. 16956+00 on the West Levee and corresponding levee segment across the river on the East Levee from Sta. 1695+00 to Sta. 1713+00. Typical levee section with riprap for the East and West levees are included in Figures 9. Riprap and bedding stone calculations are presented in Appendix B.4.

Figure 9. Typical Riprap Section



6.3 Ramps

The existing riverside levee access ramps along the levee were generally built with a 14.5-degree horizontal angle (between the centerline of the ramp and the centerline of the levee) rather than the required 7.5-degree horizontal angle per USIBWC's Standard Drawing SD-25084. All levee access ramps will be reconstructed to meet the USIBWC requirement and material used for new ramps will consist of the same fill used for the proposed levee. Riverside levee access ramps are intended for USIBWC maintenance use only. Proposed ramps will have approximately a 4.5% downslope to the riverside toe of the levee and side slopes will be maintained at 3H:1V. The levee access ramps will have a 16-foot-wide crown with a 6-inch-thick gravel base surface per USIBWC requirements. Levee ramp WR-14, at West Levee Sta. 1713+00, differs from the typical ramp design due to existing rock piles on the landside of the levee. The WR-14 ramp on the landside has a 30-foot-wide landing area rather than the typical 100-foot-wide landing area. Ramp details are provided in the contract drawings.

Nine landside ramps have been designed between the Country Club and the Sunland Park Dr. bridges to assist the Elephant Butte Irrigation District with access to the Montoya Intercepting Drain to perform maintenance. The width of the ramps is 12-foot-wide crown with a 6-inch-thick gravel base surface per USIBWC requirements. The landside ramp near Station 1685+50 is perpendicular to the levee and terminates at the existing vehicle gate. The landside ramp at the Frontera Road intersection is perpendicular to the levee and terminates at the new USIBWC vehicle gate.

6.4 Encroachments

Encroachments of fences, structures, landscaping, and other personal property on USIBWC's ROW are included in Table 9. This list, which is provided for information purposes only, is also included in the drawings and specifications.

Table 9. Encroachments along Sunland Park Levees

Encroachment Type	Approximate Start Station	Approximate End Station	Adjacent Property Owner [1]
EAST LEVEE			
Chain Link fence	1519+10	1522+85	CIC Limited, INC. Doc. 20190067227
Chain Link fence	1522+85	1523+89	CIC Limited, INC. Doc. 20140031532
Chain Link fence	1529+28	1530+02	CIC Limited, INC. Doc. 20140031532
Chain Link fence	1529+28	1530+02	CIC Limited, INC. Doc. 20190067227
Chain Link fence	1563+21	1569+08	Joseph E. & Sheryl L. Nowell Vol. 2888 Pg. 984-987 Doc. 20070110249
Rock Pile	1577+50	1578+00	Raman and Maria I. Rubalcaba Vol. 1302 Pg. 1572
Landscape Logs	1570+50	1571+75	Oscar F. Perez
Wire fence	1584+82	1588+69	Mario Okubo Doc. 20170053732
Wire fence	1588+69	1592+66	Mario Okubo Doc. 20170053732
Wire fence	1595+26	1596+19	Mario Okubo Doc. 20170053732
Rockwall	1619+74	1620+93	Jose R. Balderrama Doc. 2050053588
Chain Link Fence	1626+17	1626+74	Kamal Ken Vasadani Doc. 20150076202
Chain Link Fence	1626+74	1628+99	Hernandez Carlos Jr.
Chain Link Fence	1628+99	1629+39	Construction Control Inc. Vol. 0316 Pg. 0784
Chain Link Fence	1630+38	1632+74	Mutual Federal Savings & Loan Association of Paso Vol. 0914 Pg. 0251
Chain Link Fence	1632+74	1633+81	Richard & Dorothy Brown Vol. 1105 Pg. 1282
Chain Link Fence	1635+36	1635+54	Richard & Dorothy Brown Vol. 1105 Pg. 1282

Encroachment Type	Approximate Start Station	Approximate End Station	Adjacent Property Owner [1]
Chain Link Fence	1637+34	1637+81	Peter M. & Angela C. Koplos Vol. 3437 Pg. 1219
Chain Link Fence	1637+81	1638+98	Hileen Elfreda Mallory Doc. 20140028002
Chain Link Fence	1638+98	1640+12	Fernando Anaya Doc. 20160043540
Chain Link Fence	1640+12	1641+80	Harold A. & R. Angela Sommers Doc. 20000071600
Chain Link Fence	1696+42	1698+29	Sam Legate No. 0338274 UPC No. 4-017-167379-482
Fence	1698+29	1701+30	Sam Legate No. 0338274 UPC No. 4-017-167379-482
Fence	1701+30	1705+32	Sam & Barebara Legate No. 956799 UPC No. 40-017-167355-518
Chain Link Fence	1707+61	1714+42	Zaramuu family trust No. 0119757 UPC No. 4-017-168348-032 No. 0119757 UPC No. 4-017-168-347-071
Chain Link Fence	1714+42	1715+96	Luisa Martinez No. 2112149 UPC No. 4-017-168322-098
Rockwall Fence	1722+05	1724+63	Donald & Martha Dickason BK. 40, Pg. 827
Landscape Logs	1724+63	1727+23	Stephen Schuster No. 891184 UPC No. 4-017-168323-200
Brush & Vegetation	1741+00	-	Justin & Kimberly M. Fowlkes Doc. 20060067838
Brush & Vegetation	1742+50	1742+75	Justin & Kimberly M. Fowlkes Doc. 20060067838
Brush & Vegetation	1749+00	1749+50	Claudio & Kristi Iai Doc. 20120041863
Brush & Vegetation	1760+00	1761+35	Angela Urrea No. 1313553 UPC No. 4-017-168520-430
Brush & Vegetation	1774+50	1775+75	Diaz Family Limited Partnership may 05, 2017 Reception No. 1710641 UPC No. 4-018-168-120478
Wood Fence	1775+81	1778+67	William Levin No. 1120881 UPC No. 4-018-168143-503

Encroachment Type	Approximate Start Station	Approximate End Station	Adjacent Property Owner (1)
Chain Link Fence	1779+04	1780+31	Gerardo A Fierro et ux No. 0931419 UPC No. 4-018-168-165-511
Chain Link Fence	1904+84	1906+29	Jesus & Gloria Prieto No. 0831702 UPC No. 4-020-170042-224
Chain Link Fence	1906+29	1906+32	New Mexico Dept. of Transportation
WEST LEVEE			
Chain Link fence	1682+25	1684+89	Fernando & Aurora Silva warranty deed recorded June 15, 1977 Reception No. 772442 BK. 243, Pg. 1-5 UPC No. 4-017-167-303-311
Railroad ties and landscaping	1685+58	1687+71	David & Cheryl Warring warranty deed recorded September 22, 2004 Reception No. 0431336 BK. 552, Pg.1284 UPC No. 4-017-167-305-355
Chain Link fence	1687+82	1691+25	Lee & Rowena Marcontel warranty deed recorded January 28, 2004 Reception No. 042853 BK. 493, Pg. 1522 UPC No. 4-017-168-300-385

Note (1):

- Volume and Page, (Vol. Pg.) refers to Deed Records at the County Clerk's Office, El Paso County, Texas. Document Number, (Doc.) refers to Official Public Records, El Paso County, Texas. The document Number designation have replaced Volume and Page numbers previously used to file documents.
- The UPC number, (UPC NO.) represents a geographic identification for the location of parcels on tax maps at the Tax Assessors offices in the State of New Mexico.
- Book and Page, (BK. PG.) refers to Deed Records at the County Clerk's Office in Dona Ana County, New Mexico
- The Reception Numbers, (RECEPTION NO.) are identifiers used to file documents at the County Clerk's Office in Dona Ana County, New Mexico. The Reception Number designation have replaced Book and Page numbers previously used to file documents.

6.5 Subsurface Utility Exploration

The purpose of the subsurface utility exploration (SUE) was to obtain underground utility information. SUE was performed by FXSA at the Country Club Bridge and Montoya Lateral where penetrations are anticipated in the levee. SUE is typically performed in four levels. Descriptions of each level was obtained from the Federal Highway Administration website at https://www.fhwa.dot.gov/programadmin/sueindex.cfm.

Level D: Level “D” is the most basic level of information for utility locations. It comes solely from existing utility records or verbal recollections, both typically unreliable sources. It may provide an overall "feel" for the congestion of utilities but is often highly limited in terms of comprehensiveness and accuracy.

Level C: Level “C” includes surveying visible utility facilities (e.g., manholes, valve boxes, etc.) and correlating this information with existing utility records. When using this information, it is not unusual to find that many underground utilities have been either omitted or erroneously plotted. Its usefulness, therefore, is primarily on rural projects where utilities are not prevalent or are not too expensive to repair or relocate.

Level B. Level “B” involves the application of appropriate surface geophysical methods to determine the existence and horizontal position of all utilities within the project limits. This activity is called "designating." The information obtained in this manner is surveyed to project control. It addresses problems caused by inaccurate utility records, abandoned or unrecorded facilities, and lost references. The proper selection and application of surface geophysical techniques for achieving quality Level B data is critical. Information provided by quality Level B can enable the accomplishment of preliminary engineering goals. Decisions regarding the location of storm drainage systems, footers, foundations, and other design features can be made to successfully avoid conflicts with existing utilities. Slight adjustments in design can produce substantial cost savings by eliminating utility relocations.

Level A. Level A, also known as "locating," is the highest level of accuracy presently available and involves the full use of the subsurface utility engineering services. It provides information for the precise plan and profile mapping of underground utilities through the nondestructive exposure of underground utilities, and also provides the type, size, condition, material, and other characteristics of underground features.

The report including the findings from the SUE is included in Appendix D.

6.6 Road Crossings

Most of the road crossings will have the USIBWC Notice Signs installed on both paved and dirt crossings. The signs will be installed per USIBWC Drawing 26424 and as included in contract documents. The specific signs to be placed at each road crossing are included in Table 10.

Table 10. USIBWC Signs for Road Crossings

Road Crossing	Approximate Location	Sign
EAST LEVEE		
North of W. Borderland Rd. Bridge	Landside – 1516+79	Detail A (Levee Notice Sign in Non-Hunting Area) and Detail F (Prohibited Activities in Non-hunting Area) in Drawing 26424

Road Crossing	Approximate Location	Sign
North of W. Borderland Rd. Bridge	Landside – Sta. 1517+56	Stop Sign per Standard Drawing 26424
South of W. Borderland Rd. Bridge	Landside – Sta. 1518+03	Stop Sign per Standard Drawing 26424
South of W. Borderland Rd. Bridge	Landside – Sta. 1518+73	Detail A (Levee Notice Sign in Non-Hunting Area) and Detail F (Prohibited Activities in Non-hunting Area) in Drawing 26424
North of Country Club Rd. Bridge	Landside – Sta. 1666+26	Detail A (Levee Notice Sign in Non-Hunting Area) and Detail F (Prohibited Activities in Non-hunting Area) in Drawing 26424
North of Country Club Rd. Bridge	Landside – Sta. 1666+75	Stop Sign per Standard Drawing 26424
South of Country Club Rd. Bridge	Landside – Sta. 1667+55	Stop Sign per Standard Drawing 26424
South of Country Club Rd. Bridge	Landside – Sta. 1667+91	Detail A (Levee Notice Sign in Non-Hunting Area) and Detail F (Prohibited Activities in Non-hunting Area) in Drawing 26424
North of Sunland Park Dr. Bridge	Landside – Sta. 1905+64	Detail A (Levee Notice Sign in Non-Hunting Area) and Detail F (Prohibited Activities in Non-hunting Area) in Drawing 26424
North of Sunland Park Dr. Bridge	Landside – Sta. 1906+29	Stop Sign per Standard Drawing 26424
South of Sunland Park Dr. Bridge	Landside – Sta. 1907+06	Stop Sign per Standard Drawing 26424
South of Sunland Park Dr. Bridge	Landside – Sta. 1907+71	Detail A (Levee Notice Sign in Non-Hunting Area) and Detail F (Prohibited Activities in Non-hunting Area) in Drawing 26424
North of Racetrack Dr. Bridge	Landside – Sta. 1931+93	Detail A (Levee Notice Sign in Non-Hunting Area) and Detail F (Prohibited Activities in Non-hunting Area) in Drawing 26424
North of Racetrack Dr. Bridge	Riverside – Sta. 1932+60	Stop Sign per Standard Drawing 26424
South of Racetrack Dr. Bridge	Landside – Sta. 1933+29	Stop Sign per Standard Drawing 26424

Road Crossing	Approximate Location	Sign
South of Racetrack Dr. Bridge	Landside – Sta. 1934+48	Detail A (Levee Notice Sign in Non-Hunting Area) and Detail F (Prohibited Activities in Non-hunting Area) in Drawing 26424
WEST LEVEE		
South of Country Club Rd. Bridge	Riverside – Sta. 1667+09	Stop Sign per Standard Drawing 26424
South of Country Club Rd. Bridge	Landside - Sta. 1667+62	Detail A (Levee Notice Sign in Non-Hunting Area) and Detail F (Prohibited Activities in Non-hunting Area) in Drawing 26424
South of Country Club Rd. Bridge	Riverside – end of floodwall at Sta. 1668+38	MUTCD OM3-R

7 ENVIRONMENTAL CONSIDERATIONS

The USIBWC prepared an environmental assessment in 2007 to describe the environmentally and culturally sensitive areas that need to be protected from disturbance caused by construction activities. The environmental assessment is included in the "*Final Environmental Assessment, Flood Control Improvements to the Rio Grande Canalization Project*" document dated December 2007. Additional information about the best management practices is listed in the "*USIBWC Rio Grande Canalization Project, River Management Plan*" (USIBWC 2016). These environmental considerations are included as appropriate in the contract documents.

7.1 Environmentally Sensitive Areas

Improvements to the levee system from the construction activities will have potential impacts to air quality from disturbed soils and construction equipment emissions. Air emissions during construction will be reduced by limiting heavy equipment operation to normal working hours. Ambient noise levels will moderately increase during construction activities using trucks, heavy equipment, and fill activities. Construction noise will generally be limited to the immediate construction zone.

The construction contractor will be required to follow all best management practices listed in the "*USIBWC Rio Grande Canalization Project, River Management Plan"* (USIBWC 2016). The use of best management practices, such as soil wetting for dust suppression and proper maintenance of equipment, will reduce the impacts to the annual emissions inventory.

7.2 Culturally Sensitive Areas

Impacts to archaeological and historic properties are not anticipated. The contract documents include requirements to be followed if culturally sensitive areas are encountered during construction.

7.3 Biological Resources

According to environmental assessment, the vegetation communities impacted along the levee slopes are primarily low-quality plants dominated by invasive plants including Bermuda grass, Russian thistle, Kochia, Silverleaf nightshade, and London rocket. Short-term impact on vegetation communities along the system corridor will occur. Impacts to woody riparian vegetation are not anticipated as fill material will be added to the existing top of the levee.

No significant effects are anticipated on wildlife habitat in the vicinity of the levee system, including potential habitat for threatened and endangered species. In areas requiring levee footprint expansion, no riparian woodland communities will be impacted; impacts to vegetation on levee slopes will be to low quality vegetation of limited value as wildlife habitat. Potential suitable habitat for threatened and endangered species occurs within the river channel away from the levee corridor, and, therefore, will not be impacted by levee improvements.

7.4 Natural Resources

No impacts are anticipated on the natural resources due to the levee construction activities because improvements will occur within the existing levee footprint and away from the Rio Grande channel. No substantial wetlands and no jurisdictional waters were identified in the environmental assessment document within the project levee improvement area. USIBWC is considering a wetlands project at Montoya Drain.

7.5 Restoration Sites

USIBWC has several conceptual restoration sites along the proposed stretch of the levee repair. The Country Club East, Sunland Park and Anapra Bridge restoration sites are located along the East Levee and are currently active.

The construction contractor will be required to stay within 20 feet from landside levee toe at the restoration sites except for specifically identified areas on the project site where greater room is required to perform construction. The contract documents require the construction contractor to protect all existing planting outside of the working area and prohibit staging and temporary access through active restoration areas.

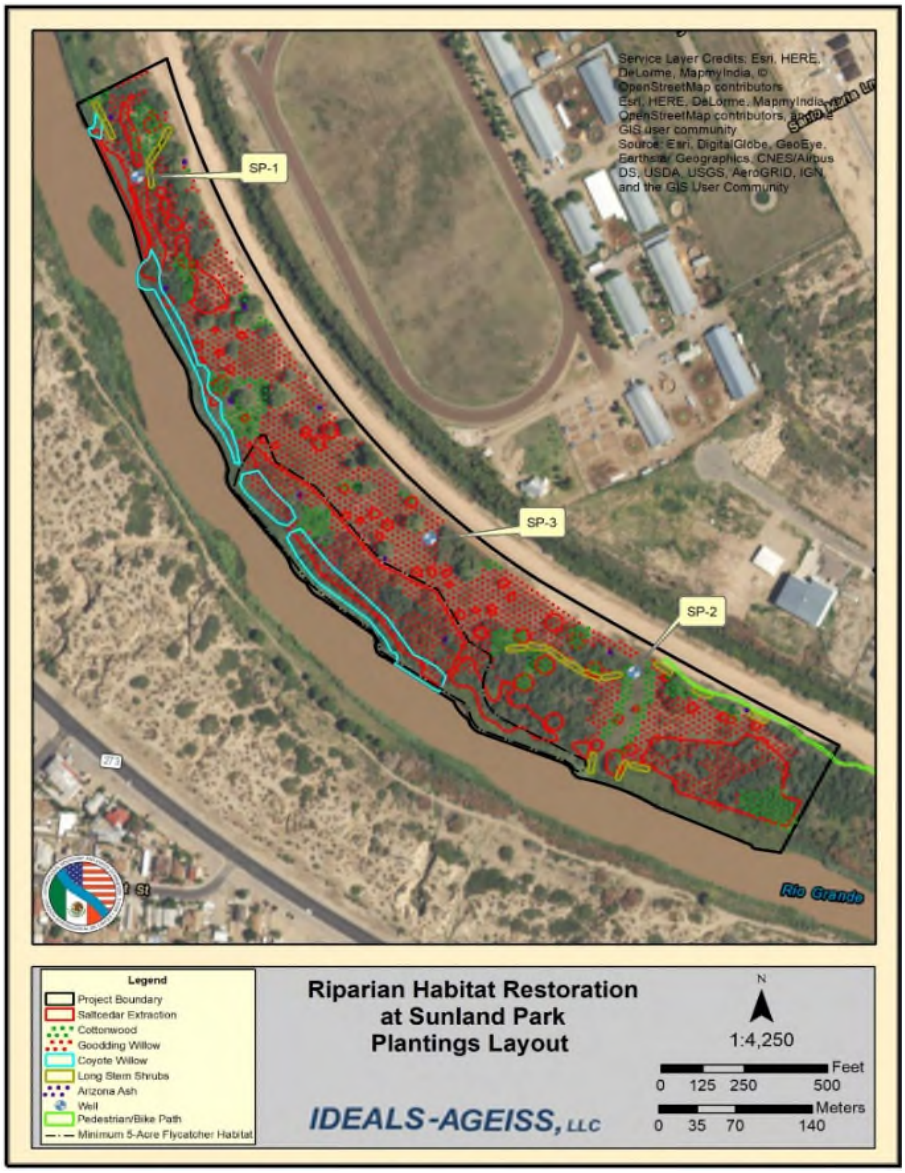
Several groundwater wells were re-established in March of 2018 at the three restoration sites. The construction contractor shall not disturb the wells. Further information on the groundwater wells can be found on final restoration report titled “Riparian Habitat Restoration at Three Sites in New Mexico and Texas: Country Club East, Sunland Park, and Anapra Bridge Restoration Sites,” by IDEALS-AGEISS, dated February 28, 2020.

The Country Club East conceptual restoration site is about a 29-acre site is on the east bank of the river as shown in Figure 10. The northern 21 acres are within New Mexico and the southern 8 acres are in Texas. The overbank inundation begins near RM 6.3 at a 3,000-cfs Caballo release with flooding extending both upstream and downstream with increasing discharges. The site would be fully inundated by Caballo releases of 4,000 cfs and higher. Vegetation consists of grassland with scattered resprouting saltcedar.

Figure 10. Country Club East Restoration Site [*Reference: IDEALS-AGEISS (2020)*]

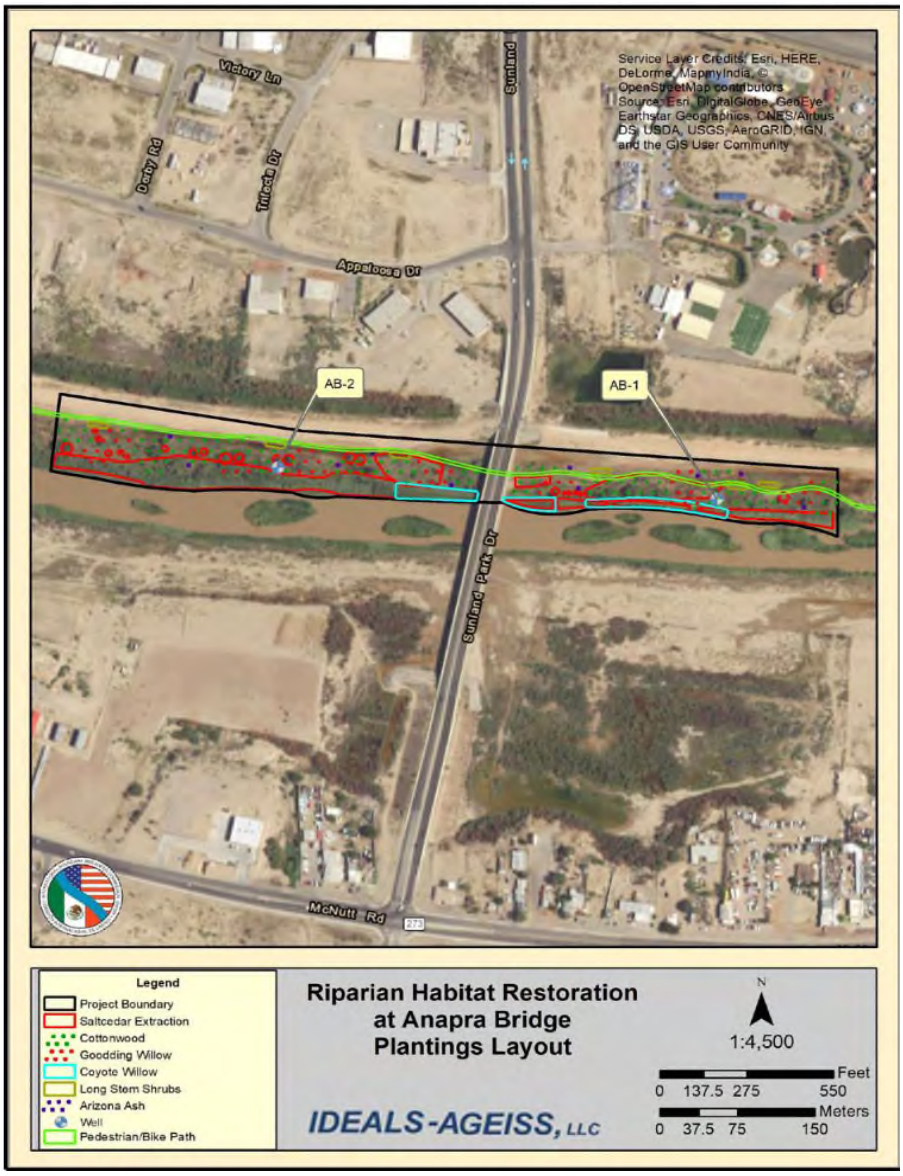
The Sunland Park restoration site covers 28.8 acres on the east bank as shown in Figure 11. This site is predicted to begin flooding with the 3,250-cfs Caballo release and be fully inundated by the 3,500-cfs release. Existing vegetation consists of relatively dense grasses (primarily saltgrass) and scattered cottonwoods and shrubs. Saltcedar resprouting occurs.

Figure 11. Sunland Park Restoration Site [*Reference:IDEALS-AGEISS (2020)]*



The Anapra Bridge restoration site is shown in Figure 12. A public parking area is located at the southern end of the site and trails extend to north. The 11-acre site is predicted to be inundated by a 2,750- to 3,500-cfs release from Caballo Dam. The site is moderately vegetated with grasses (primarily saltgrass) and mature cottonwoods, coyote willow, and screwbean mesquite.

Figure 12. Anapra Bridge Restoration Site [*Reference: IDEALS-AGEISS (2020)]*



8 STRUCTURAL DESIGN

8.1 Low Structures

The existing closure structures along the levee alignment that are currently below the 100-year flood elevation and therefore too low to be operated during flood flows, will be raised and/or modified to enable operation during the 100-year flood. These structures currently have sluice gates on the riverside with manual closure mechanisms below the flood elevation. Structures that are below the required 100-year flood elevation are included in Table 11. The structural basis of design for these structures is presented in Appendix C.1.

Table 11. Existing Low Structures on East and West Levees

Station No.	Structure	Top of Concrete Headwall Elevation, ft	100-Year Flood Elevation, ft	Recommendation
EAST LEVEE				
1535+00	4’ x 4.7’ (Montoya Lateral WW #38)	3745.02	3748.72	Design catwalk
WEST LEVEE				
1667+49	30” RCP (Schultz Lateral WW #35C)	3747.69	3753.60	Design catwalk

8.2 Levee Closure Structures

Levee closure structures must meet the 44 CFR §65.10(b) FEMA requirements. The floodgate at the Borderland Bridge on East Levee will be left in place and no modification to the floodgate is required. There is no floodgate at the Country Club Bridge along the West Levee alignment. The survey data indicates, however, that this crossing does not meet the freeboard requirement. A new flood control system will be designed at this location. East Levee alignment at the Country Club Bridge currently has a passive flood gate. USIBWC has decided to remove and replace with a closure structure similar to the West Levee alignment.

A stoplog barrier type was selected by USIBWC to provide the necessary flood protection at both East and West Levee locations at the Country Club Bridge. The floodgate alternatives for east levee were presented at the scoping meeting on November 11, 2017, and it was decided by USIBWC at the meeting that a sliding gate would be designed for the Country Club Bridge on East Levee. No discussions were made for the floodgate on West Levee during this time. The sliding gate design on the East Levee was included at the 30% and the 60% design packages. At a meeting with USIBWC on November 15, 2018, Arcadis presented four floodgate design alternatives as shown in more detail in Appendix C.2. Following the evaluation of alternatives for the floodgate on the West Levee with USIBWC on December 13, 2018, the USIBWC selected aluminum stoplogs for both sides of the Country Club Bridge. USIBWC indicated

that using the same design on both sides would be easier for operation and maintenance. Advantages and disadvantages for each design alternative are also presented in Appendix C.2.

A stoplog barrier is a deployable flood protection system, consisting of a system of fixed and/or removable column posts with stoplog planks placed horizontally into the space or bay between the two adjacent posts. The removable column posts can be installed by bolting into a prefabricated receptacle, ideally a reinforced concrete base. The removable posts and planks can be stored on/off-site depending on the project location and available facilities.

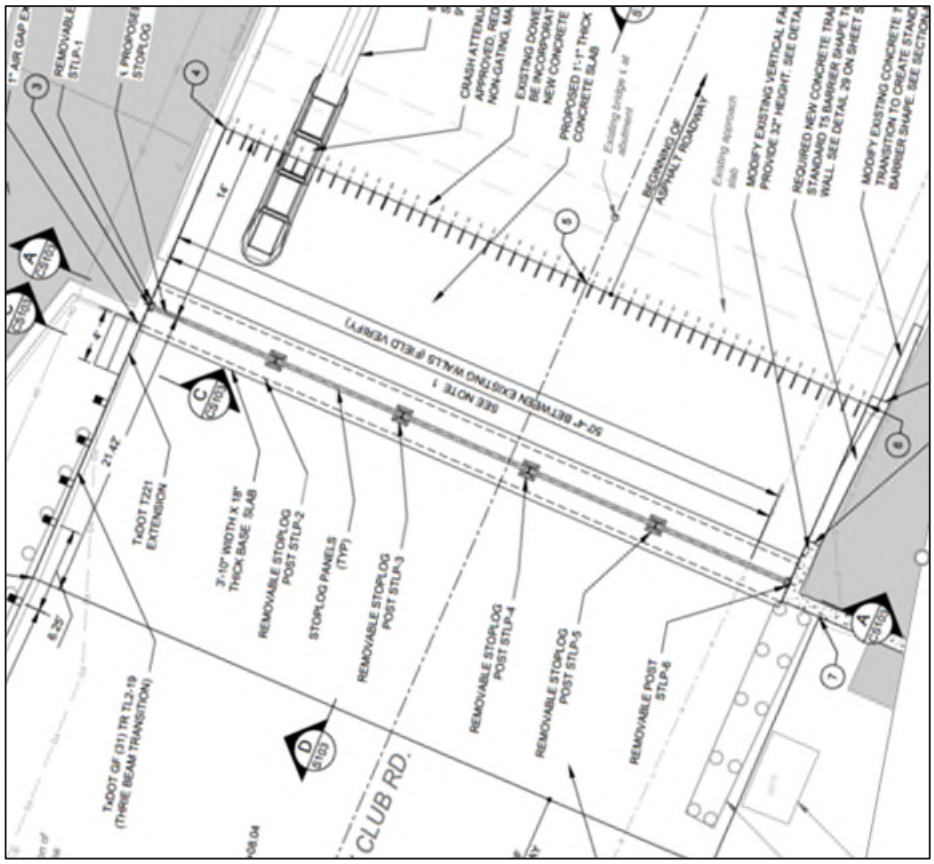
The stoplog system proposed at the Country Club Bridge is designed for a maximum height of 2.2 feet. At the East Levee alignment, the existing gate concrete housing shall be completely removed. A new slab will be poured at its place to install the removable stoplog posts that will be doweled to the approach slab. At the West Levee alignment, the existing approach slab shall be saw-cut to provide a deeper concrete base for the stoplog posts, which will also be doweled to the approach slab. The concrete base for the posts is designed to withstand the vehicular traffic moving on top of bridge. The doweled bars extending into the approach slab will prevent overturning from lateral flood loads. On either East or West Levee alignment, the width of the existing roadway, will be divided into five individual stoplog bays. Four of these stoplog bays will host stoplogs of which are close to10 feet wide, while the remaining bay in each alignment will remain custom. The final dimensions of the stoplogs will be field verified by the Contractor prior to placing order with a stoplog manufacturer. Mostly uniform lengths will make installation of the stoplog in both locations easier and faster since almost 80% of the stoplogs will be of the same length (see Figure 13). The Contractor, in coordination with the stoplog fabricator, may decide to have all five bays to have same length of stoplogs to further ease up deployment during flood event. Stoplog posts to be installed on the roadway itself are fabricated using 6061-T6 aluminum I6 X 4.69 columns, while C6x4.5 members of the same material type shall be utilized as stoplog posts on extreme ends. Both the end posts and the intermediate posts shall be removable so as not to interfere with regular traffic operations. The removable stoplog posts at the middle of the roadway are welded to a ½” thick 6061-T6 aluminum base plate and will be bolted to a ¾” stainless steel base plate embedded within the concrete slab during installation. The end posts will have both a vertical and a horizontal ½” thick 6061-T6 plate and shall be bolted to the traffic barrier and/or existing flood wall during a flood event. End posts and intermediate posts have been sized so that they can be lifted and installed by a small crew, without requiring any heavy equipment. Once the stoplog posts are removed, after the flood event, the bolt hole receptacles within the concrete base slab and vertical barrier/floodwall faces shall be plugged with dummy bolts. These dummy bolts in the middle of the roadway shall be flush with the roadway surface so as to provide a smooth riding surface. The purpose of the dummy bolts is to keep the bolt holes free of debris so that stoplog post installation can go faster in the wake of a flood event.

The removable stoplog post and base plate combination as well as the planks shall be stored onsite within a closed container to be provided by the stoplog manufacturer. The location of the onsite storage is shown on the civil drawings. The storage container shall be placed on a slab on ground and will be bolted in place to prevent theft and vandalism. Contractor shall provide separate storage containers for logs and removable posts. Approximate sizes of the containers are provided in the drawings. The height of the containers shall not exceed five feet. The containers should be able to withstand a wind gust of 116 mph with proper anchorage provided to tie them down to the slab. The stoplogs and removable posts should be retrievable from the containers without any special equipment. It is preferred to implement a rolling cart

system that stays within the container which would help bringing out the logs and posts rapidly during deployment.

At the wake of a flood event, the stoplog posts shall be taken out of the container, aligned properly on top of the designated pre-installed embedded plate, and securely placed with bolts. Once the vertical posts are installed, aluminum stoplog planks are securely placed in the bay area (area between two adjacent posts) up to the height required for flood protection. Due to roadway crown, there may be some gaps within the bottom stoplog plank and the roadway surface. To avoid substantial leakage through such gaps, the installation crew shall deploy sandbags all along the alignment of the stoplog barrier. Detailed calculations are presented in Appendix C.3.

Figure 13. Country Club East Levee – Stoplog along the width of the approach slab



8.3 Roadway Barrier Transitions at Country Club Bridge

At the Country Club Bridge, the existing roadway barriers at both East and West Levee end of the bridge and approach slab do not comply with the current TxDOT standards. In some instances, the barriers and the floodwall do not meet the standard 2'-8" requirement. In other

cases, the barriers do not have a proper transition from a rectangular cross-section to a F-shape barrier shape, mostly located on the bridge. At the approach slabs some of the existing guardrails are not bolted to adjacent concrete barriers and therefore do not provide the required terminal end protection for the rigid concrete barriers.

The design criteria used for layout and design of the new rigid barriers, flexible barriers, crash cushions, transitions, end treatments, pavements and sidewalks were based on the current versions of the TxDOT Roadway and Bridge Design Manuals, Guidelines and Standard Detail Drawings. The following assumptions were used for the design of the new project features at the Country Club Road Bridge: Urban Collector; Two-Lane/Undivided/Curbed Roadway; Posted Speed 35-mph/Design Speed 45-mph; Test Level 2 (TL-2) MASH Compliant.

The new crash cushions shall be TxDOT-approved permanent, redirective, non-gating, MASH compliant, TL-2 rated crash cushion attenuators. To fit the site constraints, the system backup shall be 24 inches and system length shall not exceed 14'-0". At both East and West bridge approaches, a 4' x 15' concrete pad or the equivalent space on existing pavements shall be provided to accommodate the noted backup width and system length. The current version (08/09/21) of TxDOTs approved crash cushion attenuators includes two products that will meet the stated constraints: SMART CUSHION (N) by Work Area Protection Corp. and QUADGUARD (M10) (N) by Trinity Highway - Energy Absorption, Inc.

The existing site constraints that affected the design of the new flood barriers across Country Club Road were:

- Existing flood walls constructed on the east and west bridge approach slabs that do not comply with current TxDOT standards for traffic barriers
- Existing skewed alignment of the levee road connections to Country Club Road at the east/upstream side and west/downstream side which limit available space for the required crash cushion protection of the existing rigid bridge barriers
- Existing underground and above-ground utility facilities and structures that could not be relocated, specifically the AT&T fiber optic manhole at the downstream side of the West Levee, and the electric cabinet and conduit mounted to the back of the barrier wall at the upstream side of the East Levee
- Existing guardrail at each of the four corners of the bridge that do not comply with current TxDOT standards

At the upstream end of the East Levee approach slab, the existing floodwall shall be raised by doweling in rebars by drilling and applying epoxy and placing concrete on top of the flood wall to attain the 2’-8” required height. This flood wall will host a removable end post for the stoplog deployment. Beyond this floodwall, towards the bridge, there is a non-standard barrier segment which shall be completely removed. A new barrier wall will be placed at this location which features a transition from rectangular cross-section, matching the existing floodwall cross-section, to an F shape barrier at the end. The rebar details for this transition segment are shown

within the plans. Since the approach slab at this location will be replaced, this transition barrier will be doweled into the new concrete during construction. Beyond this transition barrier toward the bridge, there is a short segment of existing F shape barrier which has a sloping top that needs to be raised to meet the 2'-8" height requirement. This shall also be achieved by chipping the exiting concrete, doweling in new rebars by drilling and applying epoxy and finally placing concrete on top.

At the downstream end of the East Levee, the metal barrier is currently discontinuing and is not connected with the floodwall, which acts as a barrier. A short 4'-0" long segment of ground mounted barrier shall be added adjacent to the floodwall, away from the bridge. This barrier will host mounting holes which will accept bolts from the metal barrier that will terminate at this location and as such will provide continuity of the barrier.

At the West Levee end, some of the existing concrete barrier shall be demolished and replaced with new concrete barriers on newly constructed approach slab segment. These barrier walls shall be constructed to have a 2'-8" height to meet the TxDOT standard and shall host the removable end posts for the stoplogs. The cross-section of these barriers is mostly rectangular. At the upstream end, a transition barrier segment shall be provided that will transform from a rectangular section to an F shape cross-section to match the shape of the next barrier segment on the bridge. To place this new segment, the existing barrier that is currently in place shall be removed. However, the dowels for the existing barriers shall be reused as much as possible, provided that they are not located too close to the face rebar of the new barrier segment. In such cases, these dowels will be cut flush with slab. New dowels shall be installed by drilling and applying epoxy and shall be staggered to miss the existing dowel locations. Similar to East Levee, at the upstream end of West Levee, a short 4'-0" long ground mounted rectangular shape barrier shall be placed adjacent to the new concrete barrier, away from the bridge. This segment shall host bolt holes so that the adjacent metal barrier can be transitioned and attached to this barrier providing continuity.

All new and existing barriers shall feature a 1" air gap in between, which will serve as expansion joint. Such gaps shall terminate at the bottom, at the level of the foundation slab or approach slab. The reinforcements in the newly constructed barriers and barrier transitions follow or exceed the reinforcements shown in similar standard details from TxDOT and other DOTs.

At the new East Levee flood barrier, all of the existing bridge approach slab will be demolished to enable the removal of the existing underground flood gate structure. A new concrete approach slab will be constructed to incorporate the required foundation for the new stop log flood barrier system. The new concrete approach slab will be finished with a metal-tine surface finish to match the existing concrete roadway pavement east of the bridge.

At the new West Levee flood barrier, a four-foot-wide strip of the existing concrete approach slab will be saw-cut and removed to enable the construction of the required foundation for the new stop log flood barrier system. It was decided to remove all of the existing asphalt overlay of

the concrete approach slab up to the joint with the existing bridge abutment and slab due to deteriorated condition of the existing asphalt and the amount that would be damaged by the new stop log foundation and concrete barrier wall construction. With the reconstruction of the existing downstream driveway for the levee access road due to the increased height of the levee, reconstruction of adjacent sidewalks, and installation of an integrated base slab for the new crash attenuator, a portion of the adjacent roadway shoulder concrete will also be reconstructed to provide a neatly integrated installation along the driveway extending to the bridge approach slab.

8.4 Raising of Existing Concrete Wall

The north-east corner of the East Levee alignment features a concrete wall that extends away from the roadway and provides protection against flooding. The current elevation of the top of the wall is at El. 3756.0. The top of the wall needs to be raised to El. 3758.0 to effectively resist the 100-year flood. To achieve this wall extension, the top of the existing wall shall be chipped with hammer and roughened to receive fresh concrete on top. Additional rebars shall be drilled in and epoxied on top of the existing wall. Formworks shall be placed on both sides of the wall and additional 2 feet of concrete shall be poured around the reinforcement to achieve the wall extension. Evaluation of the wall raise is presented in Appendix C.4.

8.5 Vehicular Gates

Existing vehicular gates located near the road crossings and other locations will be removed and replaced with new vehicular gates. The new gates will be built per USIBWC Drawing 26409 as included in contract documents.

Table 12. Vehicular Gates

Vehicular Gate Location	Vehicular Gate Approximate Station Location
EAST LEVEE	
North of W. Borderland Rd. Bridge	1516+99
South of W. Borderland Rd. Bridge	1518+41
North of Country Club Bridge	1666+46
South of Country Club Bridge	1667+66
Frontera Rd intersection	1840+67
West of Sunland Park Dr. Bridge	1906+05
East of Sunland Park Dr. Bridge	1907+29

Vehicular Gate Location	Vehicular Gate Approximate Station Location
Levee crest	1924+28
End of Landside Ramp ER-27 Ramp B	1924+39
End of Riverside Ramp	1931+21
West of Racetrack Dr. Bridge	1932+16
East of Racetrack Dr. Bridge	1933+80
WEST LEVEE	
South of Country Club Bridge	1667+32

9 CONSTRUCTION CONSIDERATIONS

9.1 Utility Conflicts

For the levee improvements, utilities include, but are not limited to, gas lines, water lines, sewer, electrical conduit, and fiber optics as included in Section 2.1. The construction contractor will be required to verify locations of all utilities before the start of construction. No utility conflict is known at this time.

9.2 Construction Outside USIBWC's ROW

FXSA has evaluated the available property records provided by USIWBC and has indicated that some areas have ROW constraints along the alignment. In addition, during the design, it was discovered that maintaining the levees at 3H:1V slope would require construction outside the ROW in some areas along the landside but these slopes were modified to maintain work within the USIBWC ROW as described in more detail in Section 6. The landside ramps adjacent to the Montoya Intercepting Drain requires minimal construction outside of USIBWC ROW. Arcadis attempted to steepen the landside slope to 2.5H:1V but was unable to avoid work outside of USIBWC ROW. The landside ramps allow access to the Montoya Intercepting Drain by EBID.

9.3 Long Lead Time Items

It is anticipated that material meeting the requirements for the non-dispersive clay cap needed for levee improvements may be difficult to obtain from a local borrow source. Non-dispersive clay may have to be purchased from local farmers. No long lead times are anticipated for the remainder of the construction materials to be used in this project.

9.4 Sole Source Items

No sole source items are anticipated in this project.

9.5 Specialty Contractors

No specialty contractors or unusual safety hazards are anticipated in this project. No specialty materials or products are required in our design.

9.6 Estimated Time of Construction

The time needed to complete this project will mainly depend on the weather (rain events) and embankment section working area at a time. It is envisioned that the general construction sequence will include the following steps:

- Preparing of ROW (installing traffic signage, permits, clearing and grubbing staging area)
- Clearing the construction site (clearing, grubbing, removing, and disposing of vegetation off site)
- Excavating and hauling away the upper layer of dispersive soils per construction drawings
- Placing and compacting new embankment material to the elevations shown on the construction drawings
- Constructing stoplogs
- Placing riprap, topsoil, aggregate roadway, and vegetation for erosion control
- Constructing catwalks
- Constructing new vehicular gates.

The overall construction is anticipated to take about 22 to 26 months. The items were estimated based on the production rates assumed in the Opinion of Probable Construction Cost provided in a separate submittal. The critical path items of this schedule are the levee excavation and embankment fill. The monthly rain days (obtained from the https://w2.weather.gov/climate/xmacis.php?wfo=epz website) for 0.1-inch rain event in the El Paso area are provided in Table 13. The mean annual rainy days in the El Paso area is 20 days based on the data from the last 10 years.

Table 13. Monthly Rain Days for 0.1-inch Rain Event (Mean Values in El Paso area)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Rain	1	1	1	0	1	1	4	3	3	2	1	2

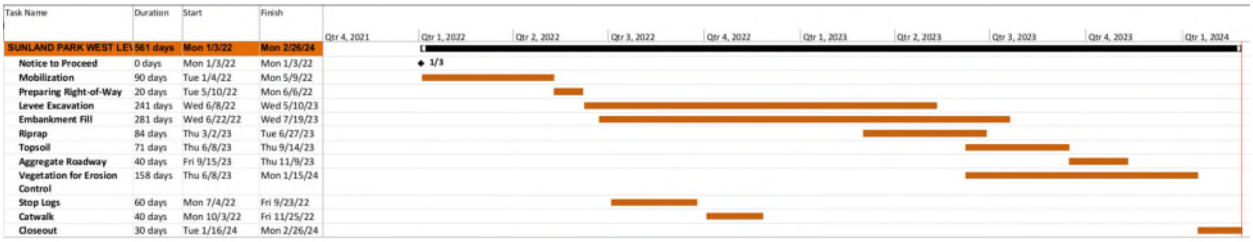
Detailed estimated construction schedule is included in Figure 14. The major items that control the construction schedule of the project include:

- Levee Excavation = 500 cubic yards per day per crew (CY/day/crew) (material in a banked state) [assume two crews]
- Embankment Fill = 800 CY/day/crew (material in a loose state) [assume two crews]
- Riprap = 50 CY/day
- Topsoil = 1,000 CY/day
- Aggregate Roadway = 5,000 SY/day (square yard/day)
- Vegetation for Erosion Control = 10 MSF/day (1,000 square feet/day)

- Working days = 5 days/week, 8 hour/day

It should be noted that the schedule in Table 14 includes time for seeding time only and does not count for any maintenance time.

Figure 14. Estimated Construction Schedule



9.7 Contractor's Technical Requirements

No specialty contractor is required for levee improvements. Experience requirements and minimum technical requirements for construction contractor's personnel are included in the contract specifications. The selected contractor will be pre-qualified for all the work items included herein based on its previous performance on similar projects. The contractor must have a minimum of 5 years of proven experience in the construction of levees, placement of concrete, installation of vehicular gates, and operation of equivalent systems required.

9.8 Traffic Plan

It is anticipated that the levee construction will cause minimal traffic disruption. However, the construction of the new concrete foundation required for the stoplogs at the Country Club Bridge would require road closures. Any closures for construction at the bridge would require coordination with the City of El Paso, NMDOT or both. The construction Contractor shall develop a traffic control plan for this work and get approval from City of El Paso and/or NMDOT. USIBWC will review any traffic plans for levee construction as required per contract documents.

10 OPERATION AND MAINTENANCE

10.1 Operation and Maintenance for Levees

Development of or updates to an operation and maintenance manual for the levees was not included in the SOW. However, it should be noted that routine visual inspection of levee slopes and crest is critical for the stability of the levees. Visual inspections should also be made during and immediately following flood events. Visual inspections are needed to identify potential problems that may require repair, including but not limited to shallow surficial slides, erosion, animal burrows, rutting, seepage, and loss of

soil. Repair will be required for any potential problem identified through routine visual inspection or through visual inspections made during and following flood events.

10.2 Operation and Maintenance of Stoplogs

10.2.1 Operation of Stoplogs

While non-operational, the stoplog post and planks should be stored in an on-site storage container. The locations of the on-site containers are shown on the civil plans. When required during a flood event, the crew shall unlock the on-site storage container, bring out the various parts of the stoplogs and shall install them using the following steps:

- Clean the dust with brush from the top of the embedded plate.
- Remove the counter sunk bolts from the embedded base.
- Place in position stoplog post and attach using anchor bolts.
- Place planks with their respective spans and position IDs
- Deploy sandbags on the protected side at least to half height of the stoplog barrier.

Tasks performed to remove the stoplogs after the storm event have passed shall be just the opposite to installation steps. Once the planks, posts, and plates are taken out, counter sunk bolts shall be placed in the embedded plate holes so that dirt or other foreign particles do not clog the holes.

Proper traffic control needs to be in place during the installation of the stoplogs following the procedures prescribed by City of El Paso. In case the stoplogs are being deployed at the wake of an imminent flood event, traffic must be completely stopped ahead of installation and proper road barrier and signage should be placed ahead of the stoplog on the protected side. It should be noted that these stoplogs, and posts are relatively light weight for easy operation and carrying and hence are not capable of withstanding any impact load from the oncoming vehicles.

10.2.2 Maintenance of Stoplogs

The maintenance of the stoplogs should be minimal. The planks will be made from aluminum and thus do not require any periodic maintenance. Since these planks shall be stored in a closed container, the probability of deterioration due to environmental effects is low. The posts shall be galvanized and thus should not have any rust issue. Once the stoplogs and posts experience a flood event, it is expected that these items be properly cleaned and dried before storing for the next flood event.

It is recommended that the stoplog system be put in place on the road and tested for proper fit at least once every six months. To reduce disruption to traffic, half of the stoplog system can be deployed while one lane of traffic can be maintained using flaggers. Such measure should be applied for both lanes to ensure proper fit of the entire stoplog system.

A dedicated crew should be assigned with the responsibility of deploying the stoplogs for both flood event and for maintenance purposes. Generally, the vendor provides a training which can be arranged at the time of procurement. Subsequent training of the new staff can be conducted by crew who are already aware of the installation procedures. Also, it is expected that the stoplog vendor shall provide an

installation manual which should be always kept with the planks when they are stored in an offsite location.

11 REFERENCES

The following is a list of references and available information that was used to develop this DDR. Other references are included in the calculation sheets as applicable.

- "Boring Logs and Laboratory Test Results, Courchesne Reach and NEMEXAS Reach Canalization Project, Area 1, BH1 through BH4," by Ninyo & Moore, dated December 14, 2012.
- City of El Paso, Texas, Title 19 – Subdivision Ordinance, 2008, Driveway Standard Detail drawings 6-15A, 6-16, and 6-17.
- "Design Report for Upper Rio Grande Flood Control Project, Sunland Park Canutillo Levee Segment," by S&B Infrastructure, dated July 23, 2010.
- "Design Scope of Work, Rio Grande Canalization, Sunland Park Levee Repair in El Paso County, TX and Doña Ana County, NM," dated March 2, 2017.
- Emily Stidworthy, J.M. Duncan, Thomas L. Brandon, and Youngjin Park, Emily Stidworthy, J.M. Duncan, Thomas L. Brandon, and Youngjin Park, "Filter Criteria Workbook Documentation," Report of a Study Performed by the Virginia Tech Center for Geotechnical Practice and Research, Filter 3.0 Excel Workbook, 2004.
- Federal Emergency Management Agency, Code of Federal Regulations, Title 44, Section 65.10 (44 CFR 65.10), Mapping of Areas Protected by Levee Systems, 2019
- Federal Emergency Management Agency, 2011, Filters for Embankment Dams
- "Final Technical Memorandum Volume I for Geotechnical Explorations of Levee System within the Rio Grande Canalization Project- Canutillo Levee Segment" by S&B Infrastructure, dated December 22, 2008.
- "Final Technical Memorandum Volume II for Geotechnical Explorations of Levee System within the Rio Grande Canalization Project- Canutillo Levee Segment" by S&B Infrastructure, dated December 22, 2008.
- *"Final Environmental Assessment, Flood Control Improvements to the Rio Grande Canalization Project*," by USIBWC, dated December 2007.
- "Riparian Habitat Restoration at Three Sites in New Mexico and Texas: Country Club East, Sunland Park, and Anapra Bridge Restoration Sites," by IDEALS-AGEISS, dated February 28, 2020.
- "Sunland Park Levee Forensic Geotechnical Investigation" by Kenall Inc., dated July 29, 2016.
- "Design Documentation Report, Sunland Park Levee Repair – West Levee," by Arcadis, Inc., dated August 14, 2019 (Revised August 21, 2019).
- U.S. Army Corps of Engineers, 2014, ER 1110-2-583, Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures.

- U.S. Army Corps of Engineers, 2006, ER 1110-1-12, Engineering and Design Quality Management.
- U.S. Army Corps of Engineers, 2005, ETL 1110-2-569, Design Guidance for Levee Underseepage.
- U.S. Army Corps of Engineers, 2005, EM 1110-2-2100, Stability Analysis of Concrete Structures.
- U.S. Army Corps of Engineers, 2004, EM 1110-2-2300, Engineering and Design - General Design and Construction Considerations for Earth and Rock-Fill Dams.
- U.S. Army Corps of Engineers, 2003, EM 1110-2-2400, Engineering Manual for Structural Design & Evaluation of Outlet Works.
- U.S. Army Corps of Engineers, 2003, EM 1110-2-1902, Slope Stability.
- U.S. Army Corps of Engineers, 2000, EM 1110-2-1913, Design and Construction of Levees.
- U.S. Army Corps of Engineers, 1999, ER 1110-1-2005, Compilation of Concrete Aggregate and Stone Riprap Test Data.
- U.S. Army Corps of Engineers, 1995, EM 1110-2-2002, Evaluation of Repair of Concrete Structures.
- U.S. Army Corps of Engineers, 1994, EM 1110-2-1601, Hydraulic Design of Flood Control Channels.
- U.S. Army Corps of Engineers, 1993, EM 1110-2-1901, Seepage Analysis and Control for Dams.
- U.S. Army Corps of Engineers, 1990, EM 1110-1-1904, Settlement Analysis.
- U.S. Army Corps of Engineers, 1989, EM 1110-2-2502, Engineering Manual for Retaining & Flood Walls.
- U.S. Army Corps of Engineers, 1984, EM 1110-3-136, Drainage and Erosion Control – Mobilization Construction.
- U.S. Department of Interior Bureau of Reclamation, 2015, Bank Stabilization Design Guidelines, Report No. SRH-2015-25.
- U.S. Department of Interior Bureau of Reclamation, 2011, Design Standards No. 13, Embankment Dams, Chapter 5: Protective Filters.
- “USIBWC Rio Grande Canalization Project, River Management Plan,” by USIBWC, dated December 2016.

Other regulatory documents and agencies pertained to this project included:

- AASHTO Guide for Design of Pavement Structures, 1993.
- AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002.
- ACI 350, Code Requirement for Environmental Engineering Concrete Structures, 2006.

- ACI Manual of Concrete Practice, 2014.
- ACI Detailing Manual, SP-66(04), 2004
- AISC 303, Code of Standard Practice for Steel Buildings and Bridges, 2016.
- AISC 360, Specifications for Structural Steel for Buildings, 2016.
- ASCE 7-10, Minimum Design Loads for Buildings and Other Structures, 3rd Printing
- ASTM International Standards.
- Executive Order 11988, Floodplain Management (https://www.fema.gov/media-library/assets/documents/235?id=1395)
- Executive Order 12088, Federal Compliance with Pollution Control Standards (http://www.archives.gov/federal-register/codification/executive-order/12088.html)
- Occupational Safety and Health Standards (29 CFR 1910) (https://www.osha.gov/law-regs.html)
- Texas Department of Transportation (TxDOT), Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, November 1, 2014.

APPENDIX A

Appendix A - Photographic Logs

Appendix A.1 General Observations – Photographic Logs

Appendix A.2 Structures – Photographic Logs

Appendix A.1: General Observations – Photographic Logs

Photo 1: East levee project limit at Montoya Drain (*Date: 10/12/2017*)

Photo 2: Levee erosion near Montoya Drain (*Date: 10/12/2017*)

Photo 3: Piping example at the crest (*Date: 10/12/2017*)

Photo 4: Erosion example (*Date: 10/12/2017*)

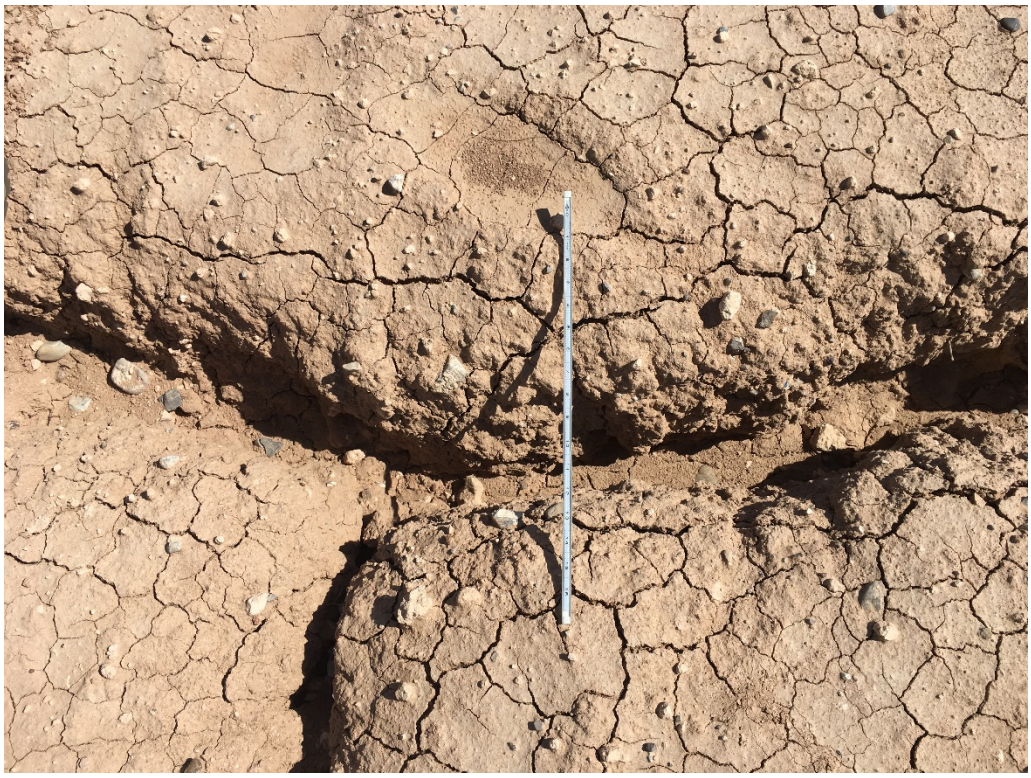


Photo 5: Rillying example about 5 inch wide (*Date: 10/12/2017*)

Photo 6: Severe rillying along the slope (*Date: 10/12/2017*)

Photo 7: Rills of 11 inch deep (*Date: 10/12/2017*)

Photo 8: Montoya Lateral (*Photo taken on 10/12/2017*)

Photo 9: Existing floodgate at the Country Club Bridge (East Levee) (*Date: 10/12/2017*)

Photo 10: Erosion north of Borderland Bridge (East Levee) (*Date: 10/12/2017*)

Photo 11: Rills of 19 inches deep (West Levee) (*Date: 10/12/2017*)

Photo 12: Rills of 36 inches deep (West Levee) (*Date: 10/12/2017*)

Photo 13: Encroachment issue. IBWC Right-of Way post within property fencing. (*Date: 10/12/2017*)

Photo 14: Encroachment issue, drain shown appears to be on IBWC property. (*Date: 10/12/2017*)

Photo 15: Encroachment issue, drain shown appears to be on IBWC property. (*Date: 10/12/2017*)

Photo 16: Nemexas siphon, looking east (*Date: 10/12/2017*)

Photo 17: Well pump looking east (*Date: 10/12/2017*)

Photo 18: Nemexas siphon (looking southwest towards levee) (*Date: 10/12/2017*)

Photo 19: Floodgate flap valve and Floodwall at Sunland Park Rd. (*Date: 10/12/2017*)

Photo 20: Floodgate flap valve and Floodwall at Sunland Park Rd has been covered by metal plates. (*Date: 10/12/2017*)

Photo 21: Floodgate at Sunland Park Rd metal plates covers and 2" asphalt on top of gates. (*Date: 10/12/2017*)

Photo 22: Metal guard rails at Sunland Park Rd can be removed during construction (*Date: 10/12/2017*)

Appendix A.2: Structures – Photographic Logs

PHOTOGRAPHIC LOG OF THE STRUCTURES (LOOKING DOWNSTREAM)

EAST SIDE LEVEE, 18’ +/- LEFT FROM CL LEVE STA 1531+83.5. DATE: 12/12/18

EAST SIDE LEVEE, 18’ +/- LEFT FROM CL LEVE STA 1543+50.95. DATE: 12/12/18

EAST SIDE LEVEE, 40’ +/- LEFT FROM CL LEVE STA 1547+39.45. DATE: 12/12/18

EAST SIDE LEVEE, 37’ +/- LEFT FROM CL LEVE STA 1553+85.42. DATE: 12/12/18

EAST SIDE LEVEE, 28’ +/- LEFT FROM CL LEVE STA 1666+76.96. DATE: 12/12/18

EAST SIDE LEVEE, 29’ +/- LEFT FROM CL LEVE STA 1667+91.9. DATE: 12/12/18

EAST SIDE LEVEE, 48’ +/- LEFT FROM CL LEVE STA 1706+43.11. DATE: 12/12/18

EAST SIDE LEVEE, 27’ +/- LEFT FROM CL LEVE STA 1761+76.6. DATE: 12/12/18

EAST SIDE LEVEE, 40’ +/- RIGHT FROM CL LEVE STA 1762+20.99. DATE: 12/12/18

EAST SIDE LEVEE, 24’ +/- RIGHT FROM CL LEVE STA 1906+11.88. DATE: 12/12/18

EAST SIDE LEVEE, 48’ +/- LEFT FROM CL LEVE STA 1932+18.49. DATE: 12/12/18

EAST SIDE LEVEE, 49’ +/- LEFT FROM CL LEVE STA 1933+82.66. DATE: 12/12/18

WEST SIDE LEVEE, 34’ +/- RIGHT FROM CL LEVE STA 1666+85.29. SCHULTZ LATERAL WW#35C. DATE: 12/12/18

WEST SIDE LEVEE, 60’ +/- LEFT FROM CL LEVE STA 1667+47.22. SCHULTZ LATERAL WW#35C. DATE: 12/12/18

APPENDIX B

Appendix B - Geotechnical Basis of Design

Appendix B.1 Geotechnical Memorandum

Appendix B.2 Seepage and Slope Stability

Appendix B.3 Settlement

Appendix B.4 Riprap Sizing

Appendix B.1: Geotechnical Memorandum

TECHNICAL MEMORANDUM

ARCADIS Design & Consultancy for natural and built assets

To:
Michael Landis, PhD, PE
Contracting Officer Representative

Copies:
Lanisa McCollum (USIBWC)
Larry Roth, P.E. (Arcadis)

Arcadis U.S., Inc.
12400 Coit Road
Suite 1200
Dallas
Texas 75251
Tel 972 934 3711
Fax 972 934 3662

ARCADIS U.S., Inc.
TX Engineering License # F-533

From:
Ardita Dushi, PE
Kirk Lowery, PE

Date:
May 17, 2019

Arcadis Project No.:
01991047.0000

Subject:
Geotechnical Basis of Design Memorandum
Design for the Sunland Park Levee Repair
Contract No. IBM15D0001 – IBM17T0010

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1 INTRODUCTION

The United States International Boundary and Water Commission (USIBWC) contracted with Arcadis U.S., Inc. (Arcadis) for the Design of the Sunland Park Levee Repair within the Rio Grande Canalization Project. In addition, Arcadis will support USIBWC during project procurement and contracting process to help make sure the construction processes comply with the intent of the submitted design and construction documents. For this project, Munoz and Dannenbaum Joint Venture (MDJV) is assisting Arcadis with the structural design and Frank X. Spencer & Associates (FXSA) is assisting with the surveying. Several studies including geotechnical investigation and laboratory testing have been performed in this area. Therefore, additional geotechnical investigation is not needed at this time for this scope of work.

For the kick off meeting on October 12, 2017 in El Paso, Texas, Arcadis presented to USIBWC eleven (11) design alternatives that were evaluated to be considered for these levee segments, which include four that were identified by Kenall, Inc. (Kenall) in its final forensic report titled “Sunland Park Levee Forensic Geotechnical Investigation,” dated July 29, 2016, and seven additional alternatives identified by Arcadis:

1. Reconstruct with non-dispersive soil (Kenall)

2. Reconstruct with lime-treated soil (Kenall)
3. Use zoned embankment (Kenall)
4. Use non-dispersive clay slope blanket (Kenall)
5. Use slope blanket (Option 4) with lime treatment (reduce haul-off, haul-on)
6. Use slope blanket (Option 4) comprising 6 inches of sand overlain by 8 inches of cobbles
7. Excavate in ~10-foot swaths and re-place excavated soil with soil amendment such as lime.
8. Treat in-place using soil amendment
9. Use soil-cement, asphalt, or Portland cement slope paving
10. Use clay slope blanket without excavation, except at crest
11. Use slope blanket comprising 6 inches of sand overlain by 8 inches of cobbles (no excavation)

In the original scope of work, the USIBWC indicated a preference for Design Option #4 above. However, at the end of the presentation, USIBWC requested that three options be evaluated for the design:

- Design Option #4: Clay blanket
- Design Option #7: Lime Treatment
- Design Option #11: Sand-Riprap Blanket

At the scoping meeting on December 13, 2017, Arcadis presented the evaluation of these three conceptual design alternatives that meet the intent of 44 CFR §65.10 requirements and their associated costs. An additional blended design option (lime treatment and sand-riprap blanket) was also presented at this meeting as discussed in the following sections. These four design options are explained in detail in Section 5.

2 PROJECT BACKGROUND

The USIBWC issued in September 2010 a construction contract to rehabilitate the Sunland Park Levee segment of the Upper Rio Grande Flood Control Project (URGFCP). Construction was completed in November 2012. However, the levee has not held up to the standards for durability that were expected after rehabilitation, and has experienced considerable erosion.

The USIBWC procured Kenall in April 2015 to perform a Forensic Geotechnical Investigation and provide the USIBWC a professional opinion on whether the newly constructed levee segment would perform satisfactorily during the 100-year flood event pursuant to 44 CFR §65.10 (FEMA). Kenall identified a series of construction-related problems, and its investigation found dispersive clay within the existing levee embankment along the entire length of this reach of the levee system. As a result of the erosion of the dispersive clay, the Sunland Park Levee Segment

does not meet the FEMA 44 CFR §65.10(b) requirements. Kenall also provided in its report the four remedial concepts for levee rehabilitation listed above.

The USIBWC initially determined that Remedial Concept #4 of Kenall's alternatives would be required to repair the entire length of the Sunland Park Levee segments. Remedial Concept #4 consists of 1) excavating and hauling off 2 feet of soil (measured normal to the slope), and 2) blanketing the slopes with a minimum of 2 feet of non-dispersive clay, which will extend over the riverside slope, top of levee and landside slope of the levee. The USIBWC procured Arcadis in September 2017 to provide a levee design that meets the FEMA requirements.

A finished gravel Operations and Maintenance (O&M) road of 16 feet wide will be provided on top of any earthen levees or on the landside of any floodwalls. The USIBWC prefers the cross section of the roadway to slope towards the flood side along the alignment. Ramps to the levee will be designed as permitted with a cut-out section doubling the finished gravel O&M road to 32 feet wide.

3 PROBLEMS WITH DISPERSIVE CLAY

The problems associated with dispersive clay are generally related to both erosion of external slopes and internal erosion. Surface erosion on embankments constructed of dispersive clay cannot generally be prevented with vegetation. Internal erosion generally must be controlled with carefully designed filters. Therefore, remedial repair through rehabilitation or removal and reconstruction of the levee is required to meet 44 CFR Section 65.10(b) criteria.

Erosion of External Slopes

Erosion of the external slopes comprising dispersive clay can occur due to rainfall and runoff and often results in severe "rilling" and "gullying" on the slopes. Kenall's forensic study indicated that rills observed on the external slopes were between 15 to 26 inches deep. Arcadis' site visit on October 12, 2017, measured rills as deep as 36 inches at one location of the west levee segment as shown in Figure 1. The continued erosion since Kenall's work has resulted in gullies in some areas and "jugging" as shown in Figure 2.

Figure 1. Example of West Levee Erosion

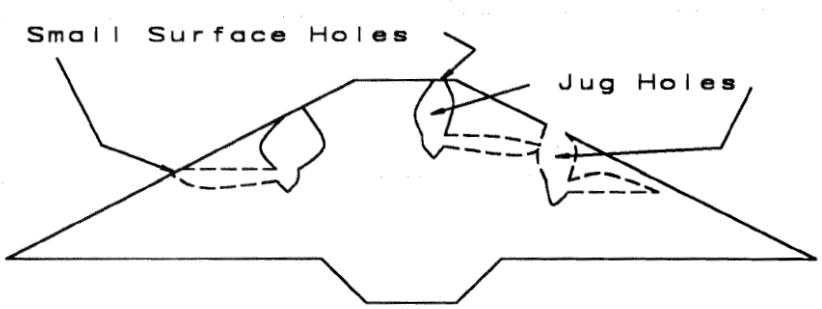

Figure 2. Jugging Example

Kenall's Ground Penetrating Radar (GPR) Survey performed in 2015 has depicted a series of jugging, gullying, loose fill, and voids throughout the levee segments during its field investigation. Tunneling and jugging can be initiated as the dispersive clay particles are carried laterally to the slope surface from the bottom of a vertical crack. Interconnected holes and rills could ultimately lead to breaching of the embankment at high water levels.

Internal Erosion

Internal erosion from dispersive clay may occur as a result of seepage through cracks, penetrations, or openings in the embankment. Water flowing through the cracks caused by desiccation, hydraulic fracturing and/or differential settlement could rapidly enlarge the crack and cause breaching of the embankment.

4 ASSUMPTIONS

Construction activities were stopped on November 2, 2012, prior to placing 6 inches of topsoil (measured normal to the slope) or vegetating the slopes. There are no as-built drawings available. Based on the existing available information, the design options evaluated in this report assume that the existing levee geometry meets the freeboard and slope requirements as shown on construction plans for *Design of Rehabilitation Improvements for the Rio Grande Canalization Protective Levee system, Sunland Park Package* dated August 25, 2010, by S&B Infrastructure, except that the side slopes of the levee prism are 6-inches lower than shown on the design because the topsoil was not placed.

Arcadis compared eleven (11) cross-sections that were surveyed by Barragan & Associates, Inc. (Barragan) in 2015 (included in Kenall's forensic report) with the proposed design cross-sections shown in the IBM10C0018 construction drawings by S&B Infrastructure dated August 25, 2010. These comparisons as presented in Attachment A show that the levee prism from the Barragan survey is similar to the S&B proposed design cross sections at most of the cross-sections and are therefore satisfactory for use in the evaluation of design options.

The following factors are assumed for the existing levees for the design option evaluation and cost estimates.

Table 1. Assumed Existing Levee Conditions

Description	Based on Existing Data
Riverside slope	3H:1V (generally)
Landside slope	2.5H:1V (generally)
Levee crest	At least 3 feet above BFE(1) and at least 4 feet above within 100 feet of structures
Crest slope	2% towards the riverside
Top surficial soil type	Sandy Lean Clay
6-inches of topsoil	Not placed
Anticipated long term settlement	Max 4 inches
Toe drain on the landside for levee height > 5.5 ft	3 ft wide and 4 feet deep filled with gravel and wrapped with geotextile fabric
Existing crown width	16 ft (measured < 16 ft at several locations)

Notes: (1) BFE = Base Flood Elevation

5 BASIS OF DESIGN

This section describes the design options selected by the USIBWC for detailed analysis.

5.1 Design Option #4: Clay Blanket

5.1.1 Remedial Concept

This alternative involves leaving the levee as is but 1) excavating a two-foot thick layer of existing soil from the slopes and crest (measured normal to the ground surface), and 2) placing a non-dispersive clay blanket. The blanket would extend over the riverside slope, top of levee and landside slope of the levee and should be a minimum of two feet thick when measured normal to the levee surface as shown in Figure 3.

To ensure that the embankment in place meets the compaction requirements, the levee slopes should be over-built and then trimmed to match pre-remediation slope angles after lift placement and compaction is complete. A minimum 2-foot thick blanket of non-dispersive clay (measured normal to the levee surface) should remain after completion of the trimming process. A 6-inch topsoil will be added on the slopes. A minimum of 6 inches of the existing foundation soils will be

scarified and recompacted. A typical section is shown in Figure 3. A simplified construction sequence for this alternative is presented in Attachment B.

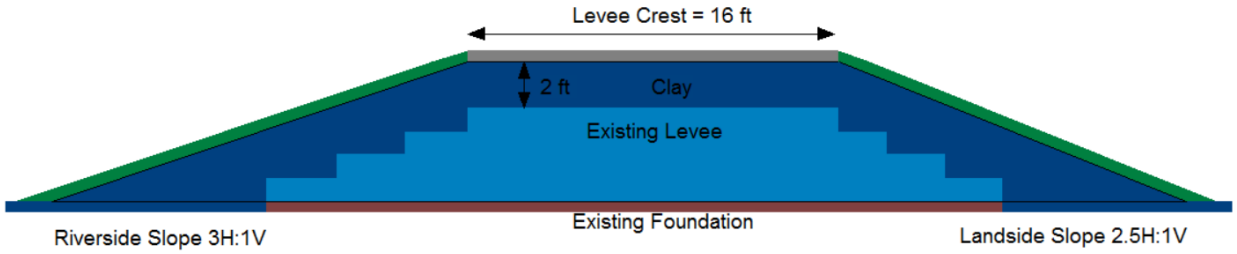

Figure 3. Clay Blanket Design Option

The following considerations are related to the clay blanket design option:

- Excavation of a 2-foot thick layer normal to the slope could be difficult and costly. For this reason, this option includes excavation in horizontal layers that are at least one equipment-width wide.
- Similarly, placement and compaction of a two-foot thick layer on a slope could be difficult, costly, and may not be successful. For this reason, non-dispersive clay will be placed and compacted in horizontal layers, overbuilt as described above.
- Placement of the non-dispersive clay is intended to mitigate the potential for excessive erosion of dispersive clay on the riverside slope during the base flood event if it occurs during construction.
- Vegetation growth would reduce erosion on the slopes. However, vegetation growth in the semi-arid climate could also be difficult.
- Long-term levee maintenance would be needed as long-term erosion will still occur but at a much slower rate compared to the currently dispersive clays.
- The possibility of desiccation cracks could lead to erosion and maintenance problems.
- During construction of repairs, a large number of samples are required to be tested to establish with certainty whether soils of the borrow source are non-dispersive.
- The availability and cost of the imported material must be assessed.
- The availability and cost of waste disposal sites and the required haul-off costs must be assessed.
- Internal erosion will still need to be addressed. If needed, the most effective design measure for preventing internal erosion of earthfills is a sand chimney filter. Sand diaphragms around the conduit are an effective method of controlling internal erosion.
- Details need to be developed at the waterside toe of the levee to ensure the underlying dispersive soils are protected from erosion.

- Details need to be developed for protecting the underlying dispersive soils from erosion at the landside toe drains.
- Details need to be developed for protecting the dispersive soils from erosion at all structures and penetrations.
- The need for and type of long-term maintenance must be assessed.

5.1.2 Materials Needed

The clay borrow should classify as CL in accordance with ASTM D2487, Unified Soil Classification System, with a plasticity index between 15 to 30 and a maximum liquid limit of 45. Fat clay should not be used due to the higher susceptibility to desiccation cracking.

The clay borrow must be non-dispersive and meet the testing requirements listed below:

- Crumb Test (ASTM D6572) - Grade 1 (non-dispersive)
- Pinhole Test (ASTM D4647) - ND1 or ND2
- Double Hydrometer (ASTM D4221) – Percent dispersive less than thirty (30)
- At least sixty percent (60%) passing a US Standard No. 200 sieve
- One hundred percent (100%) of the dry material passing the one (1) inch sieve
- Contain less than thirty five percent (35%) sand content by weight

The embankment fill shall be compacted to a minimum of 95% of maximum density at a moisture content above optimum moisture content as determined by ASTM D698, Standard Proctor.

5.1.3 Impact of BFE

This alternative does not change the levee prism and should not have any major impact on the base flood elevation (BFE).

5.1.4 Volumes and Construction Cost

Volumes and costs associated with clay blanket design option were estimated for two conditions: 1) assumes the landfill and the borrow source locations are near the project site, and 2) assumes the landfill and the borrow source locations are about 40 miles away from the project site.

5.2 Design Option #7: Lime Treatment

5.2.1 Remedial Concept

Exposed slopes of the fill can be treated with chemical amendments to alter dispersive clay and make them suitable for use on external slopes of embankments. Some chemical amendments that may be considered are hydrated lime, fly ash, gypsum, agricultural lime, and magnesium

chloride. They are added to the dispersive clay at a given rate in a percentage of dry weight basis and then the treated soil is moisture-conditioned and compacted into the embankment. Lime treatment is one of the most common treatments used for soil amendment in the region. The U.S. Army Corps of Engineers (USACE) and the Natural Resources Conservation Service (NRCS) have used lime on the levee slopes on some of their projects.

The excavation portion of this alternative would be similar to the clay blanket option described above. The excavations will need to be staged so that excavated materials from one section of levee can be hauled and taken to a staging area in the floodplain. After the soil is spread-out, lime would be added, disked to mix, moisture-conditioned, and would stay for about 24 hours to "mellow." The chemical reaction between the lime and soil happens during the mellowing phase. The treated soil would then be placed and compacted in layers in the embankment. The contractor's operation should be carefully monitored in the field.

The levee slopes should be overbuilt and trimmed as described. A minimum 2-foot thick lime-treated soil layer, when measured normal to the levee surface, should remain after completion of the trimming process. A minimum of 6 inches of the existing foundation soils will also be excavated and treated with lime as described. A typical section is shown in Figure 4, and a simplified construction sequence for this alternative is presented in Attachment B.

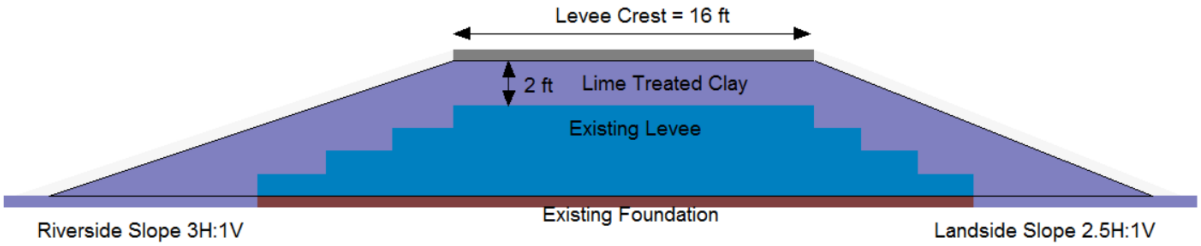

Figure 4. Lime Treatment Design Option

The following considerations are related to the lime treatment design option:

- This option may produce a long-lasting solution for treating dispersive clays if the soils can be successfully amended for placement in the compacted embankment.
- An extensive laboratory testing program will need to be developed for this design option to confirm the percentage of lime required to stabilize the existing dispersive levee soils. Lab testing will be required to: a) determine mix design for application; b) confirm that lime-treated clay is not dispersive; c) confirm soil strength for stability analyses; and (possibly) d) evaluate shrinkage (for desiccation cracking) and erodibility.
- Lime-treated soil should be tested for Atterberg limits to ensure that new shrinkage limits of the treated soil can reduce the development of the drying cracks.
- Testing for lime treatment was not included in this scope of work.
- This option reduces haul-off of the existing material.
- Chemical amendment can be used as water barrier zones in the embankment.

- Lime percentage for dispersive clay cannot be known without comprehensive testing, but could vary from 3% to as much as 8% by weight.
- Soils treated with lime have a high pH (> 11) and the establishment of the vegetation could be difficult at these pH levels. Vegetation could be established if topsoil is placed on lime-treated soils. However, vegetation growth in the semi-arid climate could also be difficult.
- Soil treated with a mixture of agricultural lime (2/3) and hydrated lime (1/3) has a lower pH after the application and may have potential for the establishment of vegetation.
- Application of hydrated lime, processing, and curing can be costly.
- The availability and cost of lime and the cost of processing the amended soil must be assessed.
- The need for and type of long-term maintenance must be assessed.
- QC inspection during construction should be thoroughly performed.

5.2.2 Materials Needed

The percentage of the lime required to treat the dispersive clay will be dependent upon the testing results. Existing embankment material could be used at the site to mix with lime. If the volume of treated clay is insufficient, imported non-dispersive clay material may be needed. Imported topsoil will be required.

5.2.3 Impact of BFE

This alternative does not change the levee prism and should not have any major impact on the BFE.

5.2.4 Volumes and Construction Cost

Volumes and costs associated with lime treatment design option were estimated.

5.3 Design Option #11: Sand-Riprap Blanket

5.3.1 Remedial Concept

Exposed slopes can be covered with sand and riprap to protect the underlying dispersive clays from erosion caused by surface water and rainfall runoff. The slope blanket of sand on the landside slope should also serve as a filter to control internal erosion of dispersive clay caused by seepage. The slope blanket may comprise 6 inches of sand overlain by about 12 inches of riprap and no excavation should be required. A filter fabric may be placed between the existing levee slopes and sand to prevent the soil migration. Erosion at the crest could be prevented by excavating the upper 1 to 2 feet of existing levee soils at the crest and either replacing with non-dispersive clay or treating the existing soil with lime. A minimum of 12 inches of the existing

foundation soils at the toes will be excavated and replaced with sand and riprap. A typical section is shown in Figure 5, and a simplified construction sequence for this alternative is presented in Attachment B.

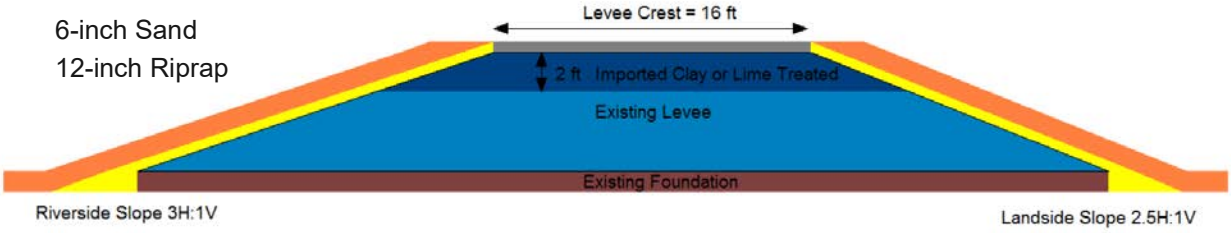

Figure 5. Sand-Riprap Blanket Design Option

The following considerations are related to the sand-riprap blanket design option:

- No excavation would be needed if it can be shown that the increased size of the levee prism does not affect the BFE.
- Reduces or eliminates haul-off and disposal, which saves costs and reduces construction traffic.
- This could be very challenging to construct. Lifts may need to be a minimum of 2-3 ft wide to be able to construct.
- Vibratory drum rollers will be rolled in place and less water may be needed for compaction.
- Sand-riprap blanket should protect underlying dispersive clay from developing drying cracks and jugging.
- In general, concrete sand can successfully filter dispersive clay, and riprap can provide needed erosion protection.
- The levee slopes are subject to traffic from people seeking access to the river bank for fishing, etc. The riprap cover layer must be built with angular rocks that will knit together and be resistant to being dislodged by traffic. Some work will be required to determine the required maximum particle size and gradation of the riprap layer to 1) be stable under traffic loads from vehicles driving on the slopes; and 2) provide the cover and filter properties necessary to hold the sand in place. The thickness of the riprap layer should be at least 1.5 times the maximum particle size.
- Details will need to be developed at the crest shoulders and the levee toes to ensure that erosion cannot create shortcuts that expose the underlying soils.
- The need for and type of long-term maintenance must be assessed.
- The availability and cost of imported sand and cobbles must be determined.

5.3.2 Materials Needed

The sand material may consist of a fine concrete aggregate (NCTCOG Item 303.2.1.2.) with a gradation as shown on Table 2. The riprap may consist of TxDOT Item 432, Class B.

Table 2. Gradation for Filter Sand (Fine Concrete Aggregate)

Gradation Sieve Size	Percent Retained (by weight), %	Percent Passing (by weight), %
3/8"	0	100
No. 4	0-5	95-100
No. 8	0 - 20	80-100
No.16	15 - 50	50-85
No. 30	35 - 75	25-65
No. 50	70 - 90	10-30
No. 100	90 - 100	0-10
No. 200	97 - 100	0-3

5.3.3 Impact of BFE

This alternative would require a change in the levee prism. However, based on a preliminary H&H analysis with the 2009 FLO-2D model, this alternative should not have any major impact on the BFE. The velocities developed from this alternative would need to be compared with USACE mean velocities requirements.

5.3.4 Volumes and Construction Cost

Volumes and costs associated with sand-riprap design option were estimated. Three conditions were assumed for the cost estimates presenting a different combination of sand, riprap and filter fabric 1) filter fabric / sand / riprap combination, 2) sand / riprap combination, and 3) filter fabric / riprap combination.

5.4 Blended Design Option: Lime Treatment and Sand-Riprap Blanket Combination

5.4.1 Remedial Concept

Due to the right-of-way limitations on the landside, a blended design option was also considered for evaluation. The blended design option would consist of lime treatment of the crest and landside slopes, and placement of sand-riprap blanket on the riverside slopes. The methodology

for lime treatment and sand-riprap would be similar as described in the above sections. A typical section is shown in Figure 6.

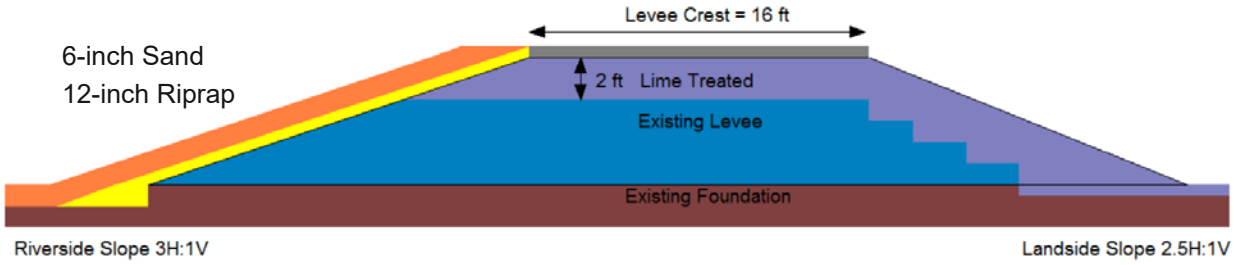

Figure 6. Blended Design Option: Sand-Riprap Blanket on Riverside and Lime Treatment on the Landside and Crest

5.4.2 Materials Needed

See Sections 5.2.2 and 5.3.2.

5.4.3 Impact of BFE

See Sections 5.2.3 and 5.3.3.

5.4.4 Volumes and Construction Cost

Volumes and costs associated with the sand-riprap and lime treatment blended design option were estimated.

5.5 Uncertainties and Issues of Concern for the Design Options

The levee conditions were assumed as discussed in Section 4. Other uncertainties and issues were considered for the evaluation and cost estimate as presented in Table 3.

Table 3. Uncertainties and Issues of Concern

Option No. 4 (Clay Blanket)	Option No. 7 (Lime Treatment)	Option No. 11 (Sand-Riprap Blanket)
1. Source of borrow 2. Cost of borrow 3. Haul distance for borrow 4. Location of waste disposal site 5. Haul distance for waste 6. Cost of waste disposal 7. Erosion protection 8. Maintenance of slopes 9. Source of topsoil (if used)	1. Success of lime treatment if no lab testing is done 2. Amount of lime required 3. Impact of zones of sandy, non-plastic materials 4. Erosion protection 5. Maintenance of slopes 6. Source of topsoil (if used) 7. Cost of topsoil (if used) 8. Haul distance for topsoil	1. Source of imported materials 2. Cost of imported materials 3. Haul distance for imported materials 4. Maintenance of slopes 5. Impact (if any) on BFE of encroaching on the channel 6. Difficulty of preparing the ground surface to receive fill

Option No. 4 (Clay Blanket)	Option No. 7 (Lime Treatment)	Option No. 11 (Sand-Riprap Blanket)
10. Cost of topsoil 11. Haul distance for topsoil 12. Impact of flood during construction 13. QC testing during construction 14. Source of water for compaction	9. Impact of flood during construction 10. QC testing during construction 11. Source of water for compaction	7. Difficulty of filling subsurface voids that may not be easily detected 8. Filling to provide a 16-foot crest roadway 9. Source of borrow for crest materials (if imported) 10. Cost of borrow 11. Haul distance for borrow

Some of the uncertainties / issues presented in Table 3 were addressed as follows in the cost estimate.

- **Location of disposal site:** Developed costs for two sites, one six miles distant (Camino Real), and one 38 miles distant (City of El Paso Landfill)
- **Borrow source:** Developed costs for 10- and 40-mile distance sites.
- **Roadway crest:** Will need to add fill to create 16-foot roadway in all locations. Has no impact on clay and lime options as levee will be re-built. Some fill required for sand-riprap blanket option.
- **Width of roadway:** We have assumed 16 feet for the cost estimate.
- **Erosion protection:** Assumed topsoil for Option Nos. 4 and 7. Potential problems with establishing and maintaining vegetation. Sand-riprap will provide superior performance.
- **QC Testing:** Extensive testing of borrow and fill required for Option No. 4. Option No. 7 will require extensive testing to determine amount of lime required to remediate dispersive clay. Should this be performed before construction, or by construction contractor?
- **Trafficability:** Concerns for traffic on side-slopes affecting performance and need for O&M.

6 ADVANTAGES AND DISADVANTAGES OF EACH ALTERNATIVE

A comparison analysis is being performed to evaluate the alternatives presented and described in Section 5. The attributes affecting the project are developed and evaluated as presented in Table 4.

Table 4. Comparison of Attributes for the Design Options

Attributes	Clay Blanket	Lime Treatment	Sand-Riprap Blanket	Blended Option
Cost	$37M to $45M(1)	$38M	$30M to $34M	$36M
Borrow Material needed	Non-dispersive Clay / Topsoil	Lime	Sand and Riprap	Lime, Sand, Riprap
Borrow Source	Hard to find	Easy to find	Easy to find	Easy to find
Long-term performance	Low	Low	Moderate	Low to Moderate
Treatment of existing gullies	Easy	Easy	Moderate	Moderate
O&M	Comprehensive	Low	Low	Low
Constructability	Moderate	Moderate	Easy	Easy to Moderate
Preliminary Estimate of Time of Construction for Earthwork	24 months	9 months	12 months	9 months
Testing required before Construction	Low	Comprehensive	Low	Moderate
Testing required during Construction	Comprehensive	Comprehensive	Moderate	Comprehensive
Testing of Borrow before Construction	Comprehensive	N/A	Moderate	Moderate
Testing of Borrow during Construction	Comprehensive	N/A	Moderate	Moderate
Impact of BFE	None	None	Low	Low
Risk of Flood during Construction	High	High	Moderate	High
Environmental Impact	High GHG(2)	Low GHG	Moderate GHG	Moderate GHG
Vegetation	Grass growth	Limited Vegetation	No grass growth	Limited Vegetation
Dust Control	High	Moderate	Low	Moderate
Traffic During Construction	High	Low	Moderate	Low

Attributes	Clay Blanket	Lime Treatment	Sand-Riprap Blanket	Blended Option
Estimated One-way Truck Trips during Construction	66,000	3,000	36,000	19,000
Hauling Material	Imported and Wasted Clay	Imported Lime	Imported Sand and Riprap	Imported Lime, Sand and Riprap
Water Needed for Construction	High	High	Low	Moderate

Notes: (1) Range of cost is dependent on haul distance.
(2) GHG = Greenhouse Gas

7 CONCLUSIONS

The cost estimates provided in this document involve levee design evaluation only at a conceptual level. The structures and other associated construction costs were not included in this evaluation. Arcadis presented the evaluations of these conceptual design options at the scoping meeting on December 13, 2017. Arcadis also presented advantages and disadvantages of each alternative at this scoping meeting. After an internal discussion, USIBWC notified us on January 11, 2018, that they selected the clay blanket option for the levee design as presented in Section 5.1.

8 REFERENCES

- "Boring Logs and Laboratory Test Results, Courchesne Reach and NEMEXAS Reach Canalization Project, Area 1, BH1 through BH4," dated December 14, 2012 by Ninyo & Moore.
- “Design Report for Upper Rio Grande Flood Control Project, Sunland Park Canutillo Levee Segment,” by S&B Infrastructure, dated July 23, 2010.
- Federal Emergency Management Agency, Code of Federal Regulations, Title 44, Section 65.10 (44 CFR 65.10), Mapping of Areas Protected by Levee Systems, 2017 *[Note: FEMA website indicates an update as of November 30, 2017. The document for Section 65.10 refers to 44 CFR Ch. I (10-1-06 Edition)].*
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- “Final Technical Memorandum Volume I for Geotechnical Explorations of Levee System within the Rio Grande Canalization Project- Canutillo Levee Segment” by Raba-Kistner Consultants, Inc., dated December 22, 2008.
- “Final Technical Memorandum Volume II for Geotechnical Explorations of Levee System within the Rio Grande Canalization Project- Canutillo Levee Segment” by Raba-Kistner Consultants, Inc., dated December 22, 2008.

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- U.S. Army Corps of Engineers, 1999, ER 1110-1-2005, Compilation of Concrete Aggregate and Stone Riprap Test Data

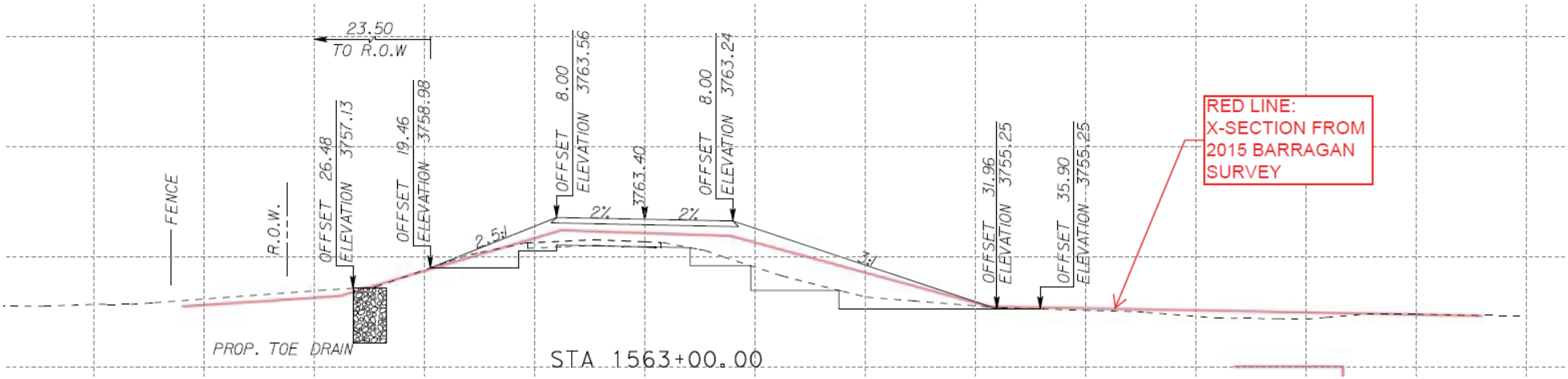
ATTACHMENT A

2015 Barragan Survey vs 2010 S&B Infrastructure Proposed Cross Sections

2010 Proposed Design vs 2015 Barragan Survey

Segment	Kenall's Bore No.	Approximate Sta. No.
CE	21	1563+00
CE	41	1614+00
CE	63	1673+00
CE	83	1722+00
CE	105	1776+00
CE	125	1826+00
CE	147	1881+00
CE	169	1937+00
CW	197	1571+00
CW	217	1622+00
CW	239	1679+00

2010 Proposed Design vs 2015 Barragan Survey



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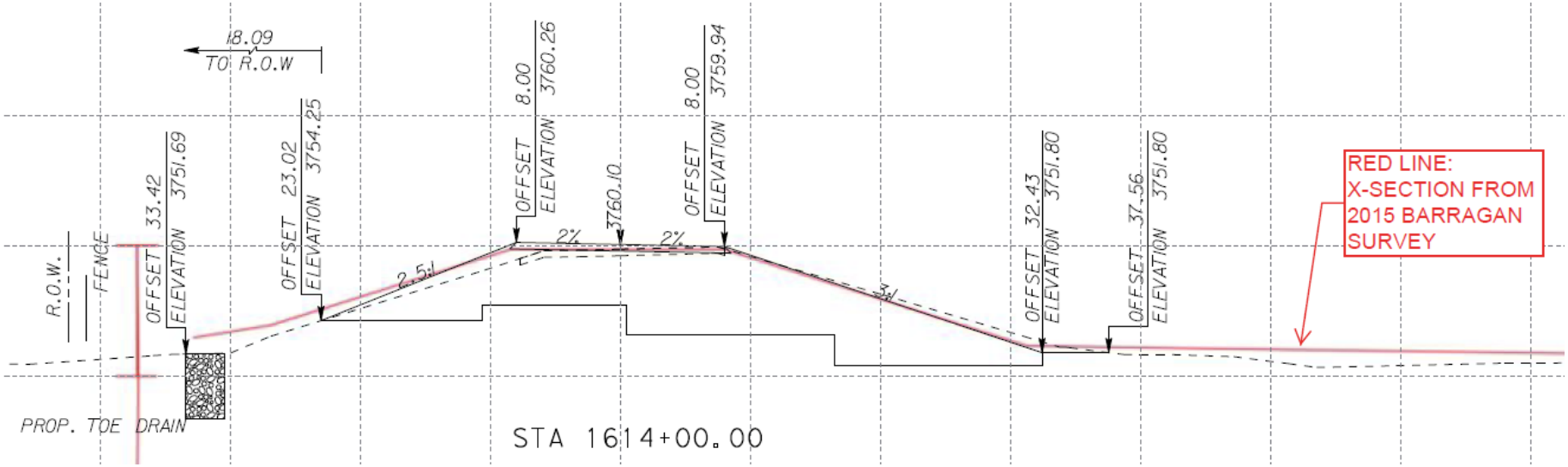

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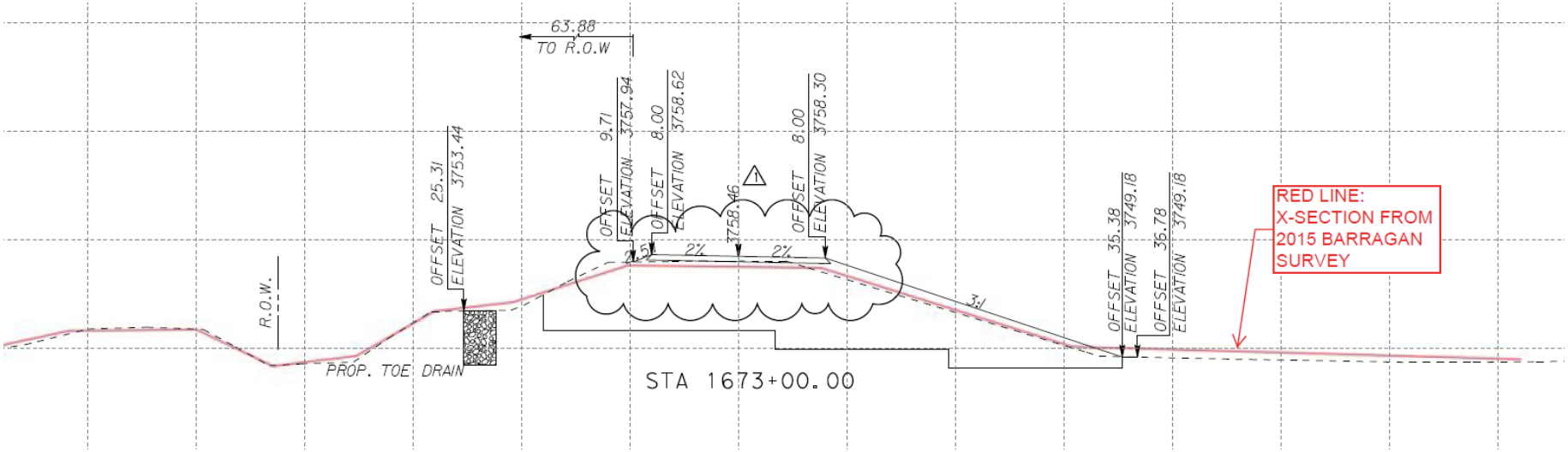

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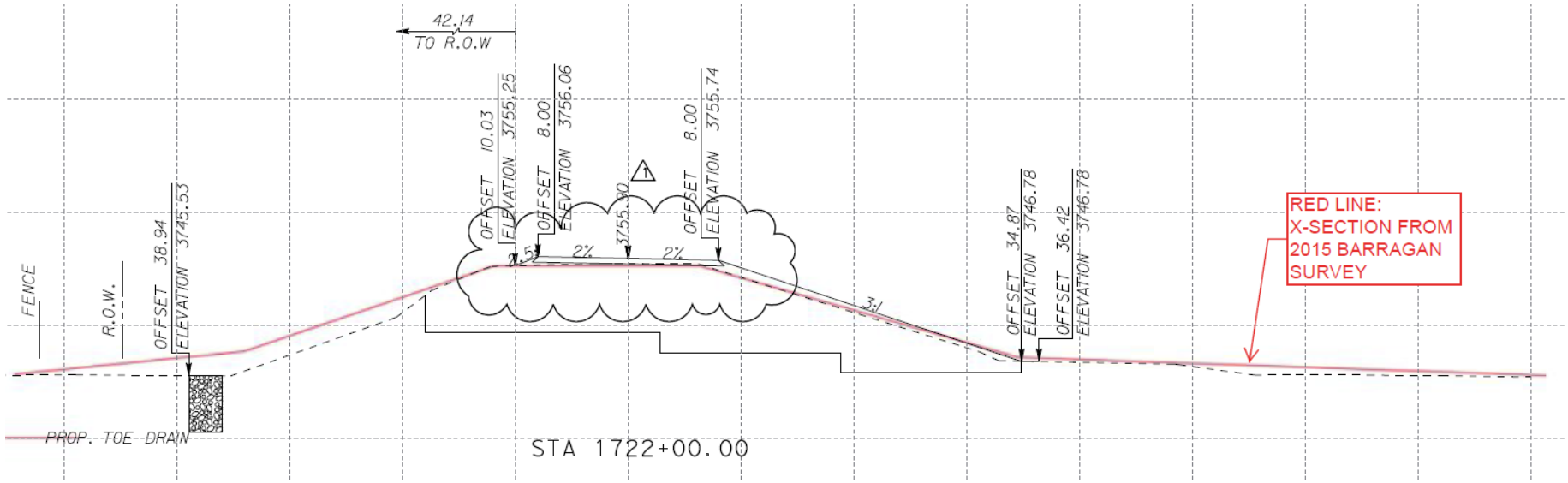

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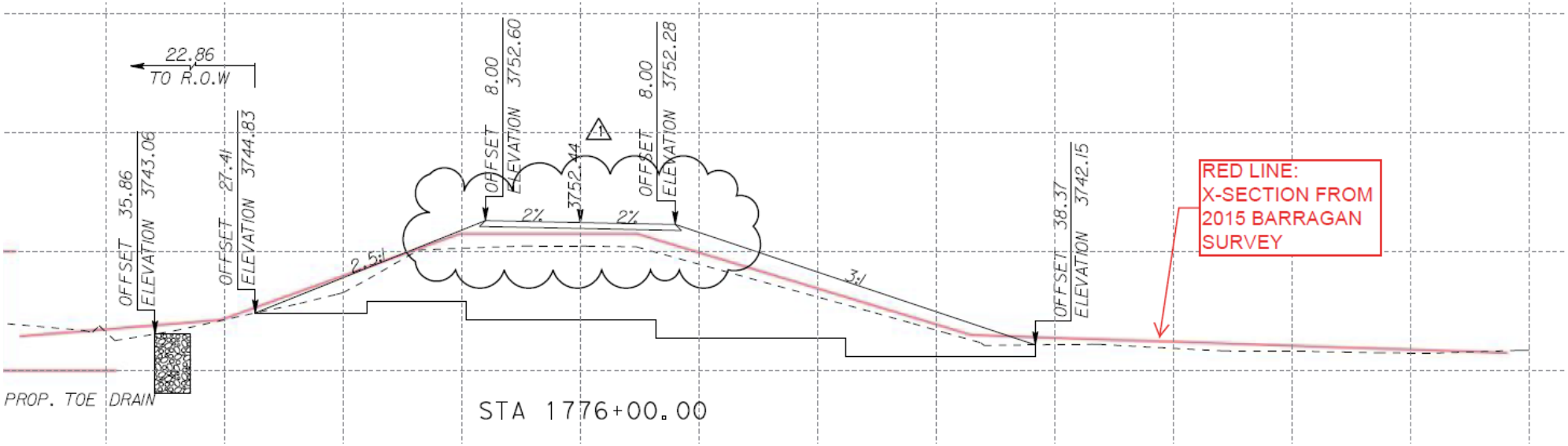

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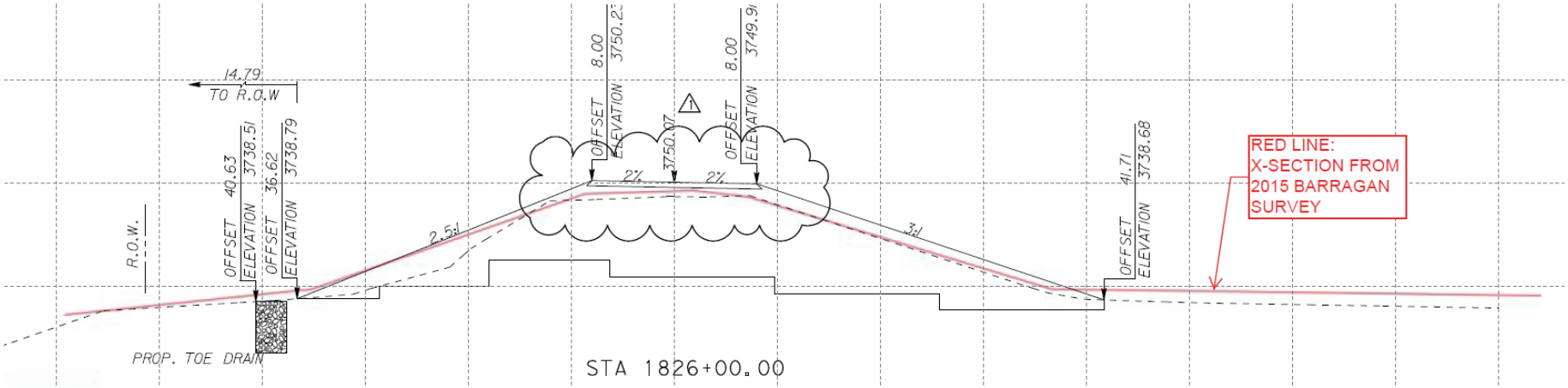

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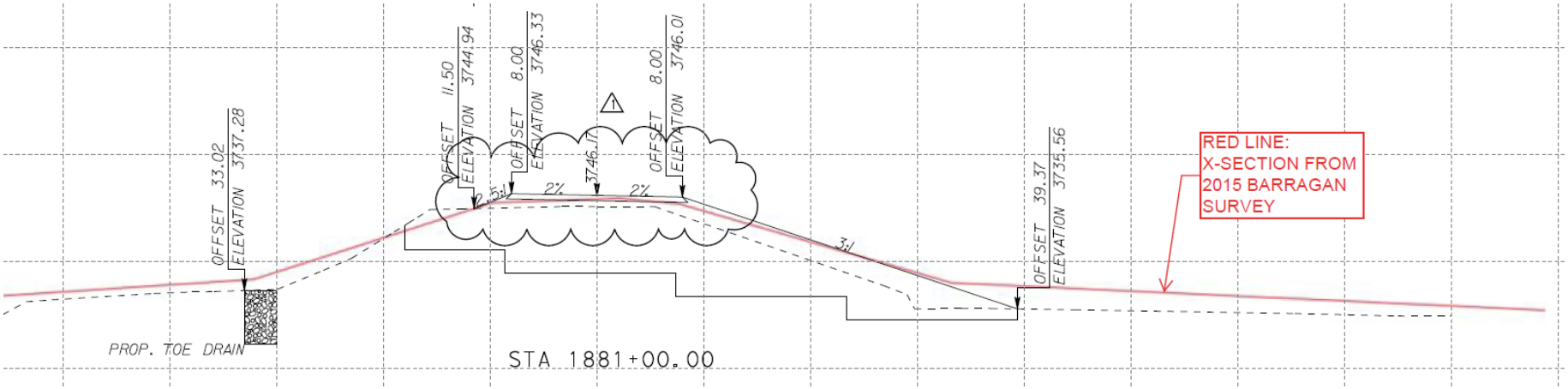

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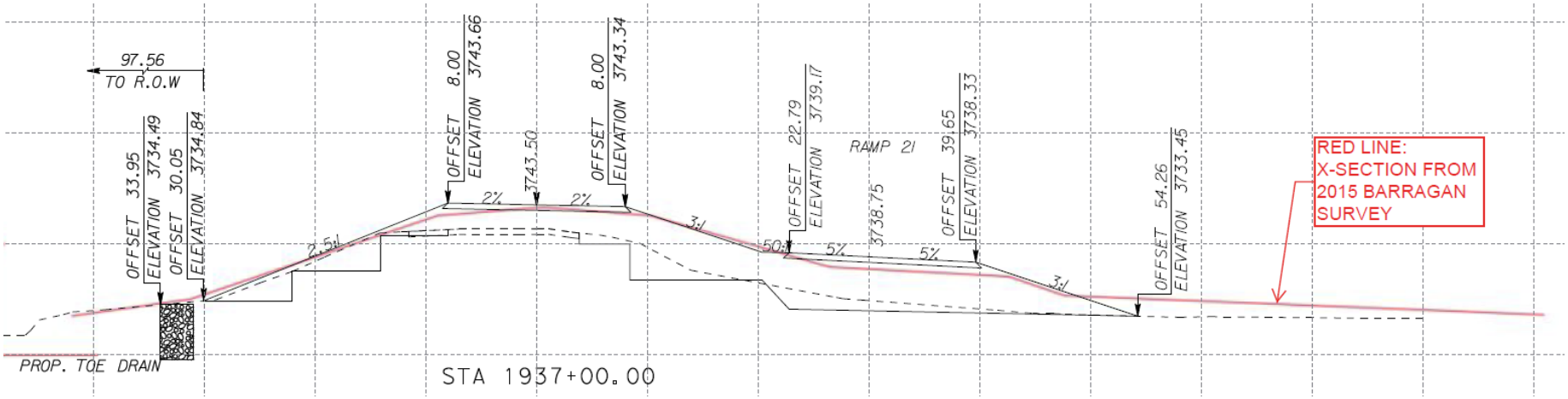

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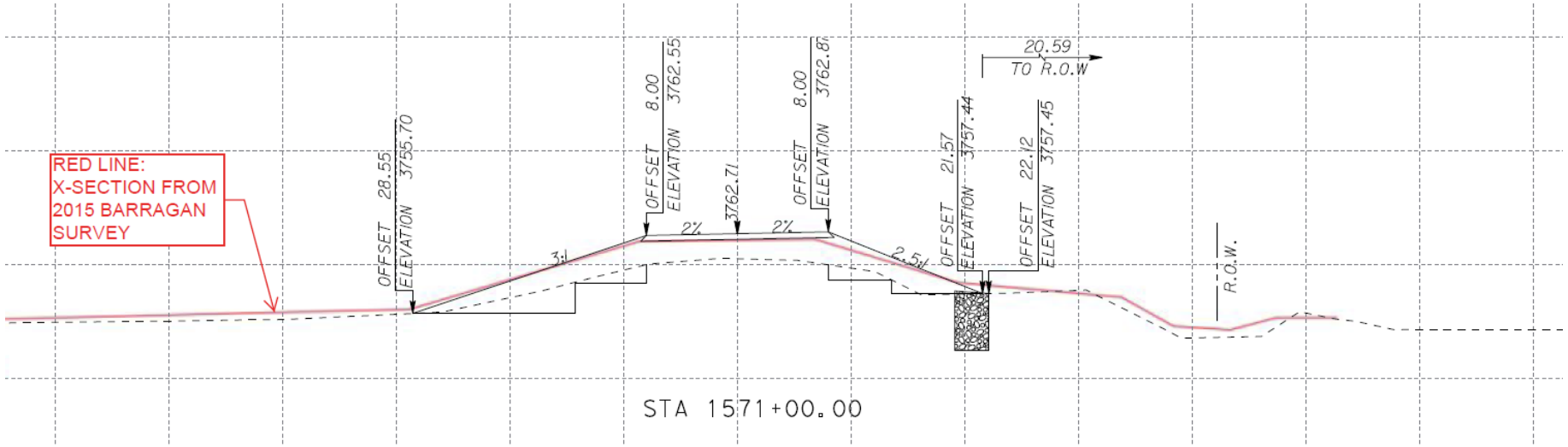

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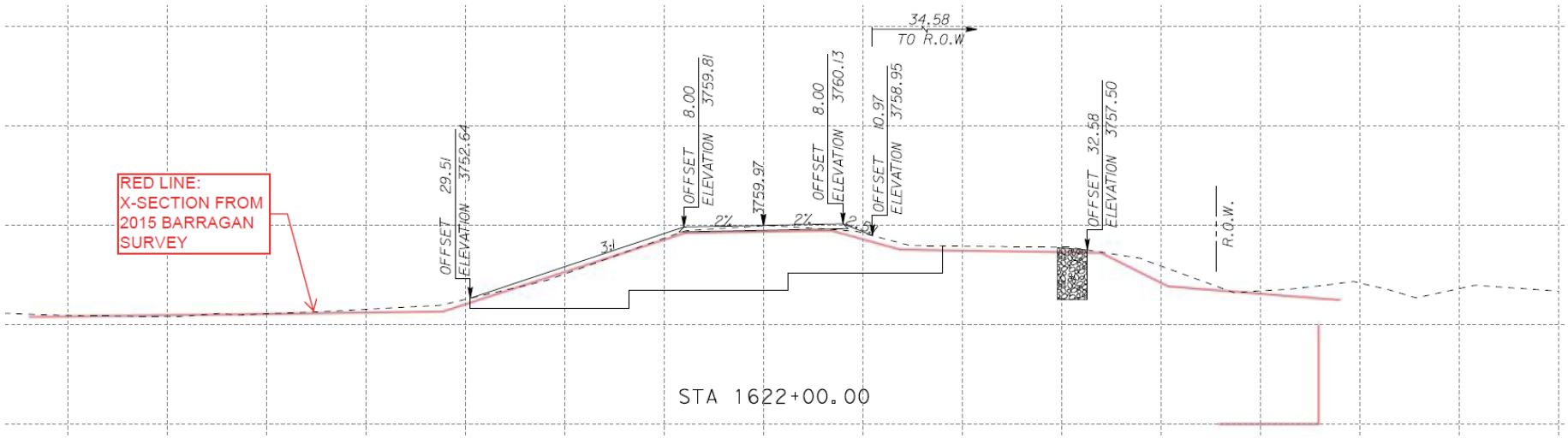

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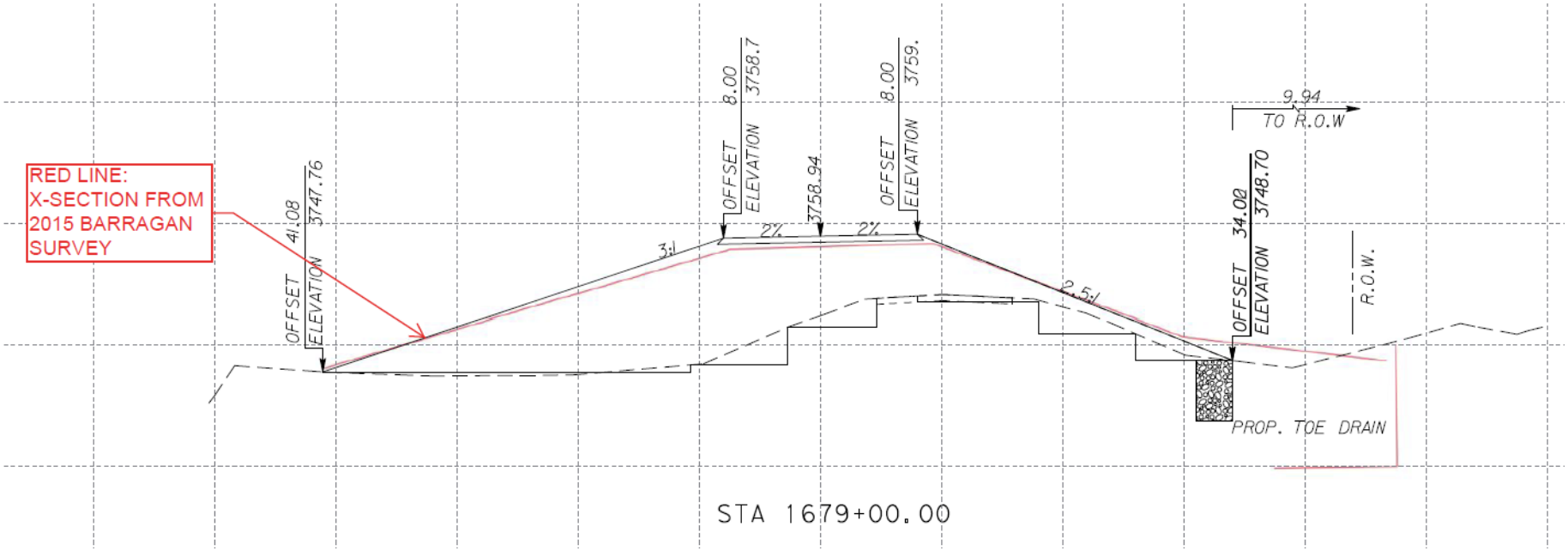
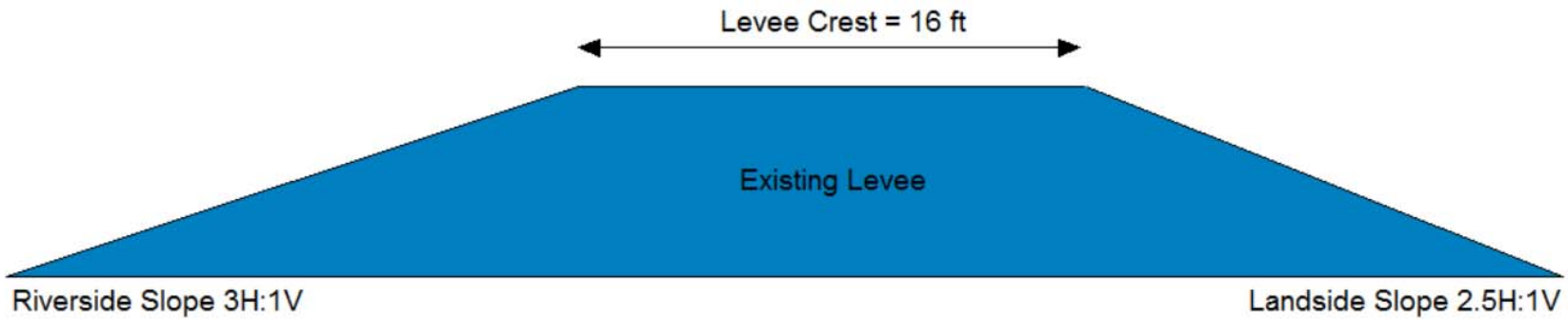


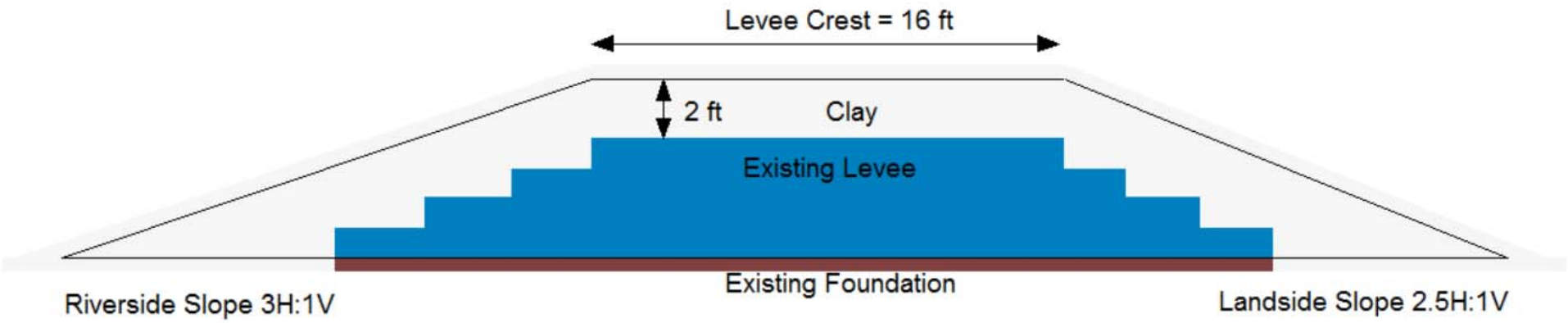
ATTACHMENT B

Design Options Construction Sequence

- Clay Blanket
- Lime Treatment
- Sand-Riprap Blanket
- Lime Treatment and Sand-Riprap Combination

Option #4 - Clay Blanket





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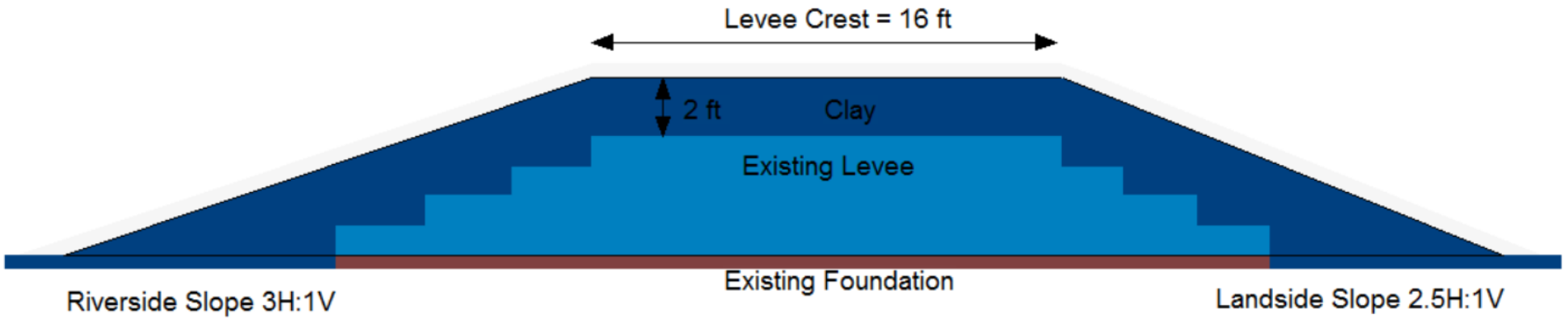
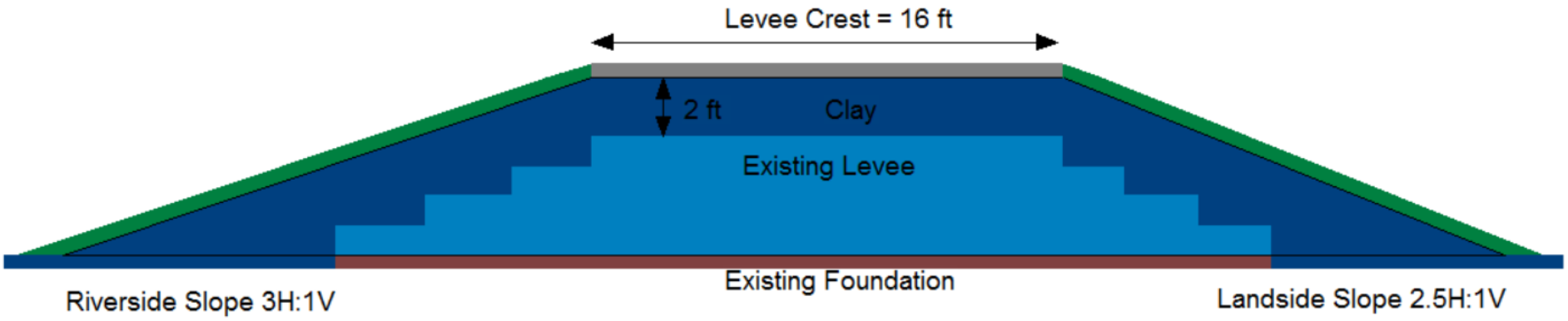
Option #4 - Clay Blanket





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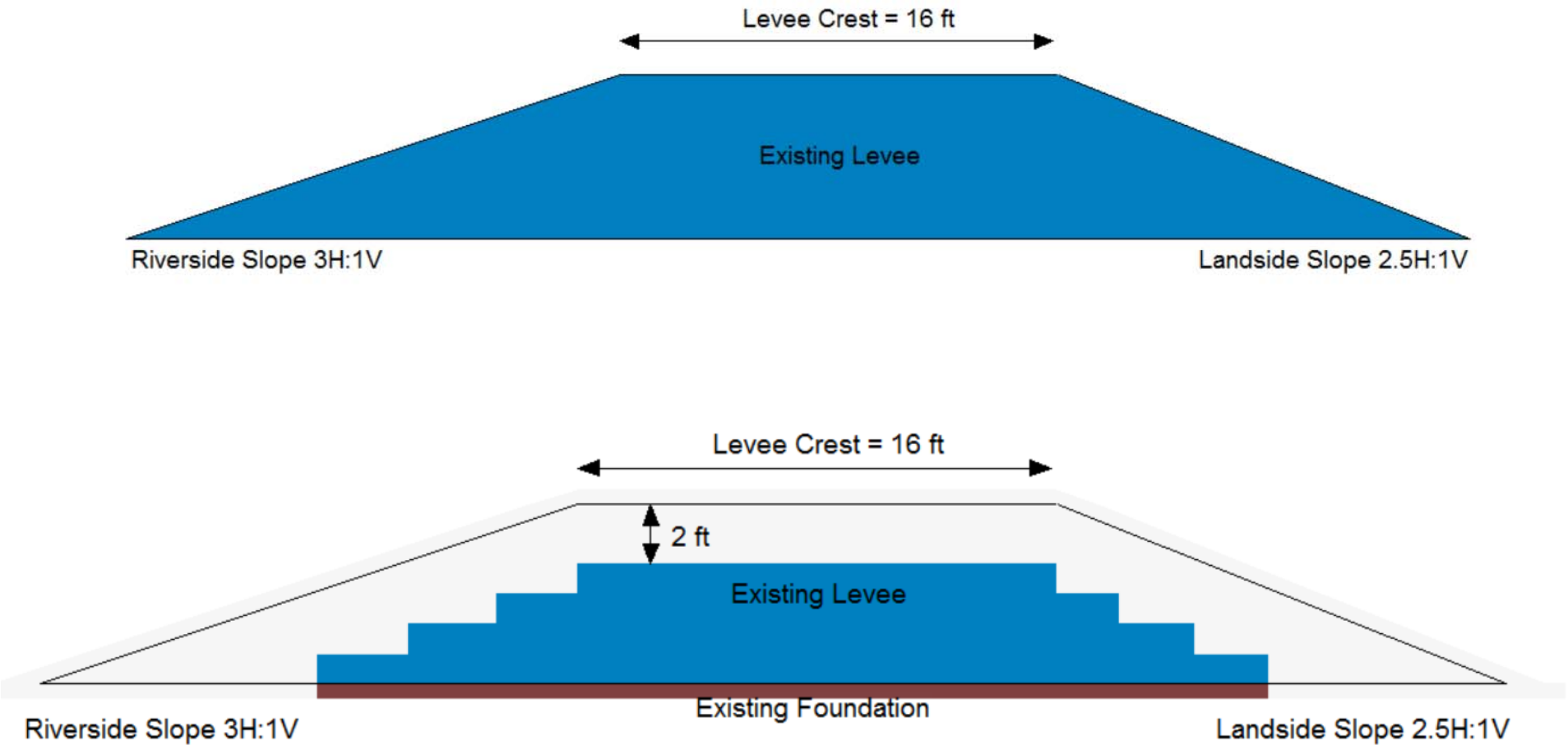
Option #7 - Lime Treatment



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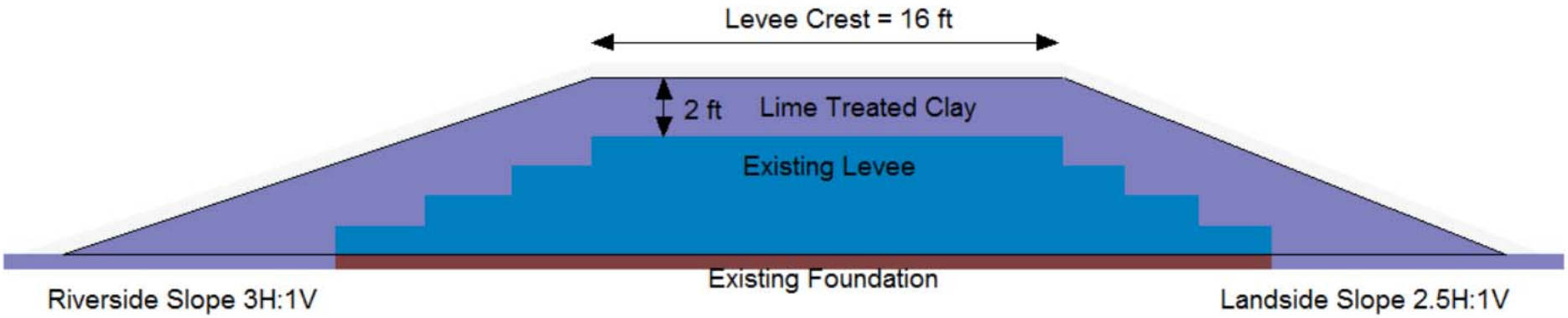
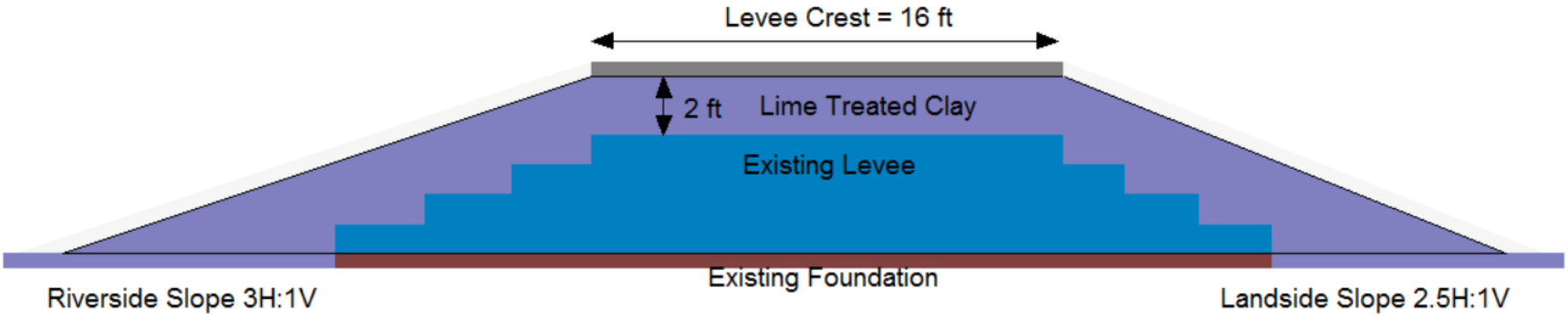
Option #7 - Lime Treatment





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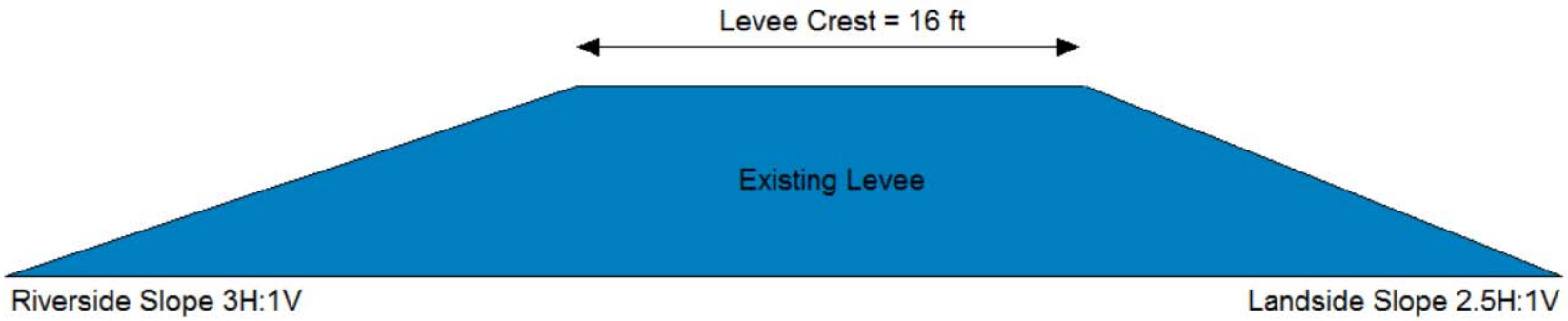
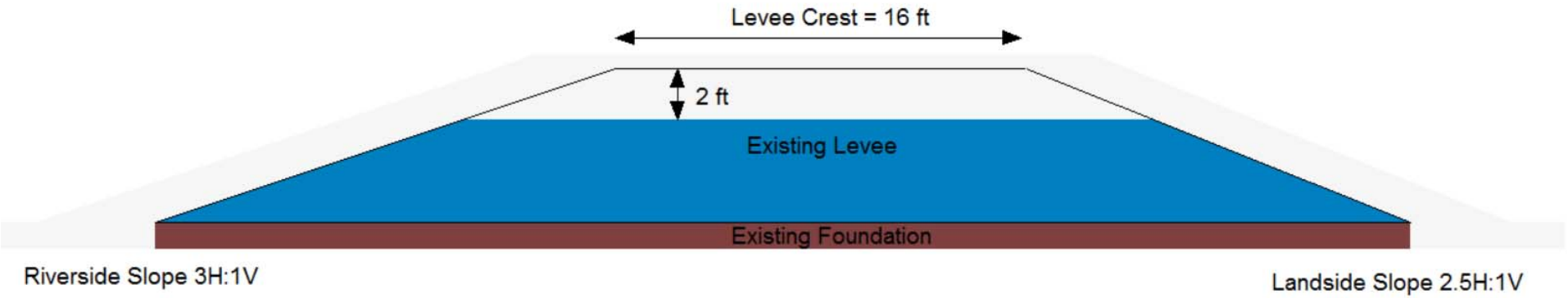
Option #11 – Sand-Riprap Blanket





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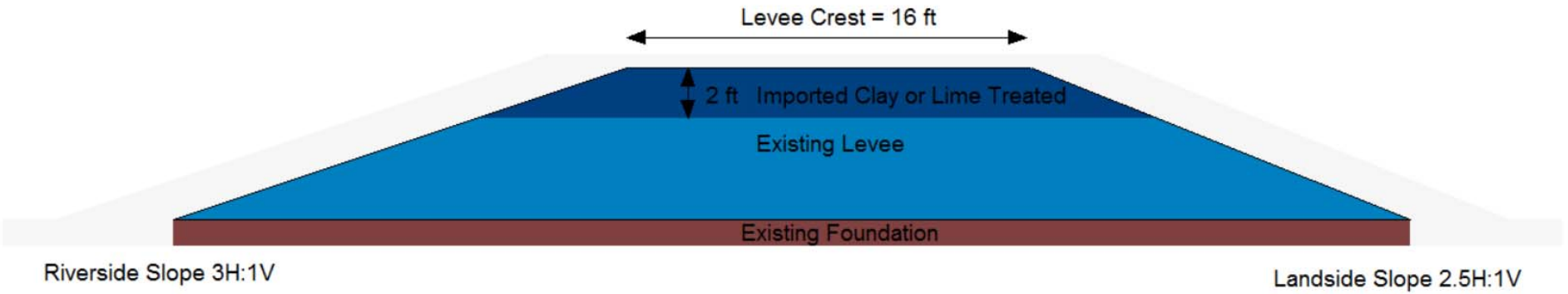
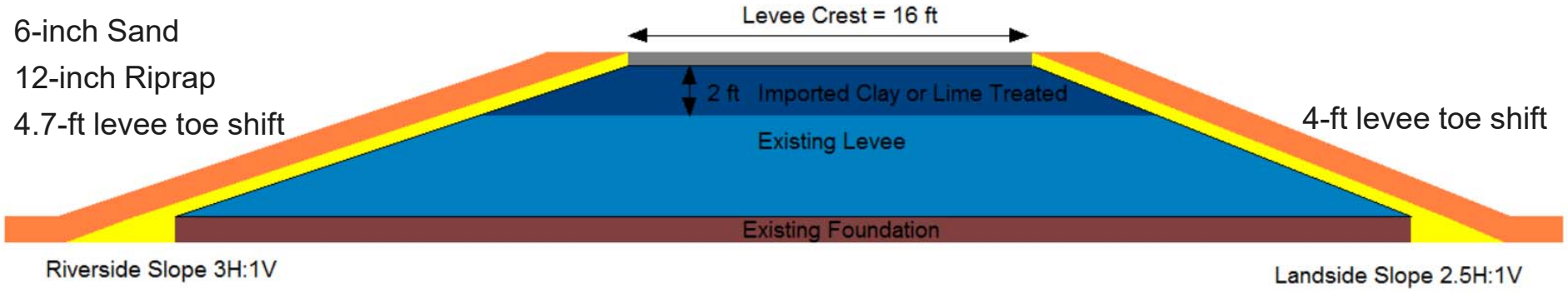
Option #11 – Sand-Riprap Blanket



6-inch Sand
12-inch Riprap
4.7-ft levee toe shift



4-ft levee toe shift

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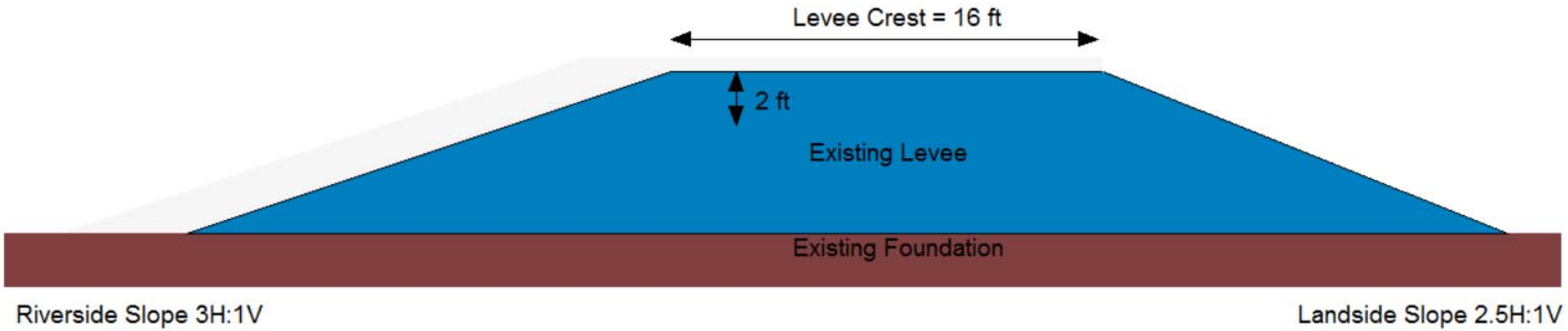
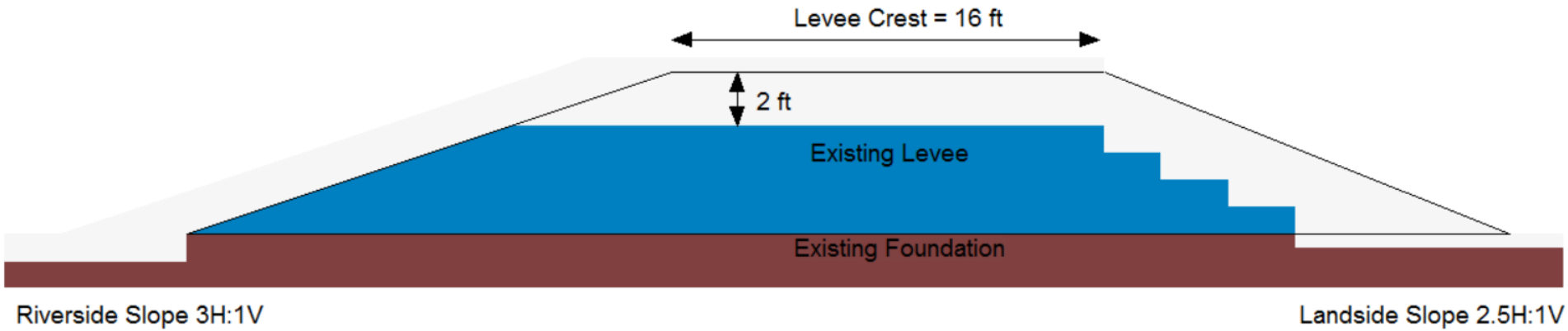
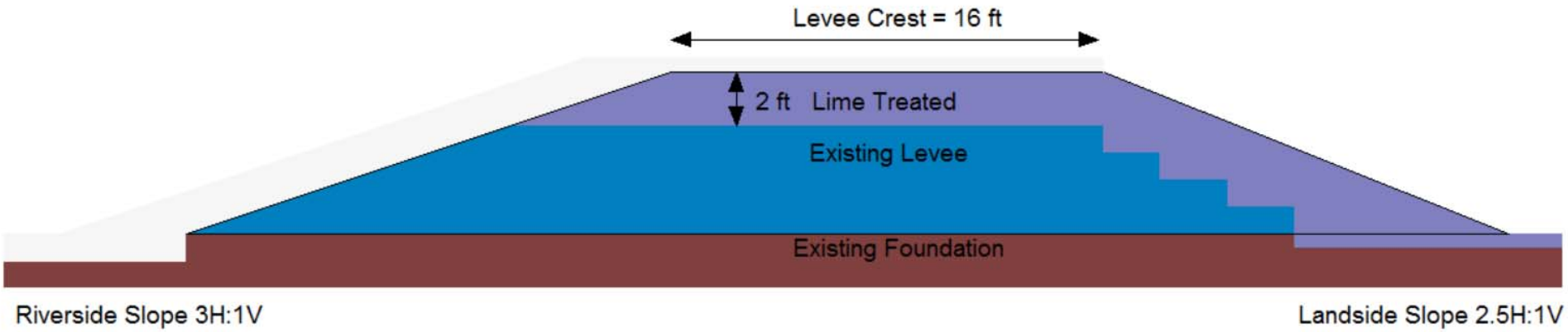
Blended Option – Lime & Sand-Riprap

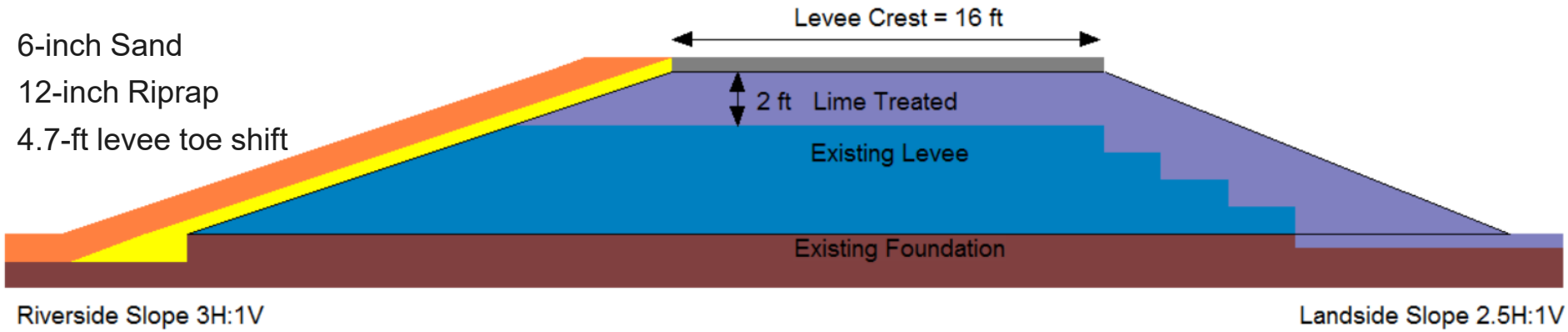




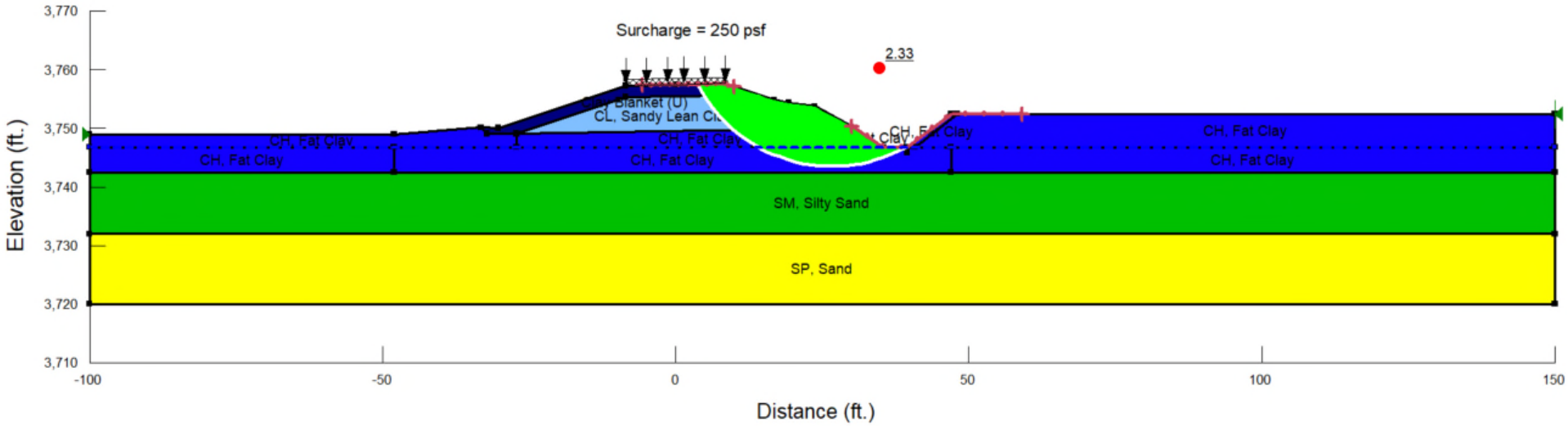
Blended Option – Lime & Sand-Riprap



6-inch Sand
12-inch Riprap
4.7-ft levee toe shift



Appendix B.2: Seepage and Slope Stability



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
Clay Blanket	127	29	50	0	1,000
Sandy Lean Clay (levee) [1]	127	29	50	18	100
Sandy Lean Clay [1]	127	28	50	11	175
Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

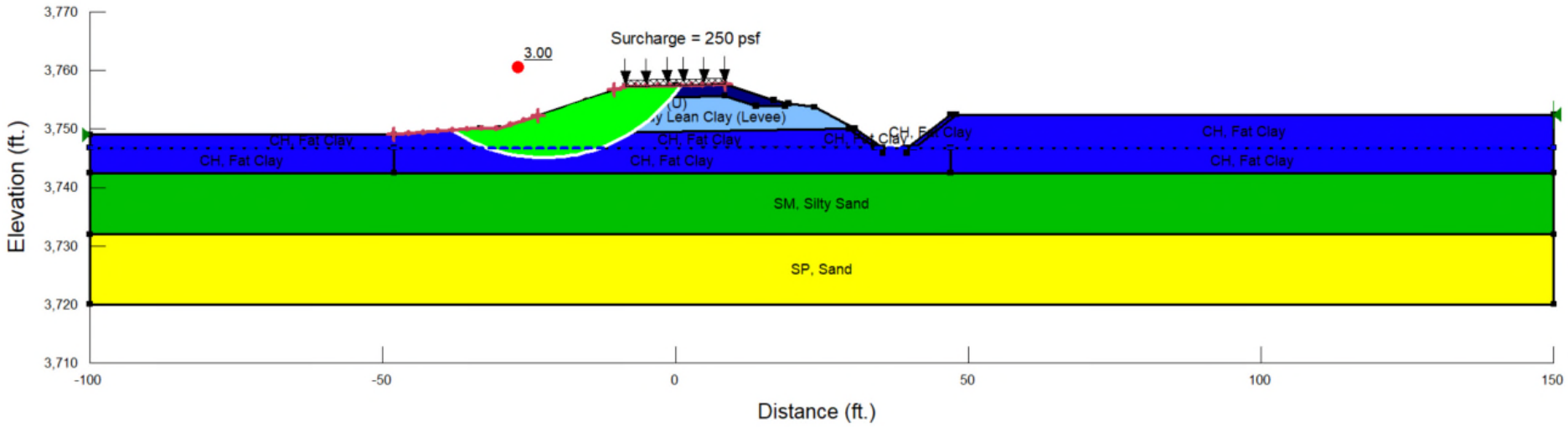
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1672+00 EAST LEVEE
END OF CONSTRUCTION – LANDSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
1/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
Clay Blanket	127	29	50	0	1,000
Sandy Lean Clay (levee) [1]	127	29	50	18	100
Sandy Lean Clay [1]	127	28	50	11	175
Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

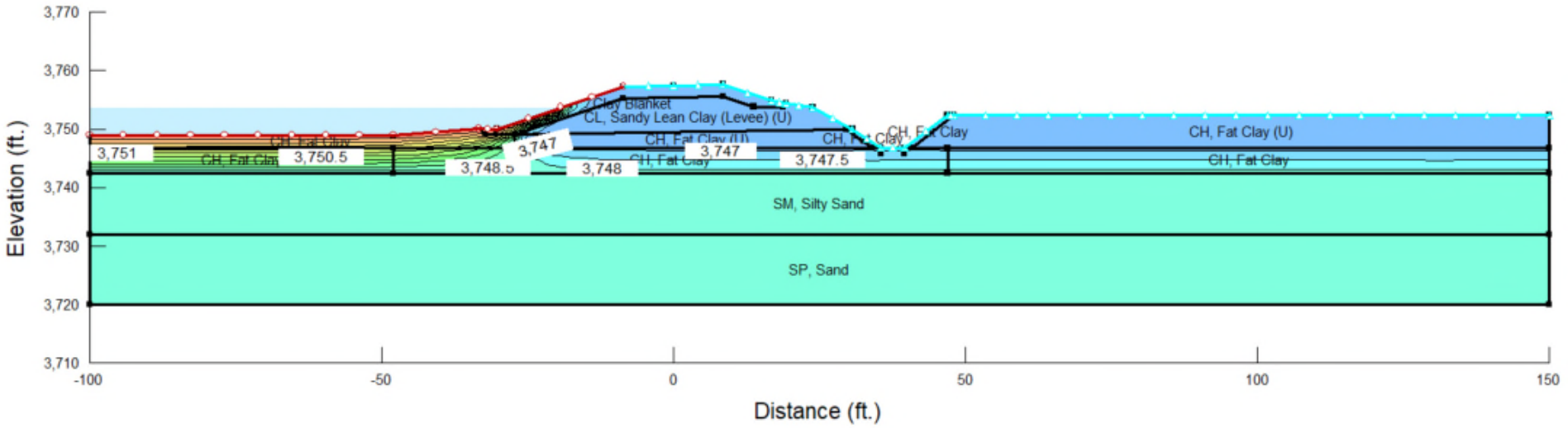
Notes: [1] Soil properties provided by USIBWC.

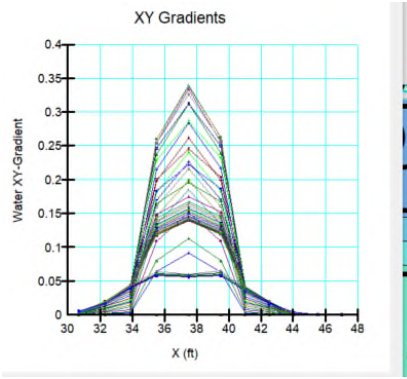
SUNLAND PARK LEVEE REPAIR

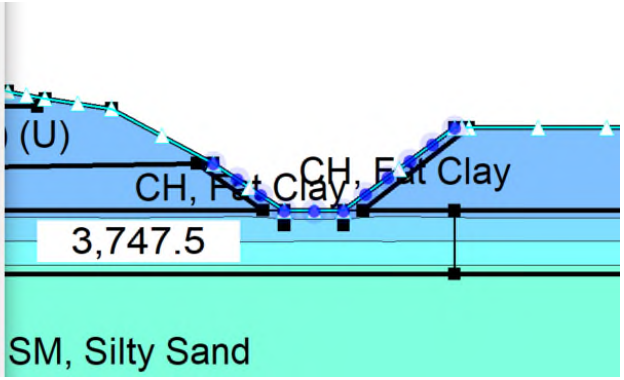
STA. 1672+00 EAST LEVEE
END OF CONSTRUCTION – RIVERSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
2/6







Material	Hydraulic Conductivity (fps)	Vertical to Horizontal Hydraulic Conductivity Ratio
Clay Blanket	4.9E-08	0.33
Sandy Lean Clay (levee) [1]	4.9E-08	0.33
Sandy Lean Clay [1]	3.0E-08	0.10
Fat Clay [1]	3.0E-08	0.10
Silty Sand [1]	5.0E-05	0.33
Poorly graded Sand (w. silt) [1]	3.0E-04	0.33

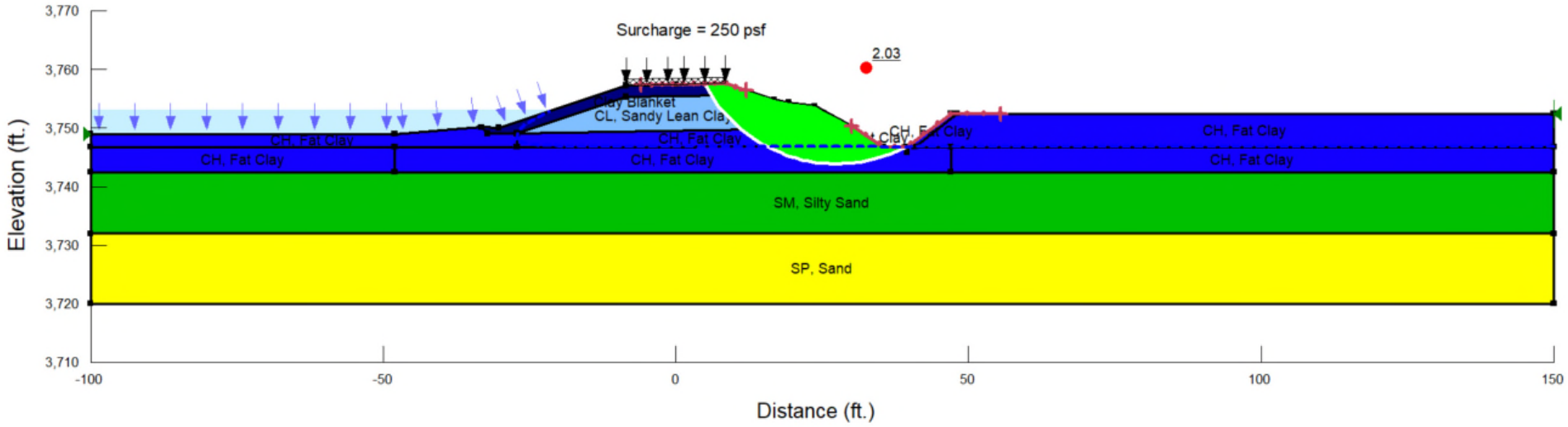
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1672+00 EAST LEVEE
SEEPAGE 100-YEAR FLOOD – EL. 3753.6
TOTAL HEAD & EXIT GRADIENT

ARCADIS Design & Consultancy for natural and built assets

FIGURE
3/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
Clay Blanket	127	29	50	0	1,000
Sandy Lean Clay (levee) [1]	127	29	50	18	100
Sandy Lean Clay [1]	127	28	50	11	175
Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

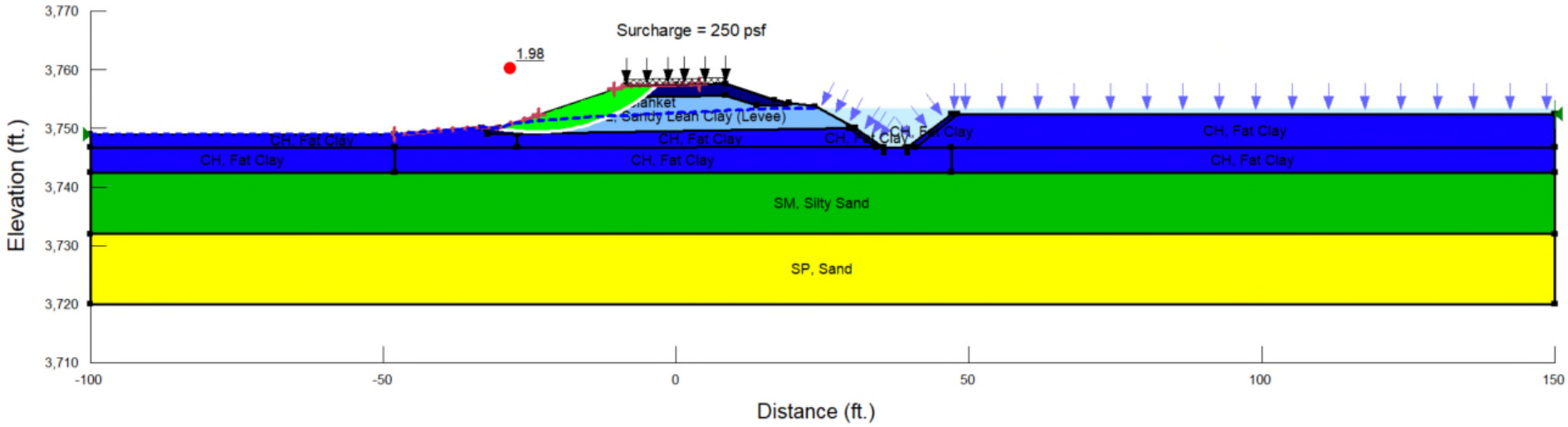
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1672+00 EAST LEVEE
100-YEAR FLOOD - EL. 3753.6
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
4/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
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Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

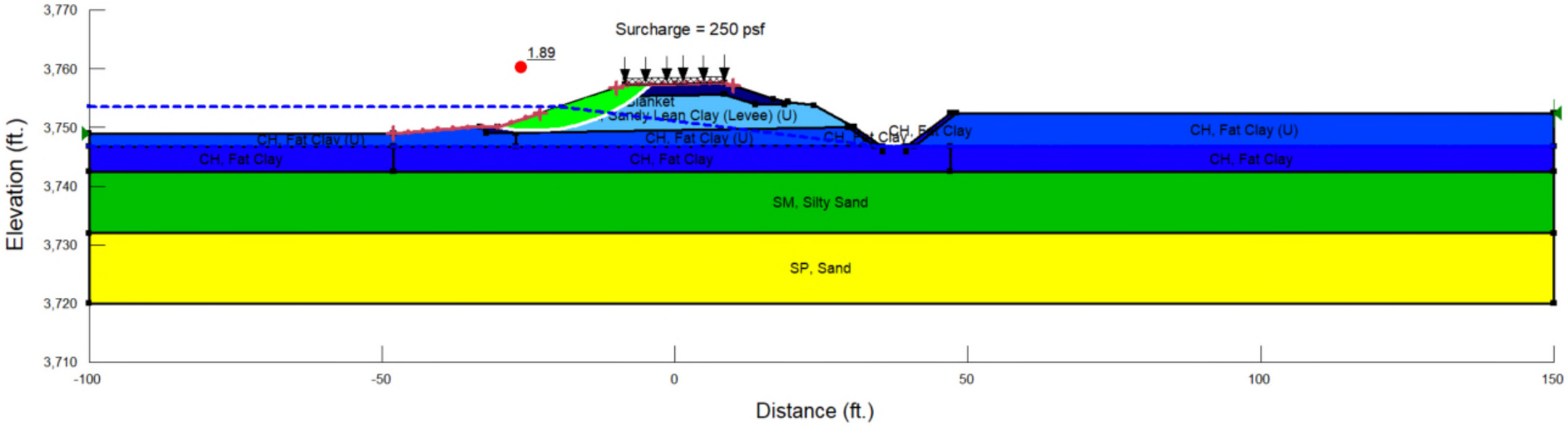
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1672+00 EAST LEVEE
1 FT OF WATER ON LANDSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
5/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
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Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

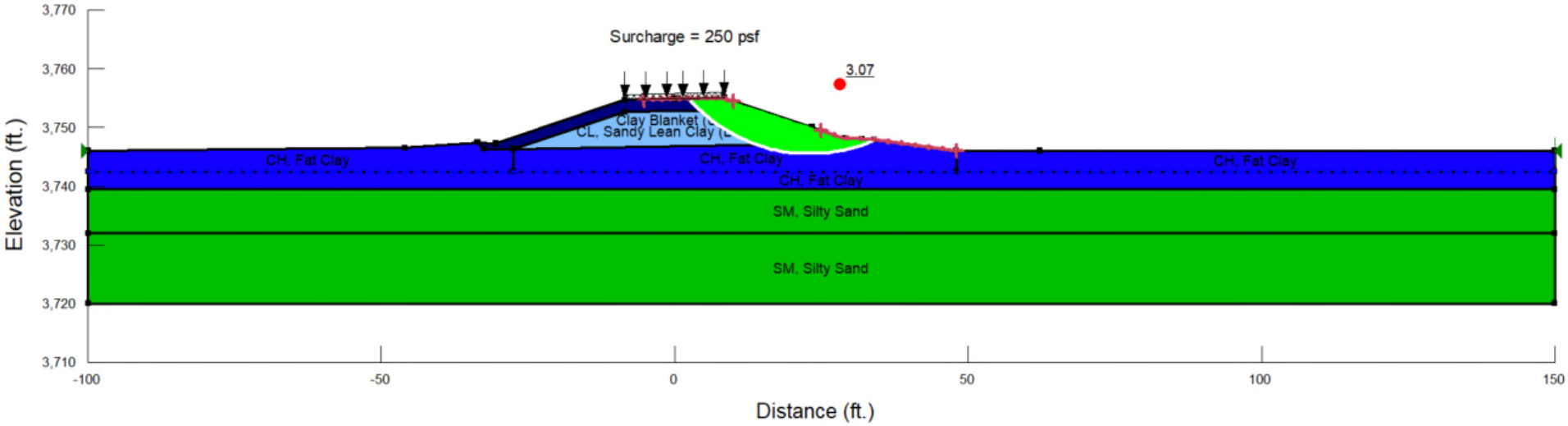
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1672+00 EAST LEVEE
RAPID DRAWDOWN
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
6/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
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Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

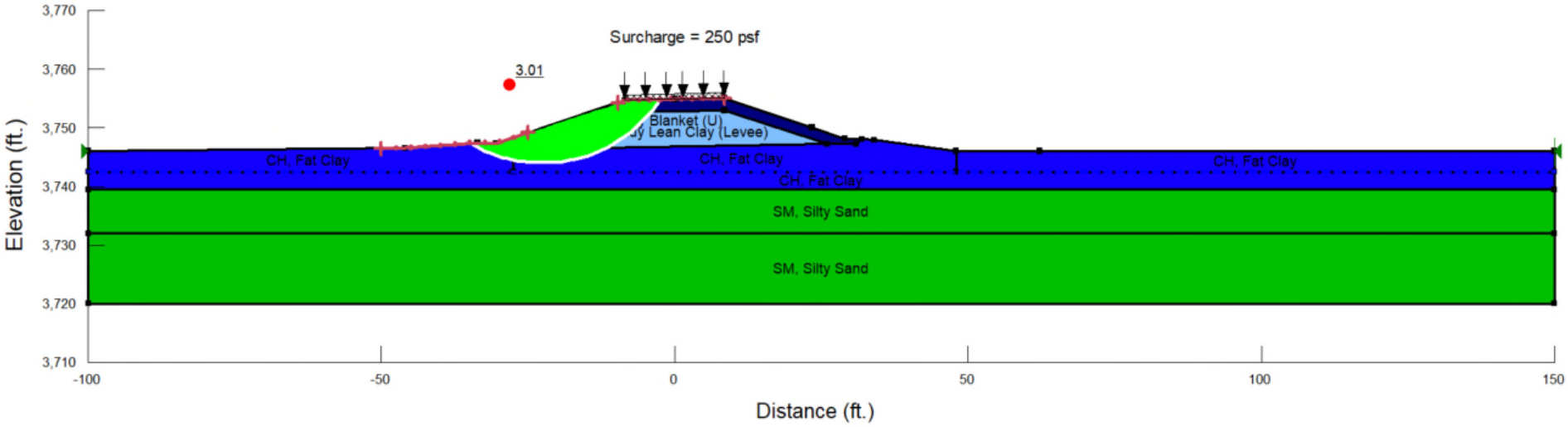
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1722+00 EAST LEVEE
END OF CONSTRUCTION – LANDSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
1/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
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Poorly graded Sand (w. silt) [1]	124	34	0	--	--

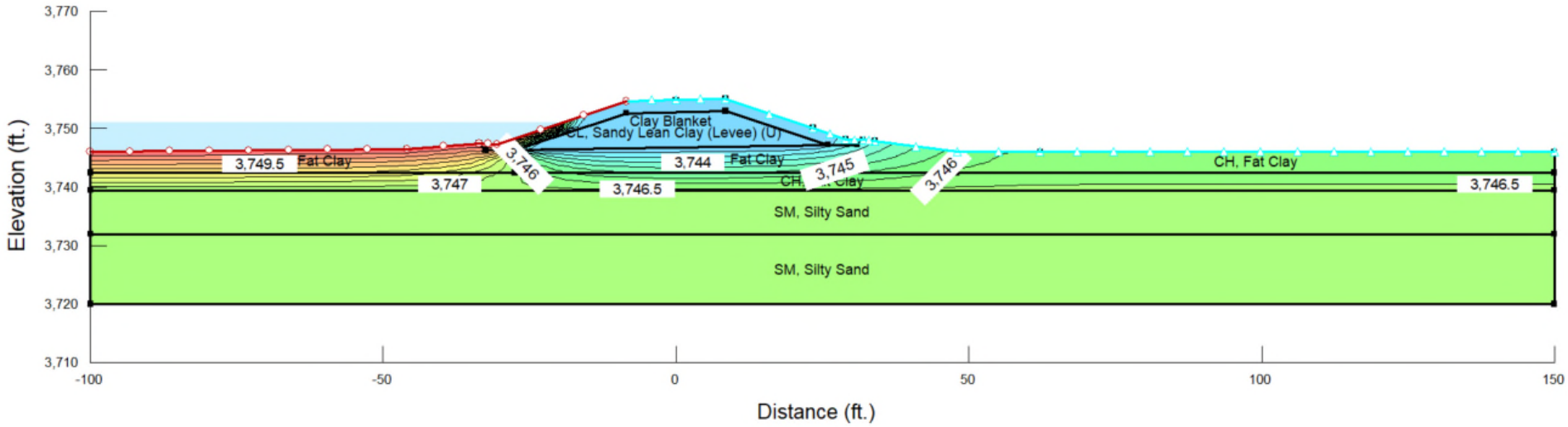
Notes: [1] Soil properties provided by USIBWC.

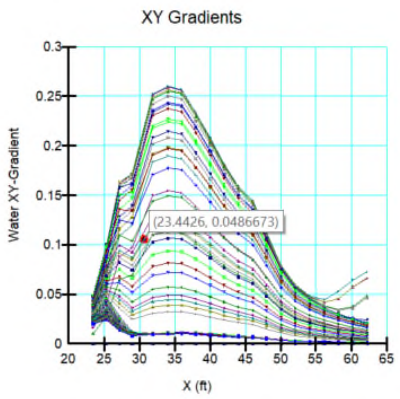
SUNLAND PARK LEVEE REPAIR

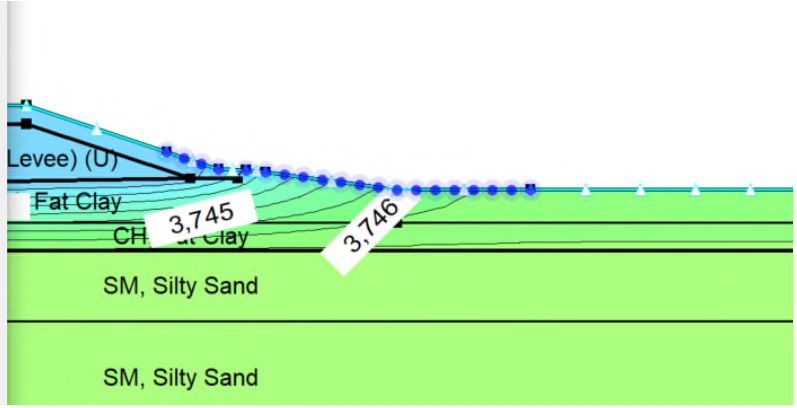
STA. 1722+00 EAST LEVEE
END OF CONSTRUCTION – RIVERSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
2/6







Material	Hydraulic Conductivity (fps)	Vertical to Horizontal Hydraulic Conductivity Ratio
Clay Blanket	4.9E-08	0.33
Sandy Lean Clay (levee) [1]	4.9E-08	0.33
Sandy Lean Clay [1]	3.0E-08	0.10
Fat Clay [1]	3.0E-08	0.10
Silty Sand [1]	5.0E-05	0.33
Poorly graded Sand (w. silt) [1]	3.0E-04	0.33

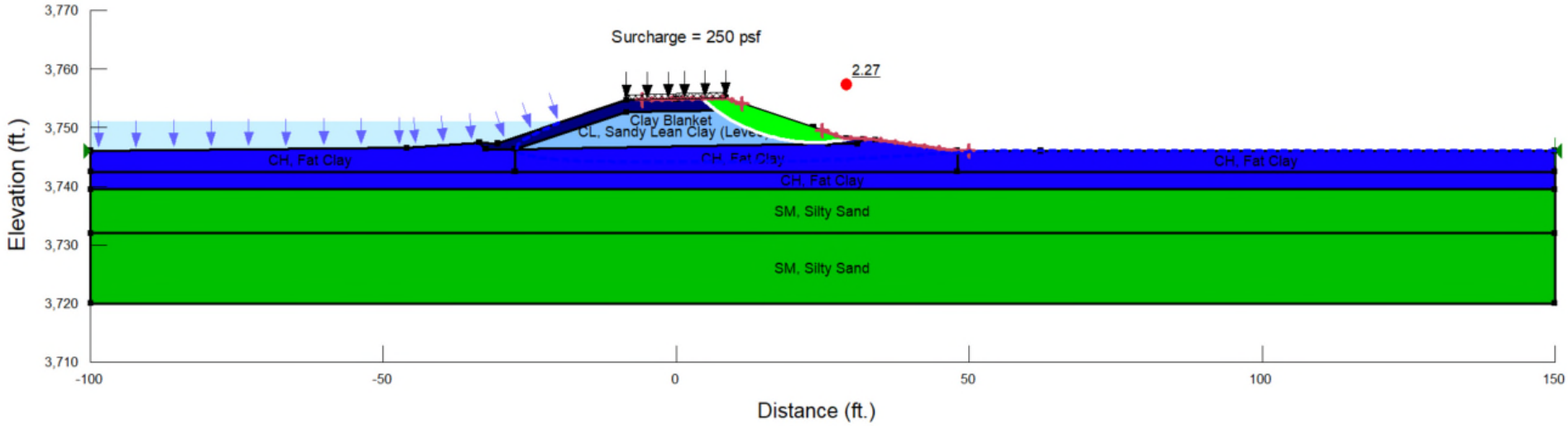
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1722+00 EAST LEVEE
SEEPAGE 100-YEAR FLOOD – EL. 3751.0
TOTAL HEAD & EXIT GRADIENT

ARCADIS Design & Consultancy for natural and built assets

FIGURE
3/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
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Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

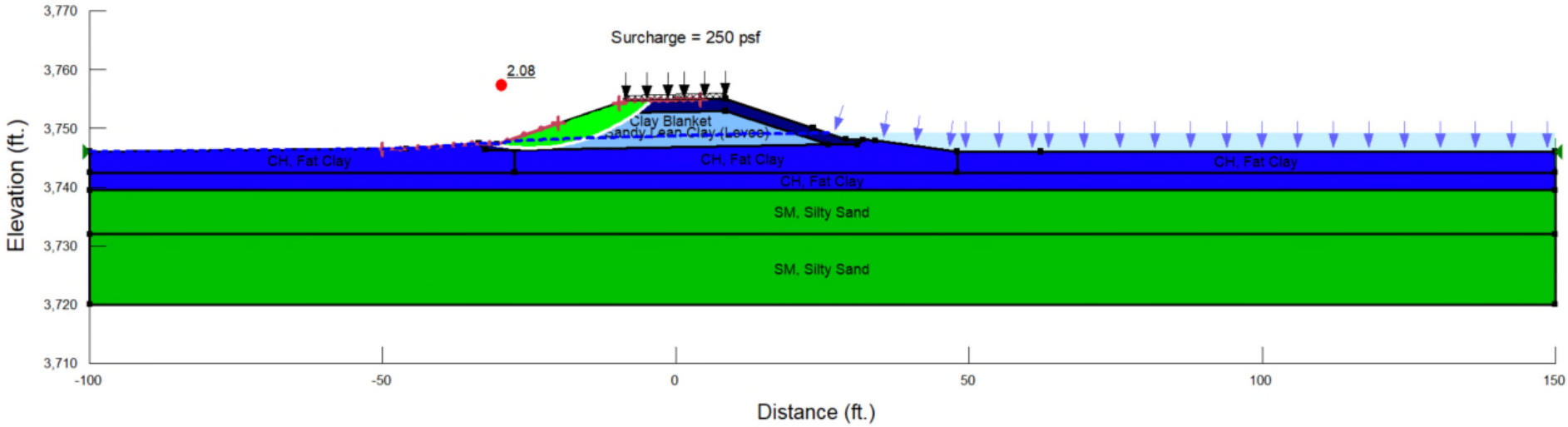
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1722+00 EAST LEVEE
100-YEAR FLOOD - EL. 3751.0
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
4/6



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Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

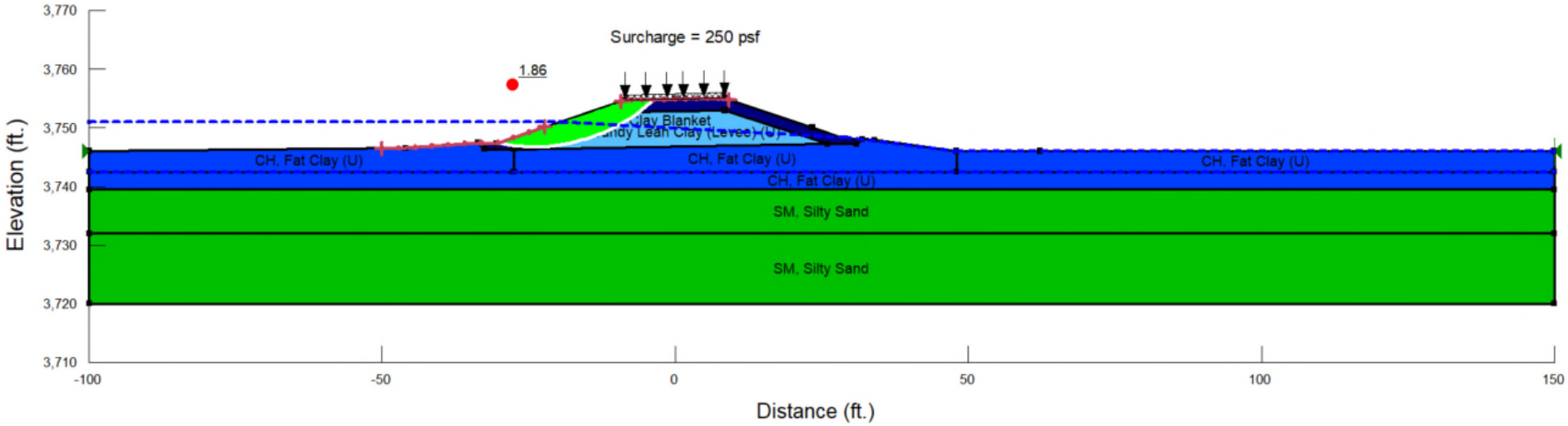
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1722+00 EAST LEVEE
1 FT OF WATER ON LANDSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
5/6



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Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

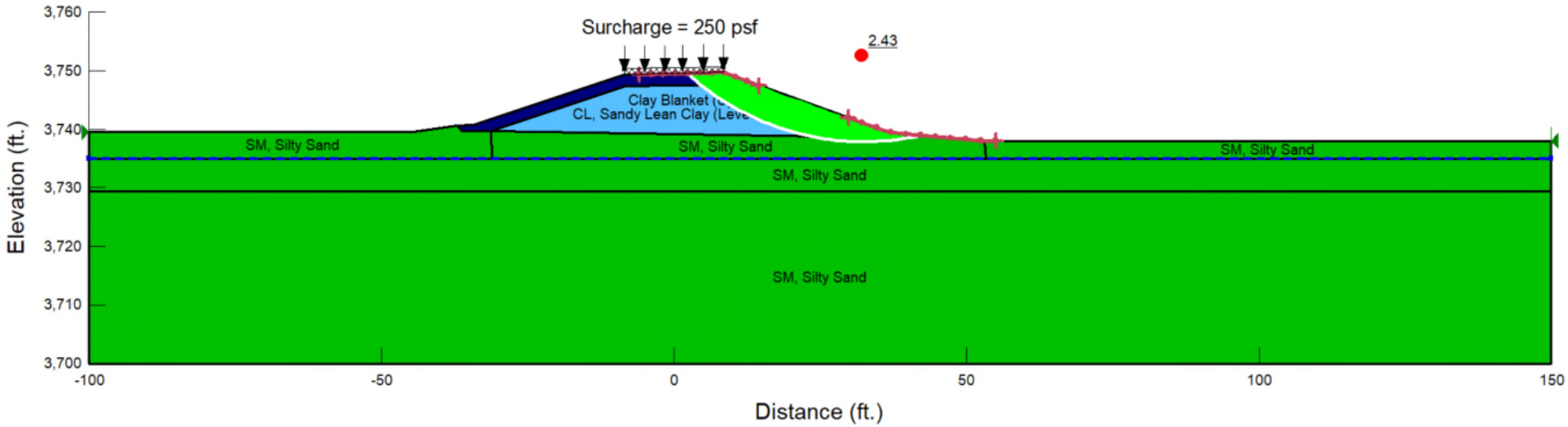
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1722+00 EAST LEVEE
RAPID DRAWDOWN
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
6/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
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Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

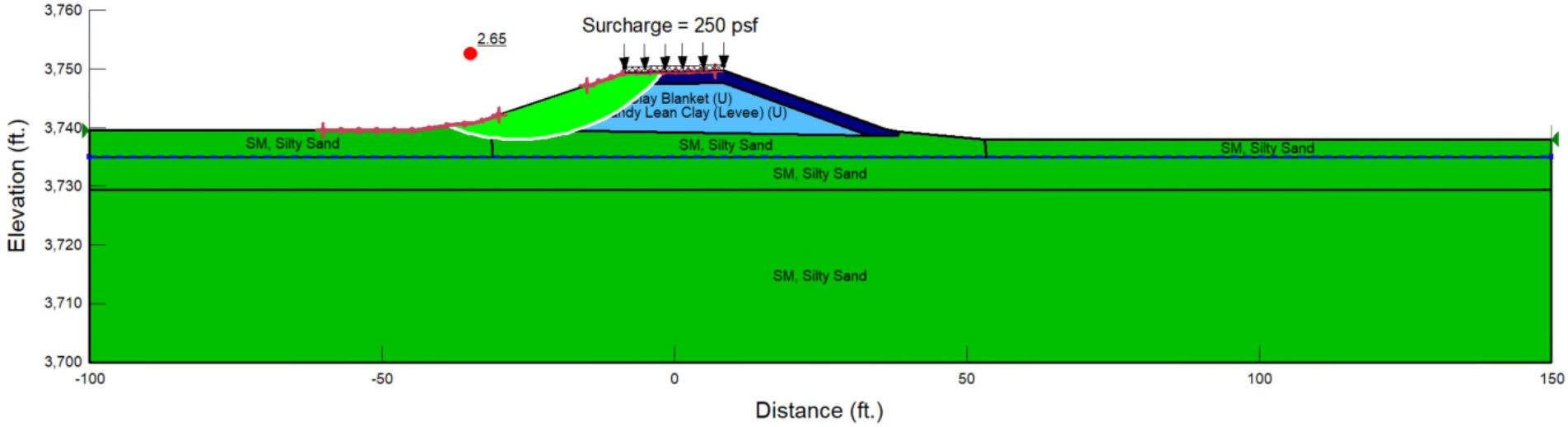
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1826+00 EAST LEVEE
END OF CONSTRUCTION – LANDSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
1/6



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Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

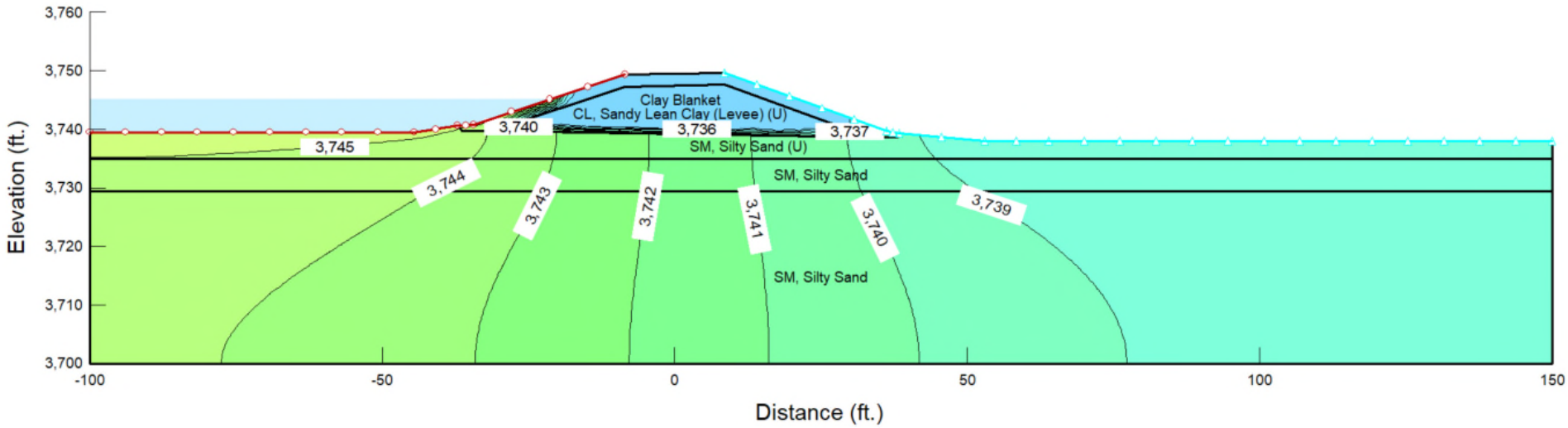
Notes: [1] Soil properties provided by USIBWC.

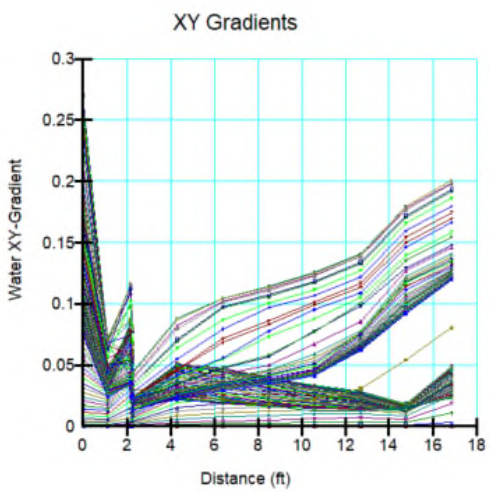
SUNLAND PARK LEVEE REPAIR

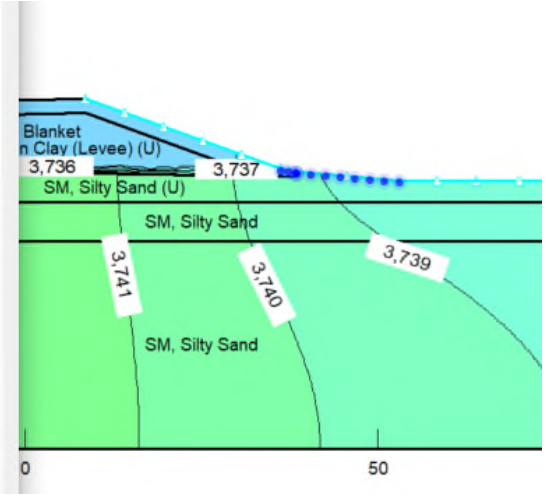
STA. 1826+00 EAST LEVEE
END OF CONSTRUCTION – RIVERSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
2/6







Material	Hydraulic Conductivity (fps)	Vertical to Horizontal Hydraulic Conductivity Ratio
Clay Blanket	4.9E-08	0.33
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Silty Sand [1]	5.0E-05	0.33
Poorly graded Sand (w. silt) [1]	3.0E-04	0.33

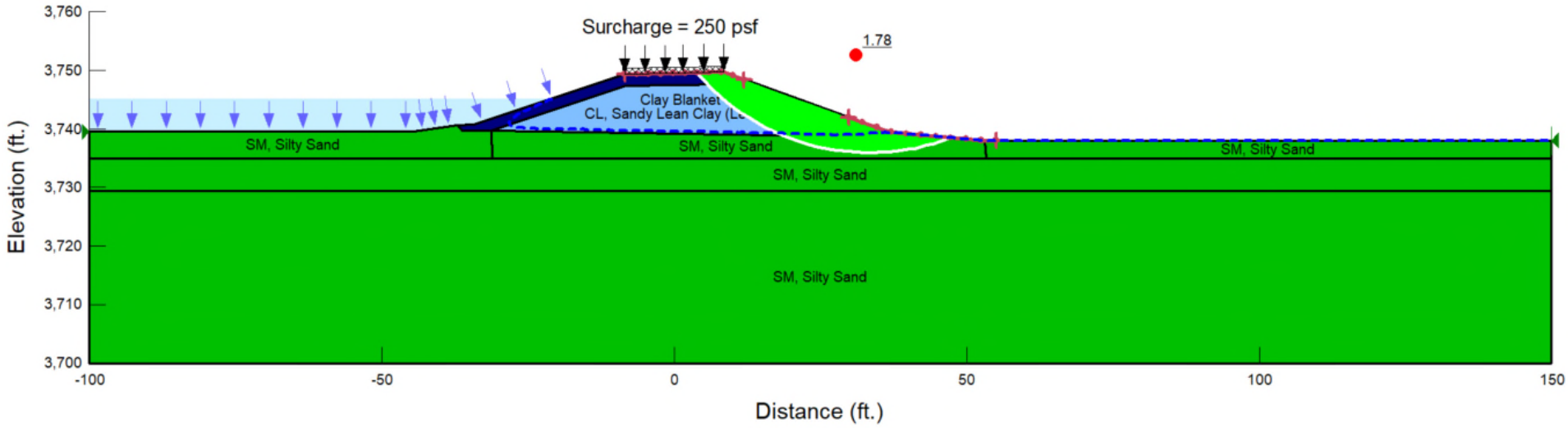
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1826+00 EAST LEVEE
SEEPAGE 100-YEAR FLOOD – EL. 3745.2
TOTAL HEAD & EXIT GRADIENT

ARCADIS Design & Consultancy for natural and built assets

FIGURE
3/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
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Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

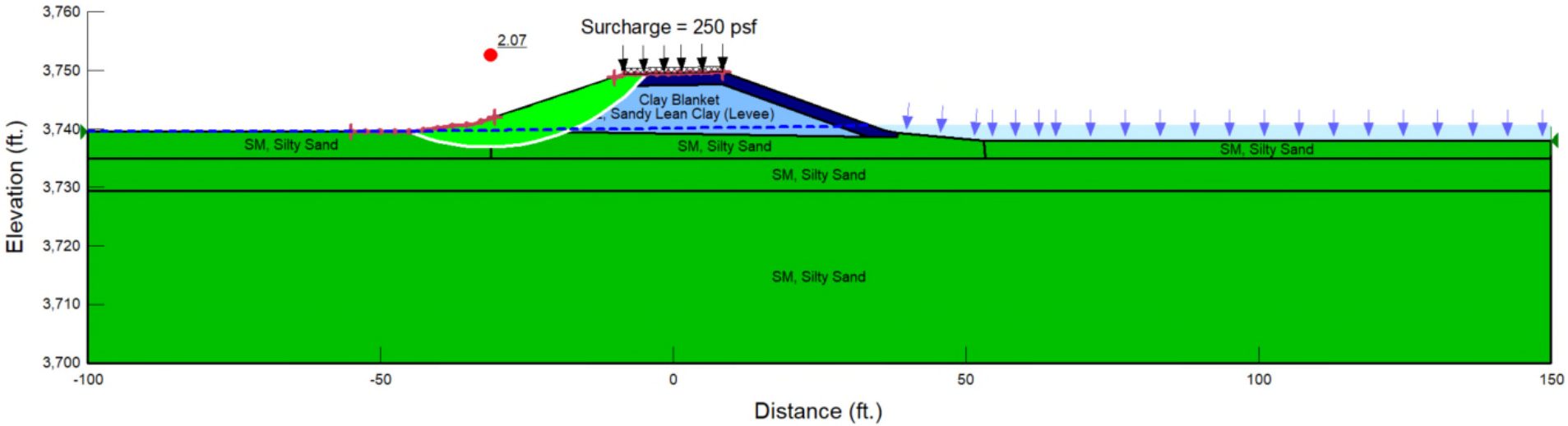
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1826+00 EAST LEVEE
100-YEAR FLOOD - EL. 3745.2
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
4/6



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Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

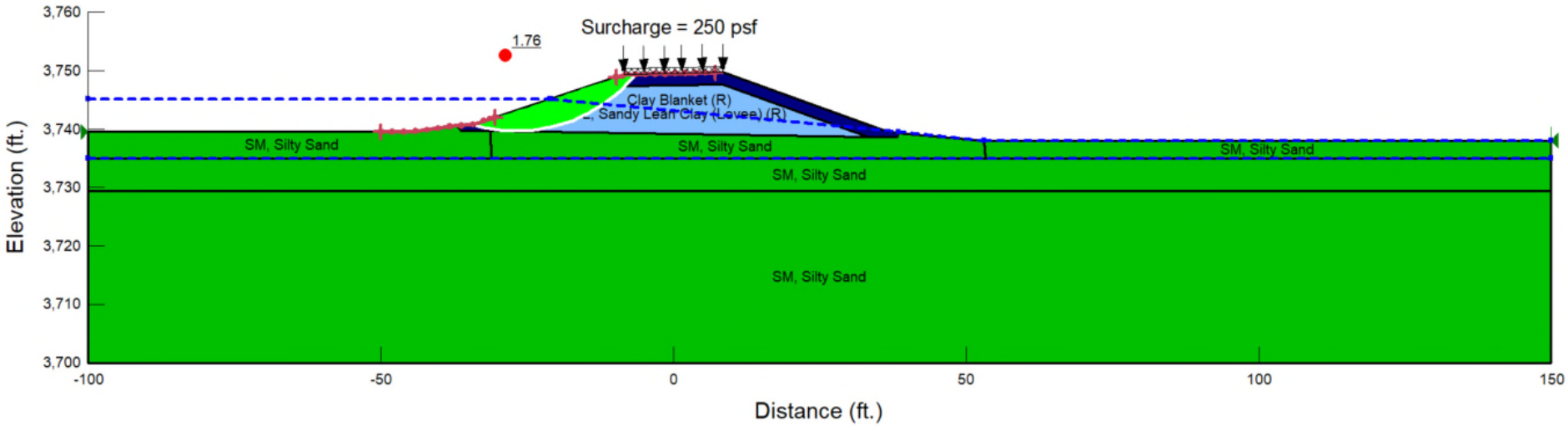
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1826+00 EAST LEVEE
1 FT OF WATER ON LANDSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
5/6



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Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

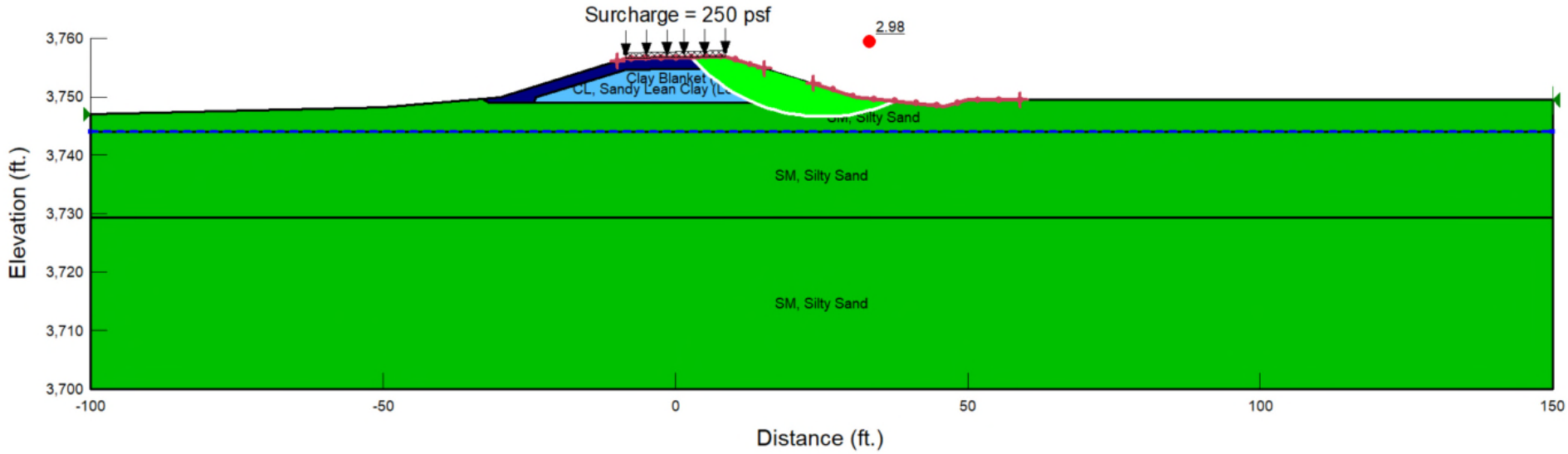
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1826+00 EAST LEVEE
RAPID DRAWDOWN
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
6/6



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Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

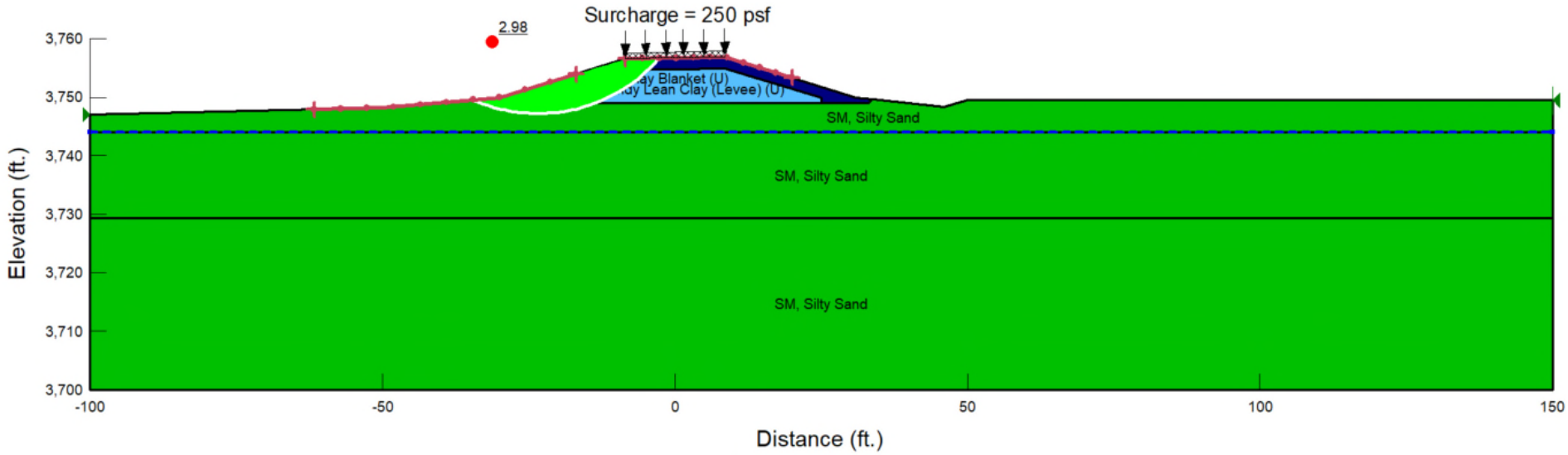
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1675+00 WEST LEVEE
END OF CONSTRUCTION – LANDSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
1/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
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Poorly graded Sand (w. silt) [1]	124	34	0	--	--

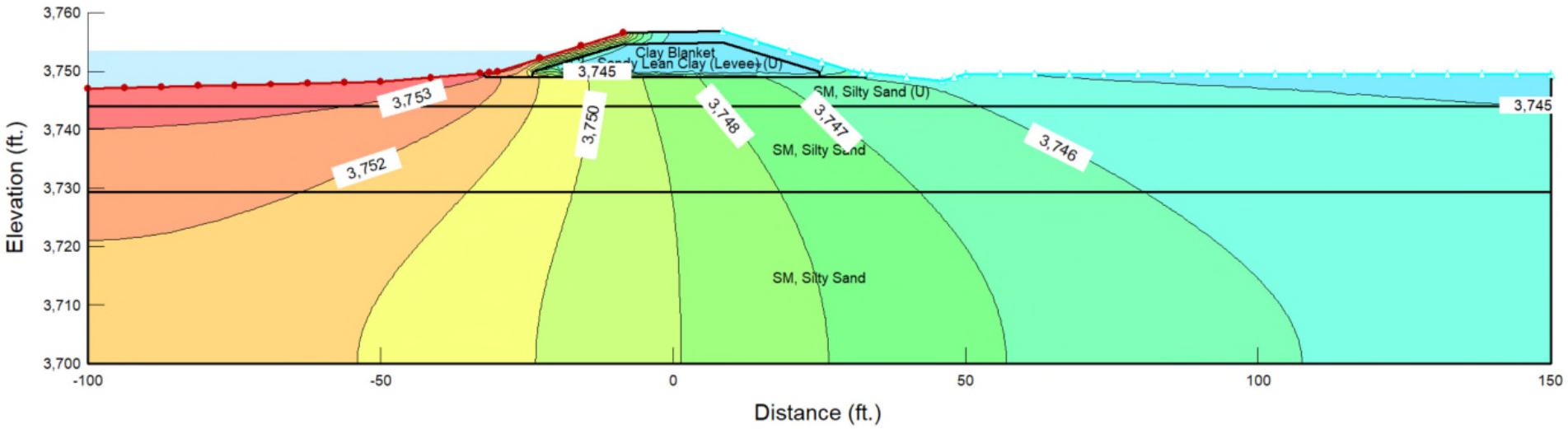
Notes: [1] Soil properties provided by USIBWC.

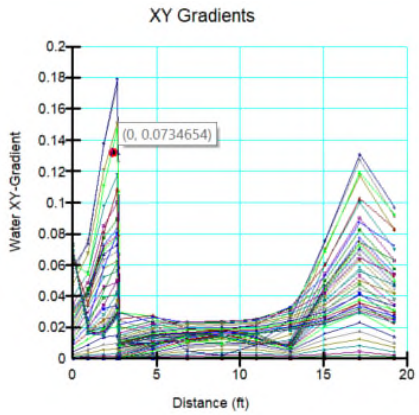
SUNLAND PARK LEVEE REPAIR

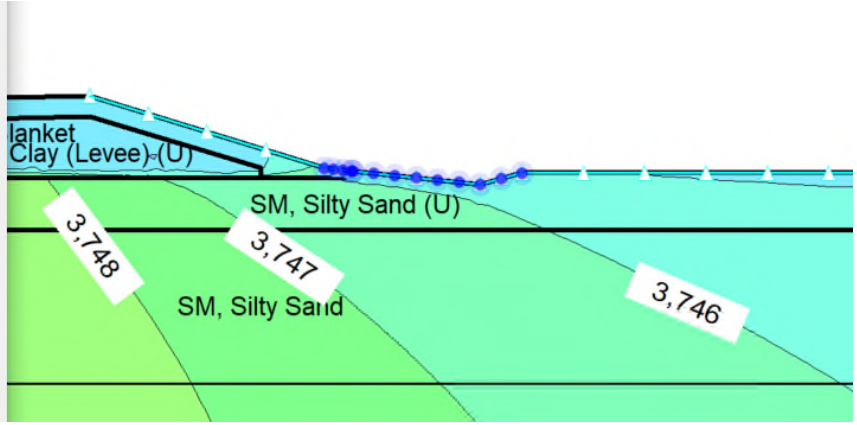
STA. 1675+00 WEST LEVEE
END OF CONSTRUCTION – RIVERSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
2/6







Material	Hydraulic Conductivity (fps)	Vertical to Horizontal Hydraulic Conductivity Ratio
Clay Blanket	4.9E-08	0.33
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Silty Sand [1]	5.0E-05	0.33
Poorly graded Sand (w. silt) [1]	3.0E-04	0.33

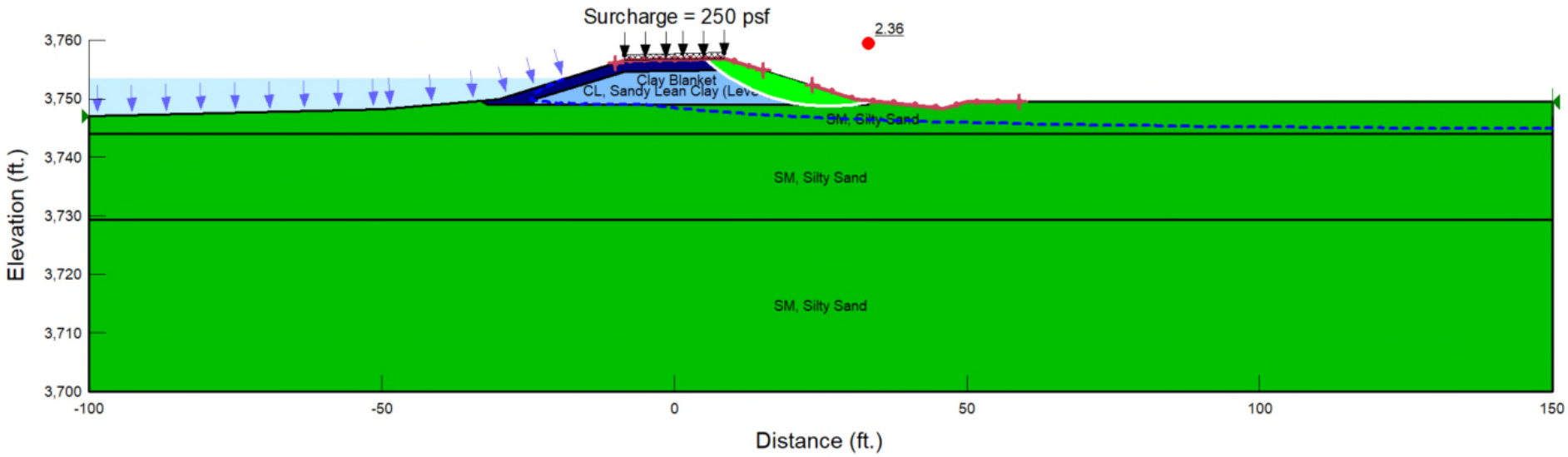
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1675+00 WEST LEVEE
SEEPAGE 100-YEAR FLOOD – EL. 3753.5
TOTAL HEAD & EXIT GRADIENT

ARCADIS Design & Consultancy for natural and built assets

FIGURE 3/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
Clay Blanket	127	29	50	0	1,000
Sandy Lean Clay (levee) [1]	127	29	50	18	100
Sandy Lean Clay [1]	127	28	50	11	175
Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

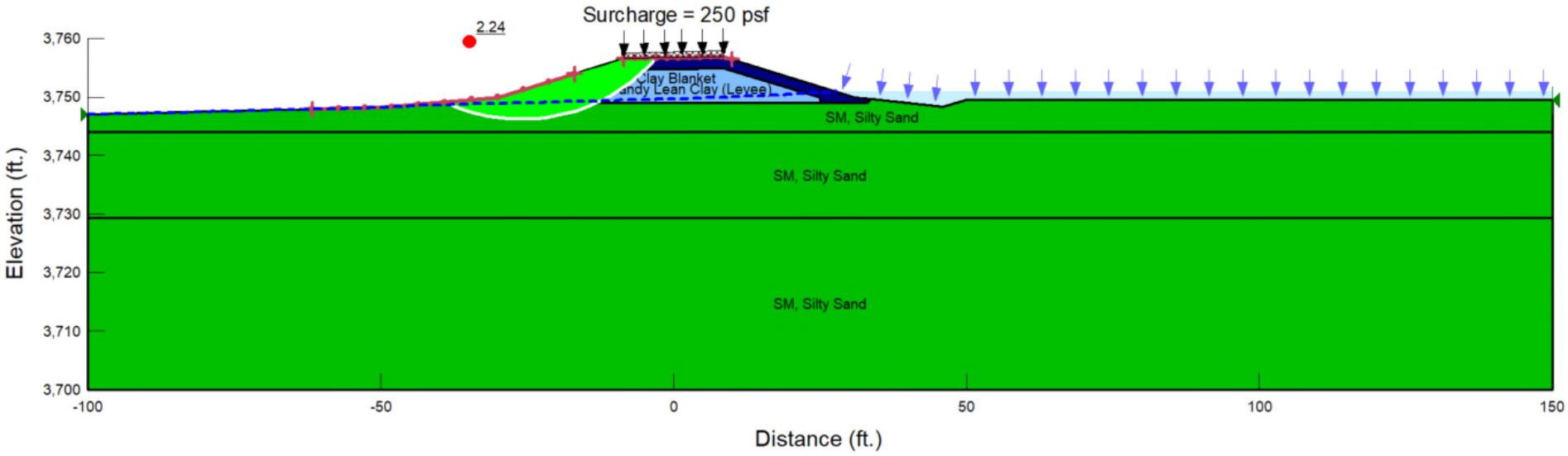
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1675+00 WEST LEVEE
100-YEAR FLOOD - EL. 3753.5
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
4/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
Clay Blanket	127	29	50	0	1,000
Sandy Lean Clay (levee) [1]	127	29	50	18	100
Sandy Lean Clay [1]	127	28	50	11	175
Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

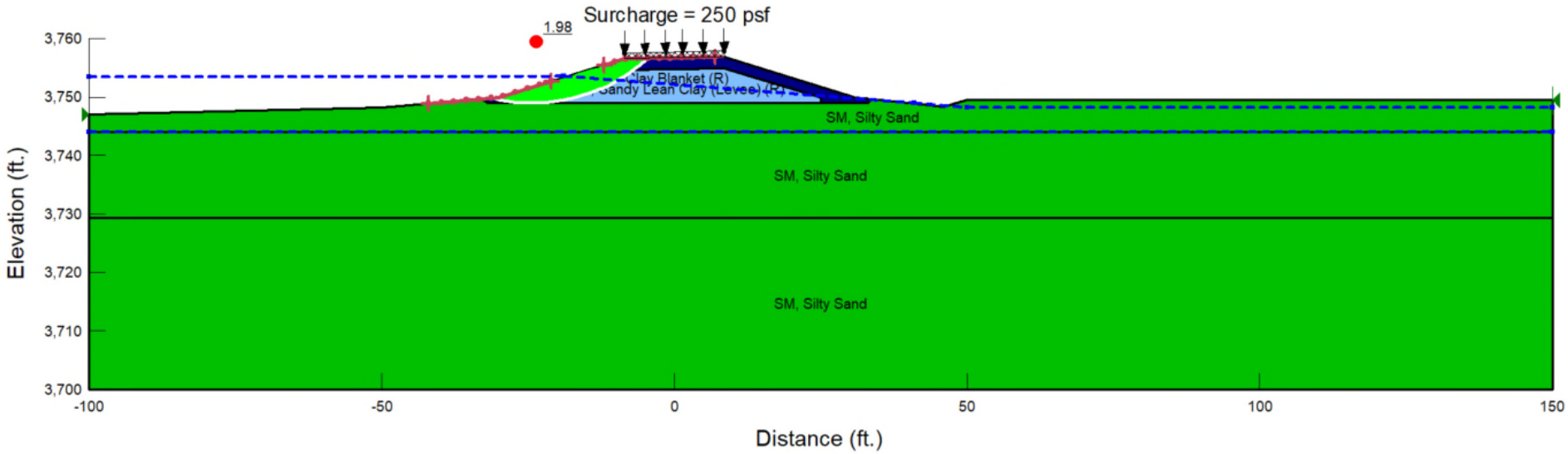
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1675+00 WEST LEVEE
1 FT OF WATER ON LANDSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
5/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
Clay Blanket	127	29	50	0	1,000
Sandy Lean Clay (levee) [1]	127	29	50	18	100
Sandy Lean Clay [1]	127	28	50	11	175
Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

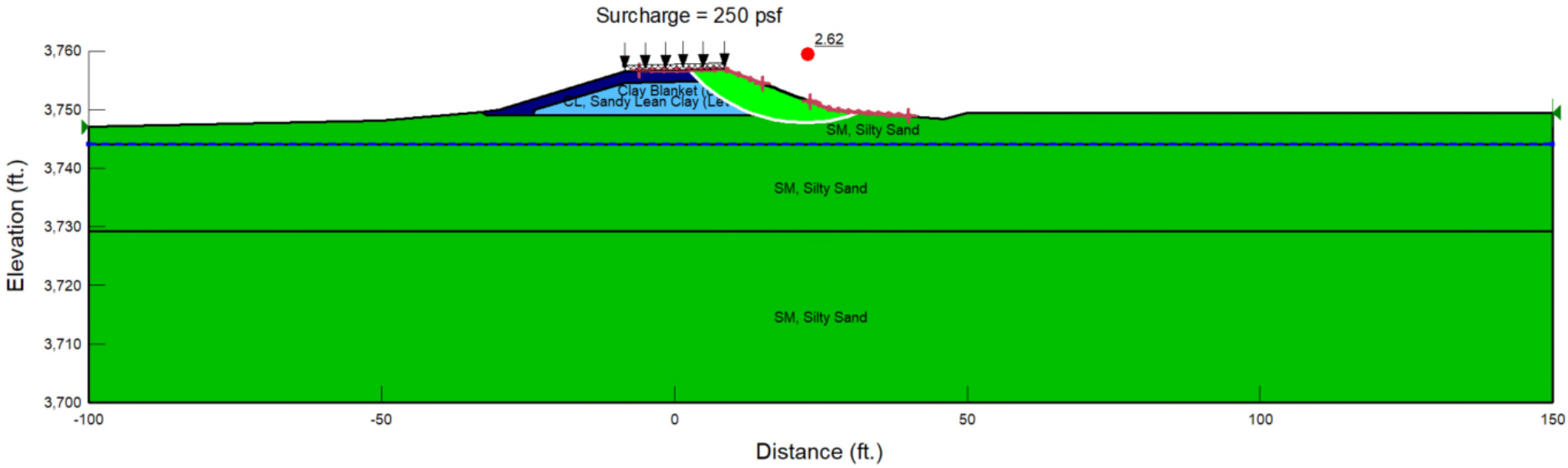
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1675+00 WEST LEVEE
RAPID DRAWDOWN
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
6/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
Clay Blanket	127	29	50	0	1,000
Sandy Lean Clay (levee) [1]	127	29	50	18	100
Sandy Lean Clay [1]	127	28	50	11	175
Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

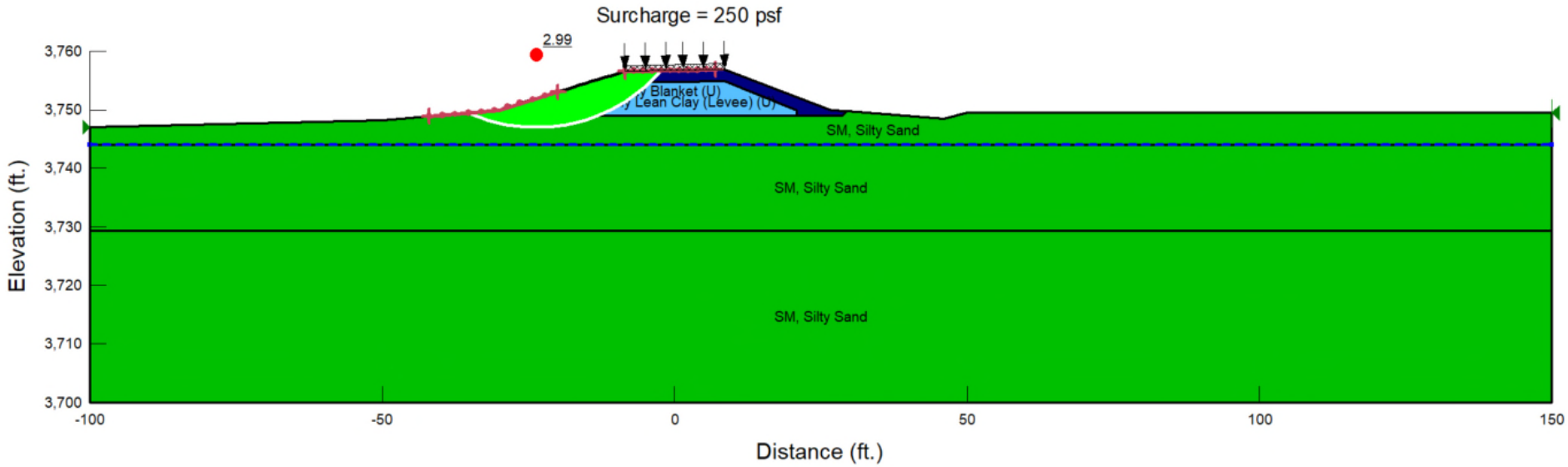
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1675+00 WEST LEVEE (2.5H:1V)
END OF CONSTRUCTION – LANDSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
1/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
Clay Blanket	127	29	50	0	1,000
Sandy Lean Clay (levee) [1]	127	29	50	18	100
Sandy Lean Clay [1]	127	28	50	11	175
Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

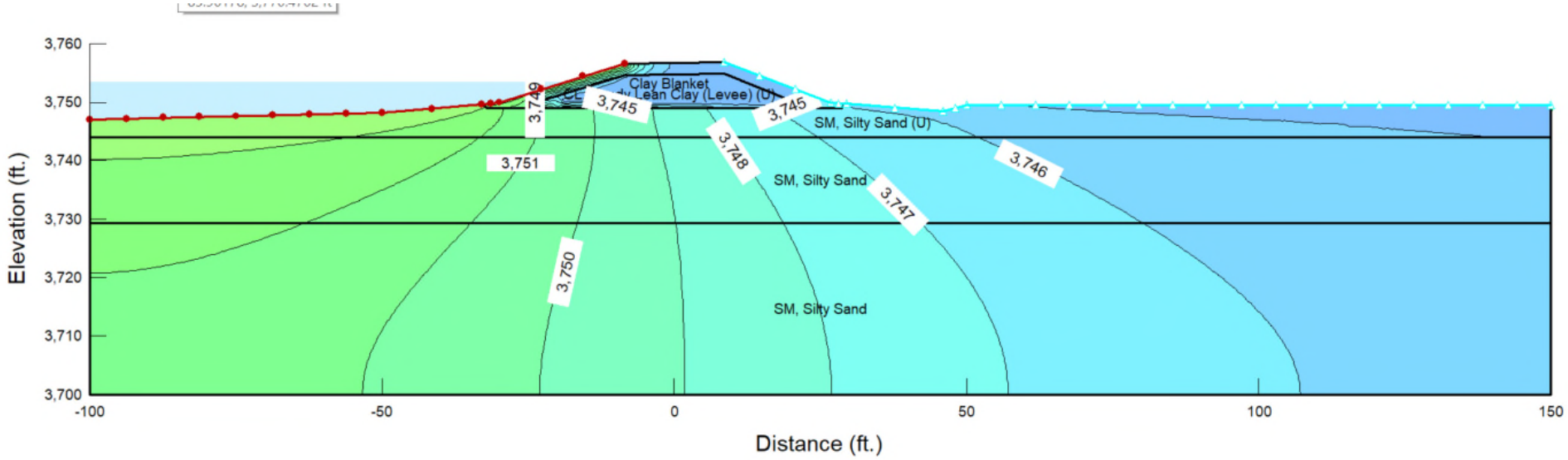
Notes: [1] Soil properties provided by USIBWC.

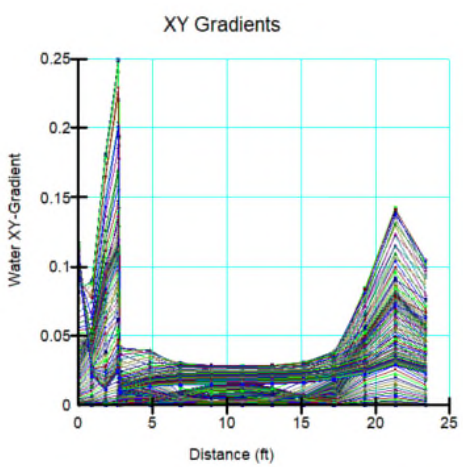
SUNLAND PARK LEVEE REPAIR

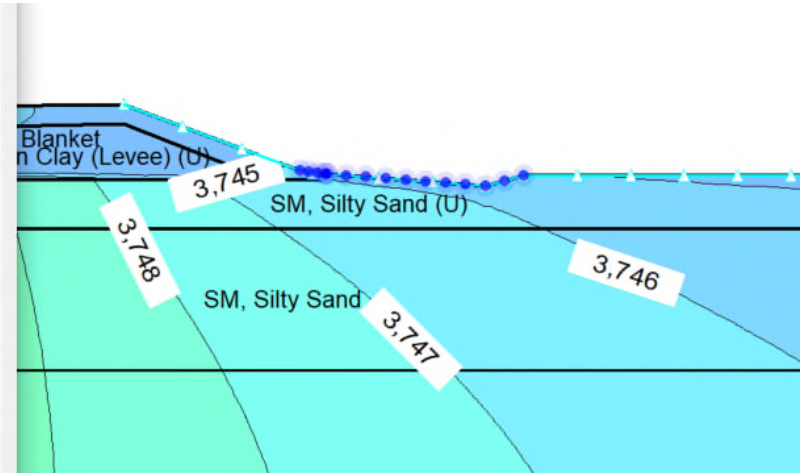
STA. 1675+00 WEST LEVEE (2.5H:1V)
END OF CONSTRUCTION – RIVERSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
2/6







Material	Hydraulic Conductivity (fps)	Vertical to Horizontal Hydraulic Conductivity Ratio
Clay Blanket	4.9E-08	0.33
Sandy Lean Clay (levee) (1)	4.9E-08	0.33
Sandy Lean Clay (1)	3.0E-08	0.10
Fat Clay (1)	3.0E-08	0.10
Silty Sand (1)	5.0E-05	0.33
Poorly graded Sand (w. silt) (1)	3.0E-04	0.33

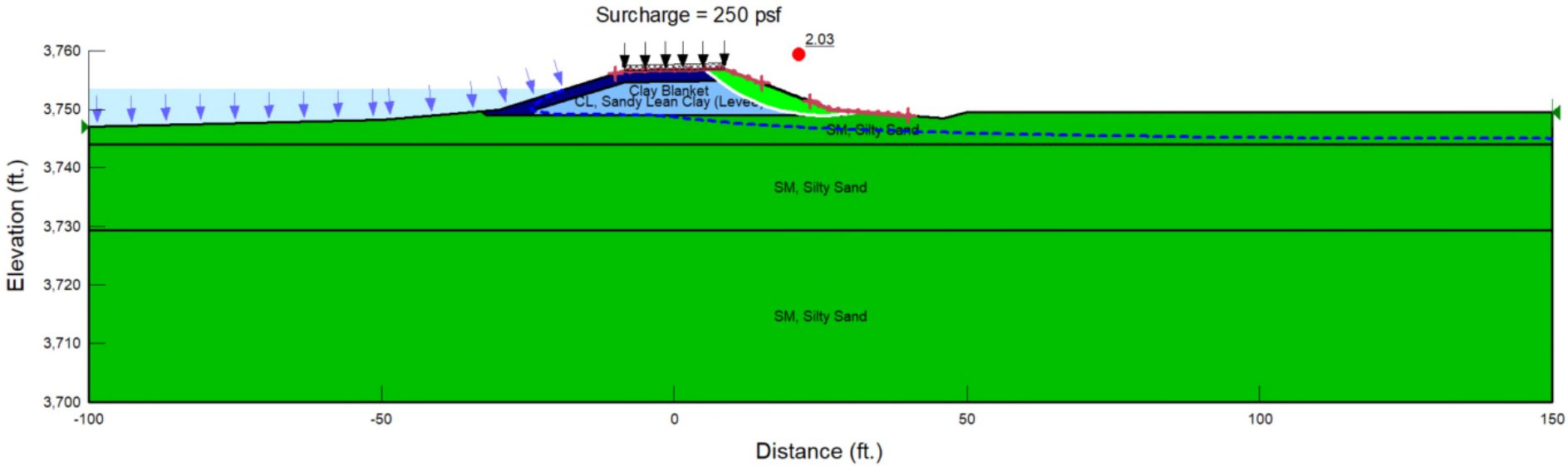
Notes: (1) Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1675+00 WEST LEVEE (2.5H:1V)
SEEPAGE 100-YEAR FLOOD – EL. 3753.5
TOTAL HEAD & EXIT GRADIENT

ARCADIS Design & Consultancy for natural and built assets

FIGURE
3/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
Clay Blanket	127	29	50	0	1,000
Sandy Lean Clay (levee) [1]	127	29	50	18	100
Sandy Lean Clay [1]	127	28	50	11	175
Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

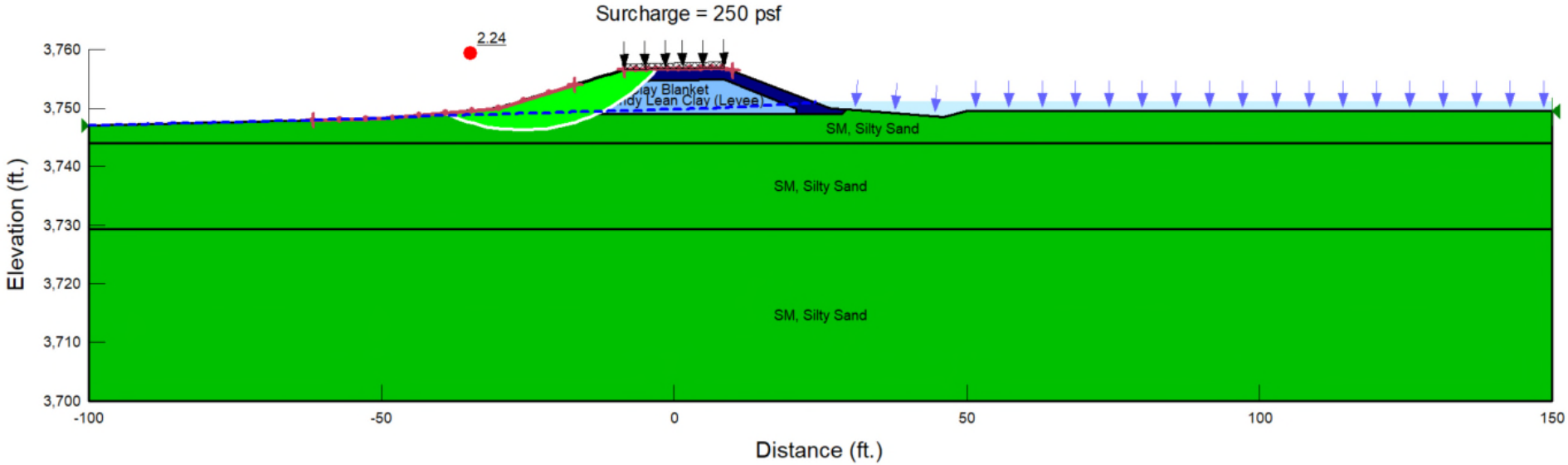
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1675+00 WEST LEVEE (2.5H:1V)
100-YEAR FLOOD - EL. 3753.5
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
4/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
Clay Blanket	127	29	50	0	1,000
Sandy Lean Clay (levee) [1]	127	29	50	18	100
Sandy Lean Clay [1]	127	28	50	11	175
Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

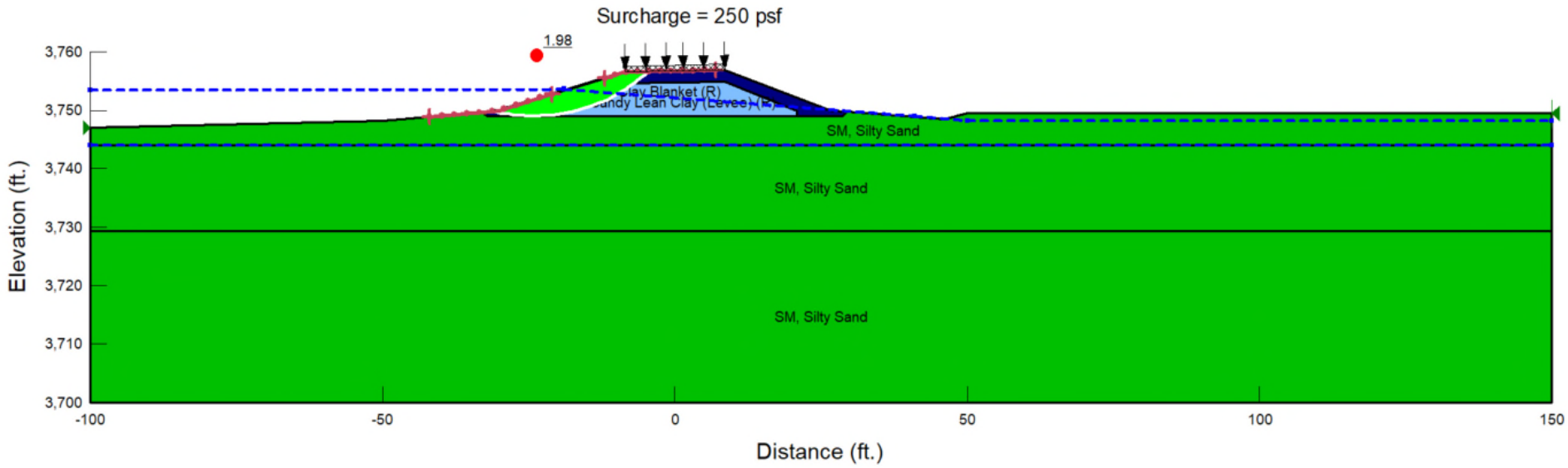
Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1675+00 WEST LEVEE (2.5H:1V)
1 FT OF WATER ON LANDSIDE
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
5/6



Material	Moist Unit Weight (pcf)	Φ, (Effective / Drained) (degree)	C, (Effective / Drained) (psf)	Φ, (Total / Undrained) (degree)	C, (Total / Undrained) (psf)
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Sandy Lean Clay (levee) [1]	127	29	50	18	100
Sandy Lean Clay [1]	127	28	50	11	175
Fat Clay [1]	118	28	100	13	325
Silty Sand [1]	131	32	0	--	--
Poorly graded Sand (w. silt) [1]	124	34	0	--	--

Notes: [1] Soil properties provided by USIBWC.

SUNLAND PARK LEVEE REPAIR

STA. 1675+00 WEST LEVEE (2.5H:1V)
RAPID DRAWDOWN
SLOPE STABILITY

ARCADIS Design & Consultancy for natural and built assets

FIGURE
6/6

Appendix B.3: Settlement

For	Sunland Park Levee Repair	Job No.	1991047.00	Sheet No.	1 of 3
Calculation	Settlement Parameters	Made By	AF	Checked by	AD
		Date	6/14/2019	Date	6/26/2019

ARCADIS Design & Consultancy for natural and built assets

Levee Soils

Parameters of Primary Consolidation and Secondary Compression for Cohesive Soils

Boring ID	Layer No	Soil Layering Information			Soils Parameters																
		Soils Description	Depth Below the Top of the Levee		Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Undrained Shear Strength, Cu (psf)	Liquid Limit (%)	Plasticity Index (%)	Moisture Content (%)	$Cc_{Liquid\ Limit}$	Cc_{Mc}	Cc_{max}	Cr	Specific Gravity	Cα	Initial Void Ratio, e_0	Degree of Saturation (%)	Pre-consolidation stress, σ'p (psf)	Coefficient of Consolidation, Cv (ft^2/day)	Time at 100 Percent of Primary Consolidation, t_{100} (days)
			Top of Layer (ft)	Bottom of Layer (ft)																	
-	1	24 inch Impervious Layer	0.00	-2.00	125	104	2500	45	30	20	0.32	0.24	0.32	0.0533	2.65	0.019	0.59	90	11312	2.10E-01	38
147	2	Levee Soil (1): Very Stiff Lean Clay (CL)	-2.00	-2.18	125	103	2250	44	26	21	0.31	0.25	0.31	0.0517	2.65	0.019	0.60	93	10912	2.40E-01	0
	3	Levee Soil (2): Stiff Lean Clay (CL)	-2.18	-5.93	125	101	1200	44	26	24	0.31	0.29	0.31	0.0517	2.65	0.019	0.64	99	5820	2.40E-01	117
	4	Levee Soil (3): Very Stiff Lean Clay (CL)	-5.93	-7.93	125	104	3500	47	30	20	0.34	0.24	0.34	0.0567	2.65	0.020	0.59	90	15837	1.80E-01	44

Foundation Soils

Parameters of Primary Consolidation and Secondary Compression for Cohesive Soils

Boring ID	Layer No	Soil Layering Information			Soils Parameters																
		Soils Description	Depth Below the Base of the Levee		Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Undrained Shear Strength, Cu (psf)	Liquid Limit (%)	Plasticity Index (%)	Moisture Content (%)	$Cc_{Liquid\ Limit}$	Cc_{Mc}	Cc_{max}	Cr	Specific Gravity	Cα	Initial Void Ratio, e_0	Degree of Saturation (%)	Pre-consolidation stress, σ'p (psf)	Coefficient of Consolidation, Cv (ft^2/day)	Time at 100 Percent of Primary Consolidation, t_{100} (days)
			Top of Layer (ft)	Bottom of Layer (ft)																	
147	1	Soft Lean Clay (CL)	0.00	-4.50	115	93	400	47	30	24	0.34	0.29	0.34	0.0567	2.65	0.020	0.78	81	1810	1.80E-01	225

Levee Soils

Elastic Paramaters and Coefficient of Structural Strength for Cohesive Soils

Boring ID	Layer No	Soil Layering Information			Soils Parameters							
		Soils Description	Depth Below the Top of the Levee		Total Unit Weight (pcf)	Avg N-Value	Es Value (ksf)			Avg Es Value (psi)	Poisson's Ratio, μs	Deformation Modulus, Ed (psi)
			Top of Layer (ft)	Bottom of Layer (ft)			500(N+15)/50	$7000N^{0.5}/50$	AASHTO			
-	1	24 inch Impervious Layer	0.00	-2.00	125	-	-	-	-	4527.45	0.40	5389.82
147	2	Levee Soil (1): Very Stiff Lean Clay (CL)	-2.00	-2.18	125	30	450.00	766.81	1000.00	5131.51	0.40	6108.94
	3	Levee Soil (2): Stiff Lean Clay (CL)	-2.18	-5.93	125	12	270.00	484.97	640.00	3229.11	0.40	3844.18
	4	Levee Soil (3): Very Stiff Lean Clay (CL)	-5.93	-7.93	125	-	-	-	-	5131.51	0.40	6108.94

Foundation Soils

Elastic Paramaters and Coefficient of Structural Strength for Non-Cohesive/Cohesive Soils

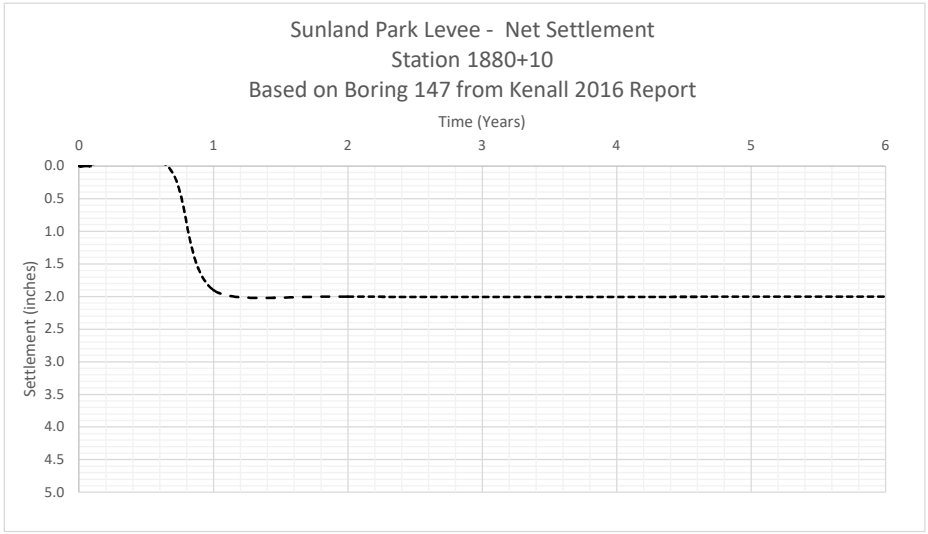
Boring ID	Layer No	Soil Layering Information			Soils Parameters							
		Soils Description	Depth Below the Base of the Levee		Total Unit Weight (pcf)	Avg N-Value	Es Value (ksf)			Avg Es Value (psi)	Poisson's Ratio, μs	Deformation Modulus, Ed (psi)
			Top of Layer (ft)	Bottom of Layer (ft)			500(N+15)/50	$7000N^{0.5}/50$	AASHTO			
147	1	Soft Lean Clay (CL)	0.00	-4.50	115	5	200.00	313.05	500.00	2345.02	0.25	2501.36
	2	Medium Silty Sand (SM)	-4.50	-6.00	111	11	260.00	464.33	620.00	3111.87	0.30	3419.64
	3	Stiff Silt Seam (ML)	-6.00	-8.00	130	-	-	-	-	3229.11	0.40	3844.18
	4	Dense Silty Sand (SM)	-8.00	-10.20	121	32	470.00	791.96	1040.00	5328.61	0.30	5855.62
	5	Medium Silty Sand (SM)	-10.20	-20.20	112	17	315.00	568.68	730.00	3735.38	0.30	4104.81
	6	Loose Silty Sand (SM)	-20.20	-22.20	103	8	230.00	395.98	560.00	2745.32	0.20	2859.71
	7	Medium Silty Sand (SM)	-22.20	-24.20	112	14	290.00	523.83	680.00	3457.94	0.30	3799.93904
	8	Loose Silty Sand (SM)	-24.20	-26.20	107	9	240.00	420.00	580.00	2870.37	0.20	2989.96914
	9	Medium Silty Sand (SM)	-26.20	-32.00	115	20.3	353.00	630.78	806.00	4143.00	0.30	4552.75056

Note:
Soil stratigraphy and parameters are obtained from "Sunland Park Levee Forensic Geotechnical Investigation" by Kenall, Inc., dated July 29, 2016.

For	Sunland Park Levee Repair	Job No.	1991047.00	Sheet No.	2 of 3
Calculation	Settlement Calculations	Made By	AF	Checked by	AD
		Date	6/14/2019	Date	6/26/2019

Boring No.	Station	Evaluation of Settlement for Static Loads	2019 Existing Levee Settlement	During of Construction			End of Construction	After Construction				
			Time (Days)									
			0	7	30	5	240	365	730	2190	3650	3653
			Time (Years)									
			0	0.019	0.082	0.014	0.658	1.000	2.000	6.000	10.000	10.008
			Maximum Settlement (inches)									
147	1880+10	Immediate Settlement for the Foundation Soils	0.1	0.1	0.1	0.2	0.2	0.3	0.3	0.3	0.3	0.3
		Consolidation for the Foundation Soils	2.7	2.6	3.1	3.2	3.6	3.8	3.8	3.8	3.8	3.8
		Consolidation for the Levee Soils	1.2	0.2	3.0	3.2	3.6	4.0	4.0	4.0	4.0	4.0
		Consolidation for the 24 inch Impervious Material	-	-	-	0.1	0.3	1.5	1.6	1.6	1.6	1.6

Time (yrs)	**0**	**0.019**	**0.082**	**0.014**	**0.658**	**1.000**	**2.000**	**6.000**	**10.000**	**10.008**
Total Settlement (in)	0.0	2.9	6.2	6.7	7.7	9.6	9.7	9.7	9.7	9.7
Net Settlement after 8 month construction (in)	0.0	0.0	0.0	0.0	0.0	1.9	2.0	2.0	2.0	2.0



Note: Soil stratigraphy and parameters are obtained from “Sunland Park Levee Forensic Geotechnical Investigation” by Kenall, Inc., dated July 29, 2016.

(b) An empirical relationship between the preconsolidation stress and liquidity index as a function of clay sensitivity, ratio of undisturbed to remolded undrained shear strength, is given in Figure 1-1. The preconsolidation stress may also be estimated from (NAVFAC DM-7.1)

$$\sigma_p' = \frac{C_u}{0.11 + 0.0037PI} \qquad (1\text{-}2)$$

where

σ_p' = preconsolidation stress, tsf
C_u = undrained shear strength, tsf
PI = plasticity index, percent

where

E_m = deformation modulus, $\frac{E_s}{1 - \nu_s^2}$, tsf

a. Evaluation of the Degree of Consolidation. Solution of the Terzaghi consolidation theory to determine U_t is provided in Table 3-10 as a function of time factor T_v for four cases of different distributions of the initial excess pore water pressure. Figure 3-16 illustrates example distributions of the initial excess pore water pressure for single (drainage from one surface) and double (drainage from top and bottom surfaces) drainage.

(1) Time factor. The time factor is given by

$$T_v = \frac{C_v t}{H_e^2} \qquad (3\text{-}26)$$

where

c_v = coefficient of consolidation of the stratum, ft/day
H_e = equivalent height of the compressible stratum, ft

EM 1110-1-1904
30 Sep 90

Table 3-7

Estimates of the Virgin Compression Index, C_c

Soil	C_c
Organic soils with sensitivity less than 4	0.009(LL - 10)
Organic soils, peat	$0.0115W_n$
Clays	$1.15(e_o - 0.35)$ $0.012W_n$ $0.01(LL - 13)$
Varved clays	$(1 + e_o) \cdot [0.1 + 0.006(W_n - 25)]$
Uniform silts	0.20
Uniform sand Loose Dense	 0.05 to 0.06 0.02 to 0.03

Note: LL = liquid limit, percent
W_n = natural water content, percent
e_o = initial void ratio

EM 1110-1-1904
30 Sep 90

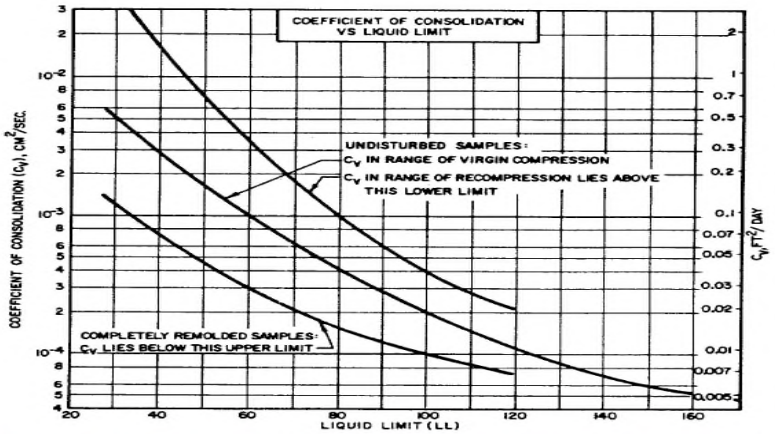


Figure 3-18. Correlations between coefficient of consolidation and liquid limit (NAVFAC DM 7.1)

Appendix B.4: Riprap Sizing

Project:	Sunland Park Levee Repair
Project Number:	01991047.0000
Description:	Riprap Calculations
Designed by:	AD **Date:** 11/30/2018

ARCADIS

RIPRAP CALCULATIONS NEAR STATION 1696+00

Reference: "Bank Stabilization Design Guidelines", Report No. SRH-2015-25 by Bureau of Reclamation, dated June 2015.

Resistances to flow for different materials that are placed on a river bank as a revetment are shown in Table 4-2.

Table 4–2. Permissible Shear and Velocity Resistance Values for Selected Lining Materials (Fischenich 2001)[1]

Boundary Category	Bank Material Type	Permissible Shear Stress (lb/sq ft)	Permissible Velocity (ft/sec)	Citation(s) ([2])
Gravel/Cobble	1-in.	0.33	2.5 – 5	A
	2-in.	0.67	3 – 6	A
	6-in.	2.0	4 – 7.5	A
	12-in.	4.0	5.5 – 12	A

The maximum velocity is 6.55 ft/s. In order to resist these maximum velocities, a minimum of 6-inch gravel / cobble material should be used on the riverside.

Using the same reference, Section 13.2 explains riprap sizing based on Isbash (1936).

$$V = C\lfloor 2g(S-1)\rfloor^{1/2}(d_{50})^{1/2} \qquad \text{(Equation 13.1)}$$

therefore,

$$D = \frac{V^2}{2gC^2(s-1)}$$

Project:	Sunland Park Levee Repair		
Project Number:	01991047.0000		
Description:	Riprap Calculations		
Designed by:	AD	**Date:**	11/30/2018

RIPRAP CALCULATIONS NEAR STATION 1696+00

V = Mean velocity against stone =	6.55	ft/s
C = Isbash constant =	0.90	
g = Acceleration of gravity =	32.17	ft/s
S = Specific gravity of stone =	2.65	
d_{50} = Median diameter of spherical stone =	0.50	ft
d_{50} = Median diameter of spherical stone =	**5.99**	in

Using Table 13-1, the nominal riprap size is 6 inches and max d(100) is 12 inches.

Table 13–1. Minimum and Maximum Allowable Particle Size in Inches

Nominal Riprap Class by Median Particle Diameter		d_{15}		d_{50}		d_{85}		d_{100}
Class	*Size*	Min	Max	Min	Max	Min	Max	Max
I	6 in	3.7	5.2	5.7	6.9	7.8	9.2	12.0
II	9 in	5.5	7.8	8.5	10.5	11.5	14.0	18.0
III	12 in	7.3	10.5	11.5	14.0	15.5	18.5	24.0
IV	15 in	9.2	13.0	14.5	17.5	19.5	23.0	30.0
V	18 in	11.0	15.5	17.0	20.5	23.5	27.5	36.0
VI	21 in	13.0	18.5	20.0	24.0	27.5	32.5	42.0
VII	24 in	14.5	21.0	23.0	27.5	31.0	37.0	48.0
VIII	30 in	18.5	26.0	28.5	34.5	39.0	46.0	60.0
IX	36 in	22.0	31.5	34.0	41.5	47.0	55.5	72.0
X	42 in	25.5	36.5	40.0	48.5	54.5	64.5	84.0

Note: Particle size corresponds to the intermediate ("B") axis of the particle.

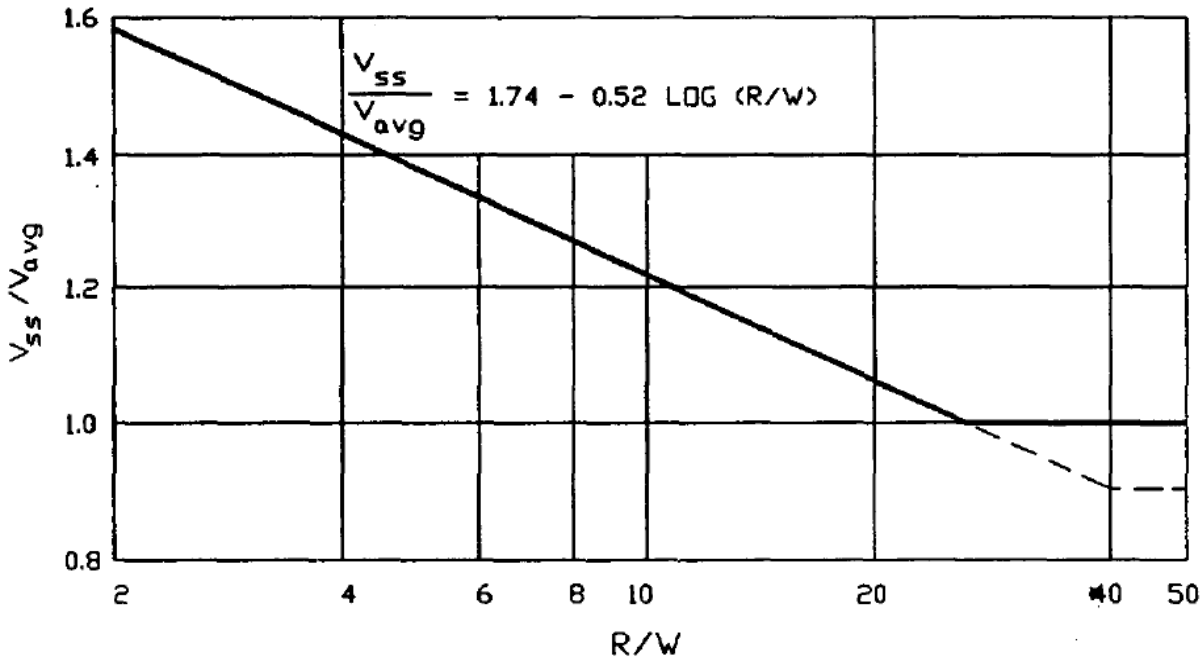
Project:	Sunland Park Levee Repair		
Project Number:	01991047.0000		
Description:	Riprap Calculations		
Designed by:	AD	**Date:**	11/30/2018

RIPRAP CALCULATIONS NEAR STATION 1696+00

Reference: USACE, 1994, Hydraulic Design of Flood Control Channels, EM 1110-2-1601

$$D_{30} = S_f C_s C_v C_t d \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{0.5} \left(\frac{V}{\sqrt{Kgd}} \right) \right]^{2.5}$$

$$K = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \varphi}} \qquad D_{50} = D_{30} \left(\frac{D_{85}}{D_{15}} \right)^{\frac{1}{3}}$$



S_f = Safety Factor =	1.30	
C_s = stability - 0.3 for angular rock =	0.30	
C_v = velocity distribution coefficient =	1.00	
C_t = thickness coefficient =	0.98	
d (depth of flow) =	7.00	ft
Θ =Side slope angle =	18	degree
ϕ = Angle of repose =	40	degree
K = Side slope correction factor =	0.88	

Project:	Sunland Park Levee Repair		
Project Number:	01991047.0000		
Description:	Riprap Calculations		
Designed by:	AD	**Date:**	11/30/2018

RIPRAP CALCULATIONS NEAR STATION 1696+00

Yw =	62.40	lbs/ft^3
Ys =	165	lbs/ft^3
g =	32.20	ft/s^2
R = Radius =	4000	ft
R/W =	7.018	
W = Width of Channel Floodplain =	570	ft
D_{85} / D_{15} =	2.0	
V_{avg}	6.55	ft/s
V_{design}	8.51	ft/s
D30	0.41	ft
D30 Minimum	**4.92**	in
D50 Minimum	**6.20**	in

The riprap sizing is selected from Table 3-1 of EM 1110-2-1601 manual for D_{100}(max) of 12 inches.

EM 1110-2-1601
Change 1
30 Jun 94

Table 3-1
Gradations for Riprap Placement in the Dry, Low-Turbulence Zones

D_{100}(max) in.	Limits of Stone Weight, lb[1], for Percent Lighter by Weight 100 Max	100 Min	50 Max[2]	50 Min	15 Max[2]	15 Min	D_{30}(min) ft	D_{90}(min) ft
Specific Weight = 155 pcf								
* 9	34	14	10	7	5	2	0.37	0.53 *
12	81	32	24	16	12	5	0.48	0.70
15	159	63	47	32	23	10	0.61	0.88
18	274	110	81	55	41	17	0.73	1.06
21	435	174	129	87	64	27	0.85	1.23
24	649	260	192	130	96	41	0.97	1.40
27	924	370	274	185	137	58	1.10	1.59
30	1,268	507	376	254	188	79	1.22	1.77
33	1,688	675	500	338	250	105	1.34	1.94
36	2,191	877	649	438	325	137	1.46	2.11
42	3,480	1,392	1,031	696	516	217	1.70	2.47
48	5,194	2,078	1,539	1,039	769	325	1.95	2.82
54	7,396	2,958	2,191	1,479	1,096	462	2.19	3.17

Project:	Sunland Park Levee Repair		
Project Number:	01991047.0000		
Description:	Riprap Calculations		
Designed by:	AD	**Date:**	11/30/2018

RIPRAP CALCULATIONS NEAR STATION 1696+00

Specific Weight = 165 pcf

*	9	36	15	11	7	5	2	0.37	0.53	*
	12	86	35	26	17	13	5	0.48	0.70	
	15	169	67	50	34	25	11	0.61	0.88	
	18	292	117	86	58	43	18	0.73	1.06	
	21	463	185	137	93	69	29	0.85	1.23	
	24	691	276	205	138	102	43	0.97	1.40	
	27	984	394	292	197	146	62	1.10	1.59	
	30	1,350	540	400	270	200	84	1.22	1.77	
	33	1,797	719	532	359	266	112	1.34	1.96	
	36	2,331	933	691	467	346	146	1.46	2.11	
	42	3,704	1,482	1,098	741	549	232	1.70	2.47	
	48	5,529	2,212	1,638	1,106	819	346	1.95	2.82	
	54	7,873	3,149	2,335	1,575	1,168	492	2.19	3.17	

Rock Riprap Thickness = 1.5(d_{50}) or a minimum of d_{100}, whichever is greater.

1.5(d_{50}) =	9.3	in
d_{100} =	12.0	in
Use riprap thickness =	**18.00**	**in**

Project:	Sunland Park Levee Repair		
Project Number:	01991047.0000		
Description:	Bedding Stone		
Designed by:	AD	**Date:**	11/30/2020

ARCADIS

Bedding Stone

Reference: "Riprap protection for Scour Holes" by USACE, Mephis District, Hydraulics Branch HN-012, dated June 2018.

A typical bedding stone used is in the Memphis Disctrict under the riprap is the gradation shown below. The bedding stone is usually placed directly on the earthen subgrade in a layer having a minimum thickness of 6 inches. The bedding stone may also be placed over a geotextile.

Table C-5. Bedding Stone Gradation

Sieve Size	Heavy Boundary Percent Passing	Light Boundary Percent Passing
3"	100	100
1 1/2"	95	100
3/4"	65	100
3/8"	40	80
# 4	30	60
# 10	20	50
#40	15	35
# 200	5	15

Appendix B.5: BFE and Freeboard Elevations

EAST LEVEE			
Station	**Existing Ground**	**100-YR WSE**	**Proposed Ground**
1513+00	3768.7	3764.3	3768.8
1514+00	3768.9	3764.3	3768.8
1515+00	3768.8	3764.3	3768.8
1516+00	3768.7	3764.2	3768.8
1517+00	3768.4	3764.0	3767.5
1518+00	3765.0	3763.2	-
1519+00	3764.5	3762.5	3765.9
1520+00	3765.0	3762.1	3765.8
1521+00	3764.8	3762.0	3765.8
1522+00	3764.9	3761.9	3765.8
1523+00	3764.8	3761.8	3765.8
1524+00	3764.6	3761.8	3765.8
1525+00	3764.8	3761.7	3765.7
1526+00	3764.7	3761.7	3765.6
1527+00	3764.5	3761.6	3765.5
1528+00	3764.6	3761.5	3765.5
1529+00	3764.5	3761.5	3765.4
1530+00	3764.5	3761.4	3765.3
1531+00	3764.4	3761.4	3765.2
1532+00	3764.3	3761.3	3765.1
1533+00	3764.4	3761.3	3765.0
1534+00	3764.1	3761.2	3764.9
1535+00	3763.9	3761.1	3764.8
1536+00	3764.0	3761.1	3764.7
1537+00	3764.0	3761.0	3764.6
1538+00	3763.9	3761.0	3764.6
1539+00	3763.7	3760.9	3764.5
1540+00	3763.6	3760.8	3764.4
1541+00	3763.8	3760.7	3764.3
1542+00	3763.6	3760.7	3764.2
1543+00	3763.6	3760.6	3764.1
1544+00	3763.1	3760.5	3764.0
1545+00	3763.2	3760.4	3763.9
1546+00	3763.1	3760.2	3763.8
1547+00	3763.1	3760.1	3763.7
1548+00	3762.9	3760.0	3763.7
1549+00	3762.8	3759.9	3763.6
1550+00	3762.6	3759.8	3763.5
1551+00	3762.4	3759.8	3763.5
1552+00	3762.5	3759.8	3763.4
1553+00	3762.4	3759.7	3763.3
1554+00	3762.4	3759.7	3763.3
1555+00	3762.5	3759.6	3763.2
1556+00	3762.2	3759.6	3763.2

EAST LEVEE			
Station	**Existing Ground**	**100-YR WSE**	**Proposed Ground**
1557+00	3762.2	3759.6	3763.1
1558+00	3762.3	3759.5	3763.1
1559+00	3762.5	3759.5	3763.0
1560+00	3762.2	3759.5	3763.0
1561+00	3762.3	3759.4	3762.9
1562+00	3762.4	3759.4	3762.9
1563+00	3762.4	3759.4	3762.8
1564+00	3762.2	3759.4	3762.8
1565+00	3762.3	3759.3	3762.7
1566+00	3762.1	3759.3	3762.7
1567+00	3762.1	3759.3	3762.6
1568+00	3761.9	3759.2	3762.6
1569+00	3761.9	3759.2	3762.5
1570+00	3761.7	3759.2	3762.5
1571+00	3761.7	3759.1	3762.5
1572+00	3761.7	3759.1	3762.5
1573+00	3761.8	3759.1	3762.5
1574+00	3761.8	3759.0	3762.5
1575+00	3761.7	3759.0	3762.5
1576+00	3761.7	3759.0	3762.5
1577+00	3761.7	3758.9	3762.5
1578+00	3761.8	3758.9	3762.5
1579+00	3761.8	3758.9	3762.5
1580+00	3761.4	3758.8	3762.4
1581+00	3761.8	3758.8	3762.4
1582+00	3761.8	3758.8	3762.3
1583+00	3761.8	3758.7	3762.2
1584+00	3761.8	3758.7	3762.2
1585+00	3761.8	3758.6	3762.1
1586+00	3761.4	3758.6	3762.0
1587+00	3761.6	3758.6	3762.0
1588+00	3761.4	3758.5	3761.9
1589+00	3761.1	3758.4	3761.9
1590+00	3761.0	3758.4	3761.8
1591+00	3761.0	3758.3	3761.7
1592+00	3760.8	3758.2	3761.7
1593+00	3760.9	3758.2	3761.6
1594+00	3760.8	3758.0	3761.5
1595+00	3760.5	3757.9	3761.5
1596+00	3760.4	3757.8	3761.4
1597+00	3760.2	3757.7	3761.3
1598+00	3760.3	3757.6	3761.3
1599+00	3760.4	3757.6	3761.2
1600+00	3760.4	3757.5	3761.1

EAST LEVEE			
Station	**Existing Ground**	**100-YR WSE**	**Proposed Ground**
1601+00	3760.5	3757.5	3761.1
1602+00	3760.4	3757.4	3761.0
1603+00	3760.1	3757.3	3760.9
1604+00	3760.0	3757.2	3760.9
1605+00	3759.8	3757.1	3760.8
1606+00	3759.9	3757.0	3760.7
1607+00	3759.9	3756.9	3760.7
1608+00	3759.8	3756.8	3760.6
1609+00	3759.8	3756.8	3760.6
1610+00	3759.7	3756.7	3760.5
1611+00	3759.6	3756.6	3760.4
1612+00	3759.3	3756.6	3760.4
1613+00	3759.2	3756.5	3760.3
1614+00	3759.2	3756.4	3760.2
1615+00	3758.9	3756.4	3760.2
1616+00	3758.9	3756.3	3760.1
1617+00	3759.3	3756.3	3760.0
1618+00	3759.6	3756.2	3760.0
1619+00	3759.7	3756.2	3759.9
1620+00	3759.6	3756.1	3759.8
1621+00	3759.6	3756.0	3759.8
1622+00	3759.7	3755.9	3759.7
1623+00	3759.4	3755.8	3759.6
1624+00	3759.2	3755.8	3759.6
1625+00	3759.2	3755.7	3759.5
1626+00	3759.0	3755.6	3759.4
1627+00	3758.7	3755.6	3759.4
1628+00	3758.8	3755.5	3759.3
1629+00	3758.9	3755.5	3759.3
1630+00	3758.7	3755.4	3759.2
1631+00	3758.6	3755.4	3759.1
1632+00	3758.4	3755.3	3759.1
1633+00	3758.4	3755.3	3759.0
1634+00	3758.2	3755.2	3758.9
1635+00	3758.3	3755.2	3758.9
1636+00	3758.4	3755.1	3758.8
1637+00	3758.2	3755.0	3758.7
1638+00	3758.3	3755.0	3758.7
1639+00	3758.4	3754.9	3758.6
1640+00	3758.2	3754.8	3758.5
1641+00	3757.9	3754.8	3758.5
1642+00	3758.0	3754.7	3758.4
1643+00	3758.1	3754.7	3758.3
1644+00	3758.0	3754.6	3758.3

EAST LEVEE			
Station	**Existing Ground**	**100-YR WSE**	**Proposed Ground**
1645+00	3758.1	3754.5	3758.2
1646+00	3758.4	3754.5	3758.1
1647+00	3757.8	3754.4	3758.1
1648+00	3757.6	3754.4	3758.0
1649+00	3757.1	3754.3	3758.0
1650+00	3756.6	3754.3	3757.9
1651+00	3756.6	3754.2	3757.8
1652+00	3756.5	3754.2	3757.8
1653+00	3756.4	3754.1	3757.7
1654+00	3756.7	3754.1	3757.6
1655+00	3756.7	3754.1	3757.6
1656+00	3756.5	3754.0	3757.5
1657+00	3756.4	3754.0	3757.4
1658+00	3756.1	3753.9	3757.4
1659+00	3756.2	3753.9	3757.3
1660+00	3756.2	3753.9	3757.2
1661+00	3756.1	3753.8	3757.1
1662+00	3756.0	3753.8	3757.0
1663+00	3756.1	3753.7	3757.0
1664+00	3756.1	3753.7	3757.0
1665+00	3755.9	3753.6	3757.0
1666+00	3755.7	3753.6	3757.0
1667+00	3755.6	3753.6	-
1668+00	3755.9	3753.7	3755.8
1669+00	3757.3	3753.7	3758.3
1670+00	3757.5	3753.6	3758.0
1671+00	3757.4	3753.6	3757.5
1672+00	3757.6	3753.6	3757.5
1673+00	3757.9	3753.5	3757.4
1674+00	3757.9	3753.5	3757.4
1675+00	3757.9	3753.4	3757.3
1676+00	3757.8	3753.4	3757.3
1677+00	3758.1	3753.3	3757.2
1678+00	3758.0	3753.3	3757.2
1679+00	3758.0	3753.2	3757.1
1680+00	3757.9	3753.2	3757.1
1681+00	3757.6	3753.1	3757.0
1682+00	3757.6	3753.1	3757.0
1683+00	3757.7	3753.0	3756.9
1684+00	3757.8	3753.0	3756.9
1685+00	3757.6	3752.8	3756.8
1686+00	3757.6	3752.8	3756.8
1687+00	3757.3	3752.7	3756.7
1688+00	3757.3	3752.7	3756.7

EAST LEVEE			
Station	**Existing Ground**	**100-YR WSE**	**Proposed Ground**
1689+00	3756.9	3752.7	3756.6
1690+00	3757.1	3752.6	3756.6
1691+00	3757.1	3752.6	3756.5
1692+00	3756.9	3752.6	3756.5
1693+00	3756.9	3752.5	3756.4
1694+00	3756.7	3752.5	3756.4
1695+00	3756.7	3752.5	3756.3
1696+00	3756.5	3752.4	3756.3
1697+00	3756.7	3752.3	3756.2
1698+00	3757	3752.1	3756.2
1699+00	3756.8	3752.1	3756.1
1700+00	3756.9	3752.0	3756.1
1701+00	3757.3	3752.0	3756.0
1702+00	3757.0	3751.9	3756.0
1703+00	3756.7	3751.8	3755.9
1704+00	3756.4	3751.8	3755.9
1705+00	3756.2	3751.7	3755.8
1706+00	3756	3751.6	3755.8
1707+00	3756.3	3751.5	3755.7
1708+00	3756.1	3751.4	3755.7
1709+00	3756.0	3751.3	3755.6
1710+00	3756.2	3751.3	3755.5
1711+00	3756.3	3751.2	3755.5
1712+00	3755.9	3751.2	3755.4
1713+00	3755.6	3751.2	3755.4
1714+00	3755.6	3751.2	3755.3
1715+00	3755.6	3751.1	3755.2
1716+00	3755.3	3751.1	3755.2
1717+00	3755.2	3751.1	3755.1
1718+00	3755.2	3751.1	3755.1
1719+00	3755.1	3751.1	3755.0
1720+00	3755.0	3751.1	3754.9
1721+00	3755.0	3751.0	3754.9
1722+00	3755.1	3751.0	3754.8
1723+00	3754.9	3751.0	3754.8
1724+00	3755.1	3750.9	3754.7
1725+00	3755.1	3750.8	3754.6
1726+00	3754.9	3750.8	3754.6
1727+00	3754.9	3750.8	3754.5
1728+00	3754.7	3750.7	3754.5
1729+00	3754.7	3750.7	3754.4
1730+00	3754.7	3750.7	3754.4
1731+00	3754.6	3750.6	3754.4
1732+00	3754.5	3750.5	3754.4

EAST LEVEE			
Station	**Existing Ground**	**100-YR WSE**	**Proposed Ground**
1733+00	3754.4	3750.5	3754.4
1734+00	3754.3	3750.4	3754.4
1735+00	3754.4	3750.4	3754.4
1736+00	3754.4	3750.3	3754.4
1737+00	3754.2	3750.3	3754.4
1738+00	3753.9	3750.2	3754.4
1739+00	3754.1	3750.2	3754.4
1740+00	3753.8	3750.1	3754.3
1741+00	3753.6	3750.0	3754.2
1742+00	3753.8	3749.9	3754.1
1743+00	3753.8	3749.8	3754.0
1744+00	3753.6	3749.8	3753.9
1745+00	3753.5	3749.7	3753.8
1746+00	3753.6	3749.6	3753.7
1747+00	3753.4	3749.6	3753.6
1748+00	3753.6	3749.5	3753.5
1749+00	3753.5	3749.5	3753.4
1750+00	3753.4	3749.4	3753.3
1751+00	3753.3	3749.4	3753.2
1752+00	3753.2	3749.3	3753.1
1753+00	3753.1	3749.2	3753.0
1754+00	3753.0	3749.2	3752.9
1755+00	3753.0	3749.1	3752.8
1756+00	3752.9	3749.1	3752.7
1757+00	3752.7	3749.0	3752.6
1758+00	3752.5	3748.9	3752.5
1759+00	3752.6	3748.9	3752.5
1760+00	3752.7	3748.8	3752.4
1761+00	3752.4	3748.7	3752.3
1762+00	3752.4	3748.6	3752.2
1763+00	3752.4	3748.5	3752.1
1764+00	3752.3	3748.5	3752.0
1765+00	3752.1	3748.4	3751.9
1766+00	3751.6	3748.4	3751.8
1767+00	3751.6	3748.3	3751.7
1768+00	3751.5	3748.2	3751.6
1769+00	3751.5	3748.1	3751.5
1770+00	3751.4	3748.1	3751.4
1771+00	3751.4	3748.0	3751.3
1772+00	3751.4	3748.0	3751.3
1773+00	3751.4	3747.9	3751.3
1774+00	3751.5	3747.9	3751.3
1775+00	3751.5	3747.8	3751.3
1776+00	3751.4	3747.8	3751.3

EAST LEVEE			
Station	**Existing Ground**	**100-YR WSE**	**Proposed Ground**
1777+00	3751.5	3747.8	3751.2
1778+00	3751.5	3747.7	3751.2
1779+00	3751.4	3747.7	3751.2
1780+00	3751.3	3747.7	3751.2
1781+00	3751.5	3747.6	3751.2
1782+00	3751.3	3747.6	3751.2
1783+00	3751.3	3747.5	3751.2
1784+00	3751.3	3747.5	3751.2
1785+00	3751.2	3747.5	3751.2
1786+00	3751.1	3747.4	3751.2
1787+00	3751.0	3747.4	3751.1
1788+00	3750.8	3747.3	3751.1
1789+00	3750.9	3747.3	3751.1
1790+00	3750.6	3747.3	3751.1
1791+00	3750.6	3747.2	3751.0
1792+00	3750.7	3747.2	3751.0
1793+00	3750.7	3747.1	3751.0
1794+00	3750.7	3747.1	3750.9
1795+00	3750.7	3747.0	3750.9
1796+00	3750.6	3747.0	3750.9
1797+00	3750.5	3746.9	3750.8
1798+00	3750.3	3746.9	3750.8
1799+00	3750.3	3746.9	3750.8
1800+00	3750.2	3746.8	3750.8
1801+00	3750.1	3746.8	3750.7
1802+00	3750.2	3746.7	3750.7
1803+00	3750.2	3746.7	3750.7
1804+00	3750.2	3746.6	3750.6
1805+00	3750.1	3746.6	3750.6
1806+00	3750.0	3746.6	3750.6
1807+00	3750.1	3746.5	3750.5
1808+00	3750.1	3746.5	3750.5
1809+00	3750.3	3746.5	3750.5
1810+00	3750.4	3746.5	3750.5
1811+00	3750.2	3746.4	3750.4
1812+00	3750.2	3746.4	3750.4
1813+00	3750.2	3746.4	3750.4
1814+00	3750.1	3746.4	3750.3
1815+00	3750.1	3746.3	3750.3
1816+00	3750.0	3746.1	3750.2
1817+00	3750.1	3746.0	3750.2
1818+00	3749.8	3745.8	3750.1
1819+00	3749.8	3745.7	3750.0
1820+00	3749.5	3745.7	3749.9

EAST LEVEE			
Station	**Existing Ground**	**100-YR WSE**	**Proposed Ground**
1821+00	3749.5	3745.6	3749.9
1822+00	3749.8	3745.5	3749.8
1823+00	3749.5	3745.4	3749.7
1824+00	3749.4	3745.3	3749.6
1825+00	3749.5	3745.3	3749.6
1826+00	3749.5	3745.2	3749.5
1827+00	3749.4	3745.1	3749.4
1828+00	3749.3	3745.1	3749.3
1829+00	3749.2	3745.0	3749.3
1830+00	3749.2	3744.9	3749.2
1831+00	3749.1	3744.9	3749.1
1832+00	3749.1	3744.8	3749.0
1833+00	3749.0	3744.8	3749.0
1834+00	3748.9	3744.7	3748.9
1835+00	3748.7	3744.6	3748.8
1836+00	3748.6	3744.6	3748.7
1837+00	3748.7	3744.5	3748.7
1838+00	3748.7	3744.4	3748.6
1839+00	3748.5	3744.4	3748.5
1840+00	3748.4	3744.3	3748.4
1841+00	3748.2	3744.3	3748.4
1842+00	3748.0	3744.2	3748.3
1843+00	3747.9	3744.2	3748.2
1844+00	3747.8	3744.1	3748.1
1845+00	3747.9	3744.0	3748.1
1846+00	3747.8	3744.0	3748.0
1847+00	3747.6	3744.0	3747.9
1848+00	3747.4	3743.9	3747.8
1849+00	3747.4	3743.9	3747.8
1850+00	3747.5	3743.8	3747.7
1851+00	3747.6	3743.8	3747.6
1852+00	3747.4	3743.7	3747.5
1853+00	3747.5	3743.6	3747.5
1854+00	3747.4	3743.6	3747.4
1855+00	3747.3	3743.6	3747.3
1856+00	3747.2	3743.5	3747.2
1857+00	3747.1	3743.5	3747.1
1858+00	3747.1	3743.4	3747.0
1859+00	3747.0	3743.2	3746.9
1860+00	3746.6	3743.0	3746.8
1861+00	3746.4	3742.9	3746.7
1862+00	3746.2	3742.8	3746.6
1863+00	3746.1	3742.8	3746.5
1864+00	3746.0	3742.7	3746.4

EAST LEVEE			
Station	**Existing Ground**	**100-YR WSE**	**Proposed Ground**
1865+00	3746.0	3742.6	3746.3
1866+00	3746.1	3742.6	3746.2
1867+00	3745.6	3742.6	3746.1
1868+00	3745.6	3742.5	3746.0
1869+00	3745.6	3742.5	3745.9
1870+00	3745.8	3742.5	3745.8
1871+00	3745.7	3742.4	3745.8
1872+00	3745.7	3742.4	3745.7
1873+00	3745.7	3742.3	3745.7
1874+00	3745.8	3742.2	3745.6
1875+00	3745.7	3742.2	3745.6
1876+00	3745.6	3742.2	3745.5
1877+00	3745.5	3742.1	3745.5
1878+00	3745.6	3742.1	3745.4
1879+00	3745.5	3742.1	3745.4
1880+00	3745.5	3742.1	3745.3
1881+00	3745.4	3742.0	3745.3
1882+00	3745.5	3742.0	3745.2
1883+00	3745.4	3741.9	3745.2
1884+00	3745.5	3741.8	3745.1
1885+00	3745.3	3741.6	3745.1
1886+00	3745.2	3741.5	3745.0
1887+00	3745.0	3741.4	3745.0
1888+00	3745.2	3741.4	3744.9
1889+00	3745.1	3741.4	3744.9
1890+00	3744.7	3741.3	3744.8
1891+00	3744.8	3741.3	3744.8
1892+00	3744.8	3741.3	3744.7
1893+00	3744.7	3741.2	3744.7
1894+00	3744.6	3741.1	3744.6
1895+00	3744.5	3741.1	3744.6
1896+00	3744.7	3741.1	3744.5
1897+00	3744.6	3741.0	3744.5
1898+00	3744.6	3741.0	3744.4
1899+00	3744.4	3741.0	3744.4
1900+00	3744.5	3741.0	3744.3
1901+00	3744.2	3740.9	3744.3
1902+00	3744.1	3740.9	3744.2
1903+00	3743.9	3740.9	3744.2
1904+00	3743.9	3740.8	3744.2
1905+00	3744.8	3740.8	3744.2
1906+00	3750.1	3740.7	3750.1
1907+00	3752.2	3740.7	-
1908+00	3748.2	3740.6	3748.7

EAST LEVEE			
Station	**Existing Ground**	**100-YR WSE**	**Proposed Ground**
1909+00	3744.5	3740.6	3745.0
1910+00	3743.5	3740.6	3744.0
1911+00	3743.1	3740.5	3744.0
1912+00	3743.2	3740.5	3743.9
1913+00	3743.2	3740.4	3743.8
1914+00	3742.9	3740.4	3743.7
1915+00	3742.9	3740.3	3743.6
1916+00	3742.8	3740.3	3743.6
1917+00	3743.0	3740.3	3743.5
1918+00	3742.9	3740.2	3743.5
1919+00	3742.7	3740.2	3743.4
1920+00	3742.6	3740.0	3743.4
1921+00	3742.4	3739.9	3743.3
1922+00	3742.3	3739.8	3743.2
1923+00	3742.1	3739.8	3743.2
1924+00	3742.3	3739.8	3743.1
1925+00	3742.2	3739.7	3743.1
1926+00	3742.1	3739.7	3743.0
1927+00	3742.1	3739.7	3743.0
1928+00	3742.1	3739.7	3743.0
1929+00	3742.2	3739.6	3743.0
1930+00	3742.3	3739.6	3743.0
1931+00	3743.6	3739.6	3743.8
1932+00	3746.2	3739.5	3746.3
1933+00	3747.2	3739.4	-
1934+00	3746.8	3739.2	3746.0
1935+00	3744.7	3739.2	3744.5
1936+00	3743.3	3739.1	3743.0
1937+00	3742.7	3739.0	3742.5
1938+00	3742.7	3739.0	3742.5
1939+00	3742.7	3739.0	3742.5
1940+00	3742.5	3739.0	3742.5
1941+00	3742.4	3738.9	3742.5
1942+00	3742.8	3738.9	3742.5
1943+00	3742.5	3738.9	3742.5
1944+00	3742.6	3738.8	3742.5
1945+00	3742.5	3738.8	3742.5
1946+00	3742.6	3738.8	3742.5
1947+00	3742.4	3738.8	3742.4
1948+00	3741.7	3738.8	3742.3
1949+00	3741.6	3738.7	3742.2
1950+00	3741.7	3738.7	3742.1
1951+00	3741.9	3738.7	3742.0
1952+00	3741.9	3738.7	3742.0

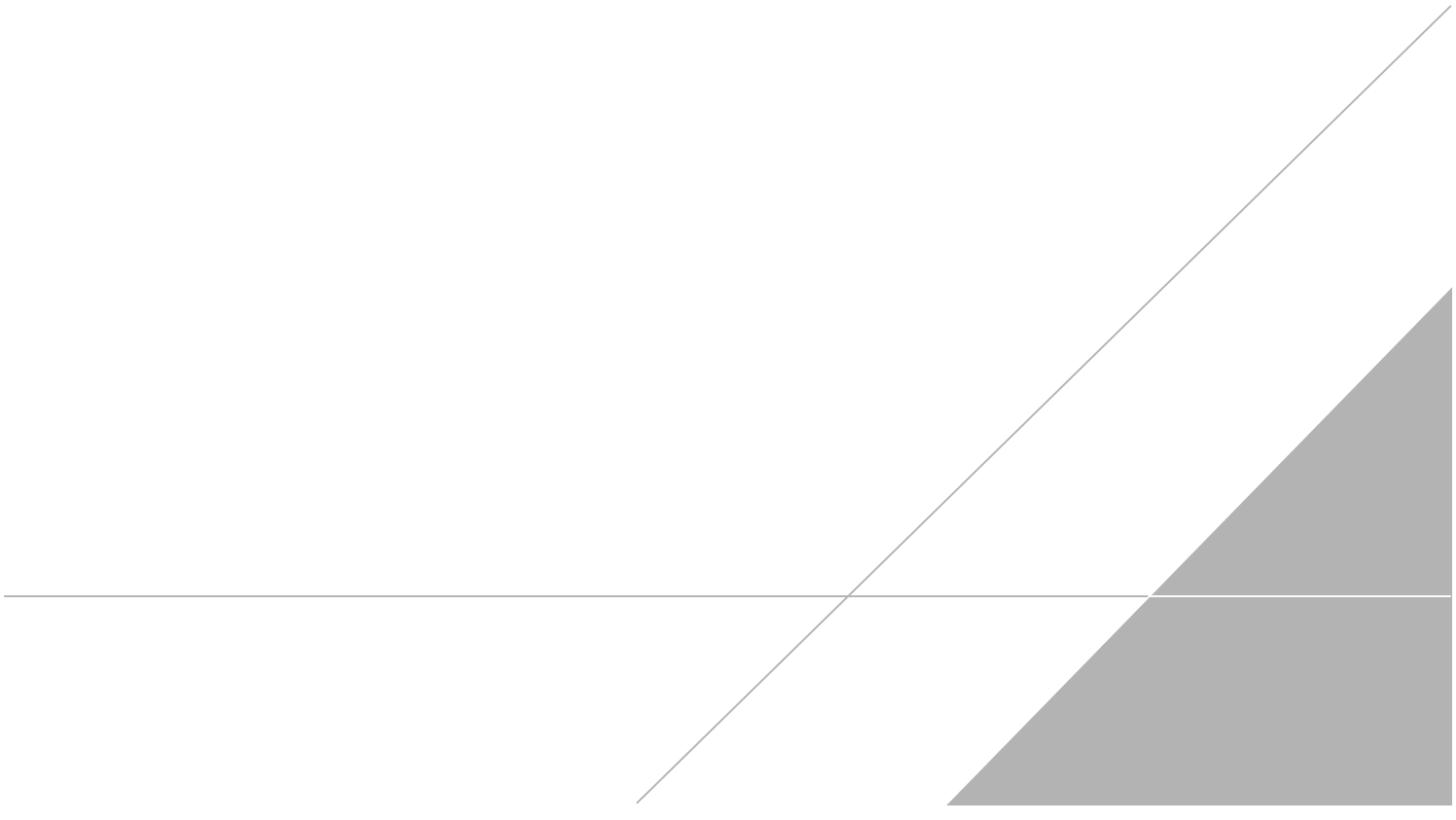
EAST LEVEE			
Station	**Existing Ground**	**100-YR WSE**	**Proposed Ground**
1953+00	3742.2	3738.6	3742.0
1954+00	3742.4	3738.5	3742.0
1955+00	3742.4	3738.4	3742.0
1956+00	3742.2	3738.4	3742.0

WEST LEVEE			
Station	**Existing Ground**	**100-YR WSE**	**Proposed Ground**
1667+00	3755.2	3753.6	3754.9
1668+00	3755.6	3753.6	3756.1
1669+00	3756.3	3753.6	3757.0
1670+00	3756.9	3753.7	3757.1
1671+00	3757.1	3753.7	3757.1
1672+00	3757.3	3753.7	3757.2
1673+00	3757.7	3753.7	3757.2
1674+00	3757.6	3753.6	3757.3
1675+00	3757.5	3753.5	3757.3
1676+00	3757.6	3753.4	3757.4
1677+00	3757.7	3753.4	3757.4
1678+00	3758.0	3753.3	3757.5
1679+00	3758.1	3753.3	3757.5
1680+00	3758.1	3753.2	3757.5
1681+00	3758.2	3753.2	3757.5
1682+00	3757.9	3753.1	3757.5
1683+00	3757.9	3753.1	3757.5
1684+00	3757.9	3753.0	3757.5
1685+00	3757.6	3753.0	3757.5
1686+00	3757.6	3752.9	3757.5
1687+00	3757.8	3752.9	3757.5
1688+00	3757.9	3752.8	3757.4
1689+00	3757.8	3752.8	3757.2
1690+00	3757.6	3752.7	3757.0
1691+00	3757.2	3752.5	3756.9
1692+00	3756.8	3752.5	3756.7
1693+00	3756.9	3752.4	3756.5
1694+00	3756.7	3752.4	3756.5
1695+00	3756.8	3752.3	3756.5
1696+00	3756.8	3752.3	3756.5
1697+00	3755.9	3752.2	3756.0

APPENDIX C

Appendix C - Structural Basis of Design

Appendix C.1 Low Structures
Appendix C.2 Levee Closure Alternatives
Appendix C.3 Levee Closure Structures
Appendix C.4 Wall Raise



Appendix C.1: Low Structures

100 % Design Report

for

USIBWC – Sunland Park Levee Rehabilitation Design Canutillo East 2 (CE2)

STATE OF TEXAS
JOSE L. REYES
91964
LICENSED
PROFESSIONAL ENGINEER

1/28/2022

28 January 2022

This Work Plan outlines the support the Munoz & Dannenbaum JV is providing Arcadis US as the Prime A-E under contract with the U.S. International Boundary & Water Commission (USIBWC) on the Sunland Park Levee Rehabilitation Design Project # IBM1704

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1 INTRODUCTION

The Sunland Park Levee Rehabilitation 100% Design Report is intended to provide more detailed documentation as it pertains to the design assistance/support the Munoz & Dannenbaum JV (MDJV) continues to provide Arcadis US, who is the prime consultant on the aforementioned levee rehabilitation design initiative, during the design phase of this project.

2 PROJECT OVERVIEW

The design of the Sunland Park Levee Repair is an integral part of the Rio Grande Canalization Project. Repair improvements of the Sunland Park Levee Segments include the Canutillo East 2 (CE2) and of Canutillo West 2 (CW2). However, this report focuses only on those structures associated with the Canutillo East (CE2) levee.

3 PROJECT GOAL

All design associated with this project shall comply with FEMA requirements in order to adhere to 44 CFR §65.10. The proposed improvements to the Canutillo East 2 (CE2) levee structures include a proposed catwalk from the levee to the Montoya Lateral Sluice Gate. The above proposed improvements are necessary in order to allow for the safe operation of the structure during a river event (100-Year). Currently the head wall is almost 2'-8" below the 100-Year Water Surface Elevation (WSE). Although the gate operator is slightly above the 100-Year WSE, the individual that would operate the gate during a flood event would be exposed to swift currents; therefore, in danger of being swept away and potentially drowning.

4 PROJECT BACKGROUND

The United States Section of the International Boundary and Water Commission (USIBWC) contracted the services of Arcadis US, to provide a design to rehabilitate the recently completed (November 2012) Sunland Park Levee segment of the Rio Grande Canalization Project. The scope of services includes rehabilitation of multiple structures along this segment of the levee in order for them to perform satisfactorily during the 100-year flood event as required per 44 CFR §65.10. The structure that is included in this report is the Montoya Lateral WW # 36.

5 PROJECT REQUIREMENTS

Repair through rehabilitation of the levee will be required along the entire length of the Sunland Park Levee segments. While Remedial Concept # 4 – Slope Blanket in Kenall's forensic report was an option considered during the concept phase of the project, the Arcadis design team was required to evaluate multiple designs that fully meet 44 CFR §65.10 and addresses the deficiencies noted in the forensic report.

The proposed design of the new levee system must meet or exceed all of the requirements of the U.S. Army Corps of Engineers' Engineering Manual 1110-2-1913, which is entitled "Design and Construction of Levees."

The reaches within the Sunland Park Levee Segment are as follows:

USIBWC stationing as reflected below were used in development of this report:

Reach Name	*Starting Station*	*Ending Station*
CE2	1510+19.16	1956+50
CW2	1518+56.27	1698+00

6 DESIGN AND OTHER REQUIREMENTS

Key assumptions/constraints associated with this 100% Design Report are as indicated below. The design criteria/standards utilized during the design phases of this project include, but are not limited to, those reflected in **Enclosure** "**A**" of this report.

6.1 DESIGN REQUIREMENTS

The tables below reflect additional information regarding this flood control structure, existing top of structure elevations according to the most recent FXSA topographic survey and also the 100-Year WSE) Recommended improvements are also included.

6.1.1 FLOOD CONTROL & IRRIGATION STRUCTURES:

In accordance with 44 CFR §65.10 structures such as these that pass through the levee, need to be controlled during flood events (100-Year WSE). The structure noted below requires that access be provided during flood events. Therefore, the proposed improvements are as described in the paragraphs below.

6.1.1.1 MONTOYA LATERAL WW# 36 (48" RCP)

Because the top of the existing sluice gate at the Montoya Lateral WW#36 on the riverside needs to be accessible from a height above the 100-Year WSE, the design proposes constructing a catwalk, a landing area and a longer sluice gate guide posts for operational purposes. During the design phase of this project, it was determined that the head frame did not need to be modified; however, the sluice gate guide posts would have to be extended. The current height of the operator mechanism is approximately 40-inches above the existing headwall.

During the design phase of this project, and through feedback provided by USIBWC, it was determined that the head frame of the structure did not need to be modified. The existing gate Operator mechanism is approximately 40-inches above the existing headwall and slightly above the 100-Year WSE. Currently, personnel needed to operate the gate would be standing in 3'-8" deep water and dangerous swift currents resulting from a 100-Year flood event which could sweep a person in the river current. A catwalk is needed at this location in order to keep individuals operating the sluice gate above the 100-Year WSE during a river event. This is required in order for USIBWC personnel to be able to safely access to gate operator. Additionally, the sluice gate guide posts need to be extended in order to be able to raise the gate operator above the rail elevation of the catwalk platform.

Table 1 Flood Control & Irrigation Structures

Description	100-Year WSE	FXSA Top of Structure Elev.	Recommended Improvements
1- 48" RCP Montoya Lateral WW# 36	**3748.72**	**3745.06**	See Para. 6.1.1.1 above

6.2 CONSTRUCTION CONSIDERATIONS

6.2.1 UTILITY CONFLICTS

None are expected that would conflict with the proposed improvements at the Montoya Lateral WW #36 structure.

6.2.2 OTHER POTENTIAL CONFLICTS

The items noted below were given consideration and found not to be of risk to the construction of the structures that are proposed for the Montoya Lateral WW #36.

- Construction outside USIBWC's ROW.
- Items that require Long Lead Time.

6.2.3 SAFETY HAZARDS

Typical with any heavy civil construction project. However, the levee construction contractor and its subcontractor(s) will have to adhere to all OSHA safety standards, USACE EM 385-1-1 (Safety and Health Requirements Manual), and other Federal Government job safety requirements.

6.2.4 ESTIMATED CONSTRUCTION DURATION

The proposed improvements associated with the Montoya Lateral WW#36 were estimated to take between 75 to 120 days to complete. This assumes that the initial 30 – 45 day period from receipt of the notice to proceed (NTP) would be dedicated to preparation of necessary submittal(s), and subsequent review and approval by the USIBWC. The remaining 45 – 75 days would include: fabrication, delivery, and the on site construction of the structure. The levee construction contractor will most likely self-perform or subcontract the work associated with this structure, which is estimated to occur concurrently with the levee construction. Therefore, production rates of equipment, etc. would not fall on critical path which will primarily consist of the overall levee rehabilitation work.

6.2.5 TECHNICAL EVALUATION REQUIREMENTS

As noted in Paragraph 6.2.4 above, the levee construction contractor will most likely self-perform or subcontract the work associated with the Montoya Lateral WW#36 structure concurrent with levee work. Therefore, the technical evaluation requirements associated with above noted structure would be to demonstrate having self-performed at least ten (10) similar structures during the last five (5) years.

ENCLOSURE A : REFERENCES & DESIGN CRITERIA/STANDARDS

The following table summarizes the design criteria/standards that will be utilized in the improvements to the flood and irrigation control and roadway structures noted above in this report.

Table 2 – Design Criteria / Standards

Document Name and Version	Description
44 CFR § 65.10 (Oct. 2006)	FEMA's Criteria for Accrediting Levee Systems on National Flood Insurance Program
EC 110-2-6067 (Aug. 2010)	USACE Engineering Circular – policy for Evaluations Performed by USACE for Accreditation of Levee System(s) Under NFIP Administered by FEMA
EM 1110-2-1913 (Apr. 2000)	USACE Engineering Manual for Design & Construction of Levees
EM 1110-2-2002 (June 1995)	USACE Engineering Manual for Evaluation of Repair of Concrete Structures
EM 1110-2-2100 (Dec. 2005 or Latest Edition)	USACE Engineering Manual for Stability analysis of Concrete Structures
EM 1110-2-2400 (Jun. 2003)	USACE Engineering Manual for Retaining & Flood Walls - Structural Design & Evaluation of Outlet Works
ACI 350 (Jan. 2006)	American Concrete Institute (ACI) – Details and Detailing of Concrete Reinforcement
AISC 303 (2016 or Latest Edition)	American Institute of Steel Construction (AISC) – Code of Standard Practice for Steel Buildings and Bridges
AISC 360 (Jun. 2018)	American Institute of Steel Construction (AISC) – Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings
ASCE 7 (2017)	Minimum Design Loads for Buildings and other Structures

ENCLOSURE B: STRUCTURAL DESIGN ASSUMPTIONS & CALCULATIONS

Sunland Park Levee Rehabilitation Project

Catwalk Design

This section presents the calculation and design of catwalks based on our scope of work defined in the modification to the Sunland park levee rehabilitation project. The calculation covers the following sections. Live load, flood load and wind load are derived from ASCE 7-10. Detailed calculation of the applied loads are presented in the following pages. Member control calculations are based on Steel construction manual AISC LRFD method.

- Live load requirement
- Flood Load Calculation
- Wind Calculation
- STAAD Input & Output
- STAAD Input Text File
- Members Controls

Table 4-1 Minimum Uniformly Distributed Live Loads, L_o, and Minimum Concentrated Live Loads

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lb (kN)
Apartments (see Residential)		
Access floor systems		
Office use	50 (2.4)	2,000 (8.9)
Computer use	100 (4.79)	2,000 (8.9)
Armories and drill rooms	150 (7.18)[a]	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)[a]	
Lobbies	100 (4.79)[a]	
Movable seats	100 (4.79)[a]	
Platforms (assembly)	100 (4.79)[a]	
Stage floors	150 (7.18)[a]	
Balconies and decks	1.5 times the live load for the occupancy served. Not required to exceed 100 psf (4.79 kN/m^2)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Other floors, same as occupancy served except as indicated		
Dining rooms and restaurants	100 (4.79)[a]	
Dwellings (see Residential)		
Elevator machine room grating (on area of 2 in. by 2 in. (50 mm by 50 mm))		300 (1.33)
Finish light floor plate construction (on area of 1 in. by 1 in. (25 mm by 25 mm))		200 (0.89)
Fire escapes	100 (4.79)	
On single-family dwellings only	40 (1.92)	
Fixed ladders	See Section 4.5	
Garages		
Passenger vehicles only	40 (1.92)[a,b,c]	
Trucks and buses	[c]	
Handrails, guardrails, and grab bars	See Section 4.5	
Helipads	60 (2.87)[d,e] Nonreducible	[e,f,g]
Hospitals		
Operating rooms, laboratories	60 (2.87)	1,000 (4.45)
Patient rooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Hotels (see Residential)		
Libraries		
Reading rooms	60 (2.87)	1,000 (4.45)
Stack rooms	150 (7.18)[a,h]	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Manufacturing		
Light	125 (6.00)[a]	2,000 (8.90)
Heavy	250 (11.97)[a]	3,000 (13.40)

Continued

CALCULATION SHEET

JOB NO. 4839-02

SIGNATURE SG	**10/16/2018**	CHECKED	DATE
PROJECT : Sunland Park-Catwalk Design			
SUBJECT: Flood Load Calculation		SHEET _____ OF _____	

Flood Load Calculation is Based on ASCE 7-10, Chapter 5

Flood load calcuation parameters:

Base Flood Elevation (BFE)	3748.72	ft	
Ground Level (G)	3739	ft	
d_s= 0.65 (BFE-G)	6.32	ft	ASCE 7-10, 5.4-3
C_D	2.25		ASCE 7-10, 5.4.4.1
D	0.96	ft	ASCE 7-10, 5.4.4.1
γ_w	64.00	pcf	ASCE 7-10, 5.4.4.1
$H_b = 0.78\ d_s$	4.93	ft	ASCE 7-10, 5.4-2

Flood Load

$F_D = 0.5*\gamma_w*C_D*D*H_b^2$	1682.99	lb

CALCULATION SHEET

JOB NO. 4839-02

SIGNATURE SG	**10/16/2018**	CHECKED	DATE
PROJECT : Sunland Park-Catwalk Design			
SUBJECT: Flood Load Calculation		SHEET _____ OF _____	

Flodd load by Individual Member:

Pile or Column Height	8.00	ft
F_D	210.37	lb/ft

CALCULATION SHEET

JOB NO. 4839-02

SIGNATURE SG	**10/16/2018**	CHECKED	DATE
PROJECT : Sunland Park-Catwalk Design			
SUBJECT: Wind Calculation		SHEET _____ OF _____	

Wind Calculation is Based on ASCE 7-10, Chapters 26 and 27

Note: Only the wind load normal to the fence will be considered in the design of concrete foundation

Wind load calcuation parameters:

Basic wind speed V:	115	mph	ASCE 7-10, P247a
Velocity Pressure Exposure Co. Kz:	1.08		ASCE 7-10, Table 27.3-1
Topographic Factor: Kzt:	1		ASCE 7-10, 26.8.2
Gust Effect G :	1		ASCE 7-10, 26.9
Wind Directionality Factor Kd:	0.85		ASCE 7-10, Table 26.6-1

Velocity Pressure:

qz=0.00256*Kz*Kzt*Kd*V^2=	31.08	lb/ft^2

Force Coeff. For single member Cf:	2	Figure 6-21	ASCE 7-05

CALCULATION SHEET

JOB NO. 4839-02

SIGNATURE SG	**10/16/2018**	CHECKED	DATE
PROJECT : Sunland Park-Catwalk Design			
SUBJECT: Wind Calculation		SHEET _____ OF _____	

Wind load by Individual Member:

qz*G*Cf=	62.16	lb/ft^2

Width of W8X40, W=	8.25	in
Wind Laod, q=	42.7	lb/ft
Width of Channel C5X9, W=	11.00	in
Wind Laod, q=	56.98	lb/ft

Main Platform Design

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 5/3/2021
JOB NAME Sunland Park_Montoya Lateral Catwalk
JOB NO 4839-02
ENGINEER NAME SG
JOB CLIENT USIBWC
END JOB INFORMATION
INPUT WIDTH 79
UNIT FEET KIP
JOINT COORDINATES
3 20 6 -2; 4 30 6 -2; 12 20 6 2; 13 30 6 2; 20 20 6 0; 21 30 6 0; 23 20 -3 0;
24 30 -3 0;
MEMBER INCIDENCES
3 3 4; 11 12 13; 18 23 20; 19 24 21; 22 3 20; 23 20 12; 24 4 21; 25 21 13;
DEFINE MATERIAL START
ISOTROPIC STEEL
E 4.176e+006
POISSON 0.3
DENSITY 0.489024
ALPHA 6.5e-006
DAMP 0.03
END DEFINE MATERIAL
MEMBER PROPERTY AMERICAN
3 11 TABLE ST C5X9
18 19 TABLE ST W8X40
22 TO 25 TABLE LD L40406 SP 0.6875
CONSTANTS
BETA 180 MEMB 3
BETA 0 MEMB 22 TO 25
MATERIAL STEEL ALL
SUPPORTS
3 23 24 FIXED
12 13 FIXED BUT MX MY MZ
MEMBER RELEASE
22 24 START MY MZ
23 25 END MY MZ
LOAD 1 LOADTYPE None  TITLE DEAD LOAD
SELFWEIGHT Y -1
****Grating load, assume 15 psf
MEMBER LOAD
3 11 UNI GY -0.016
*****Live load assume 40 psf
LOAD 2 LOADTYPE None  TITLE LIVE LOAD
MEMBER LOAD
3 11 UNI GY -0.04
*****Wind Load, assume 50 psf
LOAD 3 LOADTYPE None  TITLE WIND LOAD
MEMBER LOAD
3 UNI GZ 0.057
*
18 UNI GZ 0.042 8 12
19 UNI GZ 0.042 8 14
*
3 11 UNI GX 0.02
18 UNI GX 0.042 8 12
19 UNI GX 0.042 8 14
22 TO 25 UNI GX 0.02
LOAD 4 LOADTYPE Flood  TITLE FLOOD LOAD
MEMBER LOAD
18 19 UNI GX 0.21
LOAD 10 LOADTYPE None  TITLE 1.2DL+1.6LL_REPEATED
REPEAT LOAD
1 1.2 2 1.6
LOAD 11 LOADTYPE None  TITLE 1.2DL+LL+0.5W+FA_REPEATED
REPEAT LOAD
1 1.2 2 1.0 3 0.5 4 1.0
```

```
LOAD 12 LOADTYPE None  TITLE 0.9DL+0.5W+FA_REPEATED
REPEAT LOAD
1 0.9 3 0.5 4 1.0
LOAD 13 LOADTYPE None  TITLE 1.2DL+0.5W_REPEATED
REPEAT LOAD
1 1.2 3 0.5
LOAD COMB 5 1.2DL+1.6LL
1 1.2 2 1.6
LOAD COMB 6 1.2DL+LL+0.5W+FA
1 1.2 3 0.5 4 1.0 2 1.0
LOAD COMB 7 0.9DL+0.5W+FA
1 0.9 3 0.5 4 1.0
LOAD COMB 8 1.2DL+0.5W
1 1.2 3 0.5
LOAD COMB 9 DEFLECTION CTRL
1 1.0 2 1.0 3 1.0 4 1.0
PDELTA 3 ANALYSIS SMALLDELTA
*
*******************************
PARAMETER 1
CODE LRFD
FU 8640 ALL
FYLD 5184 ALL
KY 2.2 MEMB 18 19 22 TO 25
KZ 2.2 MEMB 18 19 22 TO 25
KY 1 MEMB 3 11
KZ 1 MEMB 3 11
*******For the C5X9, assume column space is 8ft, total 4 columns, instead of 3 columns
LY 8 MEMB 3 11
LZ 6 MEMB 3 11
RATIO 1 ALL
UNB 10 MEMB 3 11
UNT 40 MEMB 3 11
CHECK CODE ALL
***********************************
*
*********************************
*LOAD LIST 8
*PARAMETER 2
*CODE LRFD
*FU 8640 ALL
*FYLD 7200 ALL
*KY 0.5 MEMB 1 TO 4 9 TO 12
*KZ 0.5 MEMB 1 TO 4 9 TO 12
*LY 0.5 MEMB 1 TO 4 9 TO 12
*LZ 0.5 MEMB 1 TO 4 9 TO 12
*UNB 0.5 MEMB 1 TO 4 9 TO 12
*UNT 0.5 MEMB 1 TO 4 9 TO 12
**
*DFF 1000 MEMB 1 TO 4 9 TO 12
*DJ1 1 MEMB 1 TO 4
*DJ2 5 MEMB 1 TO 4
**
*DJ1 10 MEMB 9 TO 12
*DJ2 14 MEMB 9 TO 12
*CHECK CODE ALL
***************************************
PERFORM ANALYSIS PRINT STATICS CHECK
FINISH
FINISH
```

Job Information

	Engineer	Checked	Approved
Name:	SG		
Date:	5/3/2021		

Project ID	
Project Name	

Structure Type	SPACE FRAME

Number of Nodes	8	Highest Node	24
Number of Elements	8	Highest Beam	25

Number of Basic Load Cases	8
Number of Combination Load Cases	5

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	DEAD LOAD
Primary	2	LIVE LOAD
Primary	3	WIND LOAD
Primary	4	FLOOD LOAD
Primary	10	1.2DL+1.6LL_REPEATED
Primary	11	1.2DL+LL+0.5W+FA_REPEATED
Primary	12	0.9DL+0.5W+FA_REPEATED
Primary	13	1.2DL+0.5W_REPEATED
Combination	5	1.2DL+1.6LL
Combination	6	1.2DL+LL+0.5W+FA
Combination	7	0.9DL+0.5W+FA
Combination	8	1.2DL+0.5W
Combination	9	DEFLECTION CTRL

Section Properties

Prop	Section	Area (in^2)	I_{yy} (in^4)	I_{zz} (in^4)	J (in^4)	Material
1	C5X9	2.640	0.624	8.890	0.095	STEEL
2	W8X40	11.700	49.100	146.000	1.064	STEEL
3	L40406 LD	5.720	167.176	8.717	0.268	STEEL

Materials

Mat	Name	E (kip/in^2)	ν	Density (kip/in^3)	α (/°F)
1	STEEL	29E+3	0.300	0.000	6.5E -6
2	STAINLESSSTEEL	28E+3	0.300	0.000	10E -6
3	ALUMINUM	10E+3	0.330	0.000	13E -6
4	CONCRETE	3.15E+3	0.170	0.000	5E -6

Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip-ft/deg)	rY (kip-ft/deg)	rZ (kip-ft/deg)
3	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
12	Fixed	Fixed	Fixed	-	-	-
13	Fixed	Fixed	Fixed	-	-	-
23	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
24	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed

Primary Load Cases

Number	Name	Type
1	DEAD LOAD	None
2	LIVE LOAD	None
3	WIND LOAD	None
4	FLOOD LOAD	Flood
10	1.2DL+1.6LL_REPEATED	None
11	1.2DL+LL+0.5W+FA_REPEATED	None
12	0.9DL+0.5W+FA_REPEATED	None
13	1.2DL+0.5W_REPEATED	None

Combination Load Cases

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
5	1.2DL+1.6LL	1	DEAD LOAD	1.20
		2	LIVE LOAD	1.60
6	1.2DL+LL+0.5W+FA	1	DEAD LOAD	1.20
		3	WIND LOAD	0.50
		4	FLOOD LOAD	1.00
		2	LIVE LOAD	1.00
7	0.9DL+0.5W+FA	1	DEAD LOAD	0.90
		3	WIND LOAD	0.50
		4	FLOOD LOAD	1.00
8	1.2DL+0.5W	1	DEAD LOAD	1.20
		3	WIND LOAD	0.50

Combination Load Cases Cont...

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
9	DEFLECTION CTRL	1	DEAD LOAD	1.00
		2	LIVE LOAD	1.00
		3	WIND LOAD	1.00
		4	FLOOD LOAD	1.00

1 DEAD LOAD : Beam Loads

Beam	Type		Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
3	UNI	lbf/ft	GY	-16.000	-	-	-	-
11	UNI	lbf/ft	GY	-16.000	-	-	-	-

1 DEAD LOAD : Selfweight

Direction	Factor	Assigned Geometry
Y	-1.000	ALL

2 LIVE LOAD : Beam Loads

Beam	Type		Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
3	UNI	lbf/ft	GY	-40.000	-	-	-	-
11	UNI	lbf/ft	GY	-40.000	-	-	-	-

3 WIND LOAD : Beam Loads

Beam	Type		Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
3	UNI	lbf/ft	GZ	57.000	-	-	-	-
	UNI	lbf/ft	GX	20.000	-	-	-	-
11	UNI	lbf/ft	GX	20.000	-	-	-	-
18	UNI	lbf/ft	GZ	42.000	8.000	-	12.000	-
	UNI	lbf/ft	GX	42.000	8.000	-	12.000	-
19	UNI	lbf/ft	GZ	42.000	8.000	-	14.000	-
	UNI	lbf/ft	GX	42.000	8.000	-	14.000	-
22	UNI	lbf/ft	GX	20.000	-	-	-	-
23	UNI	lbf/ft	GX	20.000	-	-	-	-
24	UNI	lbf/ft	GX	20.000	-	-	-	-
25	UNI	lbf/ft	GX	20.000	-	-	-	-

4 FLOOD LOAD : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
18	UNI lbf/ft	GX	210.000	-	-	-	-
19	UNI lbf/ft	GX	210.000	-	-	-	-

10 1.2DL+1.6LL_REPEATED : Repeat Loads

Ref	Name	Factor
1	DEAD LOAD	1.200
2	LIVE LOAD	1.600

11 1.2DL+LL+0.5W+FA_REPEATED : Repeat Loads

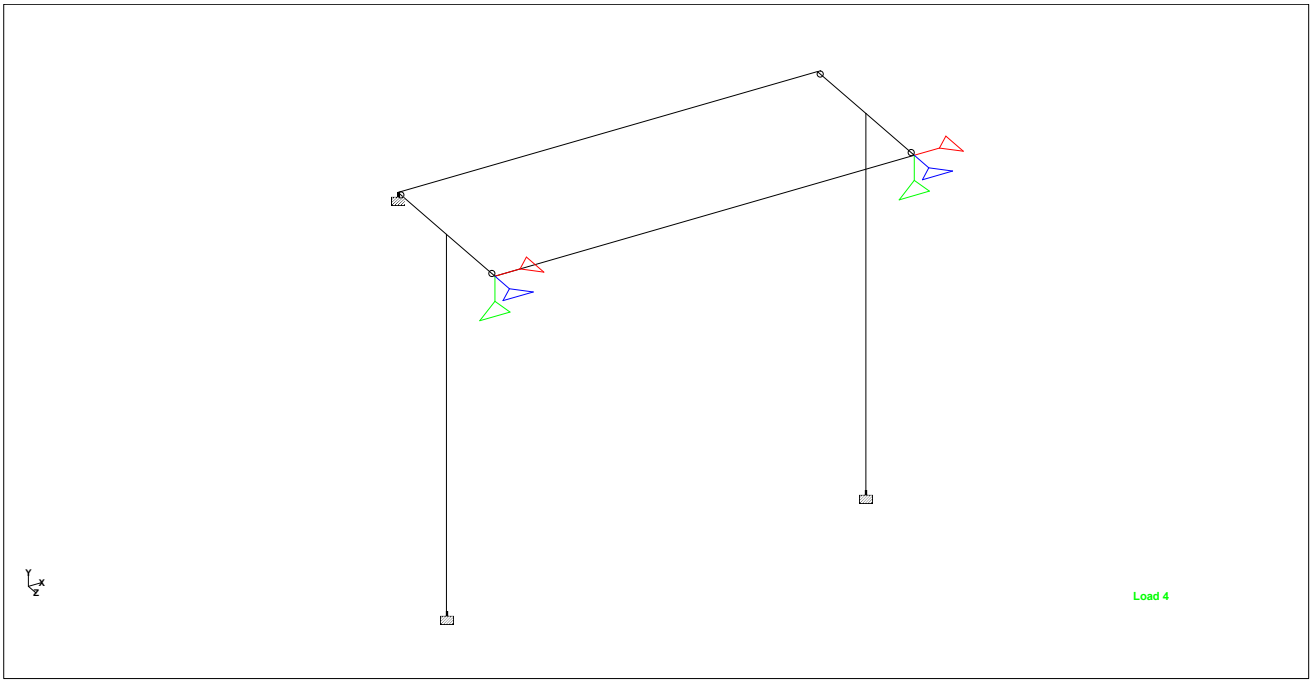
Ref	Name	Factor
1	DEAD LOAD	1.200
2	LIVE LOAD	1.000
3	WIND LOAD	0.500
4	FLOOD LOAD	1.000

12 0.9DL+0.5W+FA_REPEATED : Repeat Loads

Ref	Name	Factor
1	DEAD LOAD	0.900
3	WIND LOAD	0.500
4	FLOOD LOAD	1.000

13 1.2DL+0.5W_REPEATED : Repeat Loads

Ref	Name	Factor
1	DEAD LOAD	1.200
3	WIND LOAD	0.500



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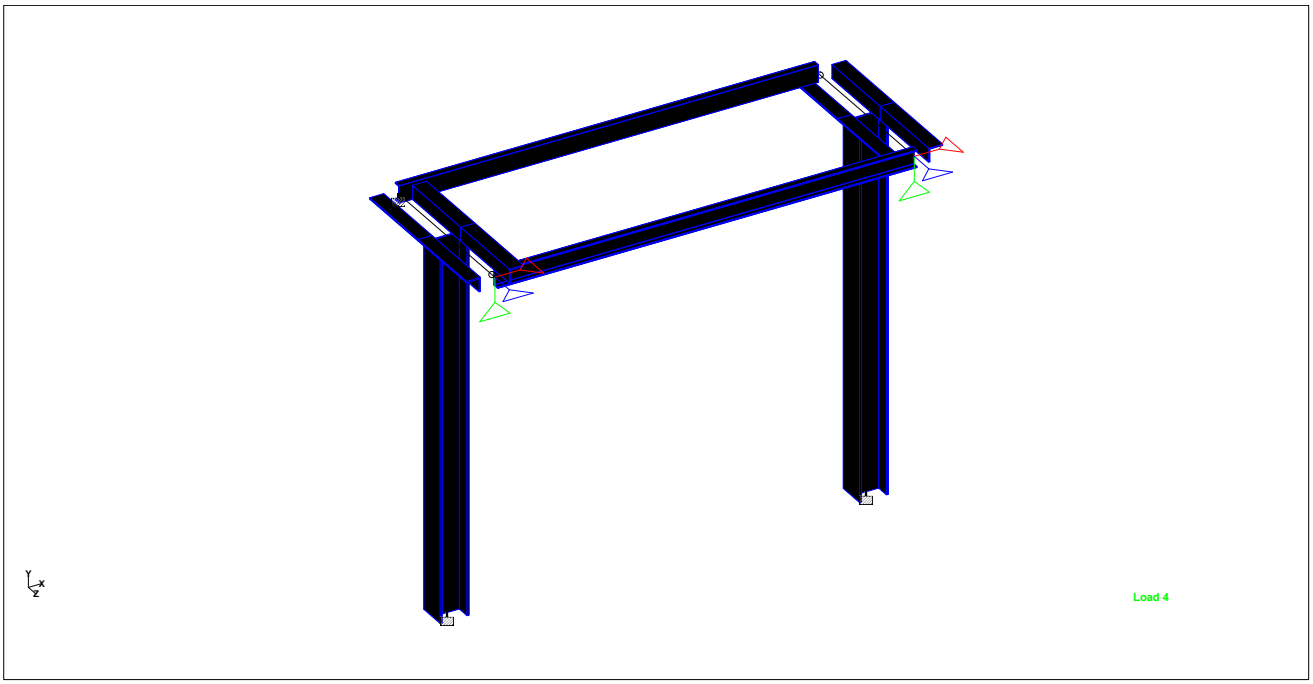

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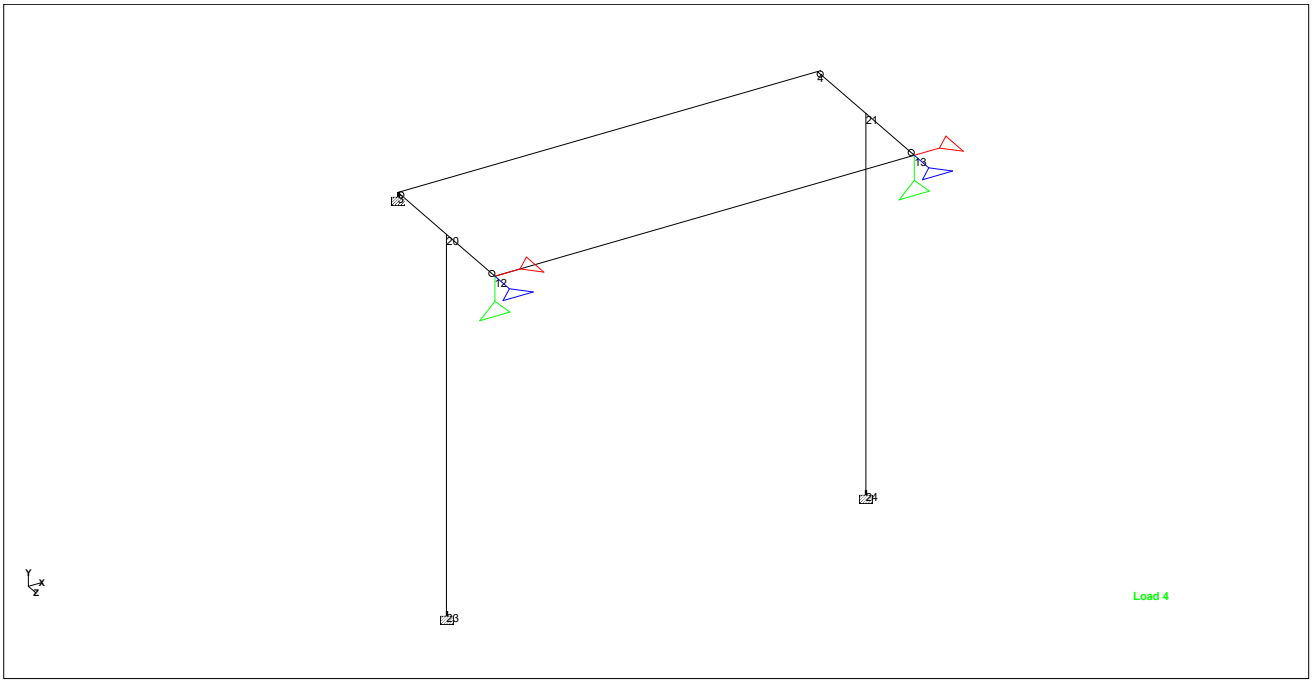

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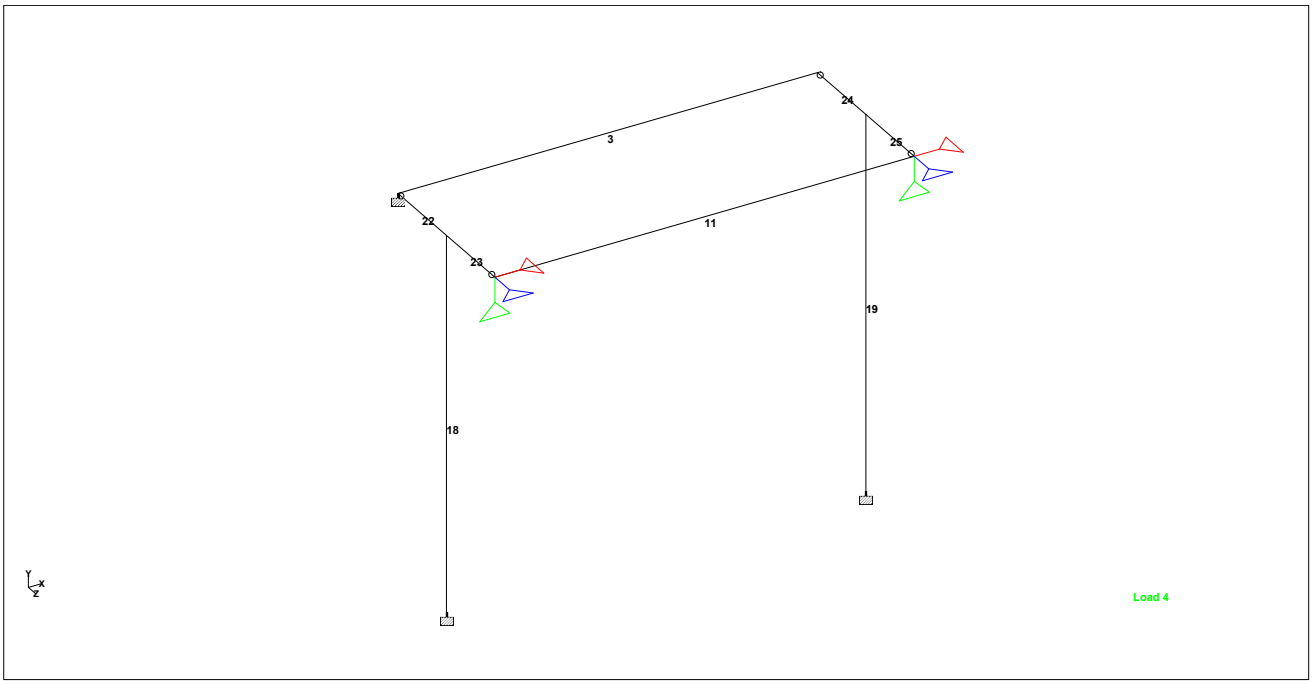

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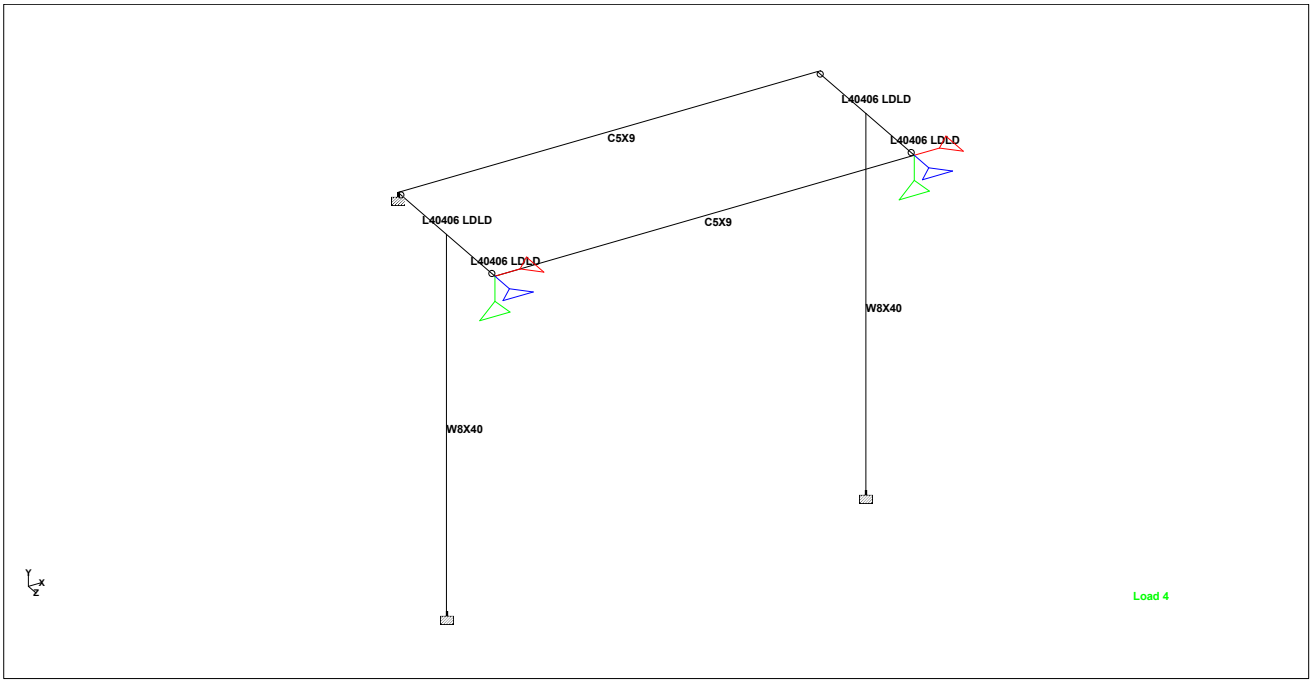


5

Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

				Axial	Shear		Torsion	Bending	
	Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
Max Fx	19	24	10:1.2DL+1.6L	**0.981**	0.005	-0.080	-0.000	2.844	0.166
Min Fx	3	3	9:DEFLECTIOI	**-0.610**	-0.408	0.356	0.000	-8.547	-10.060
Max Fy	19	24	9:DEFLECTIOI	0.753	**1.192**	-0.059	0.002	2.092	26.186
Min Fy	18	20	9:DEFLECTIOI	0.041	**-0.750**	0.040	-0.000	0.115	0.210
Max Fz	22	3	9:DEFLECTIOI	-0.020	0.021	**0.415**	-0.013	0.000	0.000
Min Fz	23	12	9:DEFLECTIOI	0.020	-0.016	**-0.415**	0.196	-0.000	-0.000
Max Mx	23	20	10:1.2DL+1.6L	0.000	0.024	-0.002	**0.265**	0.042	0.028
Min Mx	25	21	10:1.2DL+1.6L	-0.080	0.155	0.003	**-0.264**	-0.061	3.163
Max My	22	20	9:DEFLECTIOI	-0.020	-0.018	0.375	-0.013	**9.478**	-0.034
Min My	3	3	3:WIND LOAD	-0.258	-0.000	0.356	0.000	**-8.547**	-0.003
Max Mz	19	24	9:DEFLECTIOI	0.753	1.192	-0.059	0.002	2.092	**26.186**
Min Mz	3	3	10:1.2DL+1.6L	0.003	-0.590	-0.000	0.000	0.000	**-14.529**

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
3	1:DEAD LOAD	0.000	0.175	0.000	0.000	-0.000	3.879
	2:LIVE LOAD	0.000	0.251	0.000	0.000	-0.000	6.172
	3:WIND LOAD	-0.317	0.002	-0.376	0.000	8.547	0.003
	4:FLOOD LOAI	-0.708	0.000	-0.000	0.000	0.000	-0.007
	10:1.2DL+1.6LI	0.001	0.612	0.000	0.000	-0.000	14.530
	11:1.2DL+LL+0	-0.866	0.463	-0.188	0.000	4.273	10.821
	12:0.9DL+0.5W	-0.867	0.159	-0.188	0.000	4.273	3.485
	13:1.2DL+0.5W	-0.158	0.212	-0.188	0.000	4.273	4.656
	5:1.2DL+1.6LL	0.001	0.612	0.000	0.000	-0.000	14.530
	6:1.2DL+LL+0.:	-0.866	0.463	-0.188	0.000	4.273	10.821
	7:0.9DL+0.5W-	-0.867	0.159	-0.188	0.000	4.273	3.485
	8:1.2DL+0.5W	-0.158	0.212	-0.188	0.000	4.273	4.656
	9:DEFLECTIOI	-1.025	0.429	-0.376	0.000	8.547	10.046
12	1:DEAD LOAD	-0.000	0.143	0.000	0.000	0.000	0.000
	2:LIVE LOAD	-0.001	0.200	0.000	0.000	0.000	0.000
	3:WIND LOAD	-0.159	-0.002	-0.020	0.000	0.000	0.000
	4:FLOOD LOAI	-0.355	0.000	0.000	0.000	0.000	0.000
	10:1.2DL+1.6LI	-0.002	0.492	0.000	0.000	0.000	0.000
	11:1.2DL+LL+0	-0.436	0.371	-0.010	0.000	0.000	0.000
	12:0.9DL+0.5W	-0.435	0.128	-0.010	0.000	0.000	0.000
	13:1.2DL+0.5W	-0.080	0.171	-0.010	0.000	0.000	0.000
	5:1.2DL+1.6LL	-0.002	0.492	0.000	0.000	0.000	0.000
	6:1.2DL+LL+0.:	-0.436	0.371	-0.010	0.000	0.000	0.000
	7:0.9DL+0.5W-	-0.435	0.128	-0.010	0.000	0.000	0.000
	8:1.2DL+0.5W	-0.080	0.171	-0.010	0.000	0.000	0.000
	9:DEFLECTIOI	-0.515	0.341	-0.020	0.000	0.000	0.000
13	1:DEAD LOAD	0.001	0.104	0.024	0.000	0.000	0.000
	2:LIVE LOAD	0.001	0.148	0.032	0.000	0.000	0.000
	3:WIND LOAD	-0.158	-0.003	-0.253	0.000	0.000	0.000
	4:FLOOD LOAI	-0.353	-0.000	-0.000	0.000	0.000	0.000
	10:1.2DL+1.6LI	0.003	0.361	0.080	0.000	0.000	0.000
	11:1.2DL+LL+0	-0.431	0.271	-0.066	0.000	0.000	0.000
	12:0.9DL+0.5W	-0.432	0.092	-0.105	0.000	0.000	0.000
	13:1.2DL+0.5W	-0.078	0.124	-0.098	0.000	0.000	0.000
	5:1.2DL+1.6LL	0.003	0.361	0.080	0.000	0.000	0.000
	6:1.2DL+LL+0.:	-0.431	0.271	-0.066	0.000	0.000	0.000
	7:0.9DL+0.5W-	-0.432	0.092	-0.105	0.000	0.000	0.000
	8:1.2DL+0.5W	-0.078	0.124	-0.098	0.000	0.000	0.000
	9:DEFLECTIOI	-0.510	0.249	-0.197	0.000	0.000	0.000
23	1:DEAD LOAD	0.001	0.398	0.000	0.000	0.000	-0.031
	2:LIVE LOAD	0.001	0.000	0.000	0.000	0.000	-0.049
	3:WIND LOAD	-0.004	0.000	-0.002	-0.068	0.000	0.203
	4:FLOOD LOAI	-1.180	0.000	0.000	0.000	0.000	25.449
	10:1.2DL+1.6LI	0.004	0.478	0.000	0.000	0.000	-0.116

Reactions Cont...

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
	11:1.2DL+LL+0	-1.180	0.478	-0.001	-0.034	0.000	25.464
	12:0.9DL+0.5W	-1.182	0.359	-0.001	-0.034	0.000	25.523
	13:1.2DL+0.5W	-0.001	0.478	-0.001	-0.034	0.000	0.065
	5:1.2DL+1.6LL	0.004	0.478	0.000	0.000	0.000	-0.116
	6:1.2DL+LL+0.8	-1.180	0.478	-0.001	-0.034	0.000	25.464
	7:0.9DL+0.5W-	-1.182	0.359	-0.001	-0.034	0.000	25.523
	8:1.2DL+0.5W	-0.001	0.478	-0.001	-0.034	0.000	0.065
	9:DEFLECTIOI	-1.182	0.398	-0.002	-0.068	0.000	25.572
24	1:DEAD LOAD	-0.001	0.549	-0.024	-0.851	-0.000	0.044
	2:LIVE LOAD	-0.002	0.201	-0.032	-1.139	-0.000	0.071
	3:WIND LOAD	-0.005	0.003	-0.003	-0.103	0.001	0.333
	4:FLOOD LOAI	-1.183	-0.000	0.000	0.001	0.001	25.738
	10:1.2DL+1.6LI	-0.005	0.981	-0.080	-2.844	-0.000	0.166
	11:1.2DL+LL+0	-1.190	0.861	-0.062	-2.210	0.002	26.028
	12:0.9DL+0.5W	-1.187	0.495	-0.023	-0.816	0.002	25.944
	13:1.2DL+0.5W	-0.004	0.660	-0.030	-1.072	0.000	0.219
	5:1.2DL+1.6LL	-0.005	0.981	-0.080	-2.844	-0.000	0.166
	6:1.2DL+LL+0.8	-1.190	0.861	-0.062	-2.210	0.002	26.028
	7:0.9DL+0.5W-	-1.187	0.495	-0.023	-0.816	0.002	25.944
	8:1.2DL+0.5W	-0.004	0.660	-0.030	-1.072	0.000	0.219
	9:DEFLECTIOI	-1.192	0.753	-0.059	-2.092	0.002	26.186

CLIENT: USIBWC
PROJECT: SUNLAND PARK LEVEE
SUBJECT: **C8x11.5**

JOB NO.: 4839-02
DESIGNER: SG
DATE: May-21
CHECKER:
DATE:

SECTION PROPERTIES

SECTION TYPE = ***W***

SOIL CAPACITY = *0* tons (0 kips)

F_y =	***36.0*** ksi	E_s =	*29000* ksi	S_x =	*8.140* in^3
h or d	8.00 in	K_x =	*1.2*	S_y =	*0.780* in^3
b_f =	*2.26* in	K_y =	*1.2*	r_x =	*3.110* in
t_f =	*0.39* in	Lu_x =	*7.00* ft	r_y =	*0.623* in
t_w =	*0.22* in	Lu_y =	*7.00* ft	Z_x =	*9.630* in^3
A_s =	*3.38* in^2	L_b =	*7.00* ft	Z_Y =	*1.570* in^3
$b_f/2t_f$ =	*2.90*	h_0 =	*7.610* in	r_{ts} =	*0.756* in
h/tw=	*36.36*	J =	*0.130* in^4	Ix=	*32.600* in^4
A_f =	0.88 in	Cw=	16.50 in^6	Iy=	*1.310* in^4
d/A_f =	*9.08*			S_{xt}=	*0.780* in^3

AXIAL CAPACITY (AISC - LRFD Chapter E)

KL/r = **161.798** 4.71 SQRT(E/Fy) 133.68 $Fe = \pi^2 E_s/[(KL/r)^2]$ = 10.93 ksi

When KL/r =< 4.71 SQRT(E/Fy):

Fcr = [0.658^(Fy/Fe)]Fy = N/A ksi

When C_c > 4.71 SQRT(E/Fy):

Fcr = 0.877 Fe = 9.59 ksi

A.- Maximum Allowable Axial Load:

$\phi P_n = F_{cr}A_g$ = 29.17 kips

FLEXURAL CAPACITY (AISC - LRFD Chapter F)

$b_f/2t_f$ =	*2.90*	<	$0.38\sqrt{E/F_y}$ = 10.79	(Flange is Compact)
h/tw =	*36.36*	<	3.76SQRT(E/Fy) = 106.72	(Web is Compact)

DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND FLANGES

Yielding Limit State :

$M_{nx} = M_p = F_y Z_x$ = 346.68 kips-in
$M_{ny} = M_p = F_y Z_y$ = 56.52 kips-in

B.- Flexural Capacities:

ϕM_{nx}= **25.27** kips-in
ϕM_{ny}= **42.75** kips-in

COMBINED STRESSES (AISC - LRFD Chapter H)

Pr/Pc>=0.2
(H1-1a): (Pr/Pc)+8/9(Mrx/Mcx+Mry/Mcy)<1
Pr/Pc<0.2
(H1-1b): (Pr/2Pc)+(Mrx/Mcx+Mry/Mcy)<1

$P_{applied}$ =	0.50 kips
Mrx =	14.00 kip-in
Mry =	4.27 kip-in
Vu=	0.60 kip

CLIENT: USIBWC
JOB NO.: 4839-02
PROJECT: SUNLAND PARK LEVEE
DESIGNER: SG
DATE: May-21
SUBJECT: **C8x11.5**
CHECKER:
DATE:

(H1-1a): N/A
(H1-1b): 0.662

SHEAR (AISC - LRFD Chapter G)

DOUBLY SYMMETRIC I-SHAPED MEMBERS

a= 5 in

Kv= $5+5/(a/h)^2$ = 17.80

If h/tw ≤ 2.24 SQRT(E/Fy)

Cv= 1

Vn= 0.6FyAwCv= 38.02 kips > Vu, OK

Deflection CTRL

Δ_{max}= 0.056 from STAAD Model
L/480= 0.1875 OK

CLIENT: USIBWC
PROJECT: SUNLAND PARK LEVEE
SUBJECT: **L4x4x0.5**

JOB NO.: 4839-02
DESIGNER: SG
DATE: Oct-18
CHECKER:
DATE:

SECTION PROPERTIES

SECTION TYPE = ***W***

SOIL CAPACITY = *0* tons (0 kips)

F_y =	***36.0*** ksi	E_s =	*29000* ksi	S_x =	*1.960* in^3
h or d	*4.00* in	K_x =	*1.2*	S_y =	*1.960* in^3
b_f =	*4* in	K_y =	*1.2*	r_x =	*1.210* in
t_f =	*0.5* in	Lu_x =	*2.00* ft	r_y =	*1.210* in
t_w =	*0.5* in	Lu_y =	*2.00* ft	Z_x =	*3.500* in^3
A_s =	*3.75* in^2	L_b =	*2.00* ft	Z_Y =	*3.500* in^3
$b_f/2t_f$ =	*4.00*	h_0 =	*0.32* in	r_{ts} =	*2.160* in
h/tw=	*8.00*	J =	*0.32* in^4	Ix=	*5.520* in^4
A_f =	*2.00* in	Cw=	*0.37* in^6	Iy=	*5.520* in^4
d/A_f =	*2.00*			S_{xt}=	*1.960* in^3

AXIAL CAPACITY (AISC - LRFD Chapter E)

KL/r = **23.802** 4.71 SQRT(E/Fy) 133.68 $Fe = \pi^2 E_s/[(KL/r)^2]$ = 505.22 ksi

When KL/r =< 4.71 SQRT(E/Fy):

Fcr = [0.658^(Fy/Fe)]Fy = 34.94 ksi

When C_c > 4.71 SQRT(E/Fy):

Fcr = 0.877 Fe = N/A ksi

A.- Maximum Allowable Axial Load:

$\phi P_n = F_{cr}A_g$ = 117.93 kips

FLEXURAL CAPACITY (AISC - LRFD Chapter F)

$b_f/2t_f$ =	*4.00*	<	0.38*SQRT*(E/F_y) = 10.79	(Flange is Compact)
h/tw =	*8.00*	<	3.76SQRT(E/Fy) = 106.72	(Web is Compact)

B.- Flexural Capacities:

ϕMnx= **113.40** kips-in
ϕMny= **113.40** kips-in

COMBINED STRESSES (AISC - LRFD Chapter H)

Pr/Pc>=0.2
(H1-1a): (Pr/Pc)+8/9(Mrx/Mcx+Mry/Mcy)<1
Pr/Pc<0.2
(H1-1b): (Pr/2Pc)+(Mrx/Mcx+Mry/Mcy)<1

$P_{applied}$ =	0.11 kips
Mrx =	9.00 kip-in
Mry =	8.95 kip-in
Vu=	0.40 kip

(H1-1a): N/A
(H1-1b): 0.159

SHEAR (AISC - LRFD Chapter G)

DOUBLY SYMMETRIC I-SHAPED MEMBERS

CLIENT: USIBWC JOB NO.: 4839-02
PROJECT: SUNLAND PARK LEVEE DESIGNER: SG DATE: Oct-18
SUBJECT: **L4x4x0.5** CHECKER: DATE:

a= 5 in
Kv= $5+5/(a/h)^2$= 8.20

If h/tw ≤ 2.24 SQRT(E/Fy)
Cv= 1

Vn= 0.6FyAwCv= 43.20 kips > Vu, OK

CLIENT: USIBWC
PROJECT: SUNLAND PARK LEVEE
SUBJECT: **Wx40**

JOB NO.: 4839-02
DESIGNER: SG
DATE: Oct-18
CHECKER:
DATE:

SECTION PROPERTIES

SECTION TYPE = ***W***

SOIL CAPACITY = *0* tons (0 kips)

F_y =	***36.0*** ksi	E_s =	*29000* ksi	S_x =	*35.5* in^3
h or d	*8.25* in	K_x =	*1.2*	S_y =	*12.2* in^3
b_f =	*8.07* in	K_y =	*1.2*	r_x =	*3.53* in
t_f =	*0.56* in	Lu_x =	*8.00* ft	r_y =	*2.04* in
t_w =	*0.36* in	Lu_y =	*8.00* ft	Z_x =	*39.8* in^3
A_s =	*11.7* in^2	L_b =	8.00 ft	Z_Y =	*18.5* in^3
$b_f/2t_f$ =	*7.21*	h_0 =	*7.69* in	r_{ts} =	*2.31* in
h/tw=	*22.92*	J =	*1.12* in^4	Ix=	*146* in^4
A_f =	4.52 in	Cw=	*726* in^6	Iy=	*49.1* in^4
d/A_f =	*1.83*			S_{xt}=	*35.5* in^3

AXIAL CAPACITY (AISC - LRFD Chapter E)

KL/r = **56.471** 4.71 SQRT(E/Fy) 133.68 $Fe = \pi^2 E_s/[(KL/r)^2]$ = 89.75 ksi

When KL/r =< 4.71 SQRT(E/Fy):

Fcr = [0.658^(Fy/Fe)]Fy = 30.44 ksi

When C_c > 4.71 SQRT(E/Fy):

Fcr = 0.877 Fe = N/A ksi

A.- Maximum Allowable Axial Load:

$\phi P_n = F_{cr}A_g$ = 320.50 kips

FLEXURAL CAPACITY (AISC - LRFD Chapter F)

$b_f/2t_f$ =	*7.21*	<	$0.38\sqrt{E/F_y}$ = 10.79	(Flange is Compact)
h/tw =	*22.92*	<	3.76SQRT(E/Fy) = 106.72	(Web is Compact)

DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND FLANGES

Yielding Limit State :

$M_{nx} = M_p = F_y Z_x$ = 1432.80 kips-in
$M_{ny} = M_p = F_y Z_y$ = 666.00 kips-in

B.- Flexural Capacities:

ϕMnx= **1289.52** kips-in
ϕMny= **599.40** kips-in

COMBINED STRESSES (AISC - LRFD Chapter H)

Pr/Pc>=0.2
(H1-1a): (Pr/Pc)+8/9(Mrx/Mcx+Mry/Mcy)<1
Pr/Pc<0.2
(H1-1b): (Pr/2Pc)+(Mrx/Mcx+Mry/Mcy)<1

$P_{applied}$ = 1.00 kips
Mrx = 6.00 kip-in
Mry = 26.00 kip-in

CLIENT: USIBWC JOB NO.: 4839-02
PROJECT: SUNLAND PARK LEVEE DESIGNER: SG DATE: Oct-18
SUBJECT: **Wx40** CHECKER: DATE:

Vu= 1.20 kip

(H1-1a): N/A
(H1-1b): 0.050

SHEAR (AISC - LRFD Chapter G)

DOUBLY SYMMETRIC I-SHAPED MEMBERS

a= 5 in

Kv= $5+5/(a/h)^2$= 18.61

If h/tw ≤ 2.24 SQRT(E/Fy)

Cv= 1

If h/tw ≤ 1.10 SQRT(KvE/Fy)

Cv= 1

Vn= 0.6FyAwCv= 64.15 kips > Vu, OK

Ramp Design

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 5/13/2021
JOB NAME Sunland Park_Montoya Lateral Catwalk
JOB NO 4839-02
ENGINEER NAME SG
JOB CLIENT USIBWC
END JOB INFORMATION
INPUT WIDTH 79
UNIT FEET KIP
JOINT COORDINATES
3 30 6 0; 5 40 6 0; 12 30 6 2; 14 40 6 2;
MEMBER INCIDENCES
3 3 5; 11 12 14; 22 3 12; 26 5 14;
DEFINE MATERIAL START
ISOTROPIC STEEL
E 4.176e+006
POISSON 0.3
DENSITY 0.489024
ALPHA 6.5e-006
DAMP 0.03
END DEFINE MATERIAL
MEMBER PROPERTY AMERICAN
3 11 TABLE ST C8X11
26 TABLE ST L40406
22 TABLE LD L40406 SP 0.6875
CONSTANTS
BETA 180 MEMB 3
BETA 45 MEMB 26
MATERIAL STEEL ALL
SUPPORTS
3 12 FIXED
5 14 FIXED BUT MX MY MZ
MEMBER RELEASE
26 START MY MZ
26 END MY MZ
LOAD 1 LOADTYPE None  TITLE DEAD LOAD
SELFWEIGHT Y -1
SELFWEIGHT Y -1 LIST 22
SELFWEIGHT Y -1 LIST 11
SELFWEIGHT Y -1 LIST 3
****Grating load, assume 15 psf
MEMBER LOAD
11 UNI GY -0.016 10 20
11 UNI GY -0.016 0 10
3 UNI GY -0.016 10 20
3 UNI GY -0.016 0 10
*****Live load assume 40 psf
LOAD 2 LOADTYPE None  TITLE LIVE LOAD
MEMBER LOAD
3 11 UNI GY -0.04
3 UNI GY -0.04 10 20
3 UNI GY -0.04 0 10
*****Wind Load, assume 50 psf
LOAD 3 LOADTYPE None  TITLE WIND LOAD
MEMBER LOAD
3 UNI GZ 0.057 10 20
3 UNI GZ 0.057 0 10
*
*
11 UNI GX 0.02 10 20
11 UNI GX 0.02 0 10
3 UNI GX 0.02 10 20
3 UNI GX 0.02 0 10
26 UNI GX 0.02
22 UNI GX 0.02 1 2
22 UNI GX 0.02 0 1
```

```
LOAD 4 LOADTYPE Flood  TITLE FLOOD LOAD
LOAD 10 LOADTYPE None  TITLE 1.2DL+1.6LL_REPEATED
REPEAT LOAD
1 1.2 2 1.6
LOAD 11 LOADTYPE None  TITLE 1.2DL+LL+0.5W+FA_REPEATED
REPEAT LOAD
1 1.2 2 1.0 3 0.5 4 1.0
LOAD 12 LOADTYPE None  TITLE 0.9DL+0.5W+FA_REPEATED
REPEAT LOAD
1 0.9 3 0.5 4 1.0
LOAD 13 LOADTYPE None  TITLE 1.2DL+0.5W_REPEATED
REPEAT LOAD
1 1.2 3 0.5
LOAD COMB 5 1.2DL+1.6LL
1 1.2 2 1.6
LOAD COMB 6 1.2DL+LL+0.5W+FA
1 1.2 3 0.5 4 1.0 2 1.0
LOAD COMB 7 0.9DL+0.5W+FA
1 0.9 3 0.5 4 1.0
LOAD COMB 8 1.2DL+0.5W
1 1.2 3 0.5
LOAD COMB 9 DEFLECTION CTRL
1 1.0 2 1.0 3 1.0 4 1.0
PDELTA 3 ANALYSIS SMALLDELTA
*
*******************************
PARAMETER 1
CODE LRFD
FU 8640 MEMB 26
FYLD 5184 MEMB 26
KY 2.2 MEMB 26
KZ 2.2 MEMB 26
*******For the C5X9, assume column space is 8ft, total 4 columns, instead of 3 columns
RATIO 1 MEMB 26
CHECK CODE MEMB 26
***********************************
*
*********************************
*LOAD LIST 8
*PARAMETER 2
*CODE LRFD
*FU 8640 ALL
*FYLD 7200 ALL
*KY 0.5 MEMB 1 TO 4 9 TO 12
*KZ 0.5 MEMB 1 TO 4 9 TO 12
*LY 0.5 MEMB 1 TO 4 9 TO 12
*LZ 0.5 MEMB 1 TO 4 9 TO 12
*UNB 0.5 MEMB 1 TO 4 9 TO 12
*UNT 0.5 MEMB 1 TO 4 9 TO 12
**
*DFF 1000 MEMB 1 TO 4 9 TO 12
*DJ1 1 MEMB 1 TO 4
*DJ2 5 MEMB 1 TO 4
**
*DJ1 10 MEMB 9 TO 12
*DJ2 14 MEMB 9 TO 12
*CHECK CODE ALL
**************************************
PERFORM ANALYSIS PRINT STATICS CHECK
FINISH
FINISH
```

Job Information

	Engineer	Checked	Approved
Name:	SG		
Date:	5/13/2021		

Project ID	
Project Name	

Structure Type	SPACE FRAME

Number of Nodes	4	Highest Node	14
Number of Elements	4	Highest Beam	26

Number of Basic Load Cases	8
Number of Combination Load Cases	5

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	DEAD LOAD
Primary	2	LIVE LOAD
Primary	3	WIND LOAD
Primary	4	FLOOD LOAD
Primary	10	1.2DL+1.6LL_REPEATED
Primary	11	1.2DL+LL+0.5W+FA_REPEATED
Primary	12	0.9DL+0.5W+FA_REPEATED
Primary	13	1.2DL+0.5W_REPEATED
Combination	5	1.2DL+1.6LL
Combination	6	1.2DL+LL+0.5W+FA
Combination	7	0.9DL+0.5W+FA
Combination	8	1.2DL+0.5W
Combination	9	DEFLECTION CTRL

Nodes

Node	X (ft)	Y (ft)	Z (ft)
3	30.000	6.000	0.000
5	40.000	6.000	0.000
12	30.000	6.000	2.000
14	40.000	6.000	2.000

Beams

Beam	Node A	Node B	Length (ft)	Property	β (degrees)
3	3	5	10.000	1	180
11	12	14	10.000	1	0
22	3	12	2.000	3	0
26	5	14	2.000	2	45

Section Properties

Prop	Section	Area (in^2)	I_{yy} (in^4)	I_{zz} (in^4)	J (in^4)	Material
1	C8X11	3.370	1.310	32.500	0.116	STEEL
2	L40406	2.860	6.943	1.774	0.137	STEEL
3	L40406 LD	5.720	167.176	8.717	0.268	STEEL

Materials

Mat	Name	E (kip/in^2)	ν	Density (kip/in^3)	α (/°F)
1	STEEL	29E+3	0.300	0.000	6.5E -6
2	STAINLESSSTEEL	28E+3	0.300	0.000	10E -6
3	ALUMINUM	10E+3	0.330	0.000	13E -6
4	CONCRETE	3.15E+3	0.170	0.000	5E -6

Primary Load Cases

Number	Name	Type
1	DEAD LOAD	None
2	LIVE LOAD	None
3	WIND LOAD	None
4	FLOOD LOAD	Flood
10	1.2DL+1.6LL_REPEATED	None
11	1.2DL+LL+0.5W+FA_REPEATED	None
12	0.9DL+0.5W+FA_REPEATED	None
13	1.2DL+0.5W_REPEATED	None

Combination Load Cases

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
5	1.2DL+1.6LL	1	DEAD LOAD	1.20
		2	LIVE LOAD	1.60
6	1.2DL+LL+0.5W+FA	1	DEAD LOAD	1.20
		3	WIND LOAD	0.50
		4	FLOOD LOAD	1.00
		2	LIVE LOAD	1.00
7	0.9DL+0.5W+FA	1	DEAD LOAD	0.90
		3	WIND LOAD	0.50
		4	FLOOD LOAD	1.00
8	1.2DL+0.5W	1	DEAD LOAD	1.20
		3	WIND LOAD	0.50
9	DEFLECTION CTRL	1	DEAD LOAD	1.00
		2	LIVE LOAD	1.00
		3	WIND LOAD	1.00
		4	FLOOD LOAD	1.00

1 DEAD LOAD : Beam Loads

Beam	Type		Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
3	UNI	lbf/ft	GY	-16.000	0.000	-	10.000	-
	UNI	lbf/ft	GY	-16.000	10.000	-	20.000	-
11	UNI	lbf/ft	GY	-16.000	0.000	-	10.000	-
	UNI	lbf/ft	GY	-16.000	10.000	-	20.000	-

1 DEAD LOAD : Selfweight

Direction	Factor	Assigned Geometry
Y	-1.000	ALL
Y	-1.000	22
Y	-1.000	11
Y	-1.000	3

2 LIVE LOAD : Beam Loads

Beam	Type		Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
3	UNI	lbf/ft	GY	-40.000	-	-	-	-
	UNI	lbf/ft	GY	-40.000	0.000	-	10.000	-
	UNI	lbf/ft	GY	-40.000	10.000	-	20.000	-
11	UNI	lbf/ft	GY	-40.000	-	-	-	-

3 WIND LOAD : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
3	UNI lbf/ft	GZ	57.000	0.000	-	10.000	-
	UNI lbf/ft	GZ	57.000	10.000	-	20.000	-
	UNI lbf/ft	GX	20.000	0.000	-	10.000	-
	UNI lbf/ft	GX	20.000	10.000	-	20.000	-
11	UNI lbf/ft	GX	20.000	0.000	-	10.000	-
	UNI lbf/ft	GX	20.000	10.000	-	20.000	-
22	UNI lbf/ft	GX	20.000	0.000	-	1.000	-
	UNI lbf/ft	GX	20.000	1.000	-	2.000	-
26	UNI lbf/ft	GX	20.000	-	-	-	-

10 1.2DL+1.6LL_REPEATED : Repeat Loads

Ref	Name	Factor
1	DEAD LOAD	1.200
2	LIVE LOAD	1.600

11 1.2DL+LL+0.5W+FA_REPEATED : Repeat Loads

Ref	Name	Factor
1	DEAD LOAD	1.200
2	LIVE LOAD	1.000
3	WIND LOAD	0.500
4	FLOOD LOAD	1.000

12 0.9DL+0.5W+FA_REPEATED : Repeat Loads

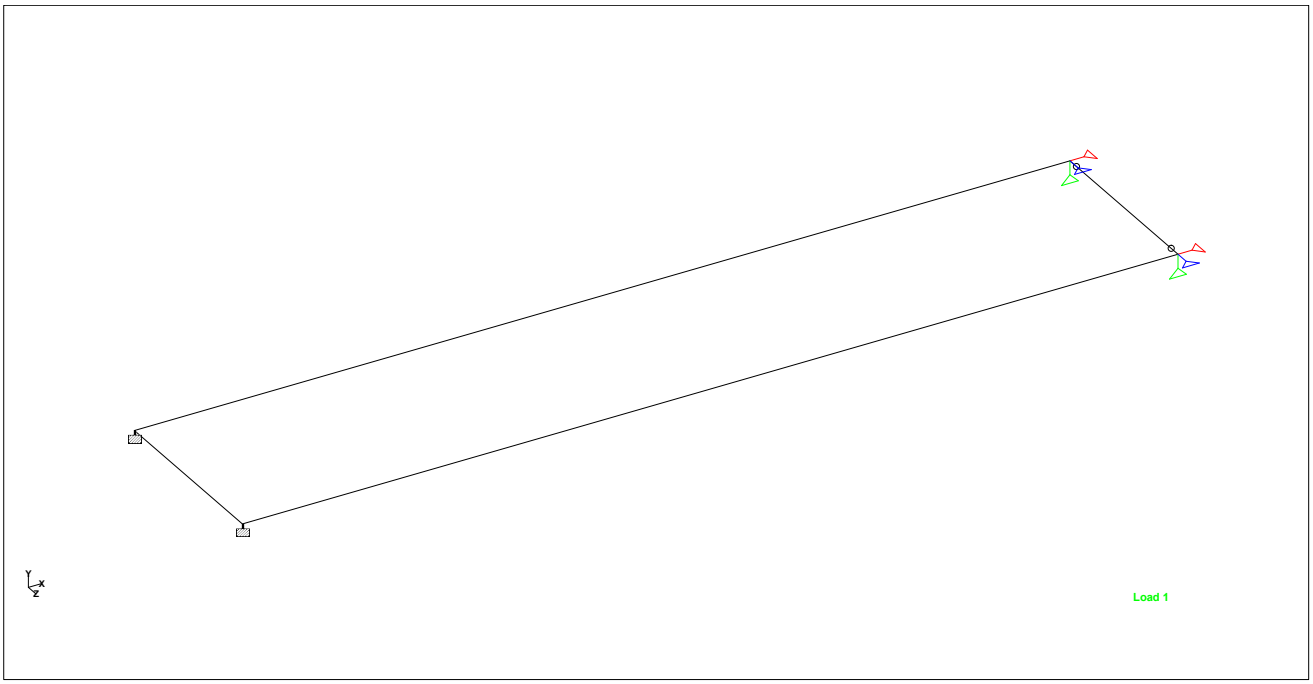
Ref	Name	Factor
1	DEAD LOAD	0.900
3	WIND LOAD	0.500
4	FLOOD LOAD	1.000

13 1.2DL+0.5W_REPEATED : Repeat Loads

Ref	Name	Factor
1	DEAD LOAD	1.200
3	WIND LOAD	0.500

Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip-ft/deg)	rY (kip-ft/deg)	rZ (kip-ft/deg)
3	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
5	Fixed	Fixed	Fixed	-	-	-
12	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
14	Fixed	Fixed	Fixed	-	-	-



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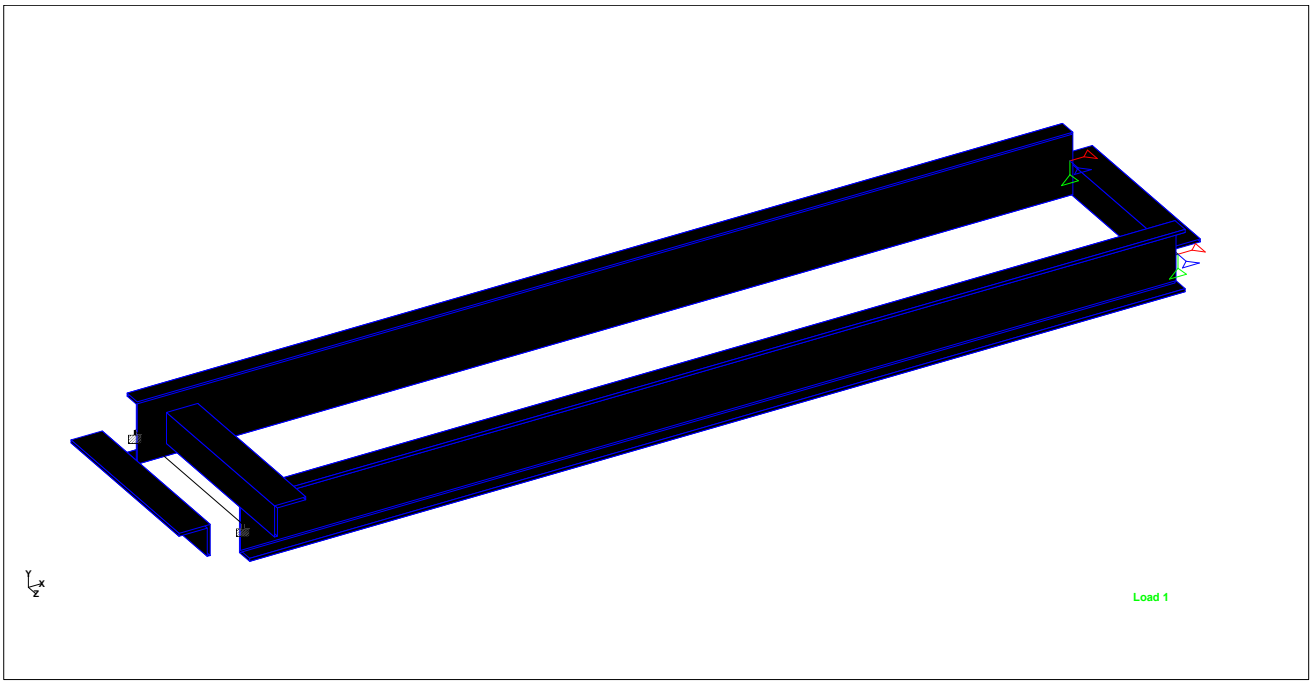

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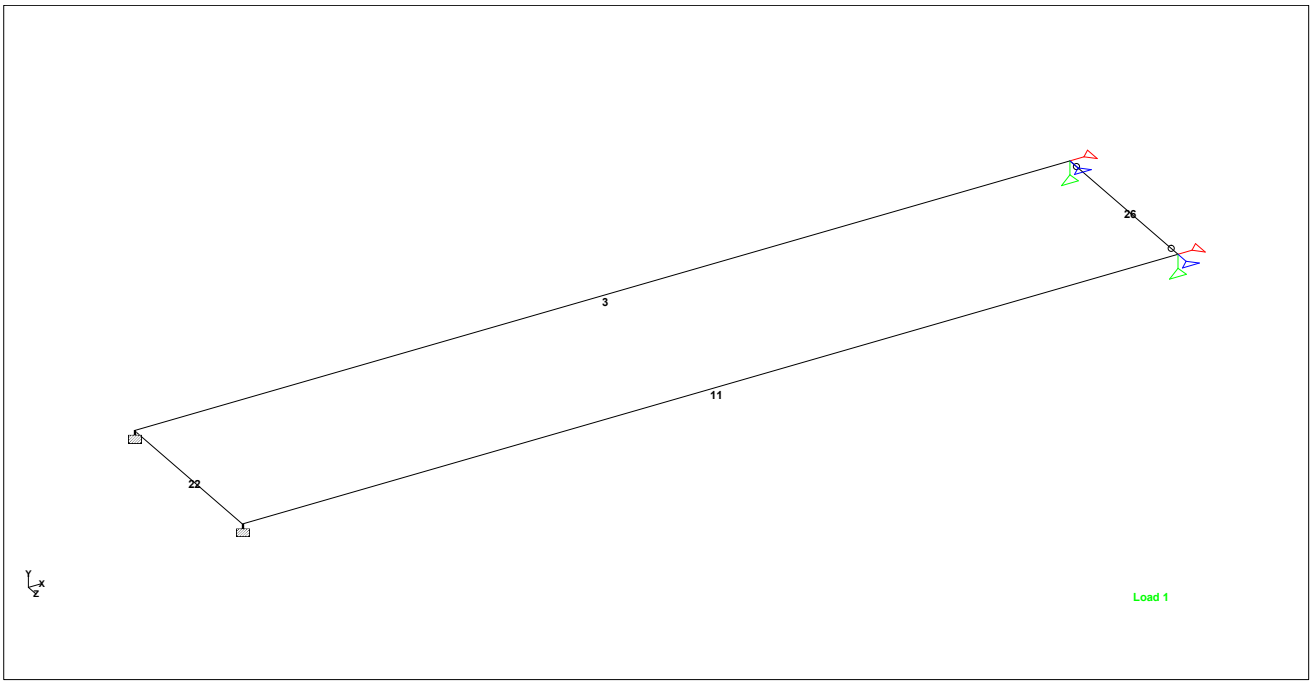

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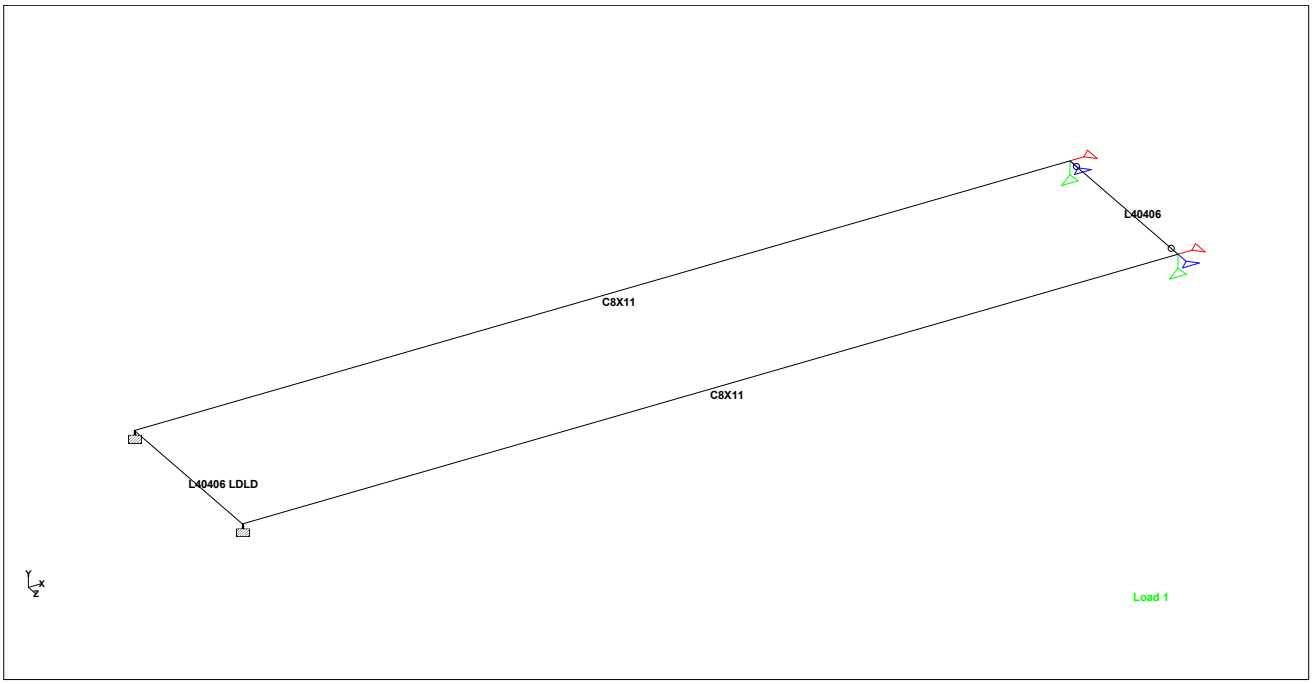


4

Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

				Axial	Shear		Torsion	Bending	
	Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
Max Fx	3	5	3:WIND LOAD	**0.100**	-0.000	-0.214	-0.000	-0.000	0.000
Min Fx	3	3	3:WIND LOAD	**-0.100**	-0.000	0.356	0.000	-8.545	-0.000
Max Fy	11	12	10:1.2DL+1.6L	0.000	**0.690**	0.000	0.000	0.000	16.442
Min Fy	3	3	10:1.2DL+1.6L	0.000	**-1.089**	0.000	0.000	-0.000	-25.934
Max Fz	3	3	3:WIND LOAD	-0.100	-0.000	**0.356**	0.000	-8.545	-0.000
Min Fz	3	5	3:WIND LOAD	0.100	-0.000	**-0.214**	-0.000	-0.000	0.000
Max Mx	26	5	10:1.2DL+1.6L	0.000	0.008	-0.008	**0.013**	0.000	0.000
Min Mx	26	5	3:WIND LOAD	0.000	0.014	0.014	**-0.000**	0.000	0.000
Max My	3	5	10:1.2DL+1.6L	-0.000	0.657	0.000	-0.000	**0.000**	-0.013
Min My	3	3	3:WIND LOAD	-0.100	-0.000	0.356	0.000	**-8.545**	-0.000
Max Mz	11	12	5:1.2DL+1.6LL	0.000	0.690	0.000	0.000	0.000	**16.442**
Min Mz	3	3	10:1.2DL+1.6L	0.000	-1.089	0.000	0.000	-0.000	**-25.934**

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
3	1:DEAD LOAD	0.000	0.281	-0.000	-0.155	0.000	5.776
	2:LIVE LOAD	0.000	0.499	-0.000	0.000	0.000	11.877
	3:WIND LOAD	-0.120	-0.000	-0.356	0.000	8.465	-0.000
	4:FLOOD LOAI	0.000	0.000	0.000	0.000	0.000	0.000
	10:1.2DL+1.6LI	0.000	1.136	-0.000	-0.186	0.000	25.934
	11:1.2DL+LL+0	-0.060	0.837	-0.178	-0.186	4.232	18.808
	12:0.9DL+0.5W	-0.060	0.253	-0.178	-0.140	4.232	5.198
	13:1.2DL+0.5W	-0.060	0.338	-0.178	-0.186	4.232	6.931
	5:1.2DL+1.6LL	0.000	1.136	-0.000	-0.186	0.000	25.934
	6:1.2DL+LL+0.	-0.060	0.837	-0.178	-0.186	4.232	18.808
	7:0.9DL+0.5W-	-0.060	0.253	-0.178	-0.140	4.232	5.198
	8:1.2DL+0.5W	-0.060	0.338	-0.178	-0.186	4.232	6.931
	9:DEFLECTIOI	-0.120	0.780	-0.356	-0.155	8.465	17.653
5	1:DEAD LOAD	0.000	0.156	-0.000	0.000	0.000	0.000
	2:LIVE LOAD	0.000	0.301	-0.000	0.000	0.000	0.000
	3:WIND LOAD	-0.120	-0.000	-0.214	0.000	0.000	0.000
	4:FLOOD LOAI	0.000	0.000	0.000	0.000	0.000	0.000
	10:1.2DL+1.6LI	0.000	0.669	-0.000	0.000	0.000	0.000
	11:1.2DL+LL+0	-0.060	0.488	-0.107	0.000	0.000	0.000
	12:0.9DL+0.5W	-0.060	0.140	-0.107	0.000	0.000	0.000
	13:1.2DL+0.5W	-0.060	0.187	-0.107	0.000	0.000	0.000
	5:1.2DL+1.6LL	0.000	0.669	-0.000	0.000	0.000	0.000
	6:1.2DL+LL+0.	-0.060	0.488	-0.107	0.000	0.000	0.000
	7:0.9DL+0.5W-	-0.060	0.140	-0.107	0.000	0.000	0.000
	8:1.2DL+0.5W	-0.060	0.187	-0.107	0.000	0.000	0.000
	9:DEFLECTIOI	-0.120	0.457	-0.214	0.000	0.000	0.000
12	1:DEAD LOAD	0.000	0.281	0.000	0.155	0.000	5.776
	2:LIVE LOAD	0.000	0.250	0.000	0.000	0.000	5.945
	3:WIND LOAD	-0.120	-0.000	0.000	0.000	0.080	-0.000
	4:FLOOD LOAI	0.000	0.000	0.000	0.000	0.000	0.000
	10:1.2DL+1.6LI	0.000	0.737	0.000	0.186	0.000	16.442
	11:1.2DL+LL+0	-0.060	0.587	0.000	0.186	0.040	12.875
	12:0.9DL+0.5W	-0.060	0.253	0.000	0.140	0.040	5.198
	13:1.2DL+0.5W	-0.060	0.338	0.000	0.186	0.040	6.931
	5:1.2DL+1.6LL	0.000	0.737	0.000	0.186	0.000	16.442
	6:1.2DL+LL+0.	-0.060	0.587	0.000	0.186	0.040	12.875
	7:0.9DL+0.5W-	-0.060	0.253	0.000	0.140	0.040	5.198
	8:1.2DL+0.5W	-0.060	0.338	0.000	0.186	0.040	6.931
	9:DEFLECTIOI	-0.120	0.531	0.000	0.155	0.080	11.720
14	1:DEAD LOAD	-0.000	0.156	0.000	0.000	0.000	0.000
	2:LIVE LOAD	0.000	0.150	0.000	0.000	0.000	0.000
	3:WIND LOAD	-0.120	0.000	0.000	0.000	0.000	0.000
	4:FLOOD LOAI	0.000	0.000	0.000	0.000	0.000	0.000
	10:1.2DL+1.6LI	0.000	0.428	0.000	0.000	0.000	0.000

Reactions Cont...

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
	11:1.2DL+LL+0	-0.060	0.338	0.000	0.000	0.000	0.000
	12:0.9DL+0.5W	-0.060	0.140	0.000	0.000	0.000	0.000
	13:1.2DL+0.5W	-0.060	0.187	0.000	0.000	0.000	0.000
	5:1.2DL+1.6LL	-0.000	0.428	0.000	0.000	0.000	0.000
	6:1.2DL+LL+0.5	-0.060	0.338	0.000	0.000	0.000	0.000
	7:0.9DL+0.5W-	-0.060	0.140	0.000	0.000	0.000	0.000
	8:1.2DL+0.5W	-0.060	0.187	0.000	0.000	0.000	0.000
	9:DEFLECTIOI	-0.120	0.306	0.000	0.000	0.000	0.000

DANNENBAUM

CLIENT: USIBWC
PROJECT: SUNLAND PARK LEVEE
SUBJECT: **C8x11.5**

JOB NO.: 4839-02
DESIGNER: SG
CHECKER:

DATE: May-21
DATE:

SECTION PROPERTIES

SECTION TYPE = ***W***

SOIL CAPACITY = *0* tons (0 kips)

F_y =	***36.0*** ksi	E_s =	*29000* ksi	S_x =	*8.140* in^3
h or d	8.00 in	K_x =	*1.2*	S_y =	*0.780* in^3
b_f =	*2.26* in	K_y =	*1.2*	r_x =	*3.110* in
t_f =	*0.39* in	Lu_x =	*7.00* ft	r_y =	*0.623* in
t_w =	*0.22* in	Lu_y =	*7.00* ft	Z_x =	*9.630* in^3
A_s =	*3.38* in^2	L_b =	*7.00* ft	Z_Y =	*1.570* in^3
$b_f/2t_f$ =	*2.90*	h_0 =	*7.610* in	r_{ts} =	*0.756* in
h/tw=	*36.36*	J =	*0.130* in^4	Ix=	*32.600* in^4
A_f =	0.88 in	Cw=	*16.50* in^6	Iy=	*1.310* in^4
d/A_f =	*9.08*			S_{xt}=	*0.780* in^3

AXIAL CAPACITY (AISC - LRFD Chapter E)

KL/r = **161.798** 4.71 SQRT(E/Fy) 133.68 $Fe = \pi^2 E_s/[(KL/r)^2]$ = 10.93 ksi

When KL/r =< 4.71 SQRT(E/Fy):

Fcr = [0.658^(Fy/Fe)]Fy = N/A ksi

When C_c > 4.71 SQRT(E/Fy):

Fcr = 0.877 Fe = 9.59 ksi

A.- Maximum Allowable Axial Load:

$\phi P_n = F_{cr}A_g$ = 29.17 kips

FLEXURAL CAPACITY (AISC - LRFD Chapter F)

$b_f/2t_f$ =	*2.90*	<	$0.38\sqrt{E/F_y}$ = 10.79	(Flange is Compact)
h/tw =	*36.36*	<	3.76SQRT(E/Fy) = 106.72	(Web is Compact)

DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND FLANGES

Yielding Limit State :

$M_{nx} = M_p = F_y Z_x$ = 346.68 kips-in
$M_{ny} = M_p = F_y Z_y$ = 56.52 kips-in

B.- Flexural Capacities:

ϕM_{nx}= **25.27** kips-in
ϕM_{ny}= **42.75** kips-in

COMBINED STRESSES (AISC - LRFD Chapter H)

Pr/Pc>=0.2
(H1-1a): (Pr/Pc)+8/9(Mrx/Mcx+Mry/Mcy)<1
Pr/Pc<0.2
(H1-1b): (Pr/2Pc)+(Mrx/Mcx+Mry/Mcy)<1

$P_{applied}$ =	0.05 kips
Mrx =	25.21 kip-in
Mry =	4.27 kip-in
Vu=	1.06 kip

CLIENT: USIBWC
JOB NO.: 4839-02
PROJECT: SUNLAND PARK LEVEE
DESIGNER: SG
DATE: May-21
SUBJECT: **C8x11.5**
CHECKER:
DATE:

(H1-1a): N/A
(H1-1b): 1.098

SHEAR (AISC - LRFD Chapter G)

DOUBLY SYMMETRIC I-SHAPED MEMBERS

a= 5 in

Kv= $5+5/(a/h)^2$ = 17.80

If h/tw ≤ 2.24 SQRT(E/Fy)

Cv= 1

Vn= 0.6FyAwCv= 38.02 kips > Vu, OK

Deflection CTRL

Δ_{max}= 0.056 from STAAD Model
L/480= 0.1875 OK

DANNENBAUM

CLIENT: USIBWC
PROJECT: SUNLAND PARK LEVEE
SUBJECT: **L4x4x0.5**

JOB NO.: 4839-02
DESIGNER: SG
DATE: Oct-18
CHECKER:
DATE:

SECTION PROPERTIES

SECTION TYPE = ***W***

SOIL CAPACITY = *0* tons (0 kips)

F_y =	***36.0*** ksi	E_s =	*29000* ksi	S_x =	*1.960* in^3
h or d	*4.00* in	K_x =	*1.2*	S_y =	*1.960* in^3
b_f =	*4* in	K_y =	*1.2*	r_x =	*1.210* in
t_f =	*0.5* in	Lu_x =	*2.00* ft	r_y =	*1.210* in
t_w =	*0.5* in	Lu_y =	*2.00* ft	Z_x =	*3.500* in^3
A_s =	*3.75* in^2	L_b =	*2.00* ft	Z_Y =	*3.500* in^3
$b_f/2t_f$ =	*4.00*	h_0 =	*0.32* in	r_{ts} =	*2.160* in
h/tw=	*8.00*	J =	*0.32* in^4	Ix=	*5.520* in^4
A_f =	*2.00* in	Cw=	*0.37* in^6	Iy=	*5.520* in^4
d/A_f =	*2.00*			S_{xt}=	*1.960* in^3

AXIAL CAPACITY (AISC - LRFD Chapter E)

KL/r = **23.802** 4.71 SQRT(E/Fy) 133.68 $Fe = \pi^2 E_s/[(KL/r)^2]$ = 505.22 ksi

When KL/r =< 4.71 SQRT(E/Fy):

Fcr = [0.658^(Fy/Fe)]Fy = 34.94 ksi

When C_c > 4.71 SQRT(E/Fy):

Fcr = 0.877 Fe = N/A ksi

A.- Maximum Allowable Axial Load:

$\phi P_n = F_{cr}A_g$ = 117.93 kips

FLEXURAL CAPACITY (AISC - LRFD Chapter F)

$b_f/2t_f$ =	*4.00*	<	0.38*SQRT*(E/F_y) = 10.79	(Flange is Compact)
h/tw =	*8.00*	<	3.76SQRT(E/Fy) = 106.72	(Web is Compact)

B.- Flexural Capacities:

ϕMnx= **113.40** kips-in
ϕMny= **113.40** kips-in

COMBINED STRESSES (AISC - LRFD Chapter H)

Pr/Pc>=0.2
(H1-1a): (Pr/Pc)+8/9(Mrx/Mcx+Mry/Mcy)<1
Pr/Pc<0.2
(H1-1b): (Pr/2Pc)+(Mrx/Mcx+Mry/Mcy)<1

$P_{applied}$ =	0.00 kips
Mrx =	0.19 kip-in
Mry =	0.04 kip-in
Vu=	0.50 kip

(H1-1a): N/A
(H1-1b): 0.002

SHEAR (AISC - LRFD Chapter G)

DOUBLY SYMMETRIC I-SHAPED MEMBERS

CLIENT: USIBWC JOB NO.: 4839-02
PROJECT: SUNLAND PARK LEVEE DESIGNER: SG DATE: Oct-18
SUBJECT: **L4x4x0.5** CHECKER: DATE:

a= 5 in
Kv= 5+5/(a/h)^2= 8.20

If h/tw ≤ 2.24 SQRT(E/Fy)
Cv= 1

Vn= 0.6FyAwCv= 43.20 kips > Vu, OK

Deflection Control

$\Delta=5wL4/384EI$	5*40/12*(10.5*12)^4)/(384*29000000*32.6)	0.012
$\Delta_{allowable}=L/360$	10.5*12/360	0.35

Company:	Dannenbaum Engineering Corp.	Date:	4/9/2019
Engineer:	Sahar Ghasemi	Page:	1/6
Project:			
Address:	3100 W Alabama St.		
Phone:	7135276363		
E-mail:	Sahar.ghasemi@dannenbaum.com		

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-11
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 55
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 3.438
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 4.81
C_{min} (inch): 3.75
S_{min} (inch): 3.75

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 220.00
State: Cracked
Compressive strength, f'_c (psi): 5000
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 16.00 x 16.00 x 0.25
Yield stress: 36000 psi

Profile type/size: W8X40

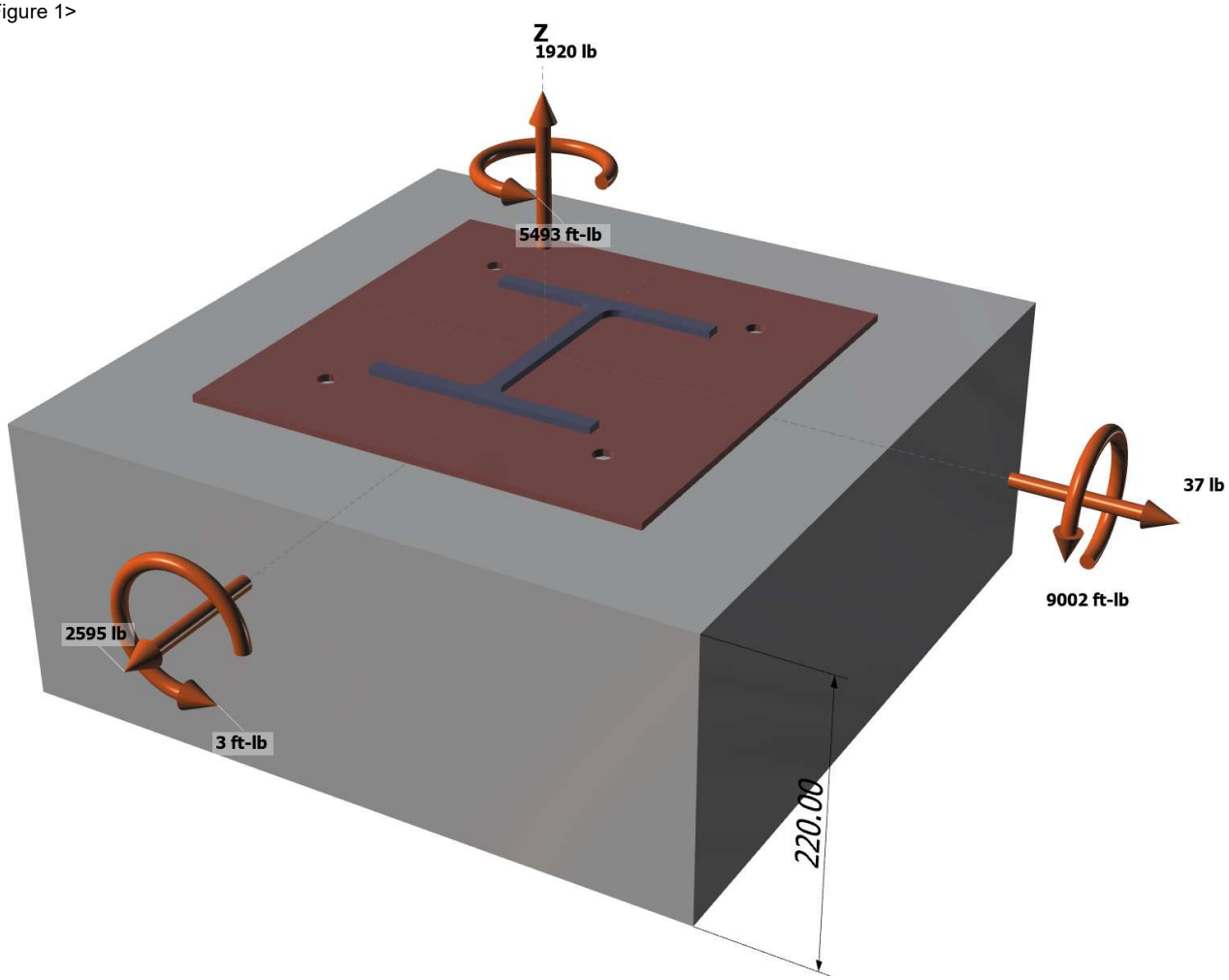
Recommended Anchor
Anchor Name: Heavy Hex Bolt - 5/8"Ø Heavy Hex Bolt, F1554 Gr. 55

Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

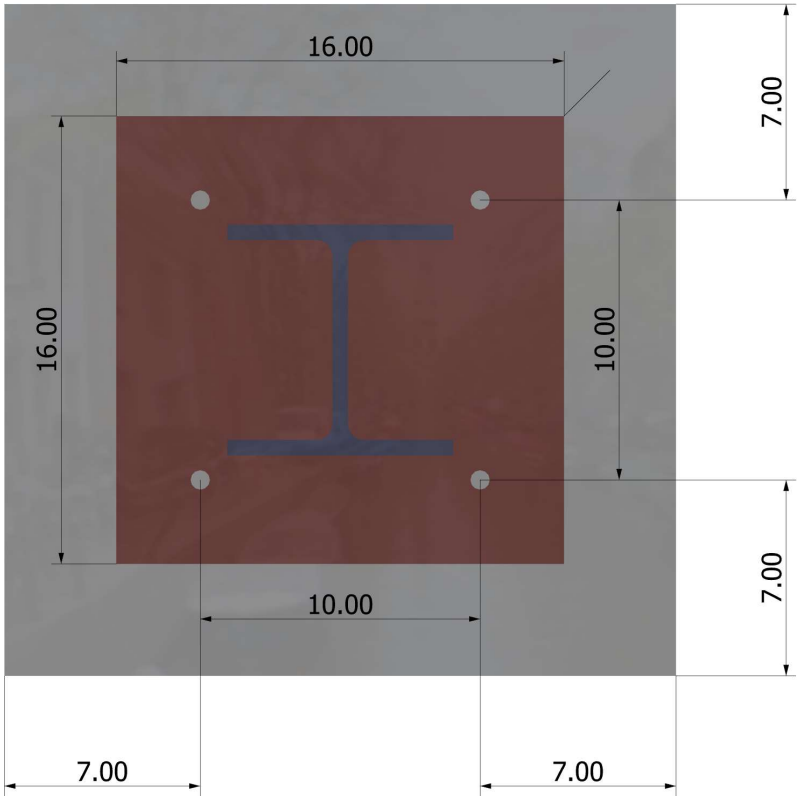
Strength level loads:

N_{ua} [lb]: 1920
V_{uax} [lb]: 2595
V_{uay} [lb]: 37
M_{ux} [ft-lb]: 3
M_{uy} [ft-lb]: 9002
M_{uz} [ft-lb]: 5493

<Figure 1>



<Figure 2>

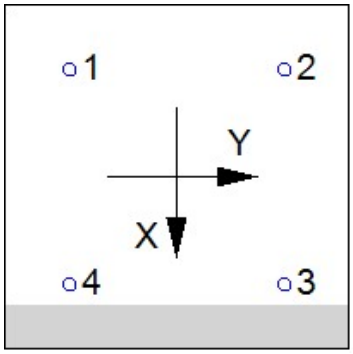


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4865.5	2296.6	-1638.6	2821.2
2	4866.2	-999.1	-1638.6	1919.2
3	443.1	-999.1	1657.1	1935.0
4	442.3	2296.6	1657.1	2832.0
Sum	10617.1	2595.0	37.0	9507.4

Maximum concrete compression strain (‰): 0.13
Maximum concrete compression stress (psi): 544
Resultant tension force (lb): 10617
Resultant compression force (lb): 8697
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 4.17
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. D.5.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
16950	0.75	12713

5. Concrete Breakout Strength of Anchor in Tension (Sec. D.5.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. D-6)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	5000	3.438	10816

$\phi N_{cbg} = \phi (A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. D.4.1 & Eq. D-4)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
412.60	106.35	7.00	0.553	1.000	1.00	1.000	10816	0.75	17405

6. Pullout Strength of Anchor in Tension (Sec. D.5.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. D.4.1, Eq. D-13 & D-14)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	0.67	5000	0.70	18788

8. Steel Strength of Anchor in Shear (Sec. D.6.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
10170	1.0	0.65	6611

9. Concrete Breakout Strength of Anchor in Shear (Sec. D.6.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
3.44	0.625	1.00	5000	17.00	38572

$\phi V_{cbgy} = \phi\,(A_{Vc} / A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{by}$ (Sec. D.4.1 & Eq. D-31)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
612.00	1300.50	1.000	0.782	1.000	1.000	38572	0.75	10651

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
3.44	0.625	1.00	5000	17.00	38572

$\phi V_{cbgx} = \phi\,(A_{Vc} / A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{bx}$ (Sec. D.4.1 & Eq. D-31)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgx} (lb)
612.00	1300.50	1.000	0.782	1.000	1.000	38572	0.75	10651

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
3.44	0.625	1.00	5000	7.00	10192

$\phi V_{cbgy} = \phi\ (2)(A_{Vc} / A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{bx}$ (Sec. D.4.1 & Eq. D-31)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
252.00	220.50	1.000	1.000	1.000	1.000	10192	0.75	17471

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
3.44	0.625	1.00	5000	7.00	10192

$\phi V_{cbgx} = \phi\ (2)(A_{Vc} / A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{by}$ (Sec. D.4.1 & Eq. D-31)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgx} (lb)
252.00	220.50	1.000	1.000	1.000	1.000	10192	0.75	17471

10. Concrete Pryout Strength of Anchor in Shear (Sec. D.6.3)

$\phi V_{cp} = \phi k_{cp}N_{cb} = \phi k_{cp}(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N}N_b$ (Sec. D.4.1 & Eq. D-40)

k_{cp}	A_{Nc} (in²)	A_{Nco} (in²)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
2.0	103.15	106.35	1.000	1.000	1.000	10816	0.70	14687

11. Results

Interaction of Tensile and Shear Forces (Sec. RD.7)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	4866	12713	0.38	Pass
Concrete breakout	**10617**	**17405**	**0.61**	**Pass (Governs)**
Pullout	4866	18788	0.26	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	2832	6611	0.43	Pass
T Concrete breakout y+	3314	10651	0.31	Pass
T Concrete breakout x+	4593	10651	0.43	Pass
|| Concrete breakout x+	3314	17471	0.19	Pass
|| Concrete breakout y-	4593	17471	0.26	Pass
Concrete breakout, combined	**-**	**-**	**0.53**	**Pass (Governs)**
Pryout	2832	14687	0.19	Pass

Interaction check	$(N_{ua}/\phi N_{ua})^{5/3}$	$(V_{ua}/\phi V_{ua})^{5/3}$	Combined Ratio	Permissible	Status
Sec. RD.7	0.44	0.35	78.8 %	1.0	Pass

5/8"Ø Heavy Hex Bolt, F1554 Gr. 55 with hef = 3.438 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

Hole No. C-30

DRILLING LOG	DIVISION: IBWC	INSTALLATION: Raba-Kistner Consultants Inc.	SHEET 1 OF 1 SHEETS

1. PROJECT	The Rio Grande Canalization Project
2. LOCATION (Coordinates or Station)	N 1533133; E 299065.98
3. DRILLING AGENCY	Southwest Engineering, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*	C-30
5. NAME OF DRILLER	Ralph Luna
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN	N/A
8. DEPTH DRILLED INTO ROCK	N/A
9. TOTAL DEPTH OF HOLE	30
10. SIZE AND TYPE OF BIT	3-3/4 Inch ID Hollow Stem Auger
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*	MSL
12. MANUFACTURER'S DESIGNATION OF DRILL	CME-85
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	DISTURBED 9 / UNDISTURBED N/A
14. TOTAL NUMBER CORE BOXES	N/A
15. ELEVATION GROUND WATER	3743.51
16. DATE HOLE	STARTED 4/16/2008 / COMPLETED 4/16/2008
17. ELEVATION TOP OF HOLE	3754.01
18. TOTAL CORE RECOVERY FOR BORING	%
19. GEOLOGIST	Jerome C. Garcia, E.I.T.

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	BLOW COUNT e	% RECOVERY f	SAMPLE INTERVAL g	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) h
+3753.8	0.3		**3-INCH COARSE BASE**	7	86	0.3 – 1.8	
			FILL: SANDY SILT (ML) loose to very loose, dry to moist, light brown, non-plastic	3	103	2.5 – 4.0	
				3	106	5.0 – 6.5	
+3746.5	7.5						
+3745.0	9.0		**FILL: SILT with SAND (ML)** loose, moist, brown, non-plastic	5	56	7.5 – 9.0	
			SANDY SILT (ML) loose to very loose, wet, brown, non-plastic	7	44	10.0 – 11.5	
				1	69	15.0 – 16.5	
+3734.0	20.0						
			SILTY SAND (SM) fine, poorly graded, very loose to loose, wet, brown, non-plastic	5	103	20.0 – 21.5	
				9	106	25.0 – 26.5	
+3724.0	30.0		- medium dense 28.5 to 30 ft	13	114	28.5 – 30.0	

NOTES:
1. Free water was observed at 10.5 ft.
2. Cement-bentonite grout used as backfill.

DRILLING LOG

DIVISION	IBWC
INSTALLATION	Raba-Kistner Consultants Inc.
SHEET	1 OF 1 SHEETS
1. PROJECT	The Rio Grande Canalization Project
2. LOCATION (Coordinates or Station)	N 1532254.44; E 299552.47
3. DRILLING AGENCY	Southwest Engineering, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*	C-31
5. NAME OF DRILLER	Ralph Luna
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN	N/A
8. DEPTH DRILLED INTO ROCK	N/A
9. TOTAL DEPTH OF HOLE	30
10. SIZE AND TYPE OF BIT	3-3/4 Inch ID Hollow Stem Auger
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*	MSL
12. MANUFACTURER'S DESIGNATION OF DRILL	CME-85
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	DISTURBED 9 / UNDISTURBED N/A
14. TOTAL NUMBER CORE BOXES	N/A
15. ELEVATION GROUND WATER	3743.981
16. DATE HOLE	STARTED 4/16/2008 / COMPLETED 4/16/2008
17. ELEVATION TOP OF HOLE	3754.981
18. TOTAL CORE RECOVERY FOR BORING	%
19. GEOLOGIST	Jerome C. Garcia, E.I.T.

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	BLOW COUNT e	% RECOVERY f	SAMPLE INTERVAL g	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) h
+3754.5	0.5		**6-INCH COARSE BASE**	17	83	0.5 – 2.0	
			FILL: SANDY SILT (ML) medium dense to loose, moist, light brown, non-plastic	7	61	2.5 – 4.0	
+3750.0	5.0		**FILL: LEAN CLAY with SAND (CL)** soft, moist to wet, brown, highly plastic	4	44	5.0 – 6.5	
+3746.5	8.5			4	39	7.5 – 9.0	
			LEAN CLAY with SAND (CL) soft, moist, brown, high to moderately plastic	3	67	10.0 – 11.5	
+3741.0	14.0		**SILTY SAND (SM)** fine, poorly graded, loose, wet, brown, non-plastic	4	108	15.0 – 16.5	
+3737.0	18.0		**SAND (SP)** fine, poorly graded, medium dense to loose, wet, brown, non-plastic	14	111	20.0 – 21.5	
				13	114	25.0 – 26.5	
+3725.0	30.0			6	119	28.5 – 30.0	

NOTES:
1. Free water was observed at 11 ft.
2. Cement-bentonite grout used as backfill.

FOUNDATION CAPACITY

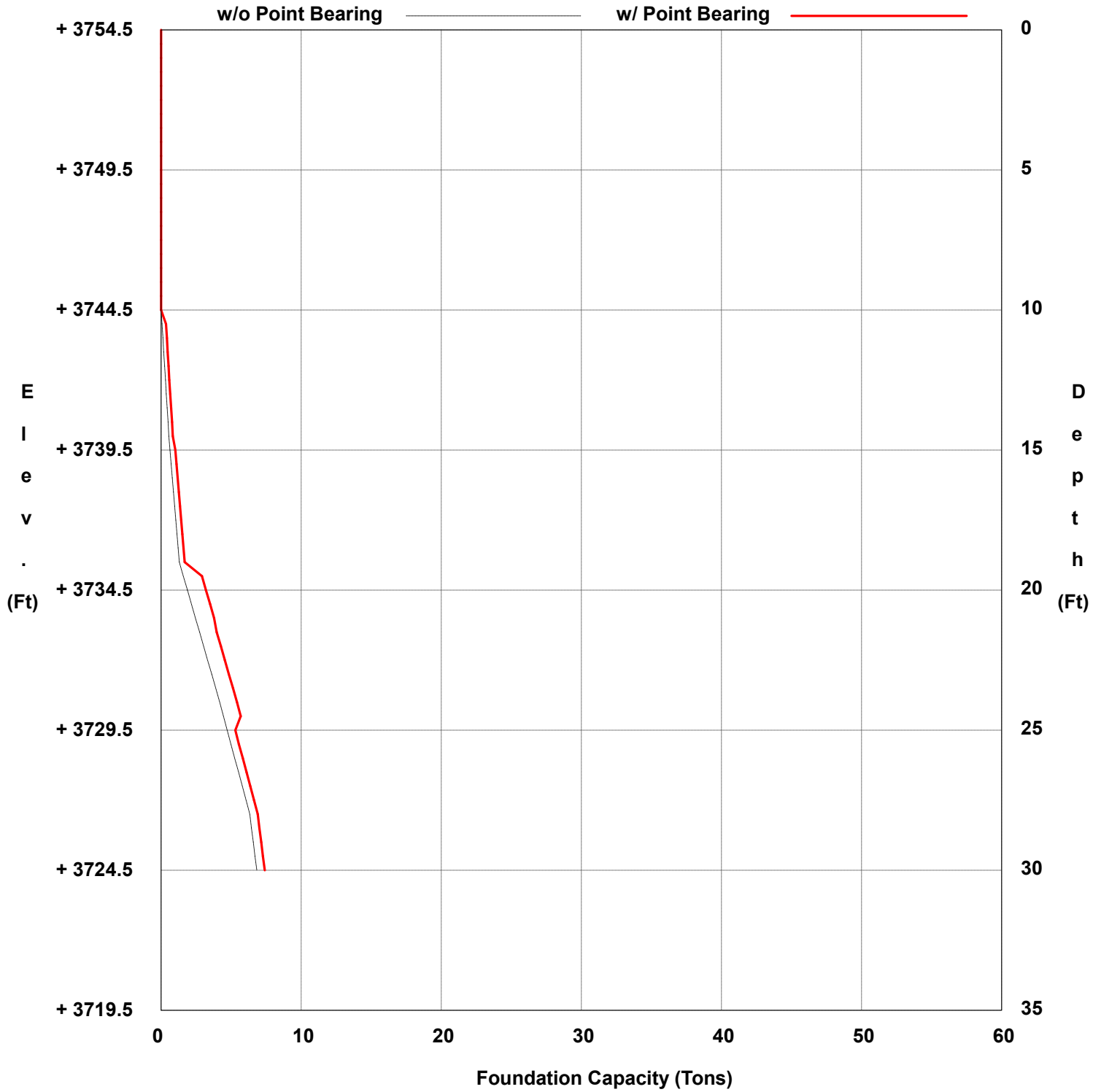
WinCore
Version 3.3

County		Hole	1	District	
Highway		Structure	Montoya	Date	
Control		Station	5.9	Grnd. Elev.	3754.50 ft
		Offset	0	GW Elev.	3743.50 ft

18 inch Drilled Shaft
3 ton Design Load
Tip Elevation = + 3734.5

+3754.5 Top Hole Elevation
+3744.5 Disregard Elevation

Disregard above hard strata disabled
Pb: 2 Diameters Below Tip Checked
TCP Capacity Values Used
0.7 Soil Reduction Factor Used



G:\1180\4839-02 Sunland Park Levee\DESIGN\100% Submittal\Foundation_4-1-19\Montoya_4-1-19\montoya-12 inch-disregard.CLG

100 % Design Report

for

USIBWC – Sunland Park Levee Rehabilitation Design
Canutillo West 2 (CW2)

01 February 2022

STATE OF TEXAS
JOSE L. REYES
91964
LICENSED
PROFESSIONAL ENGINEER
2/01/2022

This Work Plan outlines the support the Munoz & Dannenbaum JV is providing Arcadis US as the Prime A-E under contract with the U.S. International Boundary & Water Commission (USIBWC) on the Sunland Park Levee Rehabilitation Design Project # IBM1704

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1 INTRODUCTION

1.1 PURPOSE OF 100% DESIGN REPORT

The Sunland Park Levee Rehab. 100% Design Report is intended to provide more detailed documentation as it pertains to the design assistance/support the Munoz & Dannenbaum JV (MDJV) continues to provide Arcadis US, who is the prime consultant on the aforementioned levee rehabilitation design initiative, during the design phase of this project.

2 PROJECT OVERVIEW

The design of the Sunland Park Levee Repair is an integral part of the Rio Grande Canalization Project. Repair improvements of the Sunland Park Levee Segments include the Canutillo East 2 (CE2) and of Canutillo West 2 (CW2). However, this report focuses only on those structures associated with the Canutillo West (CW2) levee.

3 PROJECT GOAL

All design work on this project shall comply with FEMA requirements in order to adhere to 44 CFR §65.10.

4 PROJECT BACKGROUND

The United States Section of the International Boundary and Water Commission (USIBWC) contracted the services of Arcadis US, to provide a design to rehabilitate the recently completed (November 2012) Sunland Park Levee segment of the Rio Grande Canalization Project. The scope of services includes rehabilitation of multiple structures along this segment of the levee in order for them to perform satisfactorily during the 100-year flood event as required per 44 CFR §65.10. The structure that is included in this report is the Schultz Lateral WW # 35C.

5 PROJECT REQUIREMENTS

Repair through rehabilitation of the levee will be required along the entire length of the Sunland Park Levee segments. While Remedial Concept # 4 – Slope Blanket in Kenall's forensic report was an option considered during the concept phase of the project, the Arcadis design team is required to evaluate multiple designs that fully meets 44 CFR §65.10 and addresses the deficiencies noted in the forensic report.

The proposed design of the new levee system must meet or exceed all of the requirements of the U.S. Army Corps of Engineers' Engineering Manual 1110-2-1913, which is entitled "Design and Construction of Levees."

The reaches within the Sunland Park Levee Segment are as follows:

USIBWC stationing as reflected below were used in development of this report:

Reach Name	*Starting Station*	*Ending Station*
CE2	1510+19.16	1956+50
CW2	1518+56.27	1698+00

6 DESIGN CRITERIA/STANDARDS AND OTHER REQUIREMENTS

Key assumptions/constraints associated with this 100% Design Report are as indicated below. The design criteria/standards utilized during the design phases of this project include, but are not limited to, those reflected in Enclosure "A" of this report.

6.1 DESIGN REQUIREMENTS

The table below reflects additional information regarding the Schultz Lateral WW # 35C structure, which includes existing top of structure elevation according to the most recent FXSA topographic survey and also the 100-Year WSE. Recommended improvements are also noted below.

6.1.1 FLOOD CONTROL & IRRIGATION STRUCTURES:

In accordance with 44 CFR §65.10 structures such as these that pass through the levee need to be controlled during flood events (100-Year WSE).

6.1.1.1 Schultz Lateral WW#35C (30-inch RCP)

This part of the Schultz Lateral is a drain located south of the Country Club Bridge. The flow is controlled by a 30" sluice gate installed in 2011 by the USIBWC to manage flow during a flood event (100-Year WSE). Because the top of the existing structure on the riverside is 5.91 feet below the 100-Year WSE, the design proposes constructing a catwalk, a landing area and a longer sluice gate stem for operational purposes.

Table 1 Flood Control & Irrigation Structures

Description	100-Year WSE	FXSA Top of Structure Elev.	Recommended Improvements
1- 30" RCP Schultz Lateral WW#35C	3753.6	3747.69	See Para. 6.1.1.1 above

Below is additional information regarding major roadway structures and their tie-in to the Canutillo West (CW2) levee.

6.1.2 **ROADWAY STRUCTURES:**

All structures that pass through or under the levee shall be evaluated to determine if closure devises meet 44 CFR § 65.10. If any closure device requires maintenance or rehabilitation, it shall be included in the construction documents. Borderland Bridge – gates shall be left in place. However, the Arcadis team is required to look into "water passage" through levee if the top of levee does not meet freeboard requirements as per 44 CFR § 65.10. The proposed improvements to the Country Club Road Bridge, are also being evaluated by the Arcadis design team in consultation with representatives of the U.S. Section of the International Boundary & Water Commission. Accordingly, consideration is being given to the use of "stop logs" across Country Club Road Bridge as it ties into the Canutillo West (CW2) levee.

6.1.3 **CONSTRUCTION CONSIDERATIONS:**

6.1.3.1 – Utility Conflicts. None are expected that would conflict with the proposed improvements at the Schultz Lateral. However, this is subject to verification by the Construction Contractor conducting its own underground utility investigations as part of the 811 national call-before-you-dig campaign.

6.1.3.2 – Construction Outside of USIBWC's ROW. None expected.

6.1.3.3 – Long Lead Time Items. None expected.

6.1.3.4 – Sole Source Items. None expected.

6.1.3.5– Specialty Contractors Required. None expected.

6.1.3.6 – Safety Hazards. Typical with any heavy civil construction project. However, the levee construction contractor and its sub-contractors, will have to adhere to all OSHA safety standards, USACE EM 385-1-1 (Safety and Health Requirements Manual), and other Federal Government job safety requirements.

6.1.3.7 – Estimated Time of Construction. The proposed improvements associated with the Montoya Lateral WW#36 were estimated to take between 75 to 120 days to complete. This assumes that the initial 30 – 45 day period from receipt of the notice to proceed (NTP) would be dedicated to preparation of necessary submittal(s), and subsequent review and approval by the USIBWC. The remaining 45 – 75 days would include: fabrication, delivery, and the on site construction of the structure. The levee construction

contractor will most likely self-perform or subcontract the work associated with this structure, which is estimated to occur concurrently with the levee construction. Therefore, production rates of equipment, etc. would not fall on critical path which will primarily consist of the overall levee rehabilitation work.

The levee construction contractor will most likely subcontract the work associated with the proposed catwalk structure to a specialty sub-contractor; therefore, production rates to include labor and equipment, would not be a critical path item of the overall levee rehabilitation project construction schedule.

6.1.3.8 – Technical Evaluation Requirements of Construction Contractor. As noted in Paragraph 6.1.3.7 above, the levee construction contractor will most likely subcontract the work associated with the Schultz Lateral WW#35C to a specialty contractor. Therefore, the technical evaluation requirements associated with above noted structure would not be applicable to the specialty sub-contractor.

ENCLOSURE A : REFERENCES & DESIGN CRITERIA/STANDARDS

The following table summarizes the design criteria/standards that will be utilized in designing the improvements to the flood and irrigation control and roadway structures noted above in this report.

Document Name and Version	Description
44 CFR § 65.10 (Oct. 2006)	FEMA's Criteria for Accrediting Levee Systems on National Flood Insurance Program
EC 110-2-6067 (Aug. 2010)	USACE Engineering Circular – policy for Evaluations Performed by USACE for Accreditation of Levee System(s) Under NFIP Administered by FEMA
EM 1110-2-1913 (Apr. 2000)	USACE Engineering Manual for Design & Construction of Levees
EM 1110-2-2002 (June 1995)	USACE Engineering Manual for Evaluation of Repair of Concrete Structures
EM 1110-2-2100 (Dec. 2005)	USACE Engineering Manual for Stability analysis of Concrete Structures
EM 1110-2-2400 (Jun. 2003)	USACE Engineering Manual for Retaining & Flood Walls - Structural Design & Evaluation of Outlet Works
ACI 350 (Jan. 2006)	American Concrete Institute (ACI) – Details and Detailing of Concrete Reinforcement
AISC 303 (2016)	American Institute of Steel Construction (AISC) – Code of Standard Practice for Steel Buildings and Bridges
AISC 360 (Jun. 2018)	American Institute of Steel Construction (AISC) – Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings
ASCE 7 (2017)	Minimum Design Loads for Buildings and other Structures

ENCLOSURE B: STRUCTURAL DESIGN ASSUMPTIONS & CALCULATIONS

Sunland Park Levee Rehabilitation Project

Catwalk Design

This section presents the calculation and design of catwalks based on our scope of work defined in the modification to the Sunland park levee rehabilitation project. The calculation covers the following sections. Live load, flood load and wind load are derived from ASCE 7-10. Detailed calculation of the applied loads are presented in the following pages. Member control calculations are based on Steel construction manual AISC LRFD method.

- Live load requirement
- Flood Load Calculation
- Wind Calculation
- STAAD Input & Output
- STAAD Input Text File
- Members Controls

Table 4-1 Minimum Uniformly Distributed Live Loads, L_o, and Minimum Concentrated Live Loads

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lb (kN)
Apartments (see Residential)		
Access floor systems		
Office use	50 (2.4)	2,000 (8.9)
Computer use	100 (4.79)	2,000 (8.9)
Armories and drill rooms	150 (7.18)[a]	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)[a]	
Lobbies	100 (4.79)[a]	
Movable seats	100 (4.79)[a]	
Platforms (assembly)	100 (4.79)[a]	
Stage floors	150 (7.18)[a]	
Balconies and decks	1.5 times the live load for the occupancy served. Not required to exceed 100 psf (4.79 kN/m^2)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Other floors, same as occupancy served except as indicated		
Dining rooms and restaurants	100 (4.79)[a]	
Dwellings (see Residential)		
Elevator machine room grating (on area of 2 in. by 2 in. (50 mm by 50 mm))		300 (1.33)
Finish light floor plate construction (on area of 1 in. by 1 in. (25 mm by 25 mm))		200 (0.89)
Fire escapes	100 (4.79)	
On single-family dwellings only	40 (1.92)	
Fixed ladders	See Section 4.5	
Garages		
Passenger vehicles only	40 (1.92)[a,b,c]	
Trucks and buses	[c]	
Handrails, guardrails, and grab bars	See Section 4.5	
Helipads	60 (2.87)[d,e] Nonreducible	[e,f,g]
Hospitals		
Operating rooms, laboratories	60 (2.87)	1,000 (4.45)
Patient rooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Hotels (see Residential)		
Libraries		
Reading rooms	60 (2.87)	1,000 (4.45)
Stack rooms	150 (7.18)[a,h]	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Manufacturing		
Light	125 (6.00)[a]	2,000 (8.90)
Heavy	250 (11.97)[a]	3,000 (13.40)

Continued

CALCULATION SHEET

JOB NO. 4839-02

SIGNATURE SG	**10/16/2018**	CHECKED	DATE
PROJECT : Sunland Park-Catwalk Design			
SUBJECT: Flood Load Calculation		SHEET _____ OF _____	

Flood Load Calculation is Based on ASCE 7-10, Chapter 5

Flood load calcuation parameters:

Base Flood Elevation (BFE)	3753.6	ft	
Ground Level (G)	3749.25	ft	
d_s= 0.65 (BFE-G)	2.83	ft	ASCE 7-10, 5.4-3
C_D	2.25		ASCE 7-10, 5.4.4.1
D	0.96	ft	ASCE 7-10, 5.4.4.1
γ_w	64.00	pcf	ASCE 7-10, 5.4.4.1
$H_b = 0.78\ d_s$	2.21	ft	ASCE 7-10, 5.4-2

Flood Load

$F_D = 0.5*\gamma_w*C_D*D*H_b^2$	337.08	lb

CALCULATION SHEET

JOB NO. 4839-02

SIGNATURE SG	**10/16/2018**	CHECKED	DATE
PROJECT : Sunland Park-Catwalk Design			
SUBJECT: Flood Load Calculation		SHEET _____ OF _____	

Flodd load by Individual Member:

Pile or Column Height	8.00	ft
F_D	42.13	lb/ft

CALCULATION SHEET

JOB NO. 4839-02

SIGNATURE SG	**10/16/2018**	CHECKED	DATE
PROJECT : Sunland Park-Catwalk Design			
SUBJECT: Wind Calculation		SHEET _____ OF _____	

Wind Calculation is Based on ASCE 7-10, Chapters 26 and 27

Note: Only the wind load normal to the fence will be considered in the design of concrete foundation

Wind load calcuation parameters:

Basic wind speed V:	115	mph	ASCE 7-10, P247a
Velocity Pressure Exposure Co. Kz:	1.08		ASCE 7-10, Table 27.3-1
Topographic Factor: Kzt:	1		ASCE 7-10, 26.8.2
Gust Effect G :	1		ASCE 7-10, 26.9
Wind Directionality Factor Kd:	0.85		ASCE 7-10, Table 26.6-1

Velocity Pressure:

qz=0.00256*Kz*Kzt*Kd*V^2=	31.08	lb/ft^2

Force Coeff. For single member Cf:	2	Figure 6-21	ASCE 7-05

CALCULATION SHEET

JOB NO. 4839-02

SIGNATURE SG	**10/16/2018**	CHECKED	DATE
PROJECT : Sunland Park-Catwalk Design			
SUBJECT: Wind Calculation		SHEET _____ OF _____	

Wind load by Individual Member:

qz*G*Cf=	62.16	lb/ft^2

Width of W8X40, W=	8.25	in
Wind Laod, q=	42.7	lb/ft
Width of Channel C5X9, W=	11.00	in
Wind Laod, q=	56.98	lb/ft

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 1/14/19
JOB NAME Sunland Park_Shultz Catwalk
JOB NO 4839-02
ENGINEER NAME SG
JOB CLIENT USIBWC
END JOB INFORMATION
INPUT WIDTH 79
UNIT FEET KIP
JOINT COORDINATES
1 0 6 0; 2 10 6 0; 3 20 6 0; 4 30 6 0; 10 0 6 2; 11 10 6 2; 12 20 6 2;
13 30 6 2; 19 10 6 1; 20 20 6 1; 21 30 6 1; 22 10 0 1; 23 20 -3 1; 24 30 -3 1;
29 40 6 0; 30 40 6 2; 31 40 6 1; 32 40 -3 1; 33 50 6 0; 34 50 6 2; 35 50 6 1;
36 50 -4 1; 43 55 6 0; 44 55 6 2; 45 55 6 1; 46 55 -4 1;
MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 4; 9 10 11; 10 11 12; 11 12 13; 17 22 19; 18 23 20; 19 24 21;
20 2 19; 21 19 11; 22 3 20; 23 20 12; 24 4 21; 25 21 13; 34 32 31; 35 29 31;
36 31 30; 37 36 35; 38 33 35; 39 35 34; 47 33 43; 48 34 44; 49 46 45; 50 43 45;
51 45 44; 52 13 30; 53 4 29; 54 29 33; 55 30 34; 56 44 33; 57 33 30; 58 30 4;
59 4 12; 60 12 2; 61 2 10;
DEFINE MATERIAL START
ISOTROPIC STEEL
E 4.176e+006
POISSON 0.3
DENSITY 0.489024
ALPHA 6.5e-006
DAMP 0.03
END DEFINE MATERIAL
MEMBER PROPERTY AMERICAN
1 TO 3 9 TO 11 47 48 52 TO 55 TABLE ST C5X9
17 TO 19 34 37 49 TABLE ST W8X40
20 TO 25 35 36 38 39 50 51 56 TO 61 TABLE LD L40406 SP 0.6875
CONSTANTS
BETA 180 MEMB 1 TO 3 47 53 54
BETA 0 MEMB 20 TO 25
MATERIAL STEEL ALL
SUPPORTS
22 TO 24 32 36 46 FIXED
1 10 FIXED BUT MX MY MZ
43 44 PINNED
MEMBER RELEASE
20 22 24 35 38 START MY MZ
21 23 25 36 39 END MY MZ
LOAD 1 LOADTYPE None  TITLE DEAD LOAD
SELFWEIGHT Y -1
****Grating load, assume 15 psf
MEMBER LOAD
1 TO 3 9 TO 11 47 48 52 TO 55 UNI GY -0.016
*****Live load assume 40 psf
LOAD 2 LOADTYPE None  TITLE LIVE LOAD
MEMBER LOAD
1 TO 3 9 TO 11 47 48 52 TO 55 UNI GY -0.04
*****Wind Load, assume 50 psf
LOAD 3 LOADTYPE None  TITLE WIND LOAD
MEMBER LOAD
1 TO 3 47 53 54 UNI GZ 0.057
*
17 UNI GZ 0.042 4 6
18 19 34 UNI GZ 0.042 4 9
37 49 UNI GZ 0.042 4 10
*
1 TO 3 9 TO 11 47 48 52 TO 55 UNI GX 0.02
17 UNI GX 0.042 4 6
18 19 34 UNI GX 0.042 4 9
```

```
37 49 UNI GX 0.042 4 10
20 TO 25 35 36 38 39 50 51 UNI GX 0.02
LOAD 4 LOADTYPE Flood  TITLE FLOOD LOAD
MEMBER LOAD
17 UNI GX 0.042 4 6
17 UNI GZ 0.042 4 6
18 19 34 UNI GX 0.042 4 9
37 49 UNI GX 0.042 4 10
18 19 34 UNI GZ 0.042 4 9
37 49 UNI GZ 0.042 4 10
LOAD 10 LOADTYPE None  TITLE 1.2DL+1.6LL_REPEATED
REPEAT LOAD
1 1.2 2 1.6
LOAD 11 LOADTYPE None  TITLE 1.2DL+LL+0.5W+FA_REPEATED
REPEAT LOAD
1 1.2 2 1.0 3 0.5 4 1.0
LOAD 12 LOADTYPE None  TITLE 0.9DL+0.5W+FA_REPEATED
REPEAT LOAD
1 0.9 3 0.5 4 1.0
LOAD 13 LOADTYPE None  TITLE 1.2DL+0.5W_REPEATED
REPEAT LOAD
1 1.2 3 0.5
LOAD COMB 5 1.2DL+1.6LL
1 1.2 2 1.6
LOAD COMB 6 1.2DL+LL+0.5W+FA
1 1.2 3 0.5 4 1.0 2 1.0
LOAD COMB 7 0.9DL+0.5W+FA
1 0.9 3 0.5 4 1.0
LOAD COMB 8 1.2DL+0.5W
1 1.2 3 0.5
LOAD COMB 9 DEFLECTION CTRL
1 1.0 2 1.0 3 1.0 4 1.0
PDELTA 3 ANALYSIS SMALLDELTA
*
*********************************
PARAMETER 1
CODE LRFD
FU 8640 MEMB 1 TO 3 9 TO 11 17 TO 25 34 TO 39 48 TO 55
FYLD 5184 MEMB 1 TO 3 9 TO 11 17 TO 25 34 TO 39 48 TO 55
KY 2.2 MEMB 17 TO 25 34 TO 39 49 TO 51
KZ 2.2 MEMB 17 TO 25 34 TO 39 49 TO 51
KY 1 MEMB 1 TO 3 9 TO 11 48 52 TO 55
KZ 1 MEMB 1 TO 3 9 TO 11 48 52 TO 55
*******For the C5X9, assume column space is 8ft, total 4 columns, instead of 3 columns
LY 8 MEMB 1 TO 3 9 TO 11 48 52 TO 55
LZ 6 MEMB 1 TO 3 9 TO 11 48 52 TO 55
RATIO 1 MEMB 1 TO 3 9 TO 11 17 TO 25 34 TO 39 48 TO 55
UNB 10 MEMB 1 TO 3 9 TO 11 48 52 TO 55
UNT 40 MEMB 1 TO 3 9 TO 11 48 52 TO 55
CHECK CODE MEMB 1 TO 3 9 TO 11 17 TO 25 34 TO 39 48 TO 55
***********************************
*
*********************************
*LOAD LIST 8
*PARAMETER 2
*CODE LRFD
*FU 8640 ALL
*FYLD 7200 ALL
*KY 0.5 MEMB 1 TO 4 9 TO 12
*KZ 0.5 MEMB 1 TO 4 9 TO 12
*LY 0.5 MEMB 1 TO 4 9 TO 12
*LZ 0.5 MEMB 1 TO 4 9 TO 12
*UNB 0.5 MEMB 1 TO 4 9 TO 12
*UNT 0.5 MEMB 1 TO 4 9 TO 12
**
*DFF 1000 MEMB 1 TO 4 9 TO 12
```

```
*DJ1 1 MEMB 1 TO 4
*DJ2 5 MEMB 1 TO 4
**
*DJ1 10 MEMB 9 TO 12
*DJ2 14 MEMB 9 TO 12
*CHECK CODE ALL
************************************
PERFORM ANALYSIS PRINT STATICS CHECK
FINISH
FINISH
```

Job Information

	Engineer	Checked	Approved
Name:	SG		
Date:	1/14/19		

Structure Type	SPACE FRAME

Number of Nodes	26	Highest Node	46
Number of Elements	36	Highest Beam	61

Number of Basic Load Cases	8
Number of Combination Load Cases	5

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	DEAD LOAD
Primary	2	LIVE LOAD
Primary	3	WIND LOAD
Primary	4	FLOOD LOAD
Primary	10	1.2DL+1.6LL_REPEATED
Primary	11	1.2DL+LL+0.5W+FA_REPEATED
Primary	12	0.9DL+0.5W+FA_REPEATED
Primary	13	1.2DL+0.5W_REPEATED
Combination	5	1.2DL+1.6LL
Combination	6	1.2DL+LL+0.5W+FA
Combination	7	0.9DL+0.5W+FA
Combination	8	1.2DL+0.5W
Combination	9	DEFLECTION CTRL

Primary Load Cases

Number	Name	Type
1	DEAD LOAD	None
2	LIVE LOAD	None
3	WIND LOAD	None
4	FLOOD LOAD	Flood
10	1.2DL+1.6LL_REPEATED	None
11	1.2DL+LL+0.5W+FA_REPEATED	None
12	0.9DL+0.5W+FA_REPEATED	None
13	1.2DL+0.5W_REPEATED	None

Combination Load Cases

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
5	1.2DL+1.6LL	1	DEAD LOAD	1.20
		2	LIVE LOAD	1.60
6	1.2DL+LL+0.5W+FA	1	DEAD LOAD	1.20
		3	WIND LOAD	0.50
		4	FLOOD LOAD	1.00
		2	LIVE LOAD	1.00
7	0.9DL+0.5W+FA	1	DEAD LOAD	0.90
		3	WIND LOAD	0.50
		4	FLOOD LOAD	1.00
8	1.2DL+0.5W	1	DEAD LOAD	1.20
		3	WIND LOAD	0.50
9	DEFLECTION CTRL	1	DEAD LOAD	1.00
		2	LIVE LOAD	1.00
		3	WIND LOAD	1.00
		4	FLOOD LOAD	1.00

Nodes

Node	X (ft)	Y (ft)	Z (ft)
1	0.000	6.000	0.000
2	10.000	6.000	0.000
3	20.000	6.000	0.000
4	30.000	6.000	0.000
10	0.000	6.000	2.000
11	10.000	6.000	2.000
12	20.000	6.000	2.000
13	30.000	6.000	2.000
19	10.000	6.000	1.000
20	20.000	6.000	1.000
21	30.000	6.000	1.000
22	10.000	0.000	1.000
23	20.000	-3.000	1.000
24	30.000	-3.000	1.000
29	40.000	6.000	0.000
30	40.000	6.000	2.000
31	40.000	6.000	1.000
32	40.000	-3.000	1.000
33	50.000	6.000	0.000
34	50.000	6.000	2.000
35	50.000	6.000	1.000
36	50.000	-4.000	1.000
43	55.000	6.000	0.000
44	55.000	6.000	2.000

Nodes Cont...

Node	X (ft)	Y (ft)	Z (ft)
45	55.000	6.000	1.000
46	55.000	-4.000	1.000

Beams

Beam	Node A	Node B	Length (ft)	Property	β (degrees)
1	1	2	10.000	1	180
2	2	3	10.000	1	180
3	3	4	10.000	1	180
9	10	11	10.000	1	0
10	11	12	10.000	1	0
11	12	13	10.000	1	0
17	22	19	6.000	2	0
18	23	20	9.000	2	0
19	24	21	9.000	2	0
20	2	19	1.000	3	0
21	19	11	1.000	3	0
22	3	20	1.000	3	0
23	20	12	1.000	3	0
24	4	21	1.000	3	0
25	21	13	1.000	3	0
34	32	31	9.000	2	0
35	29	31	1.000	3	0
36	31	30	1.000	3	0
37	36	35	10.000	2	0
38	33	35	1.000	3	0
39	35	34	1.000	3	0
47	33	43	5.000	1	180
48	34	44	5.000	1	0
49	46	45	10.000	2	0
50	43	45	1.000	3	0
51	45	44	1.000	3	0
52	13	30	10.000	1	0
53	4	29	10.000	1	180
54	29	33	10.000	1	180
55	30	34	10.000	1	0
56	44	33	5.385	3	0
57	33	30	10.198	3	0
58	30	4	10.198	3	0
59	4	12	10.198	3	0
60	12	2	10.198	3	0
61	2	10	10.198	3	0

Section Properties

Prop	Section	Area (in^2)	I_{yy} (in^4)	I_{zz} (in^4)	J (in^4)	Material
1	C5X9	2.640	0.630	8.900	0.095	STEEL
2	W8X40	11.700	49.100	146.000	1.064	STEEL
3	L40406 LD	5.718	167.120	8.717	0.275	STEEL

Materials

Mat	Name	E (kip/in^2)	ν	Density (kip/in^3)	α (/°F)
1	STEEL	29E+3	0.300	0.000	6.5E -6
2	STAINLESSSTEEL	28E+3	0.300	0.000	10E -6
3	ALUMINUM	10E+3	0.330	0.000	13E -6
4	CONCRETE	3.15E+3	0.170	0.000	5E -6

Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip-ft/deg)	rY (kip-ft/deg)	rZ (kip-ft/deg)
1	Fixed	Fixed	Fixed	-	-	-
10	Fixed	Fixed	Fixed	-	-	-
22	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
23	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
24	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
32	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
36	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
43	Fixed	Fixed	Fixed	-	-	-
44	Fixed	Fixed	Fixed	-	-	-
46	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed

Releases

Beam ends not shown in this table are fixed in all directions.

Beam	Node	x	y	z	rx	ry	rz
20	2	Fixed	Fixed	Fixed	Fixed	Pin	Pin
21	11	Fixed	Fixed	Fixed	Fixed	Pin	Pin
22	3	Fixed	Fixed	Fixed	Fixed	Pin	Pin
23	12	Fixed	Fixed	Fixed	Fixed	Pin	Pin
24	4	Fixed	Fixed	Fixed	Fixed	Pin	Pin
25	13	Fixed	Fixed	Fixed	Fixed	Pin	Pin
35	29	Fixed	Fixed	Fixed	Fixed	Pin	Pin
36	30	Fixed	Fixed	Fixed	Fixed	Pin	Pin
38	33	Fixed	Fixed	Fixed	Fixed	Pin	Pin
39	34	Fixed	Fixed	Fixed	Fixed	Pin	Pin

1 DEAD LOAD : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
1	UNI lbf/ft	GY	-16.000	-	-	-	-
2	UNI lbf/ft	GY	-16.000	-	-	-	-
3	UNI lbf/ft	GY	-16.000	-	-	-	-
9	UNI lbf/ft	GY	-16.000	-	-	-	-
10	UNI lbf/ft	GY	-16.000	-	-	-	-
11	UNI lbf/ft	GY	-16.000	-	-	-	-
47	UNI lbf/ft	GY	-16.000	-	-	-	-
48	UNI lbf/ft	GY	-16.000	-	-	-	-
52	UNI lbf/ft	GY	-16.000	-	-	-	-
53	UNI lbf/ft	GY	-16.000	-	-	-	-
54	UNI lbf/ft	GY	-16.000	-	-	-	-
55	UNI lbf/ft	GY	-16.000	-	-	-	-

1 DEAD LOAD : Selfweight

Direction	Factor	Assigned Geometry
Y	-1.000	ALL

2 LIVE LOAD : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
1	UNI lbf/ft	GY	-40.000	-	-	-	-
2	UNI lbf/ft	GY	-40.000	-	-	-	-
3	UNI lbf/ft	GY	-40.000	-	-	-	-
9	UNI lbf/ft	GY	-40.000	-	-	-	-
10	UNI lbf/ft	GY	-40.000	-	-	-	-
11	UNI lbf/ft	GY	-40.000	-	-	-	-
47	UNI lbf/ft	GY	-40.000	-	-	-	-
48	UNI lbf/ft	GY	-40.000	-	-	-	-
52	UNI lbf/ft	GY	-40.000	-	-	-	-
53	UNI lbf/ft	GY	-40.000	-	-	-	-
54	UNI lbf/ft	GY	-40.000	-	-	-	-
55	UNI lbf/ft	GY	-40.000	-	-	-	-

3 WIND LOAD : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
1	UNI lbf/ft	GZ	57.000	-	-	-	-
	UNI lbf/ft	GX	20.000	-	-	-	-
2	UNI lbf/ft	GZ	57.000	-	-	-	-
	UNI lbf/ft	GX	20.000	-	-	-	-
3	UNI lbf/ft	GZ	57.000	-	-	-	-
	UNI lbf/ft	GX	20.000	-	-	-	-
9	UNI lbf/ft	GX	20.000	-	-	-	-
10	UNI lbf/ft	GX	20.000	-	-	-	-
11	UNI lbf/ft	GX	20.000	-	-	-	-
17	UNI lbf/ft	GZ	42.000	4.000	-	6.000	-
	UNI lbf/ft	GX	42.000	4.000	-	6.000	-
18	UNI lbf/ft	GZ	42.000	4.000	-	9.000	-
	UNI lbf/ft	GX	42.000	4.000	-	9.000	-
19	UNI lbf/ft	GZ	42.000	4.000	-	9.000	-
	UNI lbf/ft	GX	42.000	4.000	-	9.000	-
20	UNI lbf/ft	GX	20.000	-	-	-	-
21	UNI lbf/ft	GX	20.000	-	-	-	-
22	UNI lbf/ft	GX	20.000	-	-	-	-
23	UNI lbf/ft	GX	20.000	-	-	-	-
24	UNI lbf/ft	GX	20.000	-	-	-	-
25	UNI lbf/ft	GX	20.000	-	-	-	-
34	UNI lbf/ft	GZ	42.000	4.000	-	9.000	-
	UNI lbf/ft	GX	42.000	4.000	-	9.000	-
35	UNI lbf/ft	GX	20.000	-	-	-	-
36	UNI lbf/ft	GX	20.000	-	-	-	-
37	UNI lbf/ft	GZ	42.000	4.000	-	10.000	-
	UNI lbf/ft	GX	42.000	4.000	-	10.000	-
38	UNI lbf/ft	GX	20.000	-	-	-	-
39	UNI lbf/ft	GX	20.000	-	-	-	-
47	UNI lbf/ft	GZ	57.000	-	-	-	-
	UNI lbf/ft	GX	20.000	-	-	-	-
48	UNI lbf/ft	GX	20.000	-	-	-	-
49	UNI lbf/ft	GZ	42.000	4.000	-	10.000	-
	UNI lbf/ft	GX	42.000	4.000	-	10.000	-
50	UNI lbf/ft	GX	20.000	-	-	-	-
51	UNI lbf/ft	GX	20.000	-	-	-	-
52	UNI lbf/ft	GX	20.000	-	-	-	-
53	UNI lbf/ft	GZ	57.000	-	-	-	-
	UNI lbf/ft	GX	20.000	-	-	-	-
54	UNI lbf/ft	GZ	57.000	-	-	-	-
	UNI lbf/ft	GX	20.000	-	-	-	-
55	UNI lbf/ft	GX	20.000	-	-	-	-

4 FLOOD LOAD : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
17	UNI lbf/ft	GZ	42.000	4.000	-	6.000	-
	UNI lbf/ft	GX	42.000	4.000	-	6.000	-
18	UNI lbf/ft	GZ	42.000	4.000	-	9.000	-
	UNI lbf/ft	GX	42.000	4.000	-	9.000	-
19	UNI lbf/ft	GZ	42.000	4.000	-	9.000	-
	UNI lbf/ft	GX	42.000	4.000	-	9.000	-
34	UNI lbf/ft	GZ	42.000	4.000	-	9.000	-
	UNI lbf/ft	GX	42.000	4.000	-	9.000	-
37	UNI lbf/ft	GZ	42.000	4.000	-	10.000	-
	UNI lbf/ft	GX	42.000	4.000	-	10.000	-
49	UNI lbf/ft	GZ	42.000	4.000	-	10.000	-
	UNI lbf/ft	GX	42.000	4.000	-	10.000	-

10 1.2DL+1.6LL_REPEATED : Repeat Loads

Ref	Name	Factor
1	DEAD LOAD	1.200
2	LIVE LOAD	1.600

11 1.2DL+LL+0.5W+FA_REPEATED : Repeat Loads

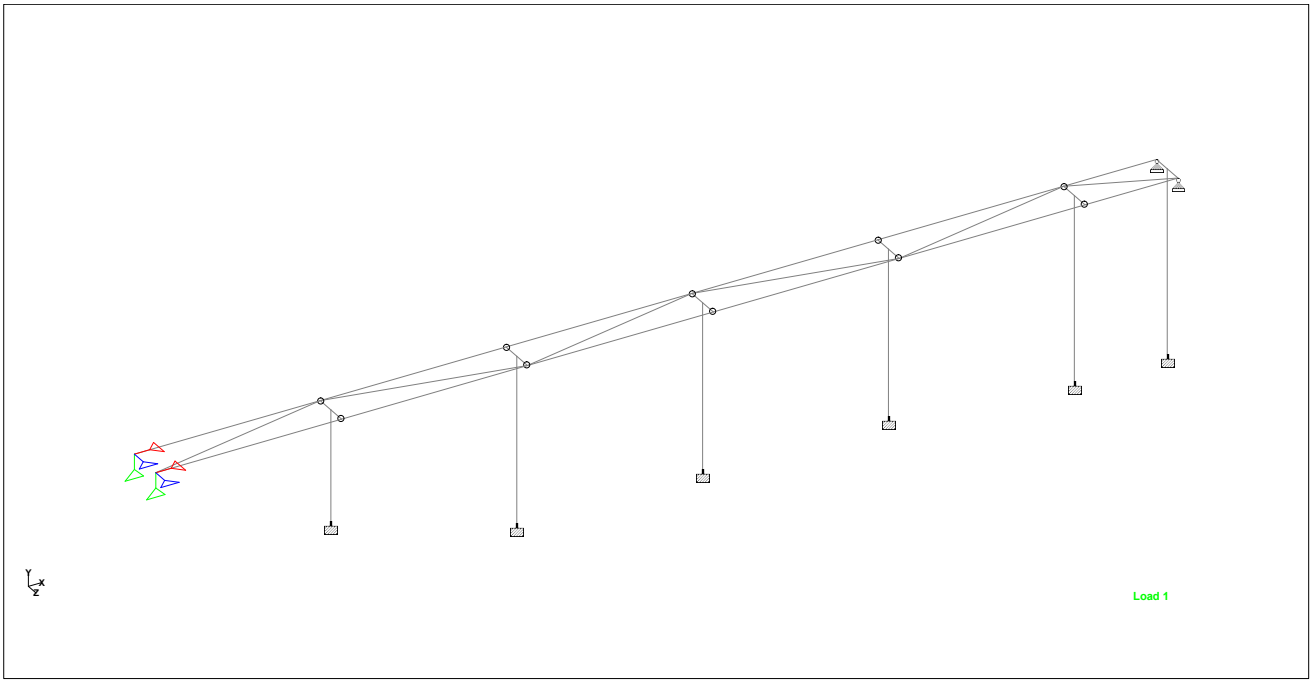
Ref	Name	Factor
1	DEAD LOAD	1.200
2	LIVE LOAD	1.000
3	WIND LOAD	0.500
4	FLOOD LOAD	1.000

12 0.9DL+0.5W+FA_REPEATED : Repeat Loads

Ref	Name	Factor
1	DEAD LOAD	0.900
3	WIND LOAD	0.500
4	FLOOD LOAD	1.000

13 1.2DL+0.5W_REPEATED : Repeat Loads

Ref	Name	Factor
1	DEAD LOAD	1.200
3	WIND LOAD	0.500



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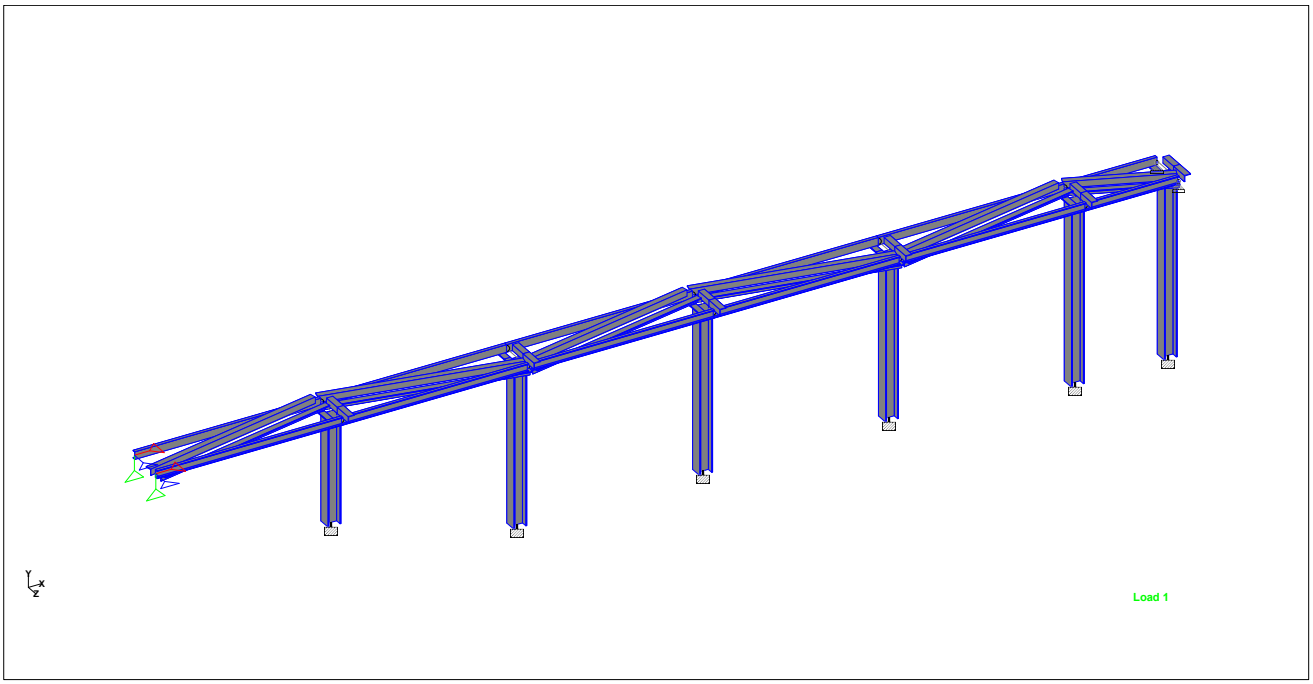

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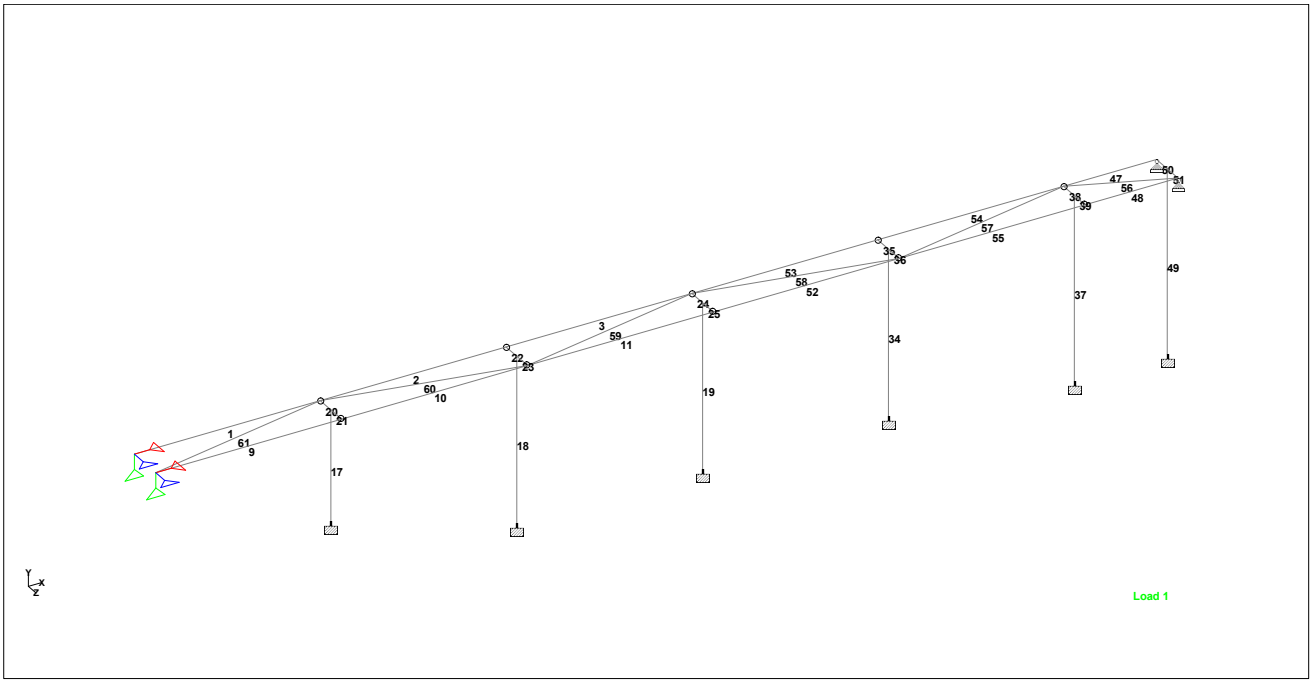

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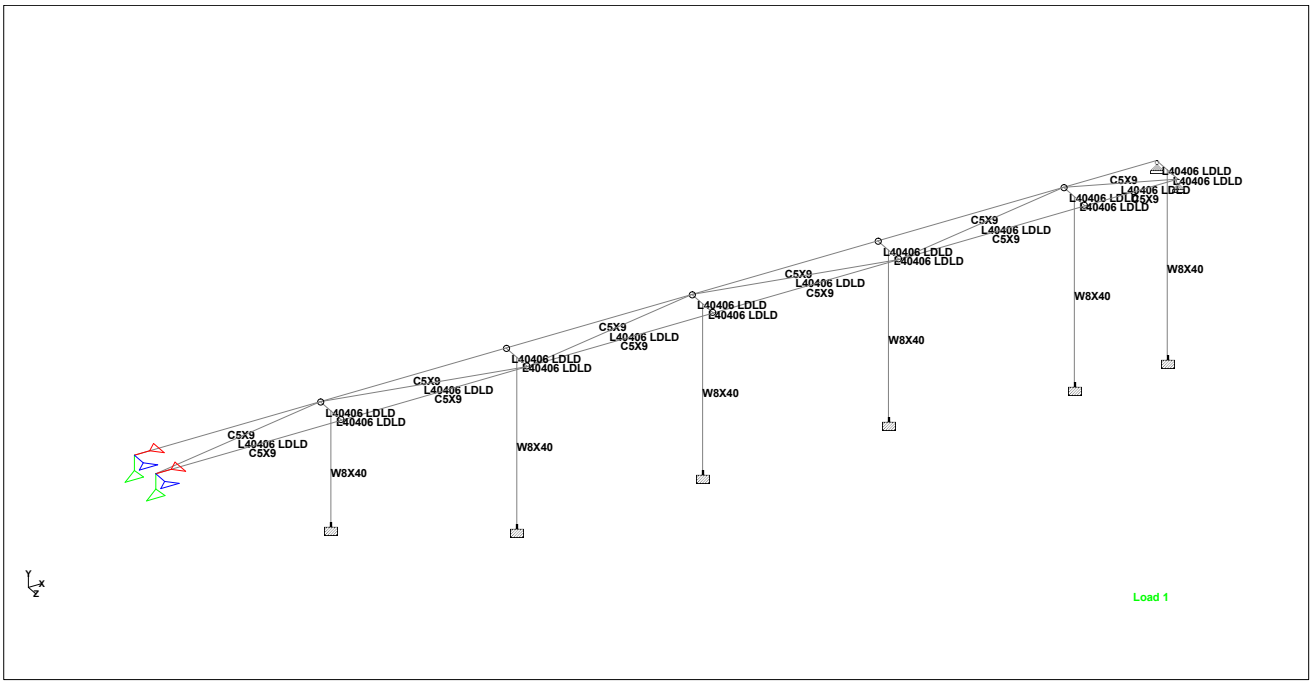

sec (Input data was modified after picture taken)

Node Displacement Summary

	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	2	9:DEFLECTIOI	**0.007**	0.009	0.052	0.053	0.003	-0.000	0.000
Min X	33	9:DEFLECTIOI	**-0.004**	-0.002	0.014	0.015	0.001	0.000	0.000
Max Y	4	3:WIND LOAD	0.001	**0.015**	0.094	0.096	0.001	0.000	-0.000
Min Y	13	9:DEFLECTIOI	0.003	**-0.019**	0.111	0.112	-0.001	0.000	0.000
Max Z	4	9:DEFLECTIOI	0.001	0.014	**0.111**	0.112	0.003	0.000	0.000
Min Z	19	10:1.2DL+1.6LI	-0.000	-0.001	**-0.001**	0.001	-0.000	0.000	0.000
Max rX	4	9:DEFLECTIOI	0.001	0.014	0.111	0.112	**0.003**	0.000	0.000
Min rX	30	10:1.2DL+1.6LI	-0.000	-0.005	0.000	0.005	**-0.002**	-0.000	-0.000
Max rY	29	9:DEFLECTIOI	-0.001	0.010	0.078	0.078	0.002	**0.000**	-0.000
Min rY	1	9:DEFLECTIOI	0.000	0.000	0.000	0.000	0.003	**-0.010**	-0.001
Max rZ	33	10:1.2DL+1.6LI	-0.000	-0.004	0.000	0.004	0.001	0.000	**0.000**
Min rZ	10	10:1.2DL+1.6LI	0.000	0.000	0.000	0.000	0.003	0.000	**-0.001**
Max Rst	13	9:DEFLECTIOI	0.003	-0.019	0.111	**0.112**	-0.001	0.000	0.000

Beam End Forces

Sign convention is as the action of the joint on the beam.

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
1	1	1:DEAD LOAD	0.023	-0.097	-0.000	0.000	-0.000	-0.000
		2:LIVE LOAD	0.002	-0.155	-0.000	-0.000	0.000	0.000
		3:WIND LOAD	-4.159	0.005	0.214	-0.000	0.000	0.000
		4:FLOOD LOAI	-0.659	0.001	-0.000	0.000	0.000	0.000
		10:1.2DL+1.6LI	0.030	-0.365	-0.000	-0.000	-0.000	-0.000
		11:1.2DL+LL+0	-2.709	-0.269	0.107	0.000	0.000	0.000
		12:0.9DL+0.5W	-2.718	-0.085	0.107	0.000	0.000	-0.000
		13:1.2DL+0.5W	-2.052	-0.114	0.107	-0.000	0.000	0.000
		5:1.2DL+1.6LL	0.030	-0.365	-0.000	-0.000	-0.000	-0.000
		6:1.2DL+LL+0.	-2.709	-0.269	0.107	-0.000	0.000	-0.000
		7:0.9DL+0.5W-	-2.718	-0.085	0.107	-0.000	0.000	-0.000
		8:1.2DL+0.5W	-2.052	-0.114	0.107	-0.000	0.000	-0.000
		9:DEFLECTIOI	-4.794	-0.247	0.214	-0.000	0.000	-0.000
	2	1:DEAD LOAD	-0.023	-0.152	0.000	0.000	0.002	3.299
		2:LIVE LOAD	-0.002	-0.245	0.000	0.000	0.000	5.402
		3:WIND LOAD	3.959	-0.005	0.356	0.000	8.542	0.568
		4:FLOOD LOAI	0.659	-0.001	0.000	0.000	0.002	0.086
		10:1.2DL+1.6LI	-0.030	-0.575	0.000	0.000	0.002	12.602
		11:1.2DL+LL+0	2.609	-0.431	0.178	-0.000	4.275	9.731
		12:0.9DL+0.5W	2.618	-0.140	0.178	-0.000	4.275	3.339
		13:1.2DL+0.5W	1.952	-0.185	0.178	0.000	4.273	4.242
		5:1.2DL+1.6LL	-0.030	-0.575	0.000	0.000	0.002	12.602
		6:1.2DL+LL+0.	2.609	-0.431	0.178	0.000	4.275	9.731
		7:0.9DL+0.5W-	2.618	-0.140	0.178	0.000	4.275	3.339

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		8:1.2DL+0.5W	1.952	-0.185	0.178	0.000	4.273	4.242
		9:DEFLECTIOI	4.594	-0.403	0.356	0.000	8.546	9.355
2	2	1:DEAD LOAD	-0.010	-0.126	0.000	0.000	-0.002	-2.705
		2:LIVE LOAD	-0.002	-0.206	-0.000	0.000	0.000	-4.569
		3:WIND LOAD	1.453	0.002	0.286	-0.002	-5.714	0.139
		4:FLOOD LOAI	0.229	-0.000	0.000	-0.000	-0.004	0.002
		10:1.2DL+1.6LI	-0.015	-0.481	0.000	0.001	-0.002	-10.556
		11:1.2DL+LL+0	0.942	-0.356	0.143	-0.000	-2.863	-7.743
		12:0.9DL+0.5W	0.947	-0.113	0.143	-0.001	-2.862	-2.363
		13:1.2DL+0.5W	0.715	-0.150	0.143	-0.001	-2.859	-3.177
		5:1.2DL+1.6LL	-0.015	-0.481	0.000	0.001	-0.002	-10.556
		6:1.2DL+LL+0.	0.942	-0.356	0.143	-0.000	-2.863	-7.743
		7:0.9DL+0.5W-	0.947	-0.113	0.143	-0.001	-2.862	-2.363
		8:1.2DL+0.5W	0.715	-0.150	0.143	-0.001	-2.859	-3.177
		9:DEFLECTIOI	1.671	-0.330	0.286	-0.001	-5.720	-7.133
	3	1:DEAD LOAD	0.010	-0.124	-0.000	-0.000	-0.001	2.570
		2:LIVE LOAD	0.002	-0.194	0.000	-0.000	0.000	3.854
		3:WIND LOAD	-1.653	-0.002	0.284	0.002	5.647	0.051
		4:FLOOD LOAI	-0.229	0.000	-0.000	0.000	-0.009	-0.012
		10:1.2DL+1.6LI	0.015	-0.459	-0.000	-0.001	-0.001	9.251
		11:1.2DL+LL+0	-1.042	-0.343	0.142	0.000	2.813	6.952
		12:0.9DL+0.5W	-1.047	-0.112	0.142	0.001	2.813	2.327
		13:1.2DL+0.5W	-0.815	-0.149	0.142	0.001	2.822	3.110
		5:1.2DL+1.6LL	0.015	-0.459	-0.000	-0.001	-0.001	9.251
		6:1.2DL+LL+0.	-1.042	-0.343	0.142	0.000	2.813	6.952
		7:0.9DL+0.5W-	-1.047	-0.112	0.142	0.001	2.813	2.327
		8:1.2DL+0.5W	-0.815	-0.149	0.142	0.001	2.822	3.110
		9:DEFLECTIOI	-1.871	-0.319	0.284	0.001	5.637	6.463
3	3	1:DEAD LOAD	-0.010	-0.127	-0.000	0.000	0.001	-2.574
		2:LIVE LOAD	-0.002	-0.199	0.000	0.000	-0.000	-3.859
		3:WIND LOAD	1.727	0.000	0.285	-0.002	-5.647	-0.043
		4:FLOOD LOAI	0.292	0.000	-0.000	-0.000	0.009	0.009
		10:1.2DL+1.6LI	-0.015	-0.470	-0.000	0.001	0.001	-9.264
		11:1.2DL+LL+0	1.142	-0.351	0.142	-0.000	-2.813	-6.960
		12:0.9DL+0.5W	1.147	-0.114	0.142	-0.001	-2.813	-2.329
		13:1.2DL+0.5W	0.852	-0.152	0.142	-0.001	-2.822	-3.111
		5:1.2DL+1.6LL	-0.015	-0.470	-0.000	0.001	0.001	-9.264
		6:1.2DL+LL+0.	1.142	-0.351	0.142	-0.000	-2.813	-6.960
		7:0.9DL+0.5W-	1.147	-0.114	0.142	-0.001	-2.813	-2.329
		8:1.2DL+0.5W	0.852	-0.152	0.142	-0.001	-2.822	-3.111
		9:DEFLECTIOI	2.008	-0.325	0.285	-0.001	-5.637	-6.467
	4	1:DEAD LOAD	0.010	-0.123	0.000	-0.000	0.001	2.310
		2:LIVE LOAD	0.002	-0.201	-0.000	-0.000	-0.000	4.020
		3:WIND LOAD	-1.927	-0.000	0.285	0.002	5.657	0.077

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		4:FLOOD LOA	-0.292	-0.000	0.000	0.000	-0.009	0.026
		10:1.2DL+1.6L	0.015	-0.469	0.000	-0.001	0.002	9.204
		11:1.2DL+LL+0	-1.242	-0.349	0.143	0.000	2.821	6.857
		12:0.9DL+0.5W	-1.247	-0.111	0.143	0.001	2.821	2.144
		13:1.2DL+0.5W	-0.952	-0.147	0.143	0.001	2.830	2.811
		5:1.2DL+1.6LL	0.015	-0.469	0.000	-0.001	0.002	9.204
		6:1.2DL+LL+0.	-1.242	-0.349	0.143	0.000	2.821	6.857
		7:0.9DL+0.5W-	-1.247	-0.111	0.143	0.001	2.821	2.144
		8:1.2DL+0.5W	-0.952	-0.147	0.143	0.001	2.830	2.811
		9:DEFLECTIOI	-2.208	-0.325	0.285	0.001	5.649	6.434
9	10	1:DEAD LOAD	0.014	0.095	0.000	0.006	-0.000	-0.260
		2:LIVE LOAD	0.001	0.161	0.000	0.009	-0.000	0.296
		3:WIND LOAD	-0.124	0.002	0.000	-0.001	-0.002	0.005
		4:FLOOD LOA	0.032	0.000	0.000	-0.000	-0.000	0.002
		10:1.2DL+1.6L	0.018	0.372	0.000	0.022	-0.000	0.162
		11:1.2DL+LL+0	-0.013	0.277	0.000	0.015	-0.001	-0.011
		12:0.9DL+0.5W	-0.018	0.087	0.000	0.005	-0.001	-0.229
		13:1.2DL+0.5W	-0.046	0.115	0.000	0.007	-0.001	-0.309
		5:1.2DL+1.6LL	0.018	0.372	0.000	0.022	-0.000	0.162
		6:1.2DL+LL+0.	-0.013	0.277	0.000	0.015	-0.001	-0.011
		7:0.9DL+0.5W-	-0.018	0.087	0.000	0.005	-0.001	-0.229
		8:1.2DL+0.5W	-0.046	0.115	0.000	0.007	-0.001	-0.309
		9:DEFLECTIOI	-0.078	0.259	0.000	0.013	-0.002	0.044
	11	1:DEAD LOAD	-0.014	0.155	-0.000	-0.006	-0.002	-3.304
		2:LIVE LOAD	-0.001	0.239	-0.000	-0.009	-0.000	-4.965
		3:WIND LOAD	-0.076	-0.002	-0.000	0.001	-0.004	0.232
		4:FLOOD LOA	-0.032	-0.000	-0.000	0.000	-0.003	0.044
		10:1.2DL+1.6L	-0.018	0.568	-0.000	-0.022	-0.002	-11.908
		11:1.2DL+LL+0	-0.087	0.423	-0.000	-0.015	-0.008	-8.770
		12:0.9DL+0.5W	-0.082	0.138	-0.000	-0.005	-0.007	-2.814
		13:1.2DL+0.5W	-0.054	0.184	-0.000	-0.007	-0.005	-3.849
		5:1.2DL+1.6LL	-0.018	0.568	-0.000	-0.022	-0.002	-11.908
		6:1.2DL+LL+0.	-0.087	0.423	-0.000	-0.015	-0.008	-8.770
		7:0.9DL+0.5W-	-0.082	0.138	-0.000	-0.005	-0.007	-2.814
		8:1.2DL+0.5W	-0.054	0.184	-0.000	-0.007	-0.005	-3.849
		9:DEFLECTIOI	-0.122	0.391	-0.000	-0.013	-0.010	-7.993
10	11	1:DEAD LOAD	0.014	0.134	-0.000	0.006	0.002	3.284
		2:LIVE LOAD	0.000	0.210	0.000	0.009	0.000	4.934
		3:WIND LOAD	0.079	-0.002	-0.000	-0.001	0.004	-0.232
		4:FLOOD LOA	0.056	-0.001	-0.000	-0.000	0.003	-0.044
		10:1.2DL+1.6L	0.017	0.498	-0.000	0.022	0.002	11.835
		11:1.2DL+LL+0	0.113	0.370	-0.000	0.015	0.008	8.714
		12:0.9DL+0.5W	0.108	0.119	-0.000	0.005	0.007	2.795
		13:1.2DL+0.5W	0.056	0.160	-0.000	0.007	0.005	3.824

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		5:1.2DL+1.6LL	0.017	0.498	-0.000	0.022	0.002	11.835
		6:1.2DL+LL+0.	0.113	0.370	-0.000	0.015	0.008	8.714
		7:0.9DL+0.5W-	0.108	0.119	-0.000	0.005	0.007	2.795
		8:1.2DL+0.5W	0.056	0.160	-0.000	0.007	0.005	3.824
		9:DEFLECTIOI	0.150	0.342	-0.001	0.013	0.010	7.942
	12	1:DEAD LOAD	-0.014	0.115	0.000	-0.006	0.001	-2.133
		2:LIVE LOAD	-0.000	0.190	-0.000	-0.009	-0.000	-3.702
		3:WIND LOAD	-0.279	0.002	0.000	0.001	0.051	-0.017
		4:FLOOD LOAI	-0.056	0.001	0.000	0.000	0.009	-0.018
		10:1.2DL+1.6LI	-0.017	0.442	0.000	-0.022	0.001	-8.483
		11:1.2DL+LL+0	-0.213	0.330	0.000	-0.015	0.036	-6.288
		12:0.9DL+0.5W	-0.208	0.105	0.000	-0.005	0.036	-1.946
		13:1.2DL+0.5W	-0.156	0.139	0.000	-0.007	0.027	-2.568
		5:1.2DL+1.6LL	-0.017	0.442	0.000	-0.022	0.001	-8.483
		6:1.2DL+LL+0.	-0.213	0.330	0.000	-0.015	0.036	-6.288
		7:0.9DL+0.5W-	-0.208	0.105	0.000	-0.005	0.036	-1.946
		8:1.2DL+0.5W	-0.156	0.139	0.000	-0.007	0.027	-2.568
		9:DEFLECTIOI	-0.350	0.308	0.001	-0.013	0.062	-5.870
11	12	1:DEAD LOAD	-0.003	0.121	0.000	-0.000	-0.001	2.212
		2:LIVE LOAD	-0.001	0.199	-0.000	0.001	0.000	3.852
		3:WIND LOAD	-2.130	0.004	0.000	0.002	-0.049	0.162
		4:FLOOD LOAI	-0.391	0.001	-0.000	0.000	-0.008	0.038
		10:1.2DL+1.6LI	-0.004	0.463	0.000	0.001	-0.001	8.817
		11:1.2DL+LL+0	-1.460	0.346	0.000	0.001	-0.034	6.625
		12:0.9DL+0.5W	-1.459	0.111	0.000	0.001	-0.034	2.110
		13:1.2DL+0.5W	-1.068	0.147	0.000	0.000	-0.026	2.736
		5:1.2DL+1.6LL	-0.004	0.463	0.000	0.001	-0.001	8.817
		6:1.2DL+LL+0.	-1.460	0.346	0.000	0.001	-0.034	6.625
		7:0.9DL+0.5W-	-1.459	0.111	0.000	0.001	-0.034	2.110
		8:1.2DL+0.5W	-1.068	0.147	0.000	0.000	-0.026	2.736
		9:DEFLECTIOI	-2.524	0.324	0.000	0.002	-0.058	6.264
	13	1:DEAD LOAD	0.003	0.129	-0.000	0.000	-0.001	-2.699
		2:LIVE LOAD	0.001	0.201	0.000	-0.001	0.000	-3.987
		3:WIND LOAD	1.930	-0.004	-0.000	-0.002	0.046	0.260
		4:FLOOD LOAI	0.391	-0.001	0.000	-0.000	0.009	0.050
		10:1.2DL+1.6LI	0.004	0.476	-0.000	-0.001	-0.002	-9.619
		11:1.2DL+LL+0	1.360	0.353	-0.000	-0.001	0.030	-7.046
		12:0.9DL+0.5W	1.359	0.114	-0.000	-0.001	0.031	-2.249
		13:1.2DL+0.5W	0.968	0.153	-0.000	-0.000	0.021	-3.109
		5:1.2DL+1.6LL	0.004	0.476	-0.000	-0.001	-0.002	-9.619
		6:1.2DL+LL+0.	1.360	0.353	-0.000	-0.001	0.030	-7.046
		7:0.9DL+0.5W-	1.359	0.114	-0.000	-0.001	0.031	-2.249
		8:1.2DL+0.5W	0.968	0.153	-0.000	-0.000	0.021	-3.109
		9:DEFLECTIOI	2.324	0.326	-0.000	-0.002	0.054	-6.376

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
17	22	1:DEAD LOAD	1.042	-0.000	-0.040	-0.000	0.639	-0.053
		2:LIVE LOAD	0.900	0.001	-0.000	-0.000	-0.009	0.015
		3:WIND LOAD	-0.002	0.121	-0.533	0.046	37.272	7.708
		4:FLOOD LOAI	-0.000	0.036	-0.099	0.008	6.131	1.571
		10:1.2DL+1.6LI	2.691	0.001	-0.048	-0.000	0.753	-0.039
		11:1.2DL+LL+0	2.150	0.097	-0.414	0.031	25.526	5.376
		12:0.9DL+0.5W	0.937	0.096	-0.402	0.031	25.343	5.377
		13:1.2DL+0.5W	1.250	0.060	-0.314	0.023	19.403	3.791
		5:1.2DL+1.6LL	2.691	0.001	-0.048	-0.000	0.753	-0.039
		6:1.2DL+LL+0.	2.150	0.097	-0.414	0.031	25.526	5.376
		7:0.9DL+0.5W-	0.937	0.096	-0.402	0.031	25.343	5.377
		8:1.2DL+0.5W	1.250	0.060	-0.314	0.023	19.403	3.791
		9:DEFLECTIOI	1.940	0.158	-0.672	0.053	44.034	9.241
	19	1:DEAD LOAD	-0.804	0.000	0.040	0.000	2.247	0.033
		2:LIVE LOAD	-0.900	-0.001	0.000	0.000	0.018	0.050
		3:WIND LOAD	0.002	-0.037	0.449	-0.046	0.076	0.016
		4:FLOOD LOAI	0.000	0.048	0.015	-0.008	0.019	0.002
		10:1.2DL+1.6LI	-2.404	-0.001	0.048	0.000	2.725	0.119
		11:1.2DL+LL+0	-1.863	0.029	0.288	-0.031	2.771	0.099
		12:0.9DL+0.5W	-0.722	0.030	0.276	-0.031	2.079	0.039
		13:1.2DL+0.5W	-0.964	-0.018	0.272	-0.023	2.734	0.047
		5:1.2DL+1.6LL	-2.404	-0.001	0.048	0.000	2.725	0.119
		6:1.2DL+LL+0.	-1.863	0.029	0.288	-0.031	2.771	0.099
		7:0.9DL+0.5W-	-0.722	0.030	0.276	-0.031	2.079	0.039
		8:1.2DL+0.5W	-0.964	-0.018	0.272	-0.023	2.734	0.047
		9:DEFLECTIOI	-1.702	0.010	0.504	-0.053	2.359	0.101
18	23	1:DEAD LOAD	1.080	-0.000	0.031	0.000	-1.174	-0.037
		2:LIVE LOAD	0.779	-0.000	-0.001	0.000	0.025	-0.005
		3:WIND LOAD	0.003	0.100	-0.372	0.020	33.870	4.531
		4:FLOOD LOAI	0.000	0.085	-0.131	0.004	7.865	2.862
		10:1.2DL+1.6LI	2.543	-0.001	0.036	0.000	-1.369	-0.054
		11:1.2DL+LL+0	2.078	0.134	-0.281	0.014	23.417	5.077
		12:0.9DL+0.5W	0.974	0.135	-0.289	0.014	23.744	5.094
		13:1.2DL+0.5W	1.298	0.050	-0.149	0.010	15.527	2.221
		5:1.2DL+1.6LL	2.543	-0.001	0.036	0.000	-1.369	-0.054
		6:1.2DL+LL+0.	2.078	0.134	-0.281	0.014	23.417	5.077
		7:0.9DL+0.5W-	0.974	0.135	-0.289	0.014	23.744	5.094
		8:1.2DL+0.5W	1.298	0.050	-0.149	0.010	15.527	2.221
		9:DEFLECTIOI	1.863	0.185	-0.473	0.024	40.587	7.350
	20	1:DEAD LOAD	-0.722	0.000	-0.031	-0.000	-2.185	-0.008
		2:LIVE LOAD	-0.779	0.000	0.001	-0.000	0.072	-0.011
		3:WIND LOAD	-0.003	0.110	0.162	-0.020	-0.008	0.008
		4:FLOOD LOAI	-0.000	0.125	-0.079	-0.004	-0.015	-0.006
		10:1.2DL+1.6LI	-2.114	0.001	-0.036	-0.000	-2.507	-0.027

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		11:1.2DL+LL+0	-1.648	0.181	-0.034	-0.014	-2.569	-0.023
		12:0.9DL+0.5W	-0.652	0.180	-0.026	-0.014	-1.985	-0.009
		13:1.2DL+0.5W	-0.868	0.055	0.044	-0.010	-2.626	-0.006
		5:1.2DL+1.6LL	-2.114	0.001	-0.036	-0.000	-2.507	-0.027
		6:1.2DL+LL+0.	-1.648	0.181	-0.034	-0.014	-2.569	-0.023
		7:0.9DL+0.5W-	-0.652	0.180	-0.026	-0.014	-1.985	-0.009
		8:1.2DL+0.5W	-0.868	0.055	0.044	-0.010	-2.626	-0.006
		9:DEFLECTIOI	-1.506	0.235	0.053	-0.024	-2.136	-0.017
19	24	1:DEAD LOAD	1.092	-0.000	-0.029	0.000	0.978	-0.022
		2:LIVE LOAD	0.799	0.000	-0.000	0.000	0.005	0.004
		3:WIND LOAD	-0.003	0.102	-0.403	-0.009	37.091	4.698
		4:FLOOD LOAI	-0.000	0.086	-0.139	-0.001	8.688	2.944
		10:1.2DL+1.6LI	2.589	-0.000	-0.035	0.000	1.182	-0.021
		11:1.2DL+LL+0	2.108	0.136	-0.376	-0.006	28.413	5.270
		12:0.9DL+0.5W	0.981	0.136	-0.367	-0.006	28.114	5.273
		13:1.2DL+0.5W	1.309	0.051	-0.237	-0.004	19.720	2.322
		5:1.2DL+1.6LL	2.589	-0.000	-0.035	0.000	1.182	-0.021
		6:1.2DL+LL+0.	2.108	0.136	-0.376	-0.006	28.413	5.270
		7:0.9DL+0.5W-	0.981	0.136	-0.367	-0.006	28.114	5.273
		8:1.2DL+0.5W	1.309	0.051	-0.237	-0.004	19.720	2.322
		9:DEFLECTIOI	1.888	0.187	-0.572	-0.010	46.763	7.623
	21	1:DEAD LOAD	-0.735	0.000	0.029	-0.000	2.182	0.003
		2:LIVE LOAD	-0.799	-0.000	0.000	-0.000	0.006	0.005
		3:WIND LOAD	0.003	0.108	0.193	0.009	0.148	0.008
		4:FLOOD LOAI	0.000	0.124	-0.071	0.001	0.029	-0.006
		10:1.2DL+1.6LI	-2.160	0.000	0.035	-0.000	2.628	0.012
		11:1.2DL+LL+0	-1.679	0.179	0.061	0.006	2.727	0.007
		12:0.9DL+0.5W	-0.659	0.179	0.052	0.006	2.067	0.001
		13:1.2DL+0.5W	-0.880	0.054	0.132	0.004	2.692	0.008
		5:1.2DL+1.6LL	-2.160	0.000	0.035	-0.000	2.628	0.012
		6:1.2DL+LL+0.	-1.679	0.179	0.061	0.006	2.727	0.007
		7:0.9DL+0.5W-	-0.659	0.179	0.052	0.006	2.067	0.001
		8:1.2DL+0.5W	-0.880	0.054	0.132	0.004	2.692	0.008
		9:DEFLECTIOI	-1.530	0.233	0.152	0.010	2.365	0.010
20	2	1:DEAD LOAD	0.040	-0.476	0.000	0.012	0.000	0.000
		2:LIVE LOAD	0.000	-0.451	-0.000	0.019	0.000	0.000
		3:WIND LOAD	0.448	-0.002	-0.001	0.016	0.000	0.000
		4:FLOOD LOAI	0.015	-0.001	0.024	0.002	0.000	0.000
		10:1.2DL+1.6LI	0.048	-1.292	-0.001	0.046	0.000	0.000
		11:1.2DL+LL+0	0.288	-1.024	0.023	0.044	0.000	0.000
		12:0.9DL+0.5W	0.275	-0.430	0.024	0.021	0.000	0.000
		13:1.2DL+0.5W	0.272	-0.573	-0.000	0.023	0.000	0.000
		5:1.2DL+1.6LL	0.048	-1.292	-0.001	0.046	0.000	0.000
		6:1.2DL+LL+0.	0.288	-1.024	0.023	0.044	0.000	0.000

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		7:0.9DL+0.5W-	0.275	-0.430	0.024	0.021	0.000	0.000
		8:1.2DL+0.5W	0.272	-0.573	-0.000	0.023	0.000	0.000
		9:DEFLECTIOI	0.504	-0.930	0.023	0.049	0.000	0.000
	19	1:DEAD LOAD	-0.040	0.496	-0.000	-0.012	-0.002	-5.831
		2:LIVE LOAD	-0.000	0.451	0.000	-0.019	0.005	-5.408
		3:WIND LOAD	-0.448	0.002	0.021	-0.016	0.127	-0.027
		4:FLOOD LOAI	-0.015	0.001	-0.024	-0.002	-0.285	-0.008
		10:1.2DL+1.6LI	-0.048	1.316	0.001	-0.046	0.007	-15.649
		11:1.2DL+LL+0	-0.288	1.047	-0.013	-0.044	-0.218	-12.426
		12:0.9DL+0.5W	-0.275	0.448	-0.014	-0.021	-0.223	-5.269
		13:1.2DL+0.5W	-0.272	0.596	0.010	-0.023	0.061	-7.010
		5:1.2DL+1.6LL	-0.048	1.316	0.001	-0.046	0.007	-15.649
		6:1.2DL+LL+0.	-0.288	1.047	-0.013	-0.044	-0.218	-12.426
		7:0.9DL+0.5W-	-0.275	0.448	-0.014	-0.021	-0.223	-5.269
		8:1.2DL+0.5W	-0.272	0.596	0.010	-0.023	0.061	-7.010
		9:DEFLECTIOI	-0.504	0.949	-0.003	-0.049	-0.155	-11.274
21	19	1:DEAD LOAD	-0.000	0.308	-0.000	-0.020	0.002	3.584
		2:LIVE LOAD	0.000	0.449	0.000	-0.031	-0.005	5.390
		3:WIND LOAD	-0.001	-0.004	0.017	-0.001	-0.081	-0.049
		4:FLOOD LOAI	-0.000	-0.001	-0.024	0.000	0.293	-0.011
		10:1.2DL+1.6LI	-0.000	1.089	0.001	-0.073	-0.007	12.925
		11:1.2DL+LL+0	-0.000	0.816	-0.016	-0.055	0.249	9.656
		12:0.9DL+0.5W	-0.000	0.275	-0.016	-0.019	0.254	3.190
		13:1.2DL+0.5W	-0.000	0.368	0.008	-0.025	-0.038	4.276
		5:1.2DL+1.6LL	-0.000	1.089	0.001	-0.073	-0.007	12.925
		6:1.2DL+LL+0.	-0.000	0.816	-0.016	-0.055	0.249	9.656
		7:0.9DL+0.5W-	-0.000	0.275	-0.016	-0.019	0.254	3.190
		8:1.2DL+0.5W	-0.000	0.368	0.008	-0.025	-0.038	4.276
		9:DEFLECTIOI	-0.001	0.753	-0.007	-0.052	0.208	8.915
	11	1:DEAD LOAD	0.000	-0.289	0.000	0.020	0.000	0.000
		2:LIVE LOAD	-0.000	-0.449	-0.000	0.031	0.000	0.000
		3:WIND LOAD	0.001	0.004	0.003	0.001	0.000	0.000
		4:FLOOD LOAI	0.000	0.001	0.024	-0.000	0.000	0.000
		10:1.2DL+1.6LI	0.000	-1.065	-0.001	0.073	0.000	0.000
		11:1.2DL+LL+0	0.000	-0.793	0.026	0.055	0.000	0.000
		12:0.9DL+0.5W	0.000	-0.257	0.026	0.019	0.000	0.000
		13:1.2DL+0.5W	0.000	-0.345	0.002	0.025	0.000	0.000
		5:1.2DL+1.6LL	0.000	-1.065	-0.001	0.073	0.000	0.000
		6:1.2DL+LL+0.	0.000	-0.793	0.026	0.055	0.000	0.000
		7:0.9DL+0.5W-	0.000	-0.257	0.026	0.019	0.000	0.000
		8:1.2DL+0.5W	0.000	-0.345	0.002	0.025	0.000	0.000
		9:DEFLECTIOI	0.001	-0.733	0.027	0.052	0.000	0.000
22	3	1:DEAD LOAD	-0.000	-0.251	0.000	-0.004	0.000	0.000
		2:LIVE LOAD	0.000	-0.393	0.000	-0.005	0.000	0.000

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		3:WIND LOAD	0.569	-0.001	0.074	0.008	0.000	0.000
		4:FLOOD LOAI	-0.000	0.000	0.062	-0.003	0.000	0.000
		10:1.2DL+1.6LI	-0.000	-0.929	0.000	-0.013	0.000	0.000
		11:1.2DL+LL+0	0.285	-0.694	0.100	-0.008	0.000	0.000
		12:0.9DL+0.5W	0.285	-0.226	0.100	-0.002	0.000	0.000
		13:1.2DL+0.5W	0.285	-0.302	0.037	-0.001	0.000	0.000
		5:1.2DL+1.6LL	-0.000	-0.929	0.000	-0.013	0.000	0.000
		6:1.2DL+LL+0.	0.285	-0.694	0.100	-0.008	0.000	0.000
		7:0.9DL+0.5W-	0.285	-0.226	0.100	-0.002	0.000	0.000
		8:1.2DL+0.5W	0.285	-0.302	0.037	-0.001	0.000	0.000
		9:DEFLECTIOI	0.569	-0.644	0.137	-0.004	0.000	0.000
	20	1:DEAD LOAD	0.000	0.270	-0.000	0.004	-0.002	-3.125
		2:LIVE LOAD	-0.000	0.393	-0.000	0.005	-0.001	-4.712
		3:WIND LOAD	-0.569	0.001	-0.054	-0.008	-0.768	-0.016
		4:FLOOD LOAI	0.000	-0.000	-0.062	0.003	-0.749	0.005
		10:1.2DL+1.6LI	0.000	0.953	-0.000	0.013	-0.004	-11.290
		11:1.2DL+LL+0	-0.285	0.717	-0.090	0.008	-1.137	-8.466
		12:0.9DL+0.5W	-0.285	0.243	-0.090	0.002	-1.135	-2.816
		13:1.2DL+0.5W	-0.285	0.325	-0.027	0.001	-0.387	-3.758
		5:1.2DL+1.6LL	0.000	0.953	-0.000	0.013	-0.004	-11.290
		6:1.2DL+LL+0.	-0.285	0.717	-0.090	0.008	-1.137	-8.466
		7:0.9DL+0.5W-	-0.285	0.243	-0.090	0.002	-1.135	-2.816
		8:1.2DL+0.5W	-0.285	0.325	-0.027	0.001	-0.387	-3.758
		9:DEFLECTIOI	-0.569	0.664	-0.117	0.004	-1.521	-7.849
23	20	1:DEAD LOAD	0.031	0.452	-0.000	0.004	0.003	5.310
		2:LIVE LOAD	-0.001	0.387	-0.000	0.006	0.001	4.641
		3:WIND LOAD	0.407	0.002	-0.056	-0.000	0.788	0.023
		4:FLOOD LOAI	0.079	0.001	-0.063	0.004	0.753	0.010
		10:1.2DL+1.6LI	0.036	1.161	-0.000	0.015	0.005	13.798
		11:1.2DL+LL+0	0.319	0.931	-0.091	0.015	1.151	11.035
		12:0.9DL+0.5W	0.311	0.409	-0.091	0.007	1.150	4.801
		13:1.2DL+0.5W	0.241	0.544	-0.028	0.005	0.397	6.384
		5:1.2DL+1.6LL	0.036	1.161	-0.000	0.015	0.005	13.798
		6:1.2DL+LL+0.	0.319	0.931	-0.091	0.015	1.151	11.035
		7:0.9DL+0.5W-	0.311	0.409	-0.091	0.007	1.150	4.801
		8:1.2DL+0.5W	0.241	0.544	-0.028	0.005	0.397	6.384
		9:DEFLECTIOI	0.517	0.842	-0.119	0.014	1.545	9.985
	12	1:DEAD LOAD	-0.031	-0.433	0.000	-0.004	0.000	0.000
		2:LIVE LOAD	0.001	-0.387	0.000	-0.006	0.000	0.000
		3:WIND LOAD	-0.407	-0.002	0.076	0.000	0.000	0.000
		4:FLOOD LOAI	-0.079	-0.001	0.063	-0.004	0.000	0.000
		10:1.2DL+1.6LI	-0.036	-1.138	0.000	-0.015	0.000	0.000
		11:1.2DL+LL+0	-0.319	-0.908	0.101	-0.015	0.000	0.000
		12:0.9DL+0.5W	-0.311	-0.391	0.101	-0.007	0.000	0.000

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		13:1.2DL+0.5W	-0.241	-0.520	0.038	-0.005	0.000	0.000
		5:1.2DL+1.6LL	-0.036	-1.138	0.000	-0.015	0.000	0.000
		6:1.2DL+LL+0.	-0.319	-0.908	0.101	-0.015	0.000	0.000
		7:0.9DL+0.5W-	-0.311	-0.391	0.101	-0.007	0.000	0.000
		8:1.2DL+0.5W	-0.241	-0.520	0.038	-0.005	0.000	0.000
		9:DEFLECTIOI	-0.517	-0.822	0.139	-0.014	0.000	0.000
24	4	1:DEAD LOAD	0.029	-0.439	0.000	0.002	0.000	0.000
		2:LIVE LOAD	0.000	-0.400	-0.000	0.002	0.000	0.000
		3:WIND LOAD	0.193	-0.005	0.074	0.001	0.000	0.000
		4:FLOOD LOAI	-0.071	-0.001	0.062	-0.003	0.000	0.000
		10:1.2DL+1.6LI	0.035	-1.166	0.000	0.006	0.000	0.000
		11:1.2DL+LL+0	0.061	-0.930	0.100	0.002	0.000	0.000
		12:0.9DL+0.5W	0.052	-0.398	0.100	-0.000	0.000	0.000
		13:1.2DL+0.5W	0.132	-0.529	0.037	0.004	0.000	0.000
		5:1.2DL+1.6LL	0.035	-1.166	0.000	0.006	0.000	0.000
		6:1.2DL+LL+0.	0.061	-0.930	0.100	0.002	0.000	0.000
		7:0.9DL+0.5W-	0.052	-0.398	0.100	-0.000	0.000	0.000
		8:1.2DL+0.5W	0.132	-0.529	0.037	0.004	0.000	0.000
		9:DEFLECTIOI	0.151	-0.844	0.137	0.002	0.000	0.000
	21	1:DEAD LOAD	-0.029	0.458	-0.000	-0.002	-0.001	-5.383
		2:LIVE LOAD	-0.000	0.400	0.000	-0.002	0.001	-4.797
		3:WIND LOAD	-0.193	0.005	-0.054	-0.001	-0.773	-0.056
		4:FLOOD LOAI	0.071	0.001	-0.062	0.003	-0.747	-0.012
		10:1.2DL+1.6LI	-0.035	1.190	-0.000	-0.006	-0.000	-14.135
		11:1.2DL+LL+0	-0.061	0.953	-0.090	-0.002	-1.135	-11.297
		12:0.9DL+0.5W	-0.052	0.416	-0.090	0.000	-1.135	-4.885
		13:1.2DL+0.5W	-0.132	0.552	-0.027	-0.004	-0.388	-6.488
		5:1.2DL+1.6LL	-0.035	1.190	-0.000	-0.006	-0.000	-14.135
		6:1.2DL+LL+0.	-0.061	0.953	-0.090	-0.002	-1.135	-11.297
		7:0.9DL+0.5W-	-0.052	0.416	-0.090	0.000	-1.135	-4.885
		8:1.2DL+0.5W	-0.132	0.552	-0.027	-0.004	-0.388	-6.488
		9:DEFLECTIOI	-0.151	0.864	-0.117	-0.002	-1.521	-10.248
25	21	1:DEAD LOAD	-0.000	0.276	-0.000	-0.001	0.001	3.201
		2:LIVE LOAD	-0.000	0.399	0.000	-0.003	-0.000	4.791
		3:WIND LOAD	-0.000	-0.008	-0.054	-0.006	0.764	-0.093
		4:FLOOD LOAI	0.000	-0.001	-0.062	0.003	0.746	-0.017
		10:1.2DL+1.6LI	-0.000	0.971	-0.000	-0.006	0.001	11.507
		11:1.2DL+LL+0	-0.000	0.726	-0.089	-0.004	1.129	8.569
		12:0.9DL+0.5W	-0.000	0.244	-0.089	-0.001	1.129	2.818
		13:1.2DL+0.5W	-0.000	0.328	-0.027	-0.004	0.383	3.795
		5:1.2DL+1.6LL	-0.000	0.971	-0.000	-0.006	0.001	11.507
		6:1.2DL+LL+0.	-0.000	0.726	-0.089	-0.004	1.129	8.569
		7:0.9DL+0.5W-	-0.000	0.244	-0.089	-0.001	1.129	2.818
		8:1.2DL+0.5W	-0.000	0.328	-0.027	-0.004	0.383	3.795

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		9:DEFLECTIOI	-0.000	0.667	-0.116	-0.008	1.511	7.883
	13	1:DEAD LOAD	0.000	-0.257	0.000	0.001	0.000	0.000
		2:LIVE LOAD	0.000	-0.399	-0.000	0.003	0.000	0.000
		3:WIND LOAD	0.000	0.008	0.074	0.006	0.000	0.000
		4:FLOOD LOAI	-0.000	0.001	0.062	-0.003	0.000	0.000
		10:1.2DL+1.6LI	0.000	-0.947	0.000	0.006	0.000	0.000
		11:1.2DL+LL+0	0.000	-0.702	0.099	0.004	0.000	0.000
		12:0.9DL+0.5W	0.000	-0.226	0.099	0.001	0.000	0.000
		13:1.2DL+0.5W	0.000	-0.305	0.037	0.004	0.000	0.000
		5:1.2DL+1.6LL	0.000	-0.947	0.000	0.006	0.000	0.000
		6:1.2DL+LL+0.	0.000	-0.702	0.099	0.004	0.000	0.000
		7:0.9DL+0.5W-	0.000	-0.226	0.099	0.001	0.000	0.000
		8:1.2DL+0.5W	0.000	-0.305	0.037	0.004	0.000	0.000
		9:DEFLECTIOI	0.000	-0.647	0.136	0.008	0.000	0.000
34	32	1:DEAD LOAD	1.109	-0.000	0.028	0.000	-0.967	-0.029
		2:LIVE LOAD	0.826	-0.000	0.002	0.000	-0.046	-0.006
		3:WIND LOAD	-0.001	0.106	-0.307	-0.032	26.691	5.128
		4:FLOOD LOAI	-0.000	0.086	-0.123	-0.005	6.951	3.029
		10:1.2DL+1.6LI	2.653	-0.001	0.036	0.000	-1.234	-0.044
		11:1.2DL+LL+0	2.157	0.139	-0.242	-0.021	19.090	5.553
		12:0.9DL+0.5W	0.998	0.139	-0.252	-0.021	19.426	5.567
		13:1.2DL+0.5W	1.331	0.052	-0.120	-0.016	12.185	2.529
		5:1.2DL+1.6LL	2.653	-0.001	0.036	0.000	-1.234	-0.044
		6:1.2DL+LL+0.	2.157	0.139	-0.242	-0.021	19.090	5.553
		7:0.9DL+0.5W-	0.998	0.139	-0.252	-0.021	19.426	5.567
		8:1.2DL+0.5W	1.331	0.052	-0.120	-0.016	12.185	2.529
		9:DEFLECTIOI	1.934	0.192	-0.401	-0.037	32.629	8.123
	31	1:DEAD LOAD	-0.752	0.000	-0.028	-0.000	-2.047	-0.010
		2:LIVE LOAD	-0.826	0.000	-0.002	-0.000	-0.118	-0.014
		3:WIND LOAD	0.001	0.104	0.097	0.032	0.179	-0.007
		4:FLOOD LOAI	0.000	0.124	-0.087	0.005	0.052	-0.009
		10:1.2DL+1.6LI	-2.224	0.001	-0.036	-0.000	-2.646	-0.033
		11:1.2DL+LL+0	-1.728	0.176	-0.073	0.021	-2.434	-0.038
		12:0.9DL+0.5W	-0.676	0.176	-0.063	0.021	-1.701	-0.021
		13:1.2DL+0.5W	-0.902	0.053	0.015	0.016	-2.367	-0.015
		5:1.2DL+1.6LL	-2.224	0.001	-0.036	-0.000	-2.646	-0.033
		6:1.2DL+LL+0.	-1.728	0.176	-0.073	0.021	-2.434	-0.038
		7:0.9DL+0.5W-	-0.676	0.176	-0.063	0.021	-1.701	-0.021
		8:1.2DL+0.5W	-0.902	0.053	0.015	0.016	-2.367	-0.015
		9:DEFLECTIOI	-1.577	0.228	-0.019	0.037	-1.935	-0.039
35	29	1:DEAD LOAD	-0.000	-0.271	0.000	-0.010	0.000	0.000
		2:LIVE LOAD	-0.000	-0.408	0.000	-0.006	0.000	0.000
		3:WIND LOAD	0.569	-0.007	0.073	-0.017	0.000	0.000
		4:FLOOD LOAI	-0.000	-0.002	0.062	-0.007	0.000	0.000

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		10:1.2DL+1.6LI	-0.000	-0.979	0.000	-0.021	0.000	0.000
		11:1.2DL+LL+C	0.284	-0.739	0.099	-0.033	0.000	0.000
		12:0.9DL+0.5W	0.284	-0.249	0.099	-0.025	0.000	0.000
		13:1.2DL+0.5W	0.284	-0.329	0.037	-0.021	0.000	0.000
		5:1.2DL+1.6LL	-0.000	-0.979	0.000	-0.021	0.000	0.000
		6:1.2DL+LL+0.	0.284	-0.739	0.099	-0.033	0.000	0.000
		7:0.9DL+0.5W-	0.284	-0.249	0.099	-0.025	0.000	0.000
		8:1.2DL+0.5W	0.284	-0.329	0.037	-0.021	0.000	0.000
		9:DEFLECTIOI	0.568	-0.688	0.136	-0.040	0.000	0.000
	31	1:DEAD LOAD	0.000	0.290	-0.000	0.010	-0.002	-3.369
		2:LIVE LOAD	0.000	0.408	-0.000	0.006	-0.001	-4.899
		3:WIND LOAD	-0.569	0.007	-0.053	0.017	-0.761	-0.085
		4:FLOOD LOAI	0.000	0.002	-0.062	0.007	-0.745	-0.024
		10:1.2DL+1.6LI	0.000	1.002	-0.000	0.021	-0.004	-11.882
		11:1.2DL+LL+C	-0.284	0.762	-0.089	0.033	-1.129	-9.009
		12:0.9DL+0.5W	-0.284	0.267	-0.089	0.025	-1.127	-3.099
		13:1.2DL+0.5W	-0.284	0.352	-0.027	0.021	-0.383	-4.086
		5:1.2DL+1.6LL	0.000	1.002	-0.000	0.021	-0.004	-11.882
		6:1.2DL+LL+0.	-0.284	0.762	-0.089	0.033	-1.129	-9.009
		7:0.9DL+0.5W-	-0.284	0.267	-0.089	0.025	-1.127	-3.099
		8:1.2DL+0.5W	-0.284	0.352	-0.027	0.021	-0.383	-4.086
		9:DEFLECTIOI	-0.568	0.708	-0.116	0.040	-1.509	-8.377
36	31	1:DEAD LOAD	0.028	0.461	-0.000	-0.001	0.002	5.416
		2:LIVE LOAD	0.002	0.418	-0.000	0.008	0.001	5.018
		3:WIND LOAD	0.472	-0.008	-0.051	-0.009	0.730	-0.094
		4:FLOOD LOAI	0.087	-0.002	-0.062	0.002	0.739	-0.028
		10:1.2DL+1.6LI	0.036	1.222	-0.000	0.012	0.004	14.528
		11:1.2DL+LL+C	0.357	0.965	-0.087	0.004	1.108	11.442
		12:0.9DL+0.5W	0.347	0.409	-0.087	-0.004	1.106	4.800
		13:1.2DL+0.5W	0.269	0.549	-0.026	-0.005	0.368	6.453
		5:1.2DL+1.6LL	0.036	1.222	-0.000	0.012	0.004	14.528
		6:1.2DL+LL+0.	0.357	0.965	-0.087	0.004	1.108	11.442
		7:0.9DL+0.5W-	0.347	0.409	-0.087	-0.004	1.106	4.800
		8:1.2DL+0.5W	0.269	0.549	-0.026	-0.005	0.368	6.453
		9:DEFLECTIOI	0.588	0.869	-0.113	-0.000	1.472	10.312
	30	1:DEAD LOAD	-0.028	-0.442	0.000	0.001	0.000	0.000
		2:LIVE LOAD	-0.002	-0.418	0.000	-0.008	0.000	0.000
		3:WIND LOAD	-0.472	0.008	0.071	0.009	0.000	0.000
		4:FLOOD LOAI	-0.087	0.002	0.062	-0.002	0.000	0.000
		10:1.2DL+1.6LI	-0.036	-1.199	0.000	-0.012	0.000	0.000
		11:1.2DL+LL+C	-0.357	-0.942	0.097	-0.004	0.000	0.000
		12:0.9DL+0.5W	-0.347	-0.391	0.097	0.004	0.000	0.000
		13:1.2DL+0.5W	-0.269	-0.526	0.036	0.005	0.000	0.000
		5:1.2DL+1.6LL	-0.036	-1.199	0.000	-0.012	0.000	0.000

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		6:1.2DL+LL+0.	-0.357	-0.942	0.097	-0.004	0.000	0.000
		7:0.9DL+0.5W-	-0.347	-0.391	0.097	0.004	0.000	0.000
		8:1.2DL+0.5W	-0.269	-0.526	0.036	0.005	0.000	0.000
		9:DEFLECTIOI	-0.588	-0.850	0.133	0.000	0.000	0.000
37	36	1:DEAD LOAD	1.022	0.001	-0.020	-0.000	0.790	0.014
		2:LIVE LOAD	0.671	0.001	0.000	-0.000	-0.004	0.025
		3:WIND LOAD	0.007	0.102	-0.132	-0.023	7.066	3.123
		4:FLOOD LOAI	0.002	0.105	-0.110	-0.004	4.273	3.529
		10:1.2DL+1.6LI	2.300	0.002	-0.024	-0.000	0.941	0.057
		11:1.2DL+LL+0	1.902	0.157	-0.199	-0.016	8.750	5.132
		12:0.9DL+0.5W	0.925	0.156	-0.194	-0.015	8.517	5.103
		13:1.2DL+0.5W	1.229	0.052	-0.090	-0.011	4.481	1.579
		5:1.2DL+1.6LL	2.300	0.002	-0.024	-0.000	0.941	0.057
		6:1.2DL+LL+0.	1.902	0.157	-0.199	-0.016	8.750	5.132
		7:0.9DL+0.5W-	0.925	0.156	-0.194	-0.015	8.517	5.103
		8:1.2DL+0.5W	1.229	0.052	-0.090	-0.011	4.481	1.579
		9:DEFLECTIOI	1.702	0.208	-0.262	-0.027	12.125	6.691
	35	1:DEAD LOAD	-0.624	-0.001	0.020	0.000	1.587	0.059
		2:LIVE LOAD	-0.671	-0.001	-0.000	0.000	-0.014	0.057
		3:WIND LOAD	-0.007	0.150	-0.120	0.023	-0.288	-0.006
		4:FLOOD LOAI	-0.002	0.147	-0.142	0.004	-0.172	-0.013
		10:1.2DL+1.6LI	-1.823	-0.002	0.024	0.000	1.882	0.161
		11:1.2DL+LL+0	-1.426	0.221	-0.179	0.016	1.574	0.111
		12:0.9DL+0.5W	-0.567	0.222	-0.184	0.015	1.112	0.037
		13:1.2DL+0.5W	-0.753	0.074	-0.036	0.011	1.760	0.068
		5:1.2DL+1.6LL	-1.823	-0.002	0.024	0.000	1.882	0.161
		6:1.2DL+LL+0.	-1.426	0.221	-0.179	0.016	1.574	0.111
		7:0.9DL+0.5W-	-0.567	0.222	-0.184	0.015	1.112	0.037
		8:1.2DL+0.5W	-0.753	0.074	-0.036	0.011	1.760	0.068
		9:DEFLECTIOI	-1.304	0.296	-0.242	0.027	1.113	0.096
38	33	1:DEAD LOAD	0.020	-0.359	-0.000	0.042	0.000	0.000
		2:LIVE LOAD	-0.000	-0.335	-0.000	0.022	0.000	0.000
		3:WIND LOAD	-0.121	0.009	0.096	-0.011	0.000	0.000
		4:FLOOD LOAI	-0.142	0.006	0.074	-0.006	0.000	0.000
		10:1.2DL+1.6LI	0.023	-0.967	-0.001	0.085	0.000	0.000
		11:1.2DL+LL+0	-0.179	-0.755	0.121	0.060	0.000	0.000
		12:0.9DL+0.5W	-0.185	-0.312	0.122	0.025	0.000	0.000
		13:1.2DL+0.5W	-0.037	-0.426	0.048	0.044	0.000	0.000
		5:1.2DL+1.6LL	0.023	-0.967	-0.001	0.085	0.000	0.000
		6:1.2DL+LL+0.	-0.179	-0.755	0.121	0.060	0.000	0.000
		7:0.9DL+0.5W-	-0.185	-0.312	0.122	0.025	0.000	0.000
		8:1.2DL+0.5W	-0.037	-0.426	0.048	0.044	0.000	0.000
		9:DEFLECTIOI	-0.244	-0.679	0.169	0.046	0.000	0.000
	35	1:DEAD LOAD	-0.020	0.378	0.000	-0.042	0.004	-4.423

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		2:LIVE LOAD	0.000	0.335	0.000	-0.022	0.004	-4.020
		3:WIND LOAD	0.121	-0.009	-0.076	0.011	-1.034	0.103
		4:FLOOD LOAI	0.142	-0.006	-0.074	0.006	-0.885	0.076
		10:1.2DL+1.6LI	-0.023	0.990	0.001	-0.085	0.011	-11.741
		11:1.2DL+LL+0	0.179	0.778	-0.111	-0.060	-1.393	-9.201
		12:0.9DL+0.5W	0.185	0.330	-0.112	-0.025	-1.398	-3.854
		13:1.2DL+0.5W	0.037	0.450	-0.038	-0.044	-0.513	-5.257
		5:1.2DL+1.6LL	-0.023	0.990	0.001	-0.085	0.011	-11.741
		6:1.2DL+LL+0.	0.179	0.778	-0.111	-0.060	-1.393	-9.201
		7:0.9DL+0.5W-	0.185	0.330	-0.112	-0.025	-1.398	-3.854
		8:1.2DL+0.5W	0.037	0.450	-0.038	-0.044	-0.513	-5.257
		9:DEFLECTIOI	0.244	0.698	-0.149	-0.046	-1.911	-8.265
39	35	1:DEAD LOAD	-0.000	0.246	0.000	-0.017	-0.004	2.836
		2:LIVE LOAD	-0.000	0.336	0.000	-0.035	-0.004	4.034
		3:WIND LOAD	-0.001	0.015	-0.074	-0.005	1.011	0.185
		4:FLOOD LOAI	-0.000	0.008	-0.073	0.007	0.881	0.096
		10:1.2DL+1.6LI	-0.000	0.833	0.001	-0.076	-0.011	9.859
		11:1.2DL+LL+0	-0.001	0.647	-0.110	-0.051	1.378	7.627
		12:0.9DL+0.5W	-0.001	0.237	-0.110	-0.011	1.383	2.742
		13:1.2DL+0.5W	-0.001	0.303	-0.037	-0.023	0.501	3.496
		5:1.2DL+1.6LL	-0.000	0.833	0.001	-0.076	-0.011	9.859
		6:1.2DL+LL+0.	-0.001	0.647	-0.110	-0.051	1.378	7.627
		7:0.9DL+0.5W-	-0.001	0.237	-0.110	-0.011	1.383	2.742
		8:1.2DL+0.5W	-0.001	0.303	-0.037	-0.023	0.501	3.496
		9:DEFLECTIOI	-0.001	0.606	-0.147	-0.050	1.884	7.152
	34	1:DEAD LOAD	0.000	-0.227	-0.000	0.017	0.000	0.000
		2:LIVE LOAD	0.000	-0.336	-0.000	0.035	0.000	0.000
		3:WIND LOAD	0.001	-0.015	0.094	0.005	0.000	0.000
		4:FLOOD LOAI	0.000	-0.008	0.073	-0.007	0.000	0.000
		10:1.2DL+1.6LI	0.000	-0.810	-0.001	0.076	0.000	0.000
		11:1.2DL+LL+0	0.001	-0.624	0.120	0.051	0.000	0.000
		12:0.9DL+0.5W	0.001	-0.220	0.120	0.011	0.000	0.000
		13:1.2DL+0.5W	0.001	-0.280	0.047	0.023	0.000	0.000
		5:1.2DL+1.6LL	0.000	-0.810	-0.001	0.076	0.000	0.000
		6:1.2DL+LL+0.	0.001	-0.624	0.120	0.051	0.000	0.000
		7:0.9DL+0.5W-	0.001	-0.220	0.120	0.011	0.000	0.000
		8:1.2DL+0.5W	0.001	-0.280	0.047	0.023	0.000	0.000
		9:DEFLECTIOI	0.001	-0.586	0.167	0.050	0.000	0.000
47	33	1:DEAD LOAD	-0.023	-0.107	0.000	0.010	-0.000	-2.695
		2:LIVE LOAD	-0.003	-0.139	-0.000	0.002	0.000	-2.361
		3:WIND LOAD	-4.142	-0.001	0.141	0.001	-1.486	-0.062
		4:FLOOD LOAI	-0.744	0.001	-0.000	-0.001	-0.006	0.084
		10:1.2DL+1.6LI	-0.033	-0.351	0.000	0.015	-0.001	-7.012
		11:1.2DL+LL+0	-2.846	-0.267	0.070	0.014	-0.750	-5.542

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		12:0.9DL+0.5W	-2.835	-0.096	0.070	0.009	-0.750	-2.372
		13:1.2DL+0.5W	-2.098	-0.129	0.071	0.012	-0.743	-3.265
		5:1.2DL+1.6LL	-0.033	-0.351	0.000	0.015	-0.001	-7.012
		6:1.2DL+LL+0.	-2.846	-0.267	0.070	0.014	-0.750	-5.542
		7:0.9DL+0.5W-	-2.835	-0.096	0.070	0.009	-0.750	-2.372
		8:1.2DL+0.5W	-2.098	-0.129	0.071	0.012	-0.743	-3.265
		9:DEFLECTIOI	-4.912	-0.246	0.141	0.012	-1.492	-5.034
	43	1:DEAD LOAD	0.023	-0.017	-0.000	-0.010	0.000	-0.000
		2:LIVE LOAD	0.003	-0.061	0.000	-0.002	0.000	0.011
		3:WIND LOAD	4.042	0.001	0.144	-0.001	1.552	-0.011
		4:FLOOD LOAI	0.744	-0.001	0.000	0.001	0.026	-0.004
		10:1.2DL+1.6LI	0.033	-0.118	-0.000	-0.015	0.000	0.018
		11:1.2DL+LL+0	2.796	-0.083	0.072	-0.014	0.803	0.002
		12:0.9DL+0.5W	2.785	-0.016	0.072	-0.009	0.802	-0.009
		13:1.2DL+0.5W	2.048	-0.020	0.072	-0.012	0.776	-0.006
		5:1.2DL+1.6LL	0.033	-0.118	-0.000	-0.015	0.000	0.018
		6:1.2DL+LL+0.	2.796	-0.083	0.072	-0.014	0.803	0.002
		7:0.9DL+0.5W-	2.785	-0.016	0.072	-0.009	0.802	-0.009
		8:1.2DL+0.5W	2.048	-0.020	0.072	-0.012	0.776	-0.006
		9:DEFLECTIOI	4.812	-0.078	0.144	-0.012	1.579	-0.004
48	34	1:DEAD LOAD	-0.015	0.107	-0.000	-0.011	0.001	2.025
		2:LIVE LOAD	-0.000	0.147	0.000	-0.001	0.000	2.945
		3:WIND LOAD	2.534	0.007	-0.000	0.005	0.084	0.437
		4:FLOOD LOAI	0.470	0.005	0.000	0.001	0.014	0.176
		10:1.2DL+1.6LI	-0.019	0.364	-0.000	-0.014	0.001	7.142
		11:1.2DL+LL+0	1.719	0.284	0.000	-0.010	0.056	5.769
		12:0.9DL+0.5W	1.724	0.105	0.000	-0.007	0.056	2.217
		13:1.2DL+0.5W	1.249	0.132	-0.000	-0.010	0.043	2.648
		5:1.2DL+1.6LL	-0.019	0.364	-0.000	-0.014	0.001	7.142
		6:1.2DL+LL+0.	1.719	0.284	0.000	-0.010	0.056	5.769
		7:0.9DL+0.5W-	1.724	0.105	0.000	-0.007	0.056	2.217
		8:1.2DL+0.5W	1.249	0.132	-0.000	-0.010	0.043	2.648
		9:DEFLECTIOI	2.989	0.266	-0.000	-0.006	0.098	5.583
	44	1:DEAD LOAD	0.015	0.018	0.000	0.011	0.000	0.648
		2:LIVE LOAD	0.000	0.053	-0.000	0.001	-0.000	-0.121
		3:WIND LOAD	-2.634	-0.007	0.000	-0.005	-0.081	-0.016
		4:FLOOD LOAI	-0.470	-0.005	-0.000	-0.001	-0.017	0.147
		10:1.2DL+1.6LI	0.019	0.106	0.000	0.014	-0.000	0.584
		11:1.2DL+LL+0	-1.769	0.065	-0.000	0.010	-0.057	0.796
		12:0.9DL+0.5W	-1.774	0.007	-0.000	0.007	-0.057	0.722
		13:1.2DL+0.5W	-1.299	0.018	0.000	0.010	-0.041	0.769
		5:1.2DL+1.6LL	0.019	0.106	0.000	0.014	-0.000	0.584
		6:1.2DL+LL+0.	-1.769	0.065	-0.000	0.010	-0.057	0.796
		7:0.9DL+0.5W-	-1.774	0.007	-0.000	0.007	-0.057	0.722

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		8:1.2DL+0.5W	-1.299	0.018	0.000	0.010	-0.041	0.769
		9:DEFLECTIOI	-3.089	0.058	0.000	0.006	-0.098	0.658
49	46	1:DEAD LOAD	0.397	0.000	0.000	-0.000	-0.008	0.006
		2:LIVE LOAD	-0.004	0.000	-0.000	-0.000	0.002	0.007
		3:WIND LOAD	-0.001	0.105	-0.077	-0.003	2.534	3.521
		4:FLOOD LOAI	0.005	0.105	-0.077	-0.001	2.531	3.599
		10:1.2DL+1.6LI	0.470	0.000	0.000	-0.000	-0.007	0.018
		11:1.2DL+LL+0	0.476	0.158	-0.115	-0.002	3.791	5.373
		12:0.9DL+0.5W	0.361	0.158	-0.115	-0.002	3.791	5.365
		13:1.2DL+0.5W	0.476	0.053	-0.038	-0.002	1.258	1.768
		5:1.2DL+1.6LL	0.470	0.000	0.000	-0.000	-0.007	0.018
		6:1.2DL+LL+0.	0.476	0.158	-0.115	-0.002	3.791	5.373
		7:0.9DL+0.5W-	0.361	0.158	-0.115	-0.002	3.791	5.365
		8:1.2DL+0.5W	0.476	0.053	-0.038	-0.002	1.258	1.768
		9:DEFLECTIOI	0.396	0.211	-0.154	-0.004	5.059	7.132
	45	1:DEAD LOAD	0.000	-0.000	-0.000	0.000	-0.016	0.014
		2:LIVE LOAD	0.004	-0.000	0.000	0.000	0.004	0.015
		3:WIND LOAD	0.001	0.147	-0.175	0.003	-2.361	-0.018
		4:FLOOD LOAI	-0.005	0.147	-0.175	0.001	-2.366	-0.011
		10:1.2DL+1.6LI	0.007	-0.000	-0.000	0.000	-0.013	0.041
		11:1.2DL+LL+0	0.001	0.220	-0.263	0.002	-3.561	0.012
		12:0.9DL+0.5W	-0.003	0.220	-0.263	0.002	-3.560	-0.008
		13:1.2DL+0.5W	0.001	0.073	-0.088	0.002	-1.199	0.007
		5:1.2DL+1.6LL	0.007	-0.000	-0.000	0.000	-0.013	0.041
		6:1.2DL+LL+0.	0.001	0.220	-0.263	0.002	-3.561	0.012
		7:0.9DL+0.5W-	-0.003	0.220	-0.263	0.002	-3.560	-0.008
		8:1.2DL+0.5W	0.001	0.073	-0.088	0.002	-1.199	0.007
		9:DEFLECTIOI	0.001	0.293	-0.350	0.004	-4.738	-0.000
50	43	1:DEAD LOAD	-0.000	0.030	-0.002	-0.000	0.000	-0.010
		2:LIVE LOAD	0.000	-0.000	-0.001	0.011	0.000	-0.002
		3:WIND LOAD	-0.087	0.098	-0.713	-0.011	1.552	-0.001
		4:FLOOD LOAI	-0.088	0.099	-0.078	-0.004	0.026	0.001
		10:1.2DL+1.6LI	-0.000	0.036	-0.003	0.018	0.000	-0.015
		11:1.2DL+LL+0	-0.131	0.184	-0.438	0.002	0.803	-0.014
		12:0.9DL+0.5W	-0.131	0.175	-0.437	-0.009	0.802	-0.009
		13:1.2DL+0.5W	-0.044	0.085	-0.359	-0.006	0.776	-0.012
		5:1.2DL+1.6LL	-0.000	0.036	-0.003	0.018	0.000	-0.015
		6:1.2DL+LL+0.	-0.131	0.184	-0.438	0.002	0.803	-0.014
		7:0.9DL+0.5W-	-0.131	0.175	-0.437	-0.009	0.802	-0.009
		8:1.2DL+0.5W	-0.044	0.085	-0.359	-0.006	0.776	-0.012
		9:DEFLECTIOI	-0.175	0.227	-0.794	-0.004	1.579	-0.012
	45	1:DEAD LOAD	0.000	-0.011	0.002	0.000	0.019	0.256
		2:LIVE LOAD	-0.000	0.000	0.001	-0.011	0.009	-0.003
		3:WIND LOAD	0.087	-0.098	0.733	0.011	7.128	1.180

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		4:FLOOD LOAI	0.088	-0.099	0.078	0.004	0.915	1.184
		10:1.2DL+1.6LI	0.000	-0.012	0.003	-0.018	0.037	0.303
		11:1.2DL+LL+0	0.131	-0.160	0.448	-0.002	4.511	2.078
		12:0.9DL+0.5W	0.131	-0.158	0.447	0.009	4.496	2.004
		13:1.2DL+0.5W	0.044	-0.062	0.369	0.006	3.587	0.897
		5:1.2DL+1.6LL	0.000	-0.012	0.003	-0.018	0.037	0.303
		6:1.2DL+LL+0.	0.131	-0.160	0.448	-0.002	4.511	2.078
		7:0.9DL+0.5W-	0.131	-0.158	0.447	0.009	4.496	2.004
		8:1.2DL+0.5W	0.044	-0.062	0.369	0.006	3.587	0.897
		9:DEFLECTIOI	0.175	-0.207	0.814	0.004	8.071	2.617
51	45	1:DEAD LOAD	0.000	0.010	-0.001	-0.014	-0.019	-0.240
		2:LIVE LOAD	-0.000	-0.004	-0.001	-0.004	-0.009	-0.001
		3:WIND LOAD	0.087	0.097	-0.881	0.007	-7.131	1.180
		4:FLOOD LOAI	0.088	0.103	-0.225	0.007	-0.916	1.182
		10:1.2DL+1.6LI	0.000	0.005	-0.003	-0.023	-0.037	-0.289
		11:1.2DL+LL+0	0.131	0.160	-0.668	-0.010	-4.513	1.483
		12:0.9DL+0.5W	0.131	0.161	-0.667	-0.002	-4.499	1.556
		13:1.2DL+0.5W	0.044	0.061	-0.442	-0.013	-3.588	0.303
		5:1.2DL+1.6LL	0.000	0.005	-0.003	-0.023	-0.037	-0.289
		6:1.2DL+LL+0.	0.131	0.160	-0.668	-0.010	-4.513	1.483
		7:0.9DL+0.5W-	0.131	0.161	-0.667	-0.002	-4.499	1.556
		8:1.2DL+0.5W	0.044	0.061	-0.442	-0.013	-3.588	0.303
		9:DEFLECTIOI	0.175	0.206	-1.107	-0.003	-8.075	2.122
	44	1:DEAD LOAD	-0.000	0.009	0.001	0.014	0.036	0.247
		2:LIVE LOAD	0.000	0.004	0.001	0.004	0.015	-0.052
		3:WIND LOAD	-0.087	-0.097	0.901	-0.007	17.817	-0.017
		4:FLOOD LOAI	-0.088	-0.103	0.225	-0.007	3.616	0.057
		10:1.2DL+1.6LI	-0.000	0.018	0.003	0.023	0.068	0.213
		11:1.2DL+LL+0	-0.131	-0.136	0.678	0.010	12.583	0.292
		12:0.9DL+0.5W	-0.131	-0.143	0.677	0.002	12.557	0.270
		13:1.2DL+0.5W	-0.044	-0.038	0.452	0.013	8.952	0.288
		5:1.2DL+1.6LL	-0.000	0.018	0.003	0.023	0.068	0.213
		6:1.2DL+LL+0.	-0.131	-0.136	0.678	0.010	12.583	0.292
		7:0.9DL+0.5W-	-0.131	-0.143	0.677	0.002	12.557	0.270
		8:1.2DL+0.5W	-0.044	-0.038	0.452	0.013	8.952	0.288
		9:DEFLECTIOI	-0.175	-0.187	1.127	0.003	21.485	0.234
52	13	1:DEAD LOAD	-0.002	0.128	-0.000	-0.000	0.001	2.698
		2:LIVE LOAD	-0.001	0.198	-0.000	0.001	-0.000	3.984
		3:WIND LOAD	-1.857	-0.004	-0.000	0.002	-0.046	-0.266
		4:FLOOD LOAI	-0.329	-0.001	-0.000	0.000	-0.009	-0.047
		10:1.2DL+1.6LI	-0.004	0.471	-0.000	0.001	0.002	9.613
		11:1.2DL+LL+0	-1.261	0.349	-0.000	0.001	-0.030	7.042
		12:0.9DL+0.5W	-1.259	0.113	-0.000	0.001	-0.031	2.248
		13:1.2DL+0.5W	-0.931	0.152	-0.000	0.000	-0.021	3.105

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		5:1.2DL+1.6LL	-0.004	0.471	-0.000	0.001	0.002	9.613
		6:1.2DL+LL+0.	-1.261	0.349	-0.000	0.001	-0.030	7.042
		7:0.9DL+0.5W-	-1.259	0.113	-0.000	0.001	-0.031	2.248
		8:1.2DL+0.5W	-0.931	0.152	-0.000	0.000	-0.021	3.105
		9:DEFLECTIOI	-2.188	0.321	-0.000	0.002	-0.054	6.369
	30	1:DEAD LOAD	0.002	0.121	0.000	0.000	0.001	-2.297
		2:LIVE LOAD	0.001	0.202	0.000	-0.001	0.000	-4.209
		3:WIND LOAD	1.657	0.004	0.000	-0.002	0.051	-0.238
		4:FLOOD LOAI	0.329	0.001	0.000	-0.000	0.009	-0.035
		10:1.2DL+1.6LI	0.004	0.469	0.000	-0.001	0.002	-9.491
		11:1.2DL+LL+0	1.161	0.350	0.000	-0.001	0.036	-7.119
		12:0.9DL+0.5W	1.159	0.112	0.000	-0.001	0.036	-2.221
		13:1.2DL+0.5W	0.831	0.148	0.000	-0.000	0.027	-2.876
		5:1.2DL+1.6LL	0.004	0.469	0.000	-0.001	0.002	-9.491
		6:1.2DL+LL+0.	1.161	0.350	0.000	-0.001	0.036	-7.119
		7:0.9DL+0.5W-	1.159	0.112	0.000	-0.001	0.036	-2.221
		8:1.2DL+0.5W	0.831	0.148	0.000	-0.000	0.027	-2.876
		9:DEFLECTIOI	1.988	0.328	0.000	-0.002	0.061	-6.778
53	4	1:DEAD LOAD	0.004	-0.119	0.000	0.006	-0.001	-2.249
		2:LIVE LOAD	0.002	-0.198	0.000	-0.000	-0.000	-3.971
		3:WIND LOAD	1.126	0.001	0.285	0.005	-5.658	-0.125
		4:FLOOD LOAI	0.224	0.000	0.000	0.001	0.008	-0.014
		10:1.2DL+1.6LI	0.008	-0.460	0.000	0.006	-0.002	-9.052
		11:1.2DL+LL+0	0.793	-0.340	0.143	0.010	-2.822	-6.746
		12:0.9DL+0.5W	0.790	-0.106	0.143	0.009	-2.822	-2.101
		13:1.2DL+0.5W	0.567	-0.142	0.143	0.010	-2.830	-2.761
		5:1.2DL+1.6LL	0.008	-0.460	0.000	0.006	-0.002	-9.052
		6:1.2DL+LL+0.	0.793	-0.340	0.143	0.010	-2.822	-6.746
		7:0.9DL+0.5W-	0.790	-0.106	0.143	0.009	-2.822	-2.101
		8:1.2DL+0.5W	0.567	-0.142	0.143	0.010	-2.830	-2.761
		9:DEFLECTIOI	1.355	-0.316	0.285	0.011	-5.651	-6.359
	29	1:DEAD LOAD	-0.004	-0.131	-0.000	-0.006	-0.001	2.964
		2:LIVE LOAD	-0.002	-0.202	-0.000	0.000	-0.000	4.158
		3:WIND LOAD	-1.326	-0.001	0.285	-0.005	5.642	0.213
		4:FLOOD LOAI	-0.224	-0.000	-0.000	-0.001	-0.011	0.054
		10:1.2DL+1.6LI	-0.008	-0.479	-0.000	-0.006	-0.002	10.210
		11:1.2DL+LL+0	-0.893	-0.359	0.142	-0.010	2.809	7.876
		12:0.9DL+0.5W	-0.890	-0.118	0.142	-0.009	2.809	2.829
		13:1.2DL+0.5W	-0.667	-0.157	0.142	-0.010	2.819	3.664
		5:1.2DL+1.6LL	-0.008	-0.479	-0.000	-0.006	-0.002	10.210
		6:1.2DL+LL+0.	-0.893	-0.359	0.142	-0.010	2.809	7.876
		7:0.9DL+0.5W-	-0.890	-0.118	0.142	-0.009	2.809	2.829
		8:1.2DL+0.5W	-0.667	-0.157	0.142	-0.010	2.819	3.664
		9:DEFLECTIOI	-1.555	-0.333	0.285	-0.011	5.630	7.390

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
54	29	1:DEAD LOAD	0.004	-0.140	-0.000	0.006	0.001	-2.975
		2:LIVE LOAD	0.002	-0.207	-0.000	-0.000	0.000	-4.164
		3:WIND LOAD	1.399	-0.006	0.284	0.005	-5.642	-0.230
		4:FLOOD LOAI	0.286	-0.002	-0.000	0.001	0.011	-0.061
		10:1.2DL+1.6LI	0.008	-0.499	-0.000	0.006	0.002	-10.232
		11:1.2DL+LL+0	0.992	-0.380	0.142	0.010	-2.809	-7.909
		12:0.9DL+0.5W	0.989	-0.131	0.142	0.009	-2.809	-2.853
		13:1.2DL+0.5W	0.704	-0.172	0.142	0.010	-2.819	-3.685
		5:1.2DL+1.6LL	0.008	-0.499	-0.000	0.006	0.002	-10.232
		6:1.2DL+LL+0.	0.992	-0.380	0.142	0.010	-2.809	-7.909
		7:0.9DL+0.5W-	0.989	-0.131	0.142	0.009	-2.809	-2.853
		8:1.2DL+0.5W	0.704	-0.172	0.142	0.010	-2.819	-3.685
		9:DEFLECTIOI	1.691	-0.355	0.284	0.011	-5.630	-7.429
	33	1:DEAD LOAD	-0.004	-0.109	0.000	-0.006	0.001	1.121
		2:LIVE LOAD	-0.002	-0.193	0.000	0.000	0.000	3.358
		3:WIND LOAD	-1.599	0.006	0.286	-0.005	5.782	-0.532
		4:FLOOD LOAI	-0.286	0.002	0.000	-0.001	0.015	-0.137
		10:1.2DL+1.6LI	-0.008	-0.441	0.000	-0.006	0.001	6.717
		11:1.2DL+LL+0	-1.092	-0.320	0.143	-0.010	2.907	4.300
		12:0.9DL+0.5W	-1.089	-0.094	0.143	-0.009	2.906	0.606
		13:1.2DL+0.5W	-0.804	-0.128	0.143	-0.010	2.892	1.079
		5:1.2DL+1.6LL	-0.008	-0.441	0.000	-0.006	0.001	6.717
		6:1.2DL+LL+0.	-1.092	-0.320	0.143	-0.010	2.907	4.300
		7:0.9DL+0.5W-	-1.089	-0.094	0.143	-0.009	2.906	0.606
		8:1.2DL+0.5W	-0.804	-0.128	0.143	-0.010	2.892	1.079
		9:DEFLECTIOI	-1.891	-0.295	0.286	-0.011	5.797	3.809
55	30	1:DEAD LOAD	-0.015	0.130	0.000	-0.011	-0.001	2.656
		2:LIVE LOAD	0.000	0.211	0.000	-0.001	-0.000	4.284
		3:WIND LOAD	2.240	-0.008	0.001	0.005	-0.064	-0.565
		4:FLOOD LOAI	0.397	-0.003	0.000	0.001	-0.011	-0.149
		10:1.2DL+1.6LI	-0.018	0.493	0.000	-0.014	-0.002	10.042
		11:1.2DL+LL+0	1.499	0.360	0.001	-0.010	-0.045	7.041
		12:0.9DL+0.5W	1.503	0.110	0.001	-0.007	-0.044	1.960
		13:1.2DL+0.5W	1.102	0.152	0.001	-0.010	-0.034	2.905
		5:1.2DL+1.6LL	-0.018	0.493	0.000	-0.014	-0.002	10.042
		6:1.2DL+LL+0.	1.499	0.360	0.001	-0.010	-0.045	7.041
		7:0.9DL+0.5W-	1.503	0.110	0.001	-0.007	-0.044	1.960
		8:1.2DL+0.5W	1.102	0.152	0.001	-0.010	-0.034	2.905
		9:DEFLECTIOI	2.622	0.330	0.001	-0.006	-0.077	6.227
	34	1:DEAD LOAD	0.015	0.120	-0.000	0.011	-0.001	-2.042
		2:LIVE LOAD	-0.000	0.189	-0.000	0.001	-0.000	-2.980
		3:WIND LOAD	-2.440	0.008	-0.001	-0.005	-0.084	-0.442
		4:FLOOD LOAI	-0.397	0.003	-0.000	-0.001	-0.014	-0.169
		10:1.2DL+1.6LI	0.018	0.446	-0.000	0.014	-0.001	-7.218

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		11:1.2DL+LL+0	-1.599	0.340	-0.001	0.010	-0.056	-5.821
		12:0.9DL+0.5W	-1.603	0.115	-0.001	0.007	-0.056	-2.228
		13:1.2DL+0.5W	-1.202	0.148	-0.001	0.010	-0.043	-2.671
		5:1.2DL+1.6LL	0.018	0.446	-0.000	0.014	-0.001	-7.218
		6:1.2DL+LL+0.	-1.599	0.340	-0.001	0.010	-0.056	-5.821
		7:0.9DL+0.5W-	-1.603	0.115	-0.001	0.007	-0.056	-2.228
		8:1.2DL+0.5W	-1.202	0.148	-0.001	0.010	-0.043	-2.671
		9:DEFLECTIOI	-2.822	0.320	-0.001	0.006	-0.098	-5.633
56	44	1:DEAD LOAD	-0.019	0.070	-0.001	0.027	-0.036	0.702
		2:LIVE LOAD	0.003	-0.008	0.000	0.006	-0.015	-0.128
		3:WIND LOAD	3.682	0.003	-0.080	0.003	-17.736	-0.026
		4:FLOOD LOAI	0.735	0.003	0.003	-0.001	-3.599	0.151
		10:1.2DL+1.6LI	-0.019	0.071	-0.001	0.041	-0.068	0.638
		11:1.2DL+LL+0	2.555	0.080	-0.038	0.037	-12.526	0.852
		12:0.9DL+0.5W	2.558	0.067	-0.038	0.024	-12.500	0.770
		13:1.2DL+0.5W	1.818	0.085	-0.041	0.033	-8.912	0.830
		5:1.2DL+1.6LL	-0.019	0.071	-0.001	0.041	-0.068	0.638
		6:1.2DL+LL+0.	2.555	0.080	-0.038	0.037	-12.526	0.852
		7:0.9DL+0.5W-	2.558	0.067	-0.038	0.024	-12.500	0.770
		8:1.2DL+0.5W	1.818	0.085	-0.041	0.033	-8.912	0.830
		9:DEFLECTIOI	4.400	0.068	-0.078	0.033	-21.387	0.699
	33	1:DEAD LOAD	0.019	0.035	0.001	-0.027	0.088	0.430
		2:LIVE LOAD	-0.003	0.008	-0.000	-0.006	0.009	-0.370
		3:WIND LOAD	-3.682	-0.003	0.080	-0.003	22.896	0.244
		4:FLOOD LOAI	-0.735	-0.003	-0.003	0.001	3.413	0.027
		10:1.2DL+1.6LI	0.019	0.054	0.001	-0.041	0.121	-0.077
		11:1.2DL+LL+0	-2.555	0.045	0.038	-0.037	14.977	0.294
		12:0.9DL+0.5W	-2.558	0.027	0.038	-0.024	14.941	0.536
		13:1.2DL+0.5W	-1.818	0.040	0.041	-0.033	11.554	0.638
		5:1.2DL+1.6LL	0.019	0.054	0.001	-0.041	0.121	-0.077
		6:1.2DL+LL+0.	-2.555	0.045	0.038	-0.037	14.977	0.294
		7:0.9DL+0.5W-	-2.558	0.027	0.038	-0.024	14.941	0.536
		8:1.2DL+0.5W	-1.818	0.040	0.041	-0.033	11.554	0.638
		9:DEFLECTIOI	-4.400	0.036	0.078	-0.033	26.407	0.331
57	33	1:DEAD LOAD	-0.045	0.107	0.003	-0.056	-0.088	1.239
		2:LIVE LOAD	-0.003	-0.005	0.000	-0.004	-0.009	-0.641
		3:WIND LOAD	-2.489	-0.000	0.261	0.018	-18.600	0.367
		4:FLOOD LOAI	-0.440	-0.001	0.048	0.003	-3.405	0.022
		10:1.2DL+1.6LI	-0.059	0.121	0.004	-0.074	-0.121	0.461
		11:1.2DL+LL+0	-1.742	0.123	0.183	-0.060	-12.820	1.052
		12:0.9DL+0.5W	-1.725	0.096	0.181	-0.039	-12.784	1.321
		13:1.2DL+0.5W	-1.299	0.129	0.134	-0.058	-9.406	1.671
		5:1.2DL+1.6LL	-0.059	0.121	0.004	-0.074	-0.121	0.461
		6:1.2DL+LL+0.	-1.742	0.123	0.183	-0.060	-12.820	1.052

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		7:0.9DL+0.5W-	-1.725	0.096	0.181	-0.039	-12.784	1.321
		8:1.2DL+0.5W	-1.299	0.129	0.134	-0.058	-9.406	1.671
		9:DEFLECTIOI	-2.978	0.102	0.313	-0.040	-22.102	0.988
	30	1:DEAD LOAD	0.045	0.091	-0.003	0.056	-0.286	-0.209
		2:LIVE LOAD	0.003	0.005	-0.000	0.004	-0.026	0.014
		3:WIND LOAD	2.489	0.000	-0.261	-0.018	-13.372	-0.377
		4:FLOOD LOAI	0.440	0.001	-0.048	-0.003	-2.468	-0.091
		10:1.2DL+1.6LI	0.059	0.117	-0.004	0.074	-0.385	-0.229
		11:1.2DL+LL+0	1.742	0.114	-0.183	0.060	-9.523	-0.517
		12:0.9DL+0.5W	1.725	0.082	-0.181	0.039	-9.411	-0.468
		13:1.2DL+0.5W	1.299	0.109	-0.134	0.058	-7.029	-0.439
		5:1.2DL+1.6LL	0.059	0.117	-0.004	0.074	-0.385	-0.229
		6:1.2DL+LL+0.	1.742	0.114	-0.183	0.060	-9.523	-0.517
		7:0.9DL+0.5W-	1.725	0.082	-0.181	0.039	-9.411	-0.468
		8:1.2DL+0.5W	1.299	0.109	-0.134	0.058	-7.029	-0.439
		9:DEFLECTIOI	2.978	0.096	-0.313	0.040	-16.152	-0.664
58	30	1:DEAD LOAD	-0.058	0.100	-0.005	0.089	0.286	0.569
		2:LIVE LOAD	-0.002	0.000	-0.000	0.003	0.026	0.055
		3:WIND LOAD	1.462	-0.004	-0.013	0.009	13.385	-0.437
		4:FLOOD LOAI	0.247	-0.001	-0.001	0.002	2.470	-0.098
		10:1.2DL+1.6LI	-0.074	0.120	-0.006	0.111	0.385	0.771
		11:1.2DL+LL+0	0.905	0.117	-0.014	0.116	9.531	0.421
		12:0.9DL+0.5W	0.925	0.087	-0.012	0.086	9.420	0.196
		13:1.2DL+0.5W	0.661	0.118	-0.012	0.111	7.036	0.465
		5:1.2DL+1.6LL	-0.074	0.120	-0.006	0.111	0.385	0.771
		6:1.2DL+LL+0.	0.905	0.117	-0.014	0.116	9.531	0.421
		7:0.9DL+0.5W-	0.925	0.087	-0.012	0.086	9.420	0.196
		8:1.2DL+0.5W	0.661	0.118	-0.012	0.111	7.036	0.465
		9:DEFLECTIOI	1.647	0.095	-0.019	0.102	16.167	0.089
	4	1:DEAD LOAD	0.058	0.098	0.005	-0.089	0.279	-0.493
		2:LIVE LOAD	0.002	-0.000	0.000	-0.003	-0.002	-0.024
		3:WIND LOAD	-1.462	0.004	0.013	-0.009	-11.820	-0.016
		4:FLOOD LOAI	-0.247	0.001	0.001	-0.002	-2.298	-0.016
		10:1.2DL+1.6LI	0.074	0.118	0.006	-0.111	0.332	-0.629
		11:1.2DL+LL+0	-0.905	0.121	0.014	-0.116	-7.874	-0.639
		12:0.9DL+0.5W	-0.925	0.091	0.012	-0.086	-7.956	-0.468
		13:1.2DL+0.5W	-0.661	0.120	0.012	-0.111	-5.574	-0.600
		5:1.2DL+1.6LL	0.074	0.118	0.006	-0.111	0.332	-0.629
		6:1.2DL+LL+0.	-0.905	0.121	0.014	-0.116	-7.874	-0.639
		7:0.9DL+0.5W-	-0.925	0.091	0.012	-0.086	-7.956	-0.468
		8:1.2DL+0.5W	-0.661	0.120	0.012	-0.111	-5.574	-0.600
		9:DEFLECTIOI	-1.647	0.103	0.019	-0.102	-13.840	-0.549
59	4	1:DEAD LOAD	-0.045	0.099	0.004	-0.090	-0.279	0.433
		2:LIVE LOAD	0.002	0.000	-0.000	0.002	0.002	-0.024

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		3:WIND LOAD	0.569	0.001	0.009	-0.001	11.819	0.068
		4:FLOOD LOAI	0.114	0.000	-0.002	-0.000	2.298	0.000
		10:1.2DL+1.6LI	-0.051	0.119	0.005	-0.106	-0.332	0.481
		11:1.2DL+LL+0	0.346	0.120	0.008	-0.107	7.873	0.530
		12:0.9DL+0.5W	0.358	0.090	0.007	-0.082	7.955	0.424
		13:1.2DL+0.5W	0.231	0.119	0.010	-0.109	5.574	0.553
		5:1.2DL+1.6LL	-0.051	0.119	0.005	-0.106	-0.332	0.481
		6:1.2DL+LL+0.	0.346	0.120	0.008	-0.107	7.873	0.530
		7:0.9DL+0.5W-	0.358	0.090	0.007	-0.082	7.955	0.424
		8:1.2DL+0.5W	0.231	0.119	0.010	-0.109	5.574	0.553
		9:DEFLECTIOI	0.640	0.101	0.011	-0.090	13.839	0.477
	12	1:DEAD LOAD	0.045	0.099	-0.004	0.090	-0.268	-0.437
		2:LIVE LOAD	-0.002	-0.000	0.000	-0.002	0.020	0.053
		3:WIND LOAD	-0.569	-0.001	-0.009	0.001	-12.884	0.105
		4:FLOOD LOAI	-0.114	-0.000	0.002	0.000	-2.094	0.012
		10:1.2DL+1.6LI	0.051	0.118	-0.005	0.106	-0.289	-0.439
		11:1.2DL+LL+0	-0.346	0.118	-0.008	0.107	-8.837	-0.407
		12:0.9DL+0.5W	-0.358	0.088	-0.007	0.082	-8.777	-0.329
		13:1.2DL+0.5W	-0.231	0.118	-0.010	0.109	-6.763	-0.472
		5:1.2DL+1.6LL	0.051	0.118	-0.005	0.106	-0.289	-0.439
		6:1.2DL+LL+0.	-0.346	0.118	-0.008	0.107	-8.837	-0.407
		7:0.9DL+0.5W-	-0.358	0.088	-0.007	0.082	-8.777	-0.329
		8:1.2DL+0.5W	-0.231	0.118	-0.010	0.109	-6.763	-0.472
		9:DEFLECTIOI	-0.640	0.097	-0.011	0.090	-15.225	-0.267
60	12	1:DEAD LOAD	-0.062	0.098	-0.006	0.093	0.268	0.513
		2:LIVE LOAD	0.001	-0.002	0.000	0.001	-0.020	0.093
		3:WIND LOAD	-1.988	-0.002	-0.123	-0.010	12.882	0.046
		4:FLOOD LOAI	-0.411	-0.000	-0.023	-0.001	2.093	0.005
		10:1.2DL+1.6LI	-0.073	0.115	-0.007	0.114	0.289	0.763
		11:1.2DL+LL+0	-1.479	0.114	-0.091	0.107	8.835	0.736
		12:0.9DL+0.5W	-1.461	0.087	-0.089	0.078	8.775	0.490
		13:1.2DL+0.5W	-1.069	0.116	-0.068	0.107	6.762	0.638
		5:1.2DL+1.6LL	-0.073	0.115	-0.007	0.114	0.289	0.763
		6:1.2DL+LL+0.	-1.479	0.114	-0.091	0.107	8.835	0.736
		7:0.9DL+0.5W-	-1.461	0.087	-0.089	0.078	8.775	0.490
		8:1.2DL+0.5W	-1.069	0.116	-0.068	0.107	6.762	0.638
		9:DEFLECTIOI	-2.460	0.094	-0.151	0.083	15.222	0.656
	2	1:DEAD LOAD	0.062	0.100	0.006	-0.093	0.432	-0.668
		2:LIVE LOAD	-0.001	0.002	-0.000	-0.001	0.002	-0.291
		3:WIND LOAD	1.988	0.002	0.123	0.010	2.112	-0.319
		4:FLOOD LOAI	0.411	0.000	0.023	0.001	0.676	-0.039
		10:1.2DL+1.6LI	0.073	0.123	0.007	-0.114	0.521	-1.268
		11:1.2DL+LL+0	1.479	0.123	0.091	-0.107	2.251	-1.291
		12:0.9DL+0.5W	1.461	0.092	0.089	-0.078	2.120	-0.800

Beam End Forces Cont...

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
		13:1.2DL+0.5W	1.069	0.121	0.068	-0.107	1.574	-0.961
		5:1.2DL+1.6LL	0.073	0.123	0.007	-0.114	0.521	-1.268
		6:1.2DL+LL+0.	1.479	0.123	0.091	-0.107	2.251	-1.291
		7:0.9DL+0.5W-	1.461	0.092	0.089	-0.078	2.120	-0.800
		8:1.2DL+0.5W	1.069	0.121	0.068	-0.107	1.574	-0.961
		9:DEFLECTIOI	2.460	0.104	0.151	-0.083	3.221	-1.317
61	2	1:DEAD LOAD	-0.096	0.098	0.004	-0.057	-0.432	0.083
		2:LIVE LOAD	-0.002	-0.002	0.000	0.049	-0.002	-0.529
		3:WIND LOAD	3.558	-0.003	-0.006	0.002	0.716	-0.388
		4:FLOOD LOAI	0.474	-0.000	0.006	0.001	-0.677	-0.049
		10:1.2DL+1.6LI	-0.118	0.114	0.004	0.011	-0.521	-0.747
		11:1.2DL+LL+0	2.137	0.113	0.007	-0.017	-0.839	-0.673
		12:0.9DL+0.5W	2.168	0.086	0.006	-0.050	-0.708	-0.169
		13:1.2DL+0.5W	1.664	0.116	0.001	-0.067	-0.160	-0.095
		5:1.2DL+1.6LL	-0.118	0.114	0.004	0.011	-0.521	-0.747
		6:1.2DL+LL+0.	2.137	0.113	0.007	-0.017	-0.839	-0.673
		7:0.9DL+0.5W-	2.168	0.086	0.006	-0.050	-0.708	-0.169
		8:1.2DL+0.5W	1.664	0.116	0.001	-0.067	-0.160	-0.095
		9:DEFLECTIOI	3.935	0.092	0.003	-0.005	-0.395	-0.884
	10	1:DEAD LOAD	0.096	0.100	-0.004	0.057	0.000	-0.253
		2:LIVE LOAD	0.002	0.002	-0.000	-0.049	0.000	0.292
		3:WIND LOAD	-3.558	0.003	0.006	-0.002	0.002	0.005
		4:FLOOD LOAI	-0.474	0.000	-0.006	-0.001	0.000	0.002
		10:1.2DL+1.6LI	0.118	0.124	-0.004	-0.011	0.000	0.163
		11:1.2DL+LL+0	-2.137	0.124	-0.007	0.017	0.001	-0.008
		12:0.9DL+0.5W	-2.168	0.092	-0.006	0.050	0.001	-0.223
		13:1.2DL+0.5W	-1.664	0.122	-0.001	0.067	0.001	-0.302
		5:1.2DL+1.6LL	0.118	0.124	-0.004	-0.011	0.000	0.163
		6:1.2DL+LL+0.	-2.137	0.124	-0.007	0.017	0.001	-0.008
		7:0.9DL+0.5W-	-2.168	0.092	-0.006	0.050	0.001	-0.223
		8:1.2DL+0.5W	-1.664	0.122	-0.001	0.067	0.001	-0.302
		9:DEFLECTIOI	-3.935	0.106	-0.003	0.005	0.002	0.045

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
1	1:DEAD LOAD	0.023	0.097	0.000	0.000	0.000	0.000
	2:LIVE LOAD	0.002	0.155	0.000	0.000	0.000	0.000
	3:WIND LOAD	-4.159	-0.005	-0.214	0.000	0.000	0.000
	4:FLOOD LOAI	-0.659	-0.001	0.000	0.000	0.000	0.000
	10:1.2DL+1.6LI	0.030	0.365	0.000	0.000	0.000	0.000
	11:1.2DL+LL+0	-2.709	0.269	-0.107	0.000	0.000	0.000
	12:0.9DL+0.5W	-2.718	0.085	-0.107	0.000	0.000	0.000
	13:1.2DL+0.5W	-2.052	0.114	-0.107	0.000	0.000	0.000
	5:1.2DL+1.6LL	0.030	0.365	0.000	0.000	0.000	0.000
	6:1.2DL+LL+0.	-2.709	0.269	-0.107	0.000	0.000	0.000
	7:0.9DL+0.5W-	-2.718	0.085	-0.107	0.000	0.000	0.000
	8:1.2DL+0.5W	-2.052	0.114	-0.107	0.000	0.000	0.000
	9:DEFLECTIOI	-4.794	0.247	-0.214	0.000	0.000	0.000
10	1:DEAD LOAD	-0.079	0.196	0.022	0.000	0.000	0.000
	2:LIVE LOAD	-0.002	0.163	0.000	0.000	0.000	0.000
	3:WIND LOAD	3.364	0.005	-0.704	0.000	0.000	0.000
	4:FLOOD LOAI	0.498	0.001	-0.088	0.000	0.000	0.000
	10:1.2DL+1.6LI	-0.098	0.495	0.027	0.000	0.000	0.000
	11:1.2DL+LL+0	2.083	0.401	-0.412	0.000	0.000	0.000
	12:0.9DL+0.5W	2.109	0.179	-0.419	0.000	0.000	0.000
	13:1.2DL+0.5W	1.587	0.237	-0.325	0.000	0.000	0.000
	5:1.2DL+1.6LL	-0.098	0.495	0.027	0.000	0.000	0.000
	6:1.2DL+LL+0.	2.083	0.401	-0.412	0.000	0.000	0.000
	7:0.9DL+0.5W-	2.109	0.179	-0.419	0.000	0.000	0.000
	8:1.2DL+0.5W	1.587	0.237	-0.325	0.000	0.000	0.000
	9:DEFLECTIOI	3.781	0.364	-0.768	0.000	0.000	0.000
22	1:DEAD LOAD	0.000	1.042	-0.040	-0.639	-0.000	-0.053
	2:LIVE LOAD	-0.001	0.900	-0.000	0.009	-0.000	0.015
	3:WIND LOAD	-0.121	-0.002	-0.533	-37.272	0.046	7.708
	4:FLOOD LOAI	-0.036	-0.000	-0.099	-6.131	0.008	1.571
	10:1.2DL+1.6LI	-0.001	2.691	-0.048	-0.753	-0.000	-0.039
	11:1.2DL+LL+0	-0.097	2.150	-0.414	-25.526	0.031	5.376
	12:0.9DL+0.5W	-0.096	0.937	-0.402	-25.343	0.031	5.377
	13:1.2DL+0.5W	-0.060	1.250	-0.314	-19.403	0.023	3.791
	5:1.2DL+1.6LL	-0.001	2.691	-0.048	-0.753	-0.000	-0.039
	6:1.2DL+LL+0.	-0.097	2.150	-0.414	-25.526	0.031	5.376
	7:0.9DL+0.5W-	-0.096	0.937	-0.402	-25.343	0.031	5.377
	8:1.2DL+0.5W	-0.060	1.250	-0.314	-19.403	0.023	3.791
	9:DEFLECTIOI	-0.158	1.940	-0.672	-44.034	0.053	9.241
23	1:DEAD LOAD	0.000	1.080	0.031	1.174	0.000	-0.037
	2:LIVE LOAD	0.000	0.779	-0.001	-0.025	0.000	-0.005
	3:WIND LOAD	-0.100	0.003	-0.372	-33.870	0.020	4.531
	4:FLOOD LOAI	-0.085	0.000	-0.131	-7.865	0.004	2.862
	10:1.2DL+1.6LI	0.001	2.543	0.036	1.369	0.000	-0.054

Reactions Cont...

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
	11:1.2DL+LL+0	-0.134	2.078	-0.281	-23.417	0.014	5.077
	12:0.9DL+0.5W	-0.135	0.974	-0.289	-23.744	0.014	5.094
	13:1.2DL+0.5W	-0.050	1.298	-0.149	-15.527	0.010	2.221
	5:1.2DL+1.6LL	0.001	2.543	0.036	1.369	0.000	-0.054
	6:1.2DL+LL+0.	-0.134	2.078	-0.281	-23.417	0.014	5.077
	7:0.9DL+0.5W-	-0.135	0.974	-0.289	-23.744	0.014	5.094
	8:1.2DL+0.5W	-0.050	1.298	-0.149	-15.527	0.010	2.221
	9:DEFLECTIOI	-0.185	1.863	-0.473	-40.587	0.024	7.350
24	1:DEAD LOAD	0.000	1.092	-0.029	-0.978	0.000	-0.022
	2:LIVE LOAD	-0.000	0.799	-0.000	-0.005	0.000	0.004
	3:WIND LOAD	-0.102	-0.003	-0.403	-37.091	-0.009	4.698
	4:FLOOD LOAI	-0.086	-0.000	-0.139	-8.688	-0.001	2.944
	10:1.2DL+1.6LI	0.000	2.589	-0.035	-1.182	0.000	-0.021
	11:1.2DL+LL+0	-0.136	2.108	-0.376	-28.413	-0.006	5.270
	12:0.9DL+0.5W	-0.136	0.981	-0.367	-28.114	-0.006	5.273
	13:1.2DL+0.5W	-0.051	1.309	-0.237	-19.720	-0.004	2.322
	5:1.2DL+1.6LL	0.000	2.589	-0.035	-1.182	0.000	-0.021
	6:1.2DL+LL+0.	-0.136	2.108	-0.376	-28.413	-0.006	5.270
	7:0.9DL+0.5W-	-0.136	0.981	-0.367	-28.114	-0.006	5.273
	8:1.2DL+0.5W	-0.051	1.309	-0.237	-19.720	-0.004	2.322
	9:DEFLECTIOI	-0.187	1.888	-0.572	-46.763	-0.010	7.623
32	1:DEAD LOAD	0.000	1.109	0.028	0.967	0.000	-0.029
	2:LIVE LOAD	0.000	0.826	0.002	0.046	0.000	-0.006
	3:WIND LOAD	-0.106	-0.001	-0.307	-26.691	-0.032	5.128
	4:FLOOD LOAI	-0.086	-0.000	-0.123	-6.951	-0.005	3.029
	10:1.2DL+1.6LI	0.001	2.653	0.036	1.234	0.000	-0.044
	11:1.2DL+LL+0	-0.139	2.157	-0.242	-19.090	-0.021	5.553
	12:0.9DL+0.5W	-0.139	0.998	-0.252	-19.426	-0.021	5.567
	13:1.2DL+0.5W	-0.052	1.331	-0.120	-12.185	-0.016	2.529
	5:1.2DL+1.6LL	0.001	2.653	0.036	1.234	0.000	-0.044
	6:1.2DL+LL+0.	-0.139	2.157	-0.242	-19.090	-0.021	5.553
	7:0.9DL+0.5W-	-0.139	0.998	-0.252	-19.426	-0.021	5.567
	8:1.2DL+0.5W	-0.052	1.331	-0.120	-12.185	-0.016	2.529
	9:DEFLECTIOI	-0.192	1.934	-0.401	-32.629	-0.037	8.123
36	1:DEAD LOAD	-0.001	1.022	-0.020	-0.790	-0.000	0.014
	2:LIVE LOAD	-0.001	0.671	0.000	0.004	-0.000	0.025
	3:WIND LOAD	-0.102	0.007	-0.132	-7.066	-0.023	3.123
	4:FLOOD LOAI	-0.105	0.002	-0.110	-4.273	-0.004	3.529
	10:1.2DL+1.6LI	-0.002	2.300	-0.024	-0.941	-0.000	0.057
	11:1.2DL+LL+0	-0.157	1.902	-0.199	-8.750	-0.016	5.132
	12:0.9DL+0.5W	-0.156	0.925	-0.194	-8.517	-0.015	5.103
	13:1.2DL+0.5W	-0.052	1.229	-0.090	-4.481	-0.011	1.579
	5:1.2DL+1.6LL	-0.002	2.300	-0.024	-0.941	-0.000	0.057
	6:1.2DL+LL+0.	-0.157	1.902	-0.199	-8.750	-0.016	5.132

Reactions Cont...

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
	7:0.9DL+0.5W-	-0.156	0.925	-0.194	-8.517	-0.015	5.103
	8:1.2DL+0.5W	-0.052	1.229	-0.090	-4.481	-0.011	1.579
	9:DEFLECTIOI	-0.208	1.702	-0.262	-12.125	-0.027	6.691
43	1:DEAD LOAD	0.024	0.048	-0.000	0.000	0.000	0.000
	2:LIVE LOAD	0.004	0.060	0.000	0.000	0.000	0.000
	3:WIND LOAD	4.756	0.097	-0.231	0.000	0.000	0.000
	4:FLOOD LOAI	0.822	0.100	-0.088	0.000	0.000	0.000
	10:1.2DL+1.6LI	0.036	0.154	-0.000	0.000	0.000	0.000
	11:1.2DL+LL+0	3.233	0.266	-0.203	0.000	0.000	0.000
	12:0.9DL+0.5W	3.222	0.191	-0.203	0.000	0.000	0.000
	13:1.2DL+0.5W	2.407	0.106	-0.116	0.000	0.000	0.000
	5:1.2DL+1.6LL	0.036	0.154	-0.000	0.000	0.000	0.000
	6:1.2DL+LL+0.	3.233	0.266	-0.203	0.000	0.000	0.000
	7:0.9DL+0.5W-	3.222	0.191	-0.203	0.000	0.000	0.000
	8:1.2DL+0.5W	2.407	0.106	-0.116	0.000	0.000	0.000
	9:DEFLECTIOI	5.606	0.305	-0.319	0.000	0.000	0.000
44	1:DEAD LOAD	0.031	0.097	0.008	0.000	0.000	0.000
	2:LIVE LOAD	-0.003	0.050	-0.001	0.000	0.000	0.000
	3:WIND LOAD	-6.982	-0.101	-1.381	0.000	0.000	0.000
	4:FLOOD LOAI	-1.376	-0.106	-0.363	0.000	0.000	0.000
	10:1.2DL+1.6LI	0.033	0.196	0.008	0.000	0.000	0.000
	11:1.2DL+LL+0	-4.833	0.010	-1.045	0.000	0.000	0.000
	12:0.9DL+0.5W	-4.839	-0.069	-1.046	0.000	0.000	0.000
	13:1.2DL+0.5W	-3.454	0.066	-0.681	0.000	0.000	0.000
	5:1.2DL+1.6LL	0.033	0.196	0.008	0.000	0.000	0.000
	6:1.2DL+LL+0.	-4.833	0.010	-1.045	0.000	0.000	0.000
	7:0.9DL+0.5W-	-4.839	-0.069	-1.046	0.000	0.000	0.000
	8:1.2DL+0.5W	-3.454	0.066	-0.681	0.000	0.000	0.000
	9:DEFLECTIOI	-8.330	-0.060	-1.737	0.000	0.000	0.000
46	1:DEAD LOAD	-0.000	0.397	0.000	0.008	-0.000	0.006
	2:LIVE LOAD	-0.000	-0.004	-0.000	-0.002	-0.000	0.007
	3:WIND LOAD	-0.105	-0.001	-0.077	-2.534	-0.003	3.521
	4:FLOOD LOAI	-0.105	0.005	-0.077	-2.531	-0.001	3.599
	10:1.2DL+1.6LI	-0.000	0.470	0.000	0.007	-0.000	0.018
	11:1.2DL+LL+0	-0.158	0.476	-0.115	-3.791	-0.002	5.373
	12:0.9DL+0.5W	-0.158	0.361	-0.115	-3.791	-0.002	5.365
	13:1.2DL+0.5W	-0.053	0.476	-0.038	-1.258	-0.002	1.768
	5:1.2DL+1.6LL	-0.000	0.470	0.000	0.007	-0.000	0.018
	6:1.2DL+LL+0.	-0.158	0.476	-0.115	-3.791	-0.002	5.373
	7:0.9DL+0.5W-	-0.158	0.361	-0.115	-3.791	-0.002	5.365
	8:1.2DL+0.5W	-0.053	0.476	-0.038	-1.258	-0.002	1.768
	9:DEFLECTIOI	-0.211	0.396	-0.154	-5.059	-0.004	7.132

Reaction Summary

			Horizontal	Vertical	Horizontal	Moment		
	Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
Max FX	43	9:DEFLECTIOI	**5.606**	0.305	-0.319	0.000	0.000	0.000
Min FX	44	9:DEFLECTIOI	**-8.330**	-0.060	-1.737	0.000	0.000	0.000
Max FY	22	10:1.2DL+1.6LI	-0.001	**2.691**	-0.048	-0.753	-0.000	-0.039
Min FY	44	4:FLOOD LOAI	-1.376	**-0.106**	-0.363	0.000	0.000	0.000
Max FZ	32	10:1.2DL+1.6LI	0.001	2.653	**0.036**	1.234	0.000	-0.044
Min FZ	44	9:DEFLECTIOI	-8.330	-0.060	**-1.737**	0.000	0.000	0.000
Max MX	23	10:1.2DL+1.6LI	0.001	2.543	0.036	**1.369**	0.000	-0.054
Min MX	24	9:DEFLECTIOI	-0.187	1.888	-0.572	**-46.763**	-0.010	7.623
Max MY	22	9:DEFLECTIOI	-0.158	1.940	-0.672	-44.034	**0.053**	9.241
Min MY	32	9:DEFLECTIOI	-0.192	1.934	-0.401	-32.629	**-0.037**	8.123
Max MZ	22	9:DEFLECTIOI	-0.158	1.940	-0.672	-44.034	0.053	**9.241**
Min MZ	23	10:1.2DL+1.6LI	0.001	2.543	0.036	1.369	0.000	**-0.054**

CLIENT: USIBWC
JOB NO.: 4839-02
PROJECT: SUNLAND PARK LEVEE
DESIGNER: SG
DATE: Oct-18
SUBJECT: **Wx40**
CHECKER:
DATE:

SECTION PROPERTIES

SECTION TYPE = *W*

SOIL CAPACITY = *0* tons (0 kips)

F_y =	*36.0*	ksi	E_s =	*29000*	ksi	S_x =	*35.5*	in^3
h or d	*8.25*	in	K_x =	*1.2*		S_y =	*12.2*	in^3
b_f =	*8.07*	in	K_y =	*1.2*		r_x =	*3.53*	in
t_f =	*0.56*	in	Lu_x =	*8.00*	ft	r_y =	*2.04*	in
t_w =	*0.36*	in	Lu_y =	*8.00*	ft	Z_x =	*39.8*	in^3
A_s =	*11.7*	in^2	L_b =	*8.00*	ft	Z_Y =	*18.5*	in^3
$b_f/2t_f$ =	*7.21*		h_0 =	*7.69*	in	r_{ts} =	*2.31*	in
h/tw=	*22.92*		J =	*1.12*	in^4	Ix=	*146*	in^4
A_f =	4.52	in	Cw=	*726*	in^6	Iy=	*49.1*	in^4
d/A_f =	*1.83*					S_{xt}=	*35.5*	in^3

AXIAL CAPACITY (AISC - LRFD Chapter E)

KL/r = **56.471** 4.71 SQRT(E/Fy) 133.68 $Fe = \pi^2 E_s/[(KL/r)^2]$ = 89.75 ksi

When KL/r =< 4.71 SQRT(E/Fy):

Fcr = [0.658^(Fy/Fe)]Fy = 30.44 ksi

When C_c > 4.71 SQRT(E/Fy):

Fcr = 0.877 Fe = N/A ksi

A.- Maximum Allowable Axial Load:

$\phi P_n = F_{cr}A_g$ = 320.50 kips

FLEXURAL CAPACITY (AISC - LRFD Chapter F)

$b_f/2t_f$ =	*7.21*	<	$0.38\sqrt{E/F_y}$ = 10.79	(Flange is Compact)
h/tw =	*22.92*	<	3.76SQRT(E/Fy) = 106.72	(Web is Compact)

DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND FLANGES

Yielding Limit State :

$M_{nx} = M_p = F_y Z_x$ = 1432.80 kips-in
$M_{ny} = M_p = F_y Z_y$ = 666.00 kips-in

B.- Flexural Capacities:

ϕMnx = **1289.52** kips-in
ϕMny = **599.40** kips-in

COMBINED STRESSES (AISC - LRFD Chapter H)

Pr/Pc>=0.2
(H1-1a): (Pr/Pc)+8/9(Mrx/Mcx+Mry/Mcy)<1
Pr/Pc<0.2
(H1-1b): (Pr/2Pc)+(Mrx/Mcx+Mry/Mcy)<1

$P_{applied}$ = 2.70 kips
Mrx = 5.57 kip-in
Mry = 28.41 kip-in

CLIENT: USIBWC JOB NO.: 4839-02
PROJECT: SUNLAND PARK LEVEE DESIGNER: SG DATE: Oct-18
SUBJECT: **Wx40** CHECKER: DATE:

Vu= 0.22 kip

(H1-1a): N/A
(H1-1b): 0.056

SHEAR (AISC - LRFD Chapter G)

DOUBLY SYMMETRIC I-SHAPED MEMBERS

a= 5 in

Kv= $5+5/(a/h)^2$= 18.61

If h/tw ≤ 2.24 SQRT(E/Fy)

Cv= 1

If h/tw ≤ 1.10 SQRT(KvE/Fy)

Cv= 1

Vn= 0.6FyAwCv= 64.15 kips > Vu, OK

DANNENBAUM

CLIENT: USIBWC
JOB NO.: 4839-02
PROJECT: SUNLAND PARK LEVEE
DESIGNER: SG
DATE: Feb-21
SUBJECT: **C8x11.5**
CHECKER:
DATE:

SECTION PROPERTIES

SECTION TYPE = ***W***

SOIL CAPACITY = *0* tons (0 kips)

F_y =	***36.0*** ksi	E_s =	*29000* ksi	S_x =	*8.140* in^3
h or d	*8.00* in	K_x =	*1.2*	S_y =	*0.780* in^3
b_f =	*2.26* in	K_y =	*1.2*	r_x =	*3.110* in
t_f =	*0.39* in	Lu_x =	*7.00* ft	r_y =	*0.623* in
t_w =	*0.22* in	Lu_y =	*7.00* ft	Z_x =	*9.630* in^3
A_s =	*3.38* in^2	L_b =	*7.00* ft	Z_Y =	*1.570* in^3
$b_f/2t_f$ =	*2.90*	h_0 =	*7.610* in	r_{ts} =	*0.756* in
h/tw=	*36.36*	J =	*0.130* in^4	Ix=	*32.600* in^4
A_f =	*0.88* in	Cw=	*16.50* in^6	Iy=	*1.310* in^4
d/A_f =	*9.08*			S_{xt}=	*0.780* in^3

AXIAL CAPACITY (AISC - LRFD Chapter E)

KL/r = **161.798** 4.71 SQRT(E/Fy) 133.68 $Fe = \pi^2 E_s/[(KL/r)^2]$ = 10.93 ksi

When KL/r =< 4.71 SQRT(E/Fy):

Fcr = [0.658^(Fy/Fe)]Fy = N/A ksi

When C_c > 4.71 SQRT(E/Fy):

Fcr = 0.877 Fe = 9.59 ksi

A.- Maximum Allowable Axial Load:

$\phi P_n = F_{cr}A_g$ = 29.17 kips

FLEXURAL CAPACITY (AISC - LRFD Chapter F)

$b_f/2t_f$ =	*2.90*	<	$0.38\sqrt{E/F_y}$ = 10.79	(Flange is Compact)
h/tw =	*36.36*	<	3.76SQRT(E/Fy) = 106.72	(Web is Compact)

DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND FLANGES

Yielding Limit State :

$M_{nx} = M_p = F_y Z_x$ = 346.68 kips-in
$M_{ny} = M_p = F_y Z_y$ = 56.52 kips-in

B.- Flexural Capacities:

ϕMnx= **25.27** kips-in
ϕMny= **42.75** kips-in

COMBINED STRESSES (AISC - LRFD Chapter H)

Pr/Pc>=0.2
(H1-1a): (Pr/Pc)+8/9(Mrx/Mcx+Mry/Mcy)<1
Pr/Pc<0.2
(H1-1b): (Pr/2Pc)+(Mrx/Mcx+Mry/Mcy)<1

$P_{applied}$ =	4.18 kips
Mrx =	12.30 kip-in
Mry =	20.21 kip-in
Vu=	0.72 kip

CLIENT: USIBWC
JOB NO.: 4839-02
PROJECT: SUNLAND PARK LEVEE
DESIGNER: SG
DATE: Feb-21
SUBJECT: **C8x11.5**
CHECKER:
DATE:

(H1-1a): N/A
(H1-1b): 1.031

SHEAR (AISC - LRFD Chapter G)

DOUBLY SYMMETRIC I-SHAPED MEMBERS

a= 5 in

Kv= $5+5/(a/h)^2$ = 17.80

If h/tw ≤ 2.24 SQRT(E/Fy)

Cv= 1

Vn= 0.6FyAwCv= 38.02 kips > Vu, OK

Deflection CTRL

Δ_{max}= 0.056 from STAAD Model
L/480= 0.1875 OK

CLIENT: USIBWC
PROJECT: SUNLAND PARK LEVEE
SUBJECT: **L4x4x0.5**

JOB NO.: 4839-02
DESIGNER: SG
DATE: Oct-18
CHECKER:
DATE:

SECTION PROPERTIES

SECTION TYPE = ***W***

SOIL CAPACITY = *0* tons (0 kips)

F_y =	***36.0*** ksi	E_s =	*29000* ksi	S_x =	*1.960* in^3
h or d	*4.00* in	K_x =	*1.2*	S_y =	*1.960* in^3
b_f =	*4* in	K_y =	*1.2*	r_x =	*1.210* in
t_f =	*0.5* in	Lu_x =	*2.00* ft	r_y =	*1.210* in
t_w =	*0.5* in	Lu_y =	*2.00* ft	Z_x =	*3.500* in^3
A_s =	*3.75* in^2	L_b =	*2.00* ft	Z_Y =	*3.500* in^3
$b_f/2t_f$ =	*4.00*	h_0 =	*0.32* in	r_{ts} =	*2.160* in
h/tw=	*8.00*	J =	*0.32* in^4	Ix=	*5.520* in^4
A_f =	*2.00* in	Cw=	*0.37* in^6	Iy=	*5.520* in^4
d/A_f =	*2.00*			S_{xt}=	*1.960* in^3

AXIAL CAPACITY (AISC - LRFD Chapter E)

KL/r = **23.802** 4.71 SQRT(E/Fy) 133.68 $Fe = \pi^2 E_s/[(KL/r)^2]$ = 505.22 ksi

When KL/r =< 4.71 SQRT(E/Fy):

Fcr = [0.658^(Fy/Fe)]Fy = 34.94 ksi

When C_c > 4.71 SQRT(E/Fy):

Fcr = 0.877 Fe = N/A ksi

A.- Maximum Allowable Axial Load:

$\phi P_n = F_{cr}A_g$ = 117.93 kips

FLEXURAL CAPACITY (AISC - LRFD Chapter F)

$b_f/2t_f$ =	*4.00*	<	0.38*SQRT*(E/F_y) = 10.79	(Flange is Compact)
h/tw =	*8.00*	<	3.76SQRT(E/Fy) = 106.72	(Web is Compact)

B.- Flexural Capacities:

ϕMnx= **113.40** kips-in
ϕMny= **113.40** kips-in

COMBINED STRESSES (AISC - LRFD Chapter H)

Pr/Pc>=0.2
(H1-1a): (Pr/Pc)+8/9(Mrx/Mcx+Mry/Mcy)<1
Pr/Pc<0.2
(H1-1b): (Pr/2Pc)+(Mrx/Mcx+Mry/Mcy)<1

$P_{applied}$ = 0.36 kips
Mrx = 15.65 kip-in
Mry = 12.58 kip-in
Vu= 1.32 kip

(H1-1a): N/A
(H1-1b): 0.250

SHEAR (AISC - LRFD Chapter G)

DOUBLY SYMMETRIC I-SHAPED MEMBERS

CLIENT: USIBWC JOB NO.: 4839-02
PROJECT: SUNLAND PARK LEVEE DESIGNER: SG DATE: Oct-18
SUBJECT: **L4x4x0.5** CHECKER: DATE:

a= 5 in
Kv= 5+5/(a/h)^2= 8.20

If h/tw ≤ 2.24 SQRT(E/Fy)
Cv= 1

Vn= 0.6FyAwCv= 43.20 kips > Vu, OK

Appendix C.2: Levee Closure Alternatives

DRAFT

MEMO

ARCADIS Design & Consultancy for natural and built assets

To:
Xochitl Aranda

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From:
Ardita Dushi

Date:
November 15, 2018

Arcadis Project No.:
01991047.0000

Subject:
Country Club Bridge – West Levee Floodgate Design Alternatives
Sunland Park Levee Repair – West Levee
Contract No. IBM15D0001, Task Order No. IBM17T0010

This memorandum presents the design options for the floodgate on the west side of the Country Club Bridge that were discussed during our phone call on November 15, 2018. Advantages and disadvantages for each alternative are also included in this memo.

FEMA requires a freeboard of 4 ft above the 100-year flood within the 100 ft of the bridge. Arcadis understands that due to a prior agreement of USIBWC with FEMA, the floodgate design shall include 3 ft of freeboard above the 100-year flood. Using the current agreement with FEMA, the top of the existing pavement at the bridge crossing is about 2 feet below the required elevation.

Arcadis recommends the following design alternatives for the floodgate design on the west levee:

1) Flex Gate
2) Aquafence
3) Steel Swing/Roller Gate
4) Roadway Raise

These design alternatives are presented in Attachment 1.

ATTACHMENT 1

WEST LEVEE FLOODGATE ALTERNATIVES

DRAFT

ARCADIS Design & Consultancy for natural and built assets

SUNLAND PARK LEVEE REPAIR

Country Club Bridge – West Levee Floodgate Alternatives

November 15, 2018

CONTRACT: IBM15D0001
ORDER NO. IBM17T0010

DRAFT

Problem Statement

- Survey data shows that a gate closure is needed on the west side of the Country Club Bridge.
- The selected alternative for the 60% design of the east side is a sliding gate (about $900K at 60% design stage).
- A similar design on west side will be even longer and more costly.
- Other alternatives are available that may be significant at lower cost.

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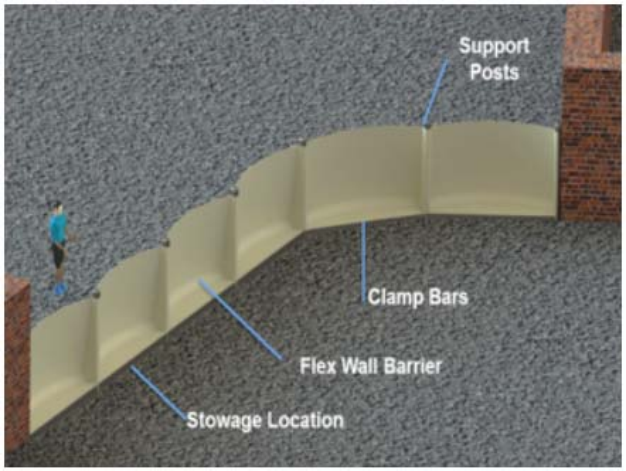
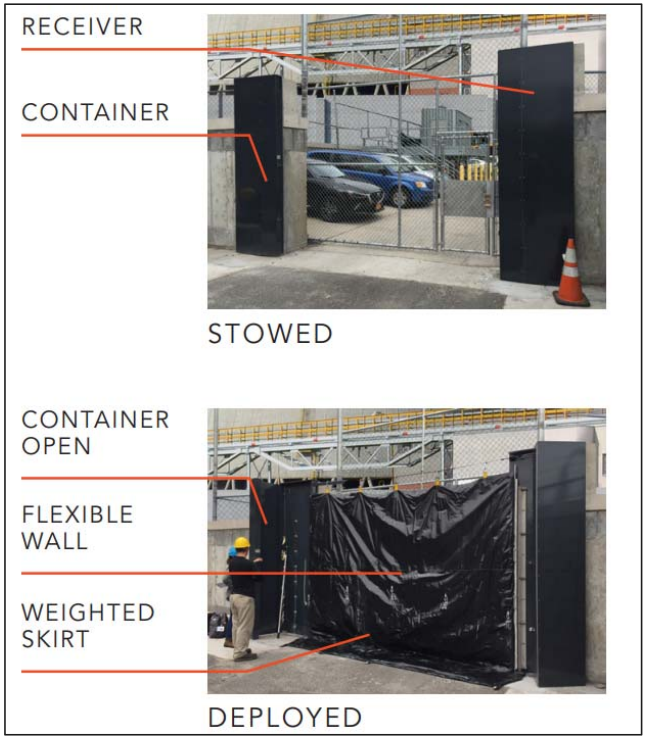
DRAFT

Alternative 1 – Flex Gate





- Lower capital cost. May require replacement more frequently than a steel gate solution.
- Stores in place.
- Easy to deploy without additional equipment.
- Conforms to roadway slopes.
- Ideal for low heights of protection.
- Can sustain debris impact. No deflection concern.
- Minimal disruption to traffic during construction.
- Easy to protect from vehicle strikes.
- https://www.ilcdover.com/product-category/flood-protection/

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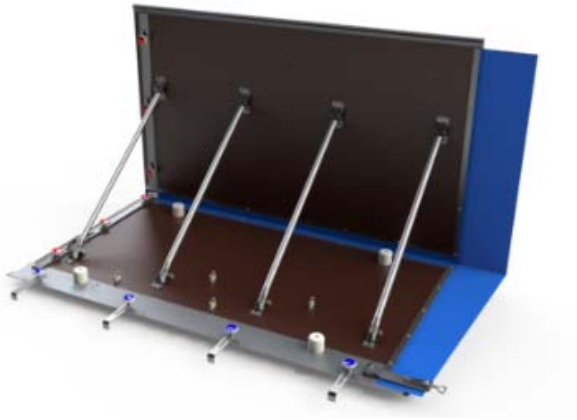
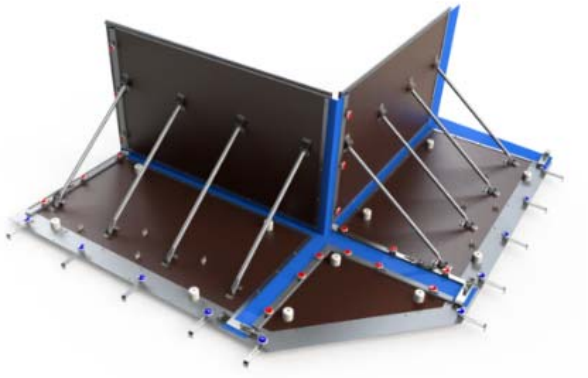
Alternative 2 – Aquafence





- Stored off-site. If stored and maintained properly, may have longer design life.
- Lightweight.
- Easy to deploy but will require more time than the Swing and Flex gates. Therefore, potential for road closures to be longer.
- Conforms to roadway slopes.
- Lower capital cost than structural gate options.
- Minimal panel available height is 4 ft.
- Minimal disruption to traffic during construction.
- Can sustain debris impact.
- https://www.aquafence.com/

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Alternative 3 – Steel Swing/Roller Gate

- Higher capital cost.
- Storage details to be determined in the 90% design, but greater space is required.
- Easy to deploy.
- May need to change roadway slope.
- Relatively large span (~90 ft.) Long spans and deflection concerns would need to be addressed during the 90% design. Center support column may be required.
- Sill plate or track on road may continue to cause noise issues if a cover plate is required. Noise issues need to be addressed in the 90% design stage.
- Longer disruption to traffic during construction.

Swing Gate

Roller Gate

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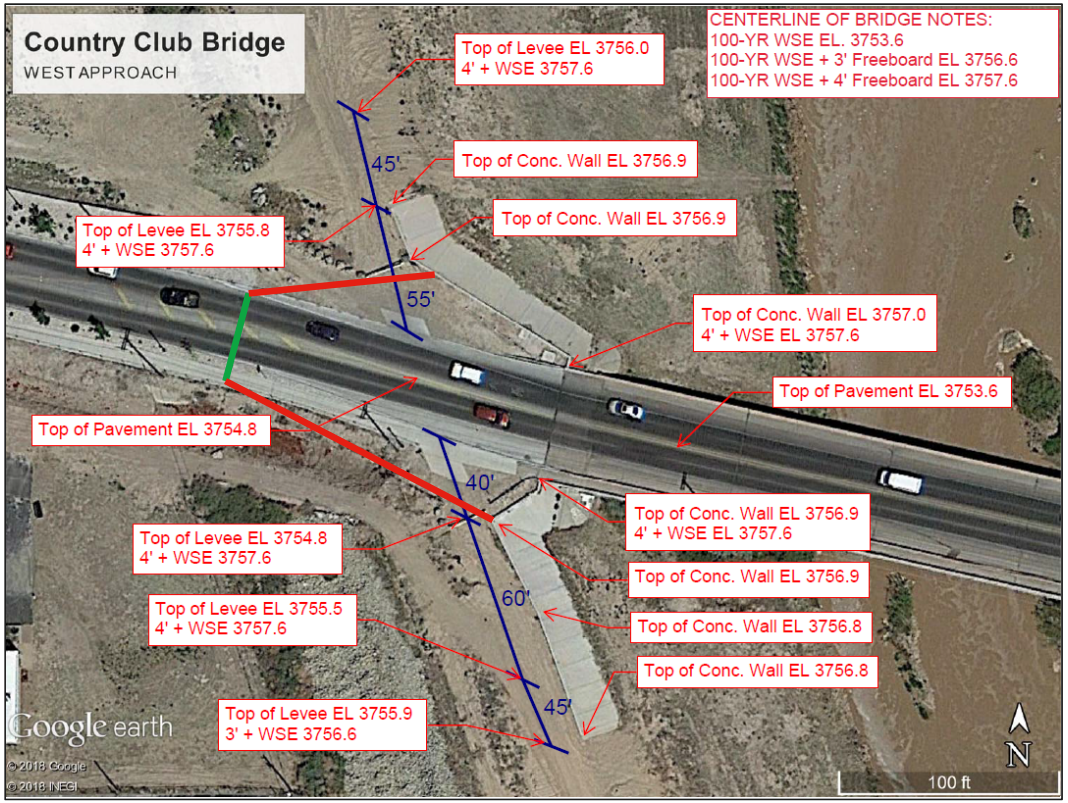
Alternative 4 – Raising the road



- Needs to be away from the bridge-abutment expansion joint.
- Will need additional grading and tie-in walls which may not be feasible.
- Passive solution, does not require deployment.
- Little to no additional maintenance required outside traditional roadwork.
- Need to evaluate cost.
- Will require additional survey, longer design time and additional time to coordinate with City of El Paso.

Advantages and Disadvantages
of
Closure Alternatives

Flex Gate:

Flex Gates are portable and easy to deploy. They may require replacement more frequently. Excerpts from the vendor's O&M manual says that "Flex-Wall™ is capable of 75 years of operation with proper routine maintenance and refurbishment of the flexible materials at planned intervals. Refurbishment intervals are dependent on operational use and environmental conditions. The limiting elements of any flood protection system are the elastomeric components (seals and coated fabrics). Our systems are designed to use the longest life materials available and protect these materials from environmental exposure when not in use. We anticipate that the flexible components of the system will need to be replaced at 20-25 year intervals."

Aquafence:

Aquafence is easy to deploy. According to the Aquafence website, the product "... is packaged and stored in custom-designed stackable wooden crates (2,1m x 1,2m, 7ft x 4ft). The crates can be stacked in columns of 4 high. For example, a typical parking spot can hold 2 crates deep and 4 high, to provide 500 linear feet of 4-foot fence. After a deployment, the panels should be cleaned and dried for storage and future use". With approximately 90 ft. of linear protection, believe the material for both east and west gate can be stored inside a reasonably sized trailer which can be hauled by a pickup truck. When not deployed, the trailer can be stored at a nearby local maintenance office.

Designed lifespan of the Aquafence product is 30+ years. The frequency of usage does not lessen the lifespan of the product, however minor damage could occur while being deployed or stored, which would mean replacing those small parts. All parts on the panels are replaceable with simple hand tools. In a recent project, we have designed and installed 8 ft. high Aquafence as a permanent flood proofing measure that is anticipated to stay in place for at least 2-3 years. The vendor has also communicated that their panels have been in use under water for over 365 days consecutively, in Basel Switzerland and have not needed any replacement.

Swing/roller gate:

Swing/Roller gate is easy to deploy. However, there are more maintenance requirements, storage requirements and may cost more than some other options.

Raising the Road:

Not at all practicable

Boxwall:

The Boxwall System (website link) is a good solution for low height deployable flood protection. However, the maximum height of protection is limited to 50 cm. or 1.62 ft. The protection height required across the roadway will vary from two to three ft. and hence eliminates the scope of using the Boxwall System.

Aluminum Stoplogs:

This is one of the earliest deployable flood control system for low to moderate height of protection. More information on this system can be found on this website. This can be a good choice for both east and west gates. Due to the large span across the roadway, three to four intermediate columns may need to be installed during deployment. These columns will be light weight because of the smaller flood height of 2-3 ft. The system will not require any heavy equipment or machinery to install. The planks are made out of aluminum and can be transported and handled easily. The following picture shows the installation of a stoplog floodwall column.

The aluminum planks can be stored in bundled stacks. An 800 sf. removable floodwall system, measured on a vertical plane, would require 100 sf. of storage space. As mentioned for AquaFence, a trailer can be used to store the planks for both east and west gate. Maintenance is very minimal. The bottom most plank comes with a rubber gasket attached which will have contact with the roadway surface. This gasket will need to be replaced every 10 to 15 years.

Appendix C.3: Levee Closure Structures

ARCADIS Design & Consultancy for natural and built assets

EL PASO COUNTY-CITY OF EL PASO

IBWC-WEST LEVEE COUNTY CLUB ROAD

Structural Design Calculation for Stop Log

Revision History

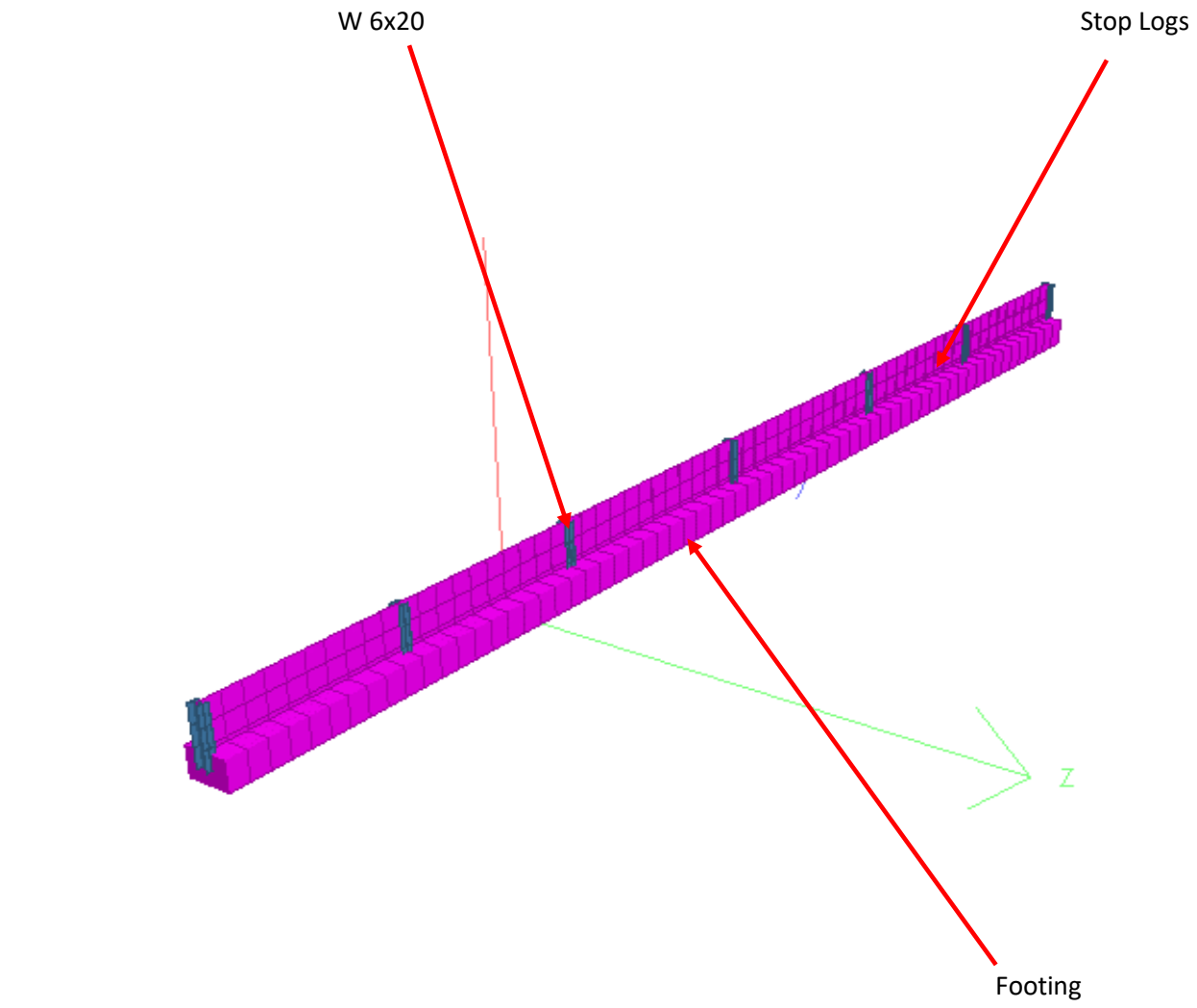
Rev	Date	Description	Author	Checker	Approver
0	25-01-2019	Issued for Review	AW	KB	

INTRODUCTION

This report consists of the structural design calculations of Stop Log at West levee county club road for minimum flood of 1'6" at the crown of the road and 2' at the road edge.

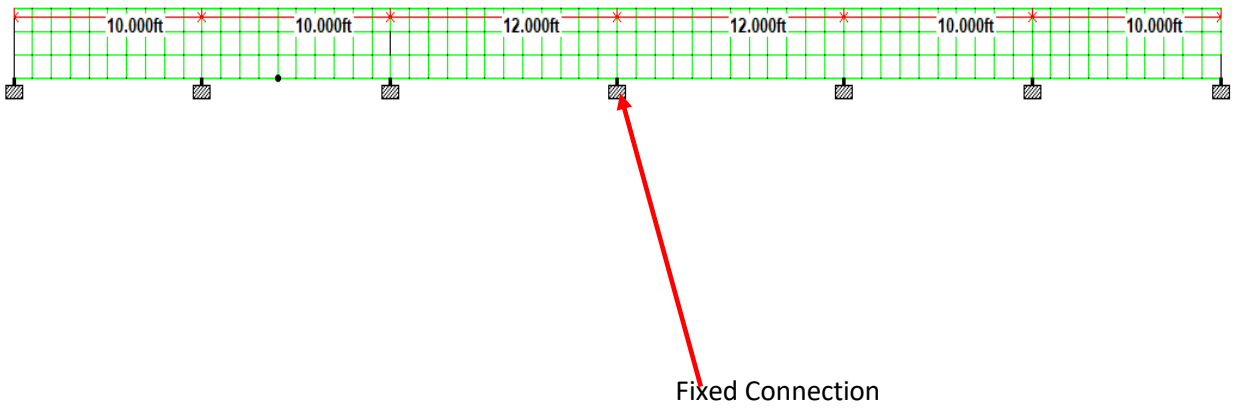
STAAD MODEL

(Staad model is analysed at the truck location and similar design is taken at other locations)



This staad model consists of a 1" thick steel plank and W section post which is analysed for Dead and Flood and designed for considering the appropriate Load Combinations.

Support condition:



ARCADIS	DOCUMENT No. (Project/Doc)	SHEET No.	
	IBWC-STOP LOG	Date.	
	SUBJECT - STRUCTURAL DESIGN CALCULATIONS	By: AW	Chk: KB

REF	CALCULATIONS	OUTPUT

LOADS:

1. Dead load(DL)

In STAAD, selfweight command is used to add dead weight of all the members.

2. Live Load (LL):

Live load = 100.00 psf

Truck load is considered in design,

AASHTO Cl.3.6.1.2 .2

Rear axle load = 32.6 kip
Front axle load = 8 kip
Load per rear wheel assembly = Load/2 = 16.3 kip
Load per Front wheel assembly = Load/2 = 4 kip

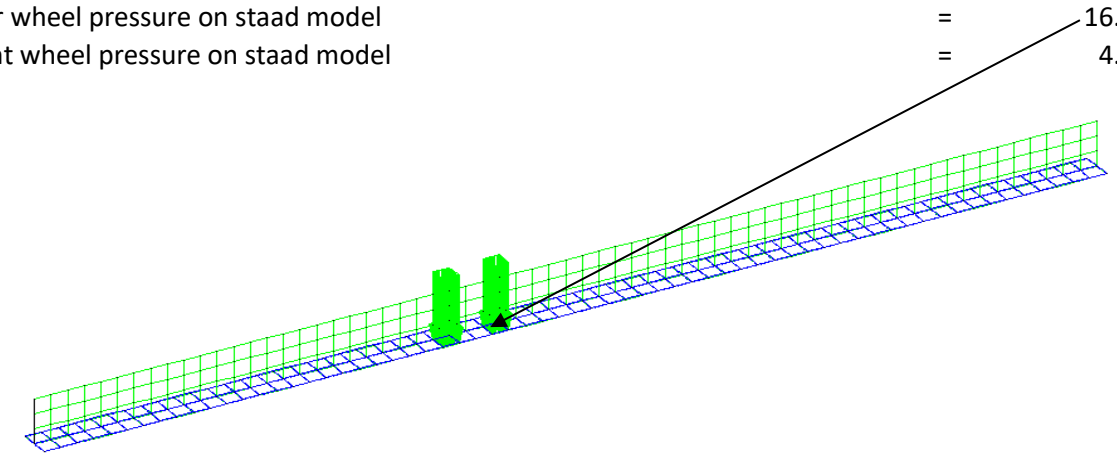
Patch load on Rear wheel assembly:

Load per rear wheel assembly / Contact Area = 16.3/1.48
= 11.01 kip/sft

Load per fornt wheel assembly / Contact Area = 4/1.48
= 2.703 kip/sft

The above force is distributed over a area of 1'-0" width & 1'-0" length on Staad model.

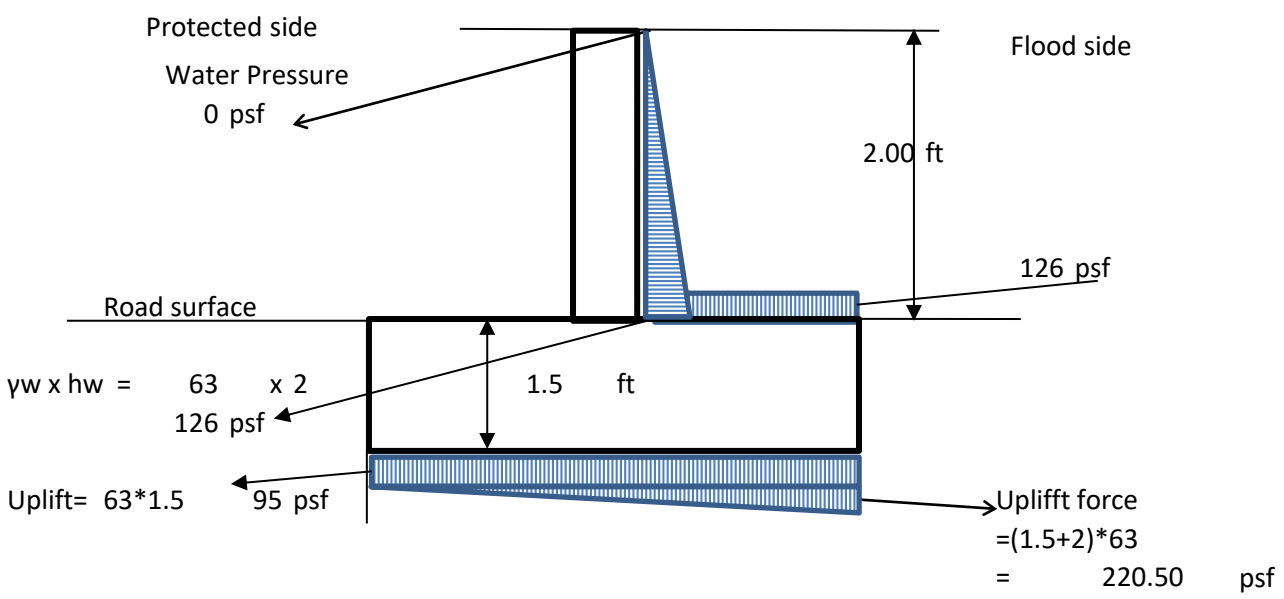
Rear wheel pressure on staad model = 16.30 ksf
Front wheel pressure on staad model = 4.00 ksf



3.Flood Load (Fa):

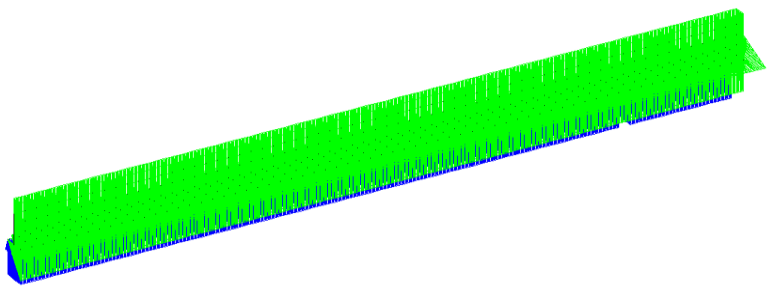
Lateral Pressure on the wall:

Height of water hw= = 2.00 ft
Density of water γw = = 63.00 pcf
Thickness of footing = 1.50 ft



Note: Water level considered upto FGL on the protected side.

REF	CALCULATIONS	OUTPUT



Load Combinations:

Allowable Stress Design : (Section 2.4 ASCE 7-10)

101 1.0D
102 1.0D+1.0LL
103 1.0DL+1.0FA

Strength Design : (Section 2.3 ASCE 7-10)

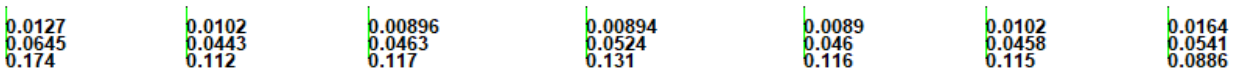
201 1.4D
202 1.2D+1.6LL
203 1.4DL+1.4FA
204 0.9DL+1.0FA

Analysis and Design results:

Steel Post

Member utilisation ratio:

Member	Utilisation ratio
W6X20	0.131
HSST4X3X0.25	0.174
C6X13	0.09



Bolt & Base Plate Design

Reaction Summary

	Node	L/C	Horizontal FX (kip)	Vertical FY (kip)	Horizontal FZ (kip)	Moment MX (kip*ft)	Moment MY (kip*ft)	Moment MZ (kip*ft)
Max FX	1288	201:1.4D	**0.382**	0.821	0.000	0.000	0.000	0.033
Min FX	1333	201:1.4D	**-0.382**	0.821	0.000	0.000	0.000	-0.033
Max FY	2400	201:1.4D	-0.000	**2.184**	0.000	0.000	0.000	0.000
Min FY	1288	204:0.9DL+1.0I	0.246	**0.528**	0.559	0.894	-0.528	0.021
Max FZ	2400	203:1.4DL+1.4I	-0.000	2.184	**2.158**	3.627	-0.000	0.000
Min FZ	1288	201:1.4D	0.382	0.821	**0.000**	0.000	0.000	0.033
Max MX	2400	203:1.4DL+1.4I	-0.000	2.184	2.158	**3.627**	-0.000	0.000
Min MX	1288	201:1.4D	0.382	0.821	0.000	**0.000**	0.000	0.033
Max MY	1333	203:1.4DL+1.4I	-0.382	0.821	0.783	1.252	**0.739**	-0.033
Min MY	1288	203:1.4DL+1.4I	0.382	0.821	0.783	1.252	**-0.739**	0.033
Max MZ	1288	201:1.4D	0.382	0.821	0.000	0.000	0.000	**0.033**
Min MZ	1333	201:1.4D	-0.382	0.821	0.000	0.000	0.000	**-0.033**

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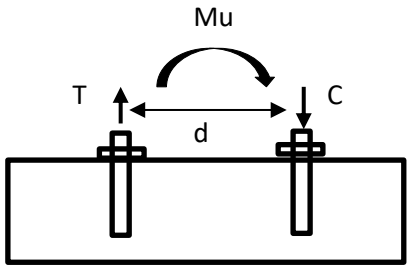
		Date.	
	SUBJECT - STRUCTURAL DESIGN CALCULATIONS	By AW	Chk KB

REF	CALCULATIONS	OUTPUT



Vertical reaction from staad		=	2.184	kip
Moment considered for anchor bolts design (couple effect on anchor bolt), Mu=			3.63	kip-ft
Min. Spacing between anchor bolts, d		=	8	in
Force acting on anchor bolts due couple moment= Mu/d =	3.63/(8/12)	=	5.45	kip-ft
Hence total tension force in anchor bolts,T =	5.445+2.184	=	7.63	kip

Shear capacity of single bolt:

Diameter of bolt	=	0.5	in
Area of bolt, A_b	=	0.196	in^2
Ultimate tensile strength, F_u	=	58	ksi
Yield strength	=	36	ksi
$F_{nv,}$ (Assuming threads are in shear plane)	=	$0.45F_u$	
F_{nv}	=	26.1	ksi
Nominal shear rupture strength of bolt, R_n	=	$F_{nv}xA_b$	
	=	5.13	kips
Hence shear capacity of bolt	=	ϕR_n	
ϕ	=	0.75	
Shear capacity of bolt for single shear= ϕRn	=	3.84	kips

Hence minimum 2no's of 1/2" connection bolts are required.

Plate Capacity:

Minimum thickness of plate, t	=	0.25	in
Minimum edge distance of plate, b = 1.5 dia of bolt	=	0.75	in
Area of bolt, Ap, = t*b	=	0.19	in^2
Ultimate tensile strength, F_u	=	70	ksi
Yield strength	=	25	ksi
Shear strength of plate, R_n	=	$0.6xF_uxAp$	
	=	7.88	kips
Hence Plate capacity	=	ϕR_n	
ϕ	=	0.75	
Shear capacity of Plate for single shear= ϕRn	=	5.91	kips

Hence minimum 1/4" connection plates are required.

Weld Capacity:

Minimum thickness of weld, w	=	0.1875	in
Ultimate tensile strength of weld, F_u	=	70	ksi
Shear strength of plate, F_{nw}	=	$0.6xF_u$	
	=	42	ksi
Strength of weld per inch	=	$wxF_{nw}x1/\sqrt{2}$	
	=	5.57	
ϕ	=	0.75	
Weld capacity	=	4.18	kips

Hence minimum 3/16" thick weld required.

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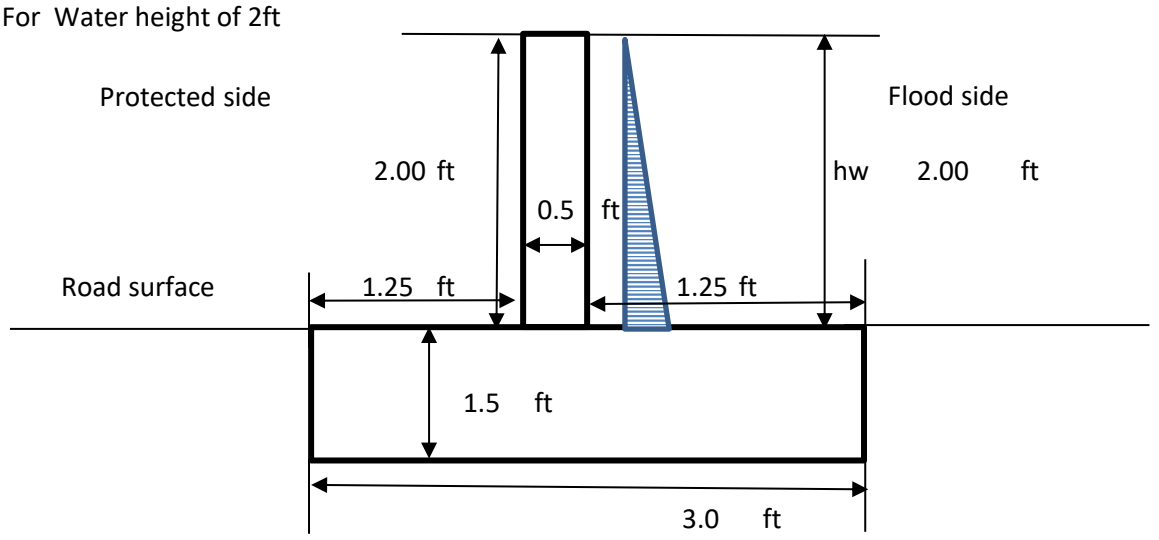
	IBWC-STOP LOG	Date.	
	SUBJECT - STRUCTURAL DESIGN CALCULATIONS	By	Chk
		AW	KB

REF	CALCULATIONS	OUTPUT

Check for sliding:

For Water height of 2ft



Density of concrete		=	150.00	pcf
Density of water = γw		=	63.00	pcf
Length of footing considered	Lf	=	1.00	ft
Panel length	L	=	1.00	ft
Area of panel		=	2.00	sft
Weight of panel per sq.ft		=	20.00	lbs
Total weight of panel	20*2*1/1000=	=	0.04	kips
Total weight of footing	(3*1.5*1*150/1000)=	=	0.68	kips
Vertical downword force, Wf=		=	0.72	kips
Co efficient of friction		=	0.35	

Resistance Force:

Resisting force= Wf x co efficient of friction=	0.35x0.72	=	0.25	kips
Passive Soil pressure=		=	300.00	psf
Lateral Soil Load,SL=	300x1.5x1/1000=	=	0.45	kips
Total resisting force=	0.45+0.25	=	0.70	kips

Sliding Force:

Hydrostatic horizontal pressure at the base =	γs x hw	=	126.00	psf
Hydrostatic horizontal force =	0.5x (γs x hw)x hw x L/1000	=	0.13	kips

Factor of safety for sliding= (Resistance Force) / (Sliding Force) = 5.56

FOS = 5.56 > 1.50 SAFE

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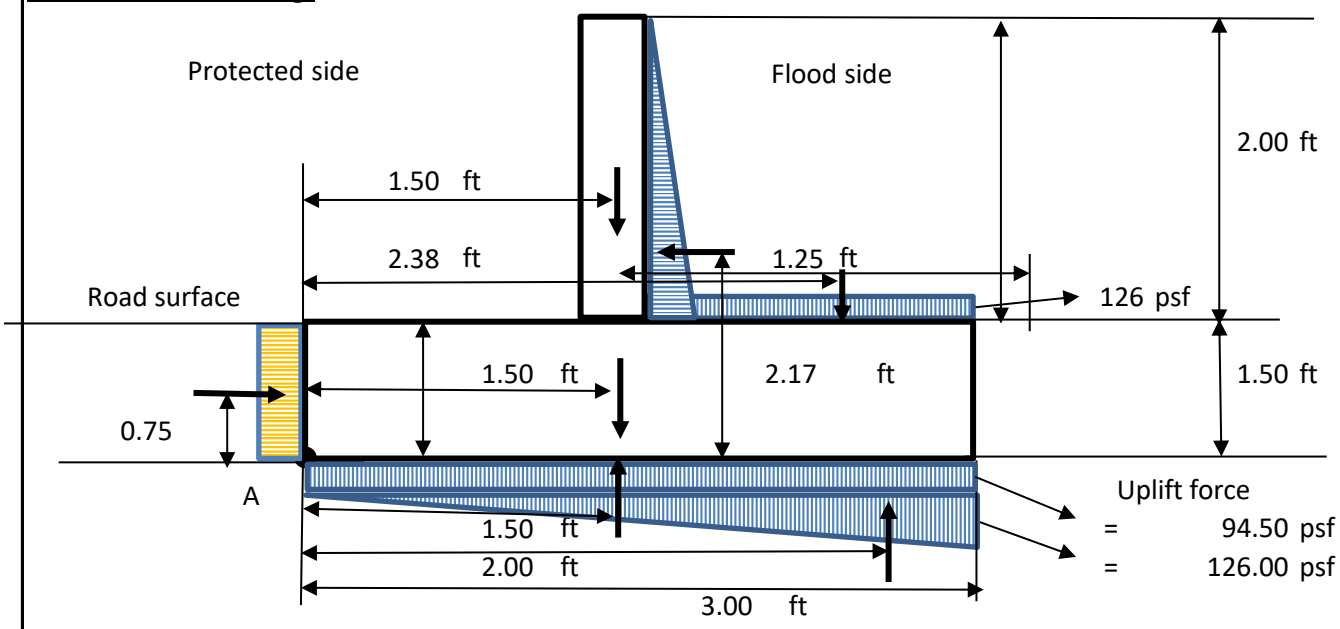
	SUBJECT - STRUCTURAL DESIGN CALCULATIONS	By: AW / Chk: KB

REF	CALCULATIONS	OUTPUT

Check for overturning:



Resisting moment

Moment due to footing weight=	0.675x1.5	=	1.01	kip-ft
Moment due to water on footing=	(1.25x126x1/1000)x2.375	=	0.37	kip-ft
Moment due to panel weight=	0.04x1.5	=	0.06	kip-ft
Moment due to passsive earth pressure=	0.45x0.75	=	0.34	kip-ft
Total resisting moment about A=	1.0125+0.37+0.06+0.3375	=	1.78	kip-ft

Overturning moment

Moment due to Hydrostatic load=	0.13x2.17	=	0.27	kip-ft
Moment due to uplift force =	(0.5*126*3*1/1000)*2+(94.5*3*1 *1.5/1000)	=	0.80	kip-ft
Total overturning moment about A=	0.27+0.8	=	1.08	kip-ft

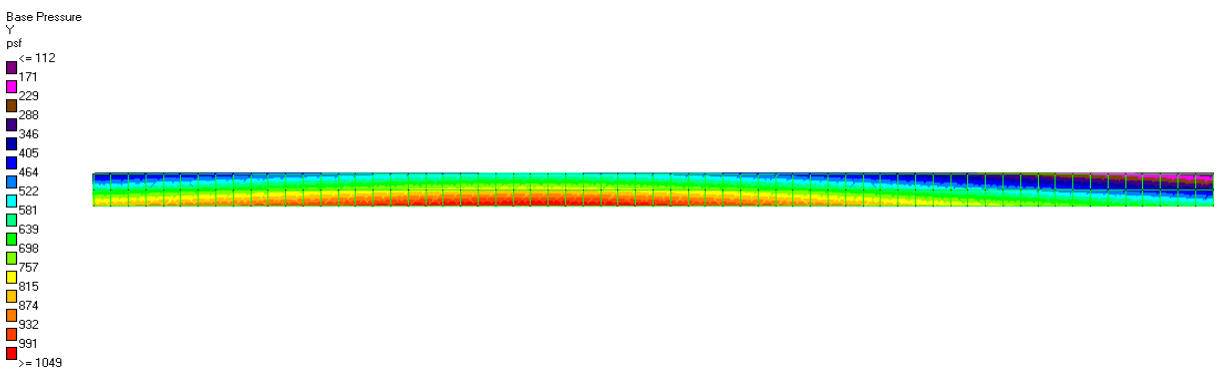
FOS= (Resisting moment about A/Overturning moment about A) = 1.66

FOS = 1.66 > 1.50 SAFE

Base Pressure Check

Allowable bearing pressure		=	2000.00	psf
Settlement		=	1.00	in
Subgrade modulus =	Allowable bearing pressure / Settlement	=	24000.00	pcf
Allowable base pressure from staad		=	1049.00	psf

Hence ok



Footing design

1.5' thick footing

Plate Center Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (psi)	Qy (psi)	Sx (psi)	Sy (psi)	Sxy (psi)	Mx (kip*ft/ft)	My (kip*ft/ft)	Mxy (kip*ft/ft)
Max Qx	1056	202:1.2D+1.6LI	**41.977**	8.010	0.000	0.000	0.000	-11.064	0.656	3.576
Min Qx	1051	202:1.2D+1.6LI	**-34.122**	7.985	0.000	0.000	0.000	-10.949	0.653	-2.429
Max Qy	1055	202:1.2D+1.6LI	17.047	**14.665**	0.000	0.000	0.000	-13.058	1.407	2.076
Min Qy	1993	202:1.2D+1.6LI	-10.083	**-4.274**	0.000	0.000	0.000	-12.096	0.668	0.950
Max Sx	1038	201:1.4D	0.051	-0.229	**0.000**	0.000	0.000	0.005	-0.024	0.000
Min Sx	1038	201:1.4D	0.051	-0.229	**0.000**	0.000	0.000	0.005	-0.024	0.000
Max Sy	1038	201:1.4D	0.051	-0.229	0.000	**0.000**	0.000	0.005	-0.024	0.000
Min Sy	1038	201:1.4D	0.051	-0.229	0.000	**0.000**	0.000	0.005	-0.024	0.000
Max Sxy	1038	201:1.4D	0.051	-0.229	0.000	0.000	**0.000**	0.005	-0.024	0.000
Min Sxy	1038	201:1.4D	0.051	-0.229	0.000	0.000	**0.000**	0.005	-0.024	0.000
Max Mx	2023	202:1.2D+1.6LI	-1.928	-1.379	0.000	0.000	0.000	**0.145**	-0.119	-0.254
Min Mx	1055	202:1.2D+1.6LI	17.047	14.665	0.000	0.000	0.000	**-13.058**	1.407	2.076
Max My	1055	202:1.2D+1.6LI	17.047	14.665	0.000	0.000	0.000	-13.058	**1.407**	2.076
Min My	2011	203:1.4DL+1.4I	0.938	1.549	0.000	0.000	0.000	0.049	**-0.210**	-0.146
Max Mxy	1056	202:1.2D+1.6LI	41.977	8.010	0.000	0.000	0.000	-11.064	0.656	**3.576**
Min Mxy	1051	202:1.2D+1.6LI	-34.122	7.985	0.000	0.000	0.000	-10.949	0.653	**-2.429**

Moment considered for footing design shown below

Footing -1'-6" thick

Mx+ (top) = 1.0 kip-ft

Mx- (bot) = 13.1 kip-ft

My+ (top) = 1.5 kip-ft

My- (bot) = 1.0 kip-ft

Sqx = 42x12x18/1000= 9.1 kips

Sqy = 15x12x18/1000= 3.2 kips

Reference:K:\WATER\US\STRUCTURAL\10028737_IBWC Floodgate - East Levee\

Two way Slab Design:

Fill in all the specific column information (all of the shaded values)

$f'_c := 4500 \cdot psi$ $M_{upos_x} := 1 \cdot ft \cdot kip$ $M_{upos_y} := 1.5 \cdot ft \cdot kip$ $E_s := 29000000 \cdot psi$ $\beta_1 := .85$

$f_y := 60000 \cdot psi$ $M_{uneg_x} := 13.1 \cdot ft \cdot kip$ $M_{uneg_y} := 1 \cdot ft \cdot kip$ assume $j := .85$ $\phi_m := .9$

$cover := 3 \cdot in$ $V_{u_x} := 42psi \cdot 12 \cdot in \cdot 18 \cdot in$ $V_{u_y} := 15psi \cdot 12 \cdot in \cdot 18 \cdot in$ $d_b := 0.625 \cdot in$ $\phi_v := 0.75$

$b := 12in$ $d_c := cover + \frac{d_b}{2}$ $d_c = 3.312 \cdot in$ $E := 1$

E= Environmental factor
1.3 If water retaining
1.0 no water

$M_u := \max(M_{upos_x}, M_{upos_y}, M_{uneg_x}, M_{uneg_y})$ $V_u := \max(V_{u_x}, V_{u_y})$

----------Sizing the member----------

$d_{mwreqd} := \sqrt{\frac{M_u}{.9 \cdot K_n \cdot b}}$ $d_{mwreqd} = 4.788 \cdot in$

$d_{vwreqd} := \frac{V_u}{0.85 \cdot 2 \cdot b \cdot \sqrt{f'_c \cdot psi}}$ $d_{vwreqd} = 6.629 \cdot in$

$d_{req} := \max(d_{mwreqd}, d_{vwreqd})$ $d_{req} = 6.629 \cdot in$

$h_{req} := \max(d_{mwreqd}, d_{vwreqd}) + d_c$ $h_{req} = 9.942 \cdot in$

Design depth:

$h := 18in$ $d := h - d_c$ $d = 14.688 \cdot in$

MINIMUM REINFORCEMENT

$\rho_{min_x} := .0030$ $\rho_{min_z} := .0030$

$h_{max} := \min(24 \cdot in, h)$ $h_{max} = 18 \cdot in$

$A_{smin_x} := \frac{(\rho_{min_x} \cdot b \cdot h_{max})}{2}$ $A_{smin_x} = 0.324 \cdot in^2$

$A_{smin_y} := \frac{(\rho_{min_z} \cdot b \cdot h_{max})}{2}$ $A_{smin_y} = 0.324 \cdot in^2$

ρ MAXIMUM

$\rho_b := .85 \cdot \beta_1 \cdot \frac{f'_c}{f_y} \cdot \left(\frac{87000 \cdot psi}{87000 \cdot psi + f_y}\right)$ $\rho_b = 0.0321$

$\rho_{max} := .75 \cdot \rho_b$ $\rho_{max} = 0.0241$

$\rho_{pref} := .5 \cdot \rho_{max}$ $\rho_{pref} = 0.012$

MAIN X Direction REINFORCEMENT(Positive Moment)

$M_{upos_x} \cdot E = 1 \cdot kip \cdot ft$

$$As_{reqd} := \frac{M_{upos_x} \cdot E}{\phi_m \cdot j \cdot d \cdot f_y}$$

$As_{reqd} = 0.018 \cdot in^2$

Try1 := "#5@10" $A_s := 0.413in^2$ $d_b := 0.625in$ $S := 9 \cdot in$

$$d_{act..xp} := h - cover - \frac{d_b}{2} - 1 \cdot in$$

$d_{act..xp} = 13.688 \cdot in$

$$a := \frac{A_s \cdot f_y}{\beta_1 \cdot f'_c \cdot b}$$

$a = 0.54 \cdot in$

$$\phi M_n := \phi_m \cdot A_s \cdot f_y \cdot \left(d_{act..xp} - \frac{a}{2} \right)$$

$\phi M_n = 24.937 \cdot kip \cdot ft$

$$\rho_{act} := \frac{A_s}{b \cdot d_{act..xp}}$$

$\rho_{act} = 0.003$

check_Mcap := "ok" if $\phi M_n \geq M_{upos_x} \cdot E$; "No Good" otherwise

check_Mcap = "ok"

check_As := "ok" if $A_s \geq A_{smin_x} \vee \phi M_n \geq 1.333 \cdot M_{upos_x} \cdot E$; "No Good" otherwise

check_As = "ok"

check_ρ := "ok" if $\rho_{act} \leq \rho_{max}$; "No Good" otherwise

check_ρ = "ok"

Crack Control:

$$M := \frac{M_{upos_x}}{1.7}$$

$M = 0.588 \cdot kip \cdot ft$

$$fs := \frac{M}{A_s \cdot \left(d_{act..xp} - \frac{a}{2} \right)}$$

$fs = 1.274 \cdot ksi$

$$z := fs \cdot \sqrt[3]{2 \cdot d_c^{\,2} \cdot S}$$

$z = 7.418 \cdot \frac{kip}{in}$

check_z := "ok" if $115 \frac{kip}{in} \geq z$; "No Good" otherwise

check_z = "ok"

MAIN X Direction REINFORCEMENT(Negative Moment)

$M_{uneg_x} \cdot E = 13.1 \cdot kip \cdot ft$

$As_{reqd} := \frac{M_{uneg_x} \cdot E}{\phi_m \cdot j \cdot d \cdot f_y}$ $\quad As_{reqd} = 0.233 \cdot in^2$

Try1 := "#5@10" $\quad A_s := 0.413in^2$ $\quad d_b := 0.625in$ $\quad S := 9in$

$d_{act..xn} := h - cover - \frac{d_b}{2} - 1 \cdot in$ $\quad d_{act..xn} = 13.688 \cdot in$

$a := \frac{A_s \cdot f_y}{\beta_1 \cdot f'_c \cdot b}$ $\quad a = 0.54 \cdot in$

$\phi M_n := \phi_m \cdot A_s \cdot f_y \cdot \left(d_{act..xn} - \frac{a}{2} \right)$ $\quad \phi M_n = 24.937 \cdot kip \cdot ft$

$\rho_{act} := \frac{A_s}{b \cdot d_{act..xn}}$ $\quad \rho_{act} = 0.003$

check_Mcap := "ok" if $\phi M_n \geq M_{uneg_x} \cdot E$; "No Good" otherwise $\quad$ check_Mcap = "ok"

check_As := "ok" if $A_s \geq A_{smin_x} \vee \phi M_n \geq 1.333 \cdot M_{uneg_x} \cdot E$; "No Good" otherwise $\quad$ check_As = "ok"

check_ρ := "ok" if $\rho_{act} \leq \rho_{max}$; "No Good" otherwise $\quad$ check_ρ = "ok"

Crack Control:

$M := \frac{M_{uneg_x}}{1.7}$ $\quad M = 7.706 \cdot kip \cdot ft$

$fs := \frac{M}{A_s \cdot \left(d_{act..xn} - \frac{a}{2} \right)}$ $\quad fs = 16.687 \cdot ksi$

$z := fs \cdot \sqrt[3]{2 \cdot d_c^2 \cdot S}$ $\quad z = 97.179 \cdot \frac{kip}{in}$

check_z := "ok" if $115 \frac{kip}{in} \geq z$; "No Good" otherwise $\quad$ check_z = "ok"

MAIN Y Direction REINFORCEMENT(Positive Moment)

$M_{upos_y} \cdot E = 1.5 \cdot kip \cdot ft$

$As_{reqd} := \frac{M_{upos_y} \cdot E}{\phi_m \cdot j \cdot d \cdot f_y}$ $\quad As_{reqd} = 0.027 \cdot in^2$

Try1 := "#5@10" $\quad A_{sv} := 0.413in^2$ $\quad d_b := 0.625in$ $\quad S := 9 \cdot in$

$d_{act..yp} := h - cover - \frac{d_b}{2}$ $\quad d_{act..yp} = 14.688 \cdot in$

$a := \frac{A_s \cdot f_y}{\beta_1 \cdot f'_c \cdot b}$ $\quad a = 0.54 \cdot in$

$\phi M_n := \phi_m \cdot A_s \cdot f_y \cdot \left(d_{act..yp} - \frac{a}{2} \right)$ $\quad \phi M_n = 26.795 \cdot kip \cdot ft$

$\rho_{act} := \frac{A_s}{b \cdot d_{act..yp}}$ $\quad \rho_{act} = 0.002$

check_Mcap := "ok" if $\phi M_n \geq M_{upos_y} \cdot E$; "No Good" otherwise $\quad$ check_Mcap = "ok"

check_A_s := "ok" if $A_s \geq A_{smin_y} \vee \phi M_n \geq 1.333 \cdot M_{upos_y} \cdot E$; "No Good" otherwise $\quad$ check_A_s = "ok"

check_ρ := "ok" if $\rho_{act} \leq \rho_{max}$; "No Good" otherwise $\quad$ check_ρ = "ok"

Crack Control:

$M := \frac{M_{upos_y}}{1.7}$ $\quad M = 0.882 \cdot kip \cdot ft$

$fs := \frac{M}{A_s \cdot \left(d_{act..yp} - \frac{a}{2} \right)}$ $\quad fs = 1.778 \cdot ksi$

$z := fs \cdot \sqrt[3]{2 \cdot d_c^2 \cdot S}$ $\quad z = 10.356 \cdot \frac{kip}{in}$

check_z := "ok" if $115 \frac{kip}{in} \geq z$; "No Good" otherwise $\quad$ check_z = "ok"

MAIN Y Direction REINFORCEMENT(Negative Moment)

$M_{uneg_y} \cdot E = 1 \cdot kip \cdot ft$

$As_{reqd} := \dfrac{M_{uneg_y} \cdot E}{\phi_m \cdot j \cdot d \cdot f_y}$

$As_{reqd} = 0.018 \cdot in^2$

Try1 := "#5@10" $A_{sv} := 0.413in^2$ $d_b := 0.625in$ $S := 9in$

$d_{act..yn} := h - cover - \dfrac{d_b}{2}$

$d_{act..yn} = 14.688 \cdot in$

$a := \dfrac{A_s \cdot f_y}{\beta_1 \cdot f'_c \cdot b}$

$a = 0.54 \cdot in$

$\phi M_n := \phi_m \cdot A_s \cdot f_y \cdot \left(d_{act..yn} - \dfrac{a}{2} \right)$

$\phi M_n = 26.795 \cdot kip \cdot ft$

$\rho_{act} := \dfrac{A_s}{b \cdot d_{act..yn}}$

$\rho_{act} = 0.002$

$check_Mcap := \begin{cases} \text{"ok"} & \text{if } \phi M_n \geq M_{uneg_y} \cdot E \\ \text{"No Good"} & \text{otherwise} \end{cases}$

check_Mcap = "ok"

$check_A_s := \begin{cases} \text{"ok"} & \text{if } A_s \geq A_{smin_y} \vee \phi M_n \geq 1.333 \cdot M_{uneg_y} \cdot E \\ \text{"No Good"} & \text{otherwise} \end{cases}$

check_As = "ok"

$check_\rho := \begin{cases} \text{"ok"} & \text{if } \rho_{act} \leq \rho_{max} \\ \text{"No Good"} & \text{otherwise} \end{cases}$

check_ρ = "ok"

Crack Control:

$M := \dfrac{M_{uneg_y}}{1.7}$

$M = 0.588 \cdot kip \cdot ft$

$fs := \dfrac{M}{A_s \cdot \left(d_{act..yn} - \dfrac{a}{2} \right)}$

$fs = 1.185 \cdot ksi$

$z := fs \cdot \sqrt[3]{2 \cdot d_c^{\,2} \cdot S}$

$z = 6.904 \cdot \dfrac{kip}{in}$

$check_z := \begin{cases} \text{"ok"} & \text{if } 115 \dfrac{kip}{in} \geq z \\ \text{"No Good"} & \text{otherwise} \end{cases}$

check_z = "ok"

ONE WAY SHEAR CHECK:

X Direction Shear:

$V_{u_x} = 9.072 \cdot kip$

$\phi_v = 0.75$

$\phi V_{c_x} := \phi_v \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot \min\left(d_{act..xp}, d_{act..xn}\right)$ $\phi V_{c_x} = 16.527 \cdot kip$

$V_x := \begin{cases} \text{"OK"} & \text{if } V_{u_x} < \phi V_{c_x} \\ \text{"See ACI 11.10.9"} & \text{otherwise} \end{cases}$ $V_x = \text{"OK"}$

Y Direction Shear:

$V_{u_y} = 3.24 \cdot kip$

$\phi_v = 0.75$

$\phi V_{c_y} := \phi_v \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot \min\left(d_{act..yp}, d_{act..yn}\right)$ $\phi V_{c_y} = 17.735 \cdot kip$

$V_y := \begin{cases} \text{"OK"} & \text{if } V_{u_y} < \phi V_{c_y} \\ \text{"See ACI 11.10.9"} & \text{otherwise} \end{cases}$ $V_y = \text{"OK"}$

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d_{mreqd} is based on the K_n corresponding to $p_{preferred}$ for a given fc.

$$K_n := \begin{cases} 580psi & \text{if } f'_c = 4000psi \\ 435psi & \text{if } f'_c = 3000psi \\ 635psi & \text{if } f'_c = 4500psi \end{cases}$$

Stop Log Aluminum Post Design:

Stop log post design shall be carried out using Aluminum Design Manual (ADM) 2005. Design shall be carried out using ASD method utilizing 6061-T6 aluminum standard structural shapes.

Initial phase of the post design was carried out using AISC LRFD method utilizing steel material. However, the weight of the post assembly was much higher than the OSHA certified safe limit of lifting weight for a single person due to density of steel. The load analysis was performed using LRFD method. Since the aluminum design follows ASD, the moment and shear requirement calculated by LRFD method shall be used without modification which should be conservative. The following is the loading result from STAAD Pro for single post.

Reaction Summary

			Horizontal	Vertical	Horizontal	Moment		
	Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip ft)	MY (kip ft)	MZ (kip ft)
Max FX	1288	201:1.4D	**0.382**	0.821	0.000	0.000	0.000	0.033
Min FX	1333	201:1.4D	**-0.382**	0.821	0.000	0.000	0.000	-0.033
Max FY	2400	201:1.4D	-0.000	**2.184**	0.000	0.000	0.000	0.000
Min FY	1288	204:0.9DL+1.0I	0.246	**0.528**	0.559	0.894	-0.528	0.021
Max FZ	2400	203:1.4DL+1.4I	-0.000	2.184	**2.158**	3.627	-0.000	0.000
Min FZ	1288	201:1.4D	0.382	0.821	**0.000**	0.000	0.000	0.033
Max MX	2400	203:1.4DL+1.4I	-0.000	2.184	2.158	**3.627**	-0.000	0.000
Min MX	1288	201:1.4D	0.382	0.821	0.000	**0.000**	0.000	0.033
Max MY	1333	203:1.4DL+1.4I	-0.382	0.821	0.783	1.252	**0.739**	-0.033
Min MY	1288	203:1.4DL+1.4I	0.382	0.821	0.783	1.252	**-0.739**	0.033
Max MZ	1288	201:1.4D	0.382	0.821	0.000	0.000	0.000	**0.033**
Min MZ	1333	201:1.4D	-0.382	0.821	0.000	0.000	0.000	**-0.033**

Design moment, $M := 3.627 \cdot kip \cdot ft$

Design shear $V := 2.158 \cdot kip$

From ADM, Part VII, Design Aids, Table 2-22 (attached later), conservatively using the lesser of 3.4.2 (for the flanges) and 3.4.4 (for the web), Section 3.4.2, allowable tensile stress,

$F := 19ksi$

Required section modulus,

$$S_{required} := \frac{M}{F} = 2.291 \cdot in^3$$

From ADM Part VI, Table 8, using I 6X4.69 as the post section, provided section modulus is,

$S_{provided} := 8.50 \cdot in^3$ $> S_{required}$ OK

Applied bending stress,

$$f := \frac{M}{S_{provided}} = 5.12 \cdot ksi$$

Checking allowable flang stress:

Width	$b := 4 \cdot in$
Web thickness,	$t_w := 0.21 \cdot in$
Flange thickness,	$t_f := 0.35 \cdot in$
Depth,	$d := 6 \cdot in$

$$slenderness_ratio := \frac{b - t_w}{2 \cdot t_f} = 5.414$$

Since slenderness ratio is less than 6.5, as per ADM Part VII, Table 2-22, Section 3.4.15, allowable flange stress is 21 ksi, which is greater than 19 ksi assumed before. OK.

Checking web shear:

$$Slenderness := \frac{d - 2 \cdot t_f}{t_w} = 25.238$$

Since slenderness is less than 36, as per ADM Part VII, Table 2-22, Section 3.4.20, allowable web shear stress is,

$$F_v := 12 \cdot ksi$$

Web area required is,

$$A_{required} := \frac{V}{F_v} = 0.18 \cdot in^2$$

Web area provided is,

$$A_{provided} := \left(b - t_f\right) \cdot d = 21.9 \cdot in^2 \qquad > A_{required} \text{ OK.}$$

Applied shear stress,

$$v := \frac{V}{A_{provided}} = 0.099 \cdot ksi$$

Checking utilization:

$$\frac{f}{F} + \frac{v}{F_v} = 0.278 \qquad << 1.0 \text{ OK}$$

Calculating post weight:

Length of post, $L_{post} := 2 \cdot ft + 2 \cdot in$

Weight of post, $W_{post} := L_{post} \cdot 4.69 \cdot \frac{lb}{ft} = 10.162 \cdot lb$

Length of base plate, $L_{base_plate} := 14 \cdot in$

Width of base plate, $B_{base_plate} := 14 \cdot in$

Thickness of base plate, $t_{base_plate} := 0.5 \cdot in$

Unit of aluminum, $\gamma_{aluminum} := 0.1 \cdot \frac{lb}{in^3}$

Weight of base plate,

$$W_{base_plate} := L_{base_plate} \cdot B_{base_plate} \cdot t_{base_plate} \cdot \gamma_{aluminum} = 9.8 \cdot lb$$

Total weight of post assembly, including 15% allowance for welds, handles etc.,

$$W_{post_assembly} := 1.5 \cdot \left(W_{post} + W_{base_plate} \right) = 29.943 \cdot lb$$

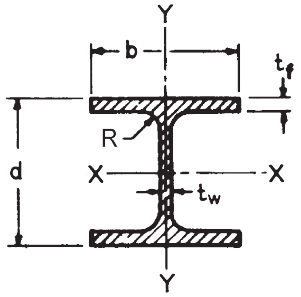


TABLE 8 – ALUMINUM ASSOCIATION STANDARD I-BEAMS

Designation	Depth d in.	Width b in.	Flange Thickness t_f in.	Web Thickness t_w in.	Fillet Radius R in.	Area A in^2	Axis x-x			Axis y-y					
							I_x in^4	S_x in^3	r_x in.	I_y in^4	S_y in^3	r_y in.	C_w in^6	J in^4	r_0 in.
I 3 × 1.64	3.000	2.500	0.200	0.130	0.250	1.39	2.24	1.49	1.27	0.522	0.418	0.613	1.02	0.0192	1.41
I 3 × 2.03	3.000	2.500	0.260	0.150	0.250	1.73	2.71	1.81	1.25	0.679	0.543	0.627	1.27	0.0374	1.40
I 4 × 2.31	4.000	3.000	0.230	0.150	0.250	1.96	5.62	2.81	1.69	1.04	0.691	0.727	3.68	0.0333	1.84
I 4 × 2.79	4.000	3.000	0.290	0.170	0.250	2.38	6.71	3.36	1.68	1.31	0.872	0.742	4.50	0.0608	1.84
I 5 × 3.70	5.000	3.500	0.320	0.190	0.300	3.15	13.9	5.58	2.11	2.29	1.31	0.853	12.5	0.0984	2.27
I 6 × 4.03	6.000	4.000	0.290	0.190	0.300	3.43	22.0	7.33	2.53	3.10	1.55	0.951	25.3	0.0888	2.71
I 6 × 4.69	6.000	4.000	0.350	0.210	0.300	3.99	25.5	8.50	2.53	3.74	1.87	0.968	29.8	0.145	2.71
I 7 × 5.80	7.000	4.500	0.380	0.230	0.300	4.93	42.9	12.3	2.95	5.78	2.57	1.08	63.3	0.206	3.14
I 8 × 6.18	8.000	5.000	0.350	0.230	0.300	5.26	59.7	14.9	3.37	7.30	2.92	1.18	107	0.188	3.57
I 8 × 7.02	8.000	5.000	0.410	0.250	0.300	5.97	67.8	16.9	3.37	8.55	3.42	1.20	123	0.286	3.57
I 9 × 8.36	9.000	5.500	0.440	0.270	0.300	7.11	102	22.7	3.79	12.2	4.44	1.31	224	0.386	4.01
I 10 × 8.65	10.000	6.000	0.410	0.250	0.400	7.35	132	26.4	4.24	14.8	4.93	1.42	340	0.360	4.47
I 10 × 10.3	10.000	6.000	0.500	0.290	0.400	8.75	156	31.2	4.22	18.0	6.01	1.44	407	0.620	4.46
I 12 × 11.7	12.000	7.000	0.470	0.290	0.400	9.92	256	42.6	5.07	26.9	7.69	1.65	894	0.621	5.33
I 12 × 14.3	12.000	7.000	0.620	0.310	0.400	12.2	317	52.9	5.11	35.5	10.1	1.71	1149	1.26	5.39
I 14 × 16.0[1]	14.000	8.000	0.600	0.300	0.400	14.2	489	69.9	6.00	51.2	12.8	1.94	2300	1.31	6.31

1. New shape; check availability with suppliers.
2. Tolerances for extruded shapes are given in *Aluminum Standards and Data*.

Table 2-22
ALLOWABLE STRESSES FOR BUILDING TYPE STRUCTURES

6061-T6 , -T6510, -T6511 Extrusions
6061-T6 Standard Structural Shapes, Pipe
6351-T5 Extrusions

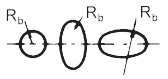
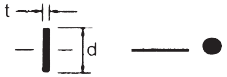
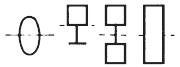
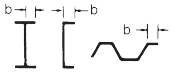
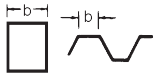
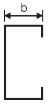
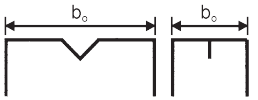
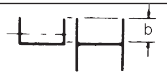
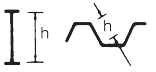
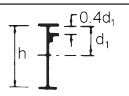
White bars apply to unwelded metal

Shaded bars apply to all thicknesses with fillers 5183, 5356, or 5556 and thicknesses ≤ 0.375 in. with fillers 4043, 5554, or 5654

For tubes with circumferential welds, Sections 3.4.10, 3.4.12, and 3.4.16.1 apply for $R_b / t < 20$

Type of Stress	Type of Member or Element		Sec. 3.4.	Allowable Stress	
TENSION, axial	Any tension member	gross section net section	1	21 19	12.5
TENSION IN BEAMS, extreme fiber, net section	Flat elements in uniform tension		2	19	9
	Round or oval tubes		3	24	10.5
	Flat elements in bending in their own plane, symmetric shapes		4	28	12
BEARING	On rivets and bolts		5	39	25
	On flat surfaces and pins and on bolts in slotted holes		6	26	16

Type of Stress	Type of Member or Element	Sec. 3.4.	Allowable Stress, $S \le S_1$	S_1	Allowable Stress, $S_1 < S < S_2$	S_2	Allowable Stress, $S \ge S_2$
COMPRESSION IN COLUMNS, axial	All columns	7	–	0	$20.2 - 0.126\ kL/r$	66	$51100\ /(kL/r)^2$
			–	0	$8.6 - 0.043\ kL/r$	133	$51100\ /(kL/r)^2$
COMPRESSION IN COLUMN ELEMENTS, gross section	Flat elements supported on one edge – columns buckling about a symmetry axis	8	21	2.4	$23.1 - 0.787\ b/t$	10	$154\ /(b/t)$
			9	3.9	$10.2 - 0.282\ b/t$	18	$92\ /(b/t)$
	Flat elements supported on one edge – columns not buckling about a symmetry axis	8.1	21	2.4	$23.1 - 0.787\ b/t$	12	$1970\ /(b/t)^2$
			9	3.9	$10.2 - 0.282\ b/t$	24	$1970\ /(b/t)^2$
	Flat elements supported on both edges	9	21	7.6	$23.1 - 0.247\ b/t$	33	$491\ /(b/t)$
			9	12	$10.2 - 0.089\ b/t$	58	$293\ /(b/t)$
	Flat elements supported on one edge and with stiffener on other edge	9.1	see Part IA Section 3.4.9.1				
	Flat elements supported on both edges and with an intermediate stiffener	9.2	see Part IA Section 3.4.9.2				
	Curved elements supported on both edges	10	21	1.4	$22.1 - 0.799\ \sqrt{R_b/t}$	141	$3190\ /\left(\frac{R_b}{t}\right)\left(1 + \frac{\sqrt{R_b/t}}{35}\right)^2$
			9	6.9	$10.0 - 0.335\ \sqrt{R_b/t}$	390	$3190\ /\left(\frac{R_b}{t}\right)\left(1 + \frac{\sqrt{R_b/t}}{35}\right)^2$

COMPRESSION IN BEAMS, extreme fiber, gross section	Single web shapes	11	21	21	$23.9 - 0.124\ L_b/r_y$	79	$87000\ /(L_b/r_y)^2$
			9	26	$10.2 - 0.043\ L_b/r_y$	160	$87000\ /(L_b/r_y)^2$
	Round or oval tubes 	12	25	29	$39.3 - 2.70\ \sqrt{R_b/t}$	81	Same as
			10.5	57	$17.7 - 0.932\ \sqrt{R_b/t}$	167	Section 3.4.10
	Solid rectangular and round sections 	13	28	14	$40.5 - 0.927\ \frac{d}{t}\sqrt{\frac{L_b}{d}}$	29	$11400\ /\left(\frac{d}{t}\right)^2\frac{L_b}{d}$
			12	18	$16.0 - 0.230\ \frac{d}{t}\sqrt{\frac{L_b}{d}}$	46	$11400\ /\left(\frac{d}{t}\right)^2\frac{L_b}{d}$
	Tubular shapes 	14	21	123	$23.9 - 0.238\ \sqrt{\frac{2L_bS_c}{\sqrt{I_yJ}}}$	1680	$23600\ /\frac{2L_bS_c}{\sqrt{I_yJ}}$
			9	186	$10.2 - 0.082\ \sqrt{\frac{2L_bS_c}{\sqrt{I_yJ}}}$	6940	$23600\ /\frac{2L_bS_c}{\sqrt{I_yJ}}$
COMPRESSION IN BEAM ELEMENTS, (element in uniform compression), gross section	Flat elements supported on one edge 	15	21	6.5	$27.3 - 0.930\ b/t$	10	$182\ /(b/t)$
			9	9	$12.0 - 0.334\ b/t$	18	$109\ /(b/t)$
	Flat elements supported on both edges 	16	21	21	$27.3 - 0.292\ b/t$	33	$580\ /(b/t)$
			9	28	$12.0 - 0.105\ b/t$	58	$346\ /(b/t)$
	Curved elements supported on both edges	16.1	25	2.1	$26.2 - 0.944\ \sqrt{R_b/t}$	141	$3780\ /\left(\frac{R_b}{t}\right)\left(1+\frac{\sqrt{R_b/t}}{35}\right)^2$
			10.5	8.4	$11.8 - 0.396\ \sqrt{R_b/t}$	390	$3780\ /\left(\frac{R_b}{t}\right)\left(1+\frac{\sqrt{R_b/t}}{35}\right)^2$
	Flat elements supported on one edge and with stiffener on other edge 	16.2	see Part IA Section 3.4.16.2				
	Flat elements supported on both edges and with an intermediate stiffener 	16.3	see Part IA Section 3.4.16.3				
COMPRESSION IN BEAM ELEMENTS, (element in bending in own plane), gross section	Flat elements supported on tension edge, compression edge free 	17	28	9.1	$40.5 - 1.41\ b/t$	19	$4930\ /(b/t)^2$
			12	12	$16.0 - 0.350\ b/t$	30	$4930\ /(b/t)^2$
	Flat elements supported on both edges 	18	28	48	$40.5 - 0.270\ h/t$	75	$1520\ /(h/t)$
			12	62	$16.0 - 0.067\ h/t$	119	$952\ /(h/t)$
	Flat elements supported on both edges and with a longitudinal stiffener 	19	28	110	$40.5 - 0.117\ h/t$	173	$3500\ /(h/t)$
			12	144	$16.0 - 0.029\ h/t$	280	$2200\ /(h/t)$
SHEAR IN ELEMENTS, gross section	Unstiffened flat elements supported on both edges	20	12	36	$15.8 - 0.101\ h/t$	64	$38700\ /(h/t)^2$
			5	48	$7.0 - 0.036\ h/t$	129	$38700\ /(h/t)^2$
	Stiffened flat elements supported on both edges	21	12	–	12	66	$53200\ /(a_e/t)^2$
			5	88	$9.6 - 0.050\ a_e/t$	129	$53200\ /(a_e/t)^2$

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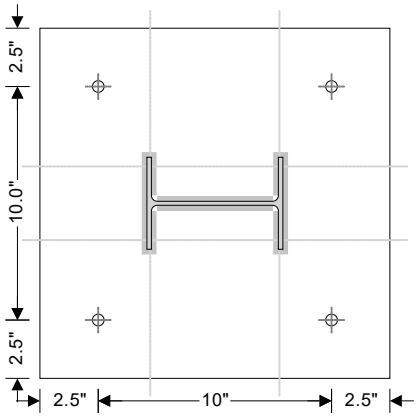
ALUMINUM POST BASE PLATE DESIGN

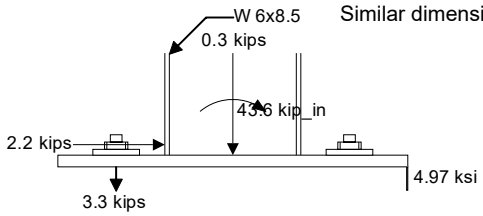
Column base plate design in accordance with AISC Steel Design Guide 1 and AISC 360-10

Tedds calculation version 2.1.02



Plan on baseplate

Bolt diameter - 0.5"
Bolt embedment - 4.0"
Flange/base weld - 0.3"
Web/base weld - 0.3"



Similar dimension to aluminum I6X4.69

Elevation on baseplate

Design forces and moments

Axial force	P_u = **0.3** kips (Compression)
Bending moment	M_u = **43.6** kip_in
Shear force	F_v = **2.2** kips
Eccentricity	e = ABS(M_u / P_u) = **145.200** in
Anchor bolt to center of plate	f = N/2 - e_1 = **5.000** in

Column details

Column section	W 6x8.5
Depth	d = **5.830** in
Breadth	b_f = **3.940** in
Flange thickness	t_f = **0.195** in
Web thickness	t_w = **0.170** in

Baseplate details

Depth	N = **15.000** in
Breadth	B = **15.000** in
Thickness	t_p = **0.500** in
Design strength	F_y = **35.0** ksi Yield strength for 6061-T6 Aluminum

Foundation geometry

Member thickness	h_a = **18.000** in
Dist center of baseplate to left edge foundation	x_{ce1} = **18.000** in
Dist center of baseplate to right edge foundation	x_{ce2} = **18.000** in
Dist center of baseplate to bot edge foundation	y_{ce1} = **18.000** in
Dist center of baseplate to top edge foundation	y_{ce2} = **18.000** in

Holding down bolt and anchor plate details

Total number of bolts	N_{bolt} = **4**
Bolt diameter	d_o = **0.500** in
Bolt spacing	s_{bolt} = **10.000** in

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Edge distance	e_1 = **2.500** in	
Minimum tensile strength, base plate	F_y = **35** ksi	Yield strength for 6061-T6 Aluminum
Minimum tensile strength, column	F_{yCol} = **35** ksi	Yield strength for 6061-T6 Aluminum
Compressive strength of concrete	f'_c = **4.5** ksi	

Strength reduction factors

Compression	ϕ_c = **0.65**
Flexure	ϕ_b = **0.90**
Weld shear	ϕ_v = **0.75**

Plate cantilever dimensions

Area of base plate	$A_1 = B \times N$ = **225.000** in^2
Maximum area of supporting surface	$A_2 = (N + 2 \times l_{min}) \times (B + 2 \times l_{min})$ = **1296.000** in^2
Nominal strength of concrete under base plate	$P_p = 0.85 \times f'_c \times A_1 \times \min(\sqrt{(A_2 / A_1)}, 2)$ = **1721.3** kips
Plate bending coefficient X	$X = (4 \times d \times b_f) / (d + b_f)^2 \times \min(1.0, P_u / (P_p \times \phi_c))$ = **0.00**
Plate bending coefficient λ	$\lambda_l = \min(1, 2 \times \sqrt{(X)} / (1 + \sqrt{(1 - X)}))$ = **0.02**
Bending line cantilever distance m	$m = (N - 0.95 \times d) / 2$ = **4.731** in
Bending line cantilever distance n	$n = (B - 0.8 \times b_f) / 2$ = **5.924** in
Yield line theory cantilever distance n'	$n' = \sqrt{(d \times b_f)} / 4$ = **1.198** in
Maximum bending line cantilever	$l = \max(m, n, \lambda_l \times n')$ = **5.924** in

Check eccentricity

Maximum bearing stress	$f_{p,max} = 0.85 \times f'_c \times \phi_c \times \min(\sqrt{(A_2 / A_1)}, 2)$ = **4.97** ksi
Maximum bearing pressure	$q_{max} = f_{p,max} \times B$ = **74.59** kips/in
Critical eccentricity	$e_{crit} = N / 2 - P_u / (2 \times q_{max})$ = **7.498** in

e > e_{crit} so loads cannot be resisted by bearing alone. Therefore consider as a large moment

Plate dimensions adequate as $(f + N/2)^2 >= (2 \times P_u \times (e + f))/q_{max}$ and a real solution for bearing length exists

Bearing length - quadratic solution 1	$Y_1 = (f + N/2) + \sqrt{((f + N/2)^2 - (2 \times P_u \times (e + f))/q_{max})}$ = **24.952** in
Bearing length - quadratic solution 2	$Y_2 = (f + N/2) - \sqrt{((f + N/2)^2 - (2 \times P_u \times (e + f))/q_{max})}$ = **0.048** in
Bearing length	$Y = \min(Y_1, Y_2)$ = **0.048** in
Tension force in bolts	$T_u = q_{max} \times Y - P_u$ = **3.3** kips
Max tension in single bolt	$T_{rod} = T_u / (N_{bolt}/2)$ = **1.7** kips

Base plate yielding limit at bearing interface

Required plate thickness	$t_{p,req} = \sqrt{((4 \times f_{p,max} \times Y \times (l - Y/2))/(\phi_b \times F_y))}$ = **0.425** in

PASS - Thickness of plate exceeds required thickness

Base plate yielding limit at tension interface

Distance from bolt CL to plate bending lines	$x = \text{abs}(m - e_1)$ = **2.231** in
Plate thickness required	$t_{p,req} = 2.11 \times \sqrt{((T_u \times x)/(B \times F_y))}$ = **0.250** in

PASS - Thickness of plate exceeds required thickness

Combined tension and bending in anchor bolts

Gross cross sectional area of anchor	$A_{bolt} = \pi \times d_a^2 / 4$ = **0.196** in^2
Thickness of washer	t_{wash} = **0.250** in
Shear stress in anchor	$f_v = F_v / (N_{boltV} \times A_{bolt})$ = **2.75** ksi
Lever arm	$z = t_p + t_{wash} / 2$ = **0.625** in
Bending in anchor	$M_l = F_v \times z / N_{boltV}$ = **0.03** kips_ft

Plastic modulus of anchor	$Z_{bolt} = d_a^3 / 6 =$ **0.021** in^3
Stress in anchor due to bending	$f_{tb} = M_l / Z_{bolt} =$ **16.2** ksi
Axial stress in anchor	$f_{ta} = T_{rod} / A_{bolt} =$ **8.4** ksi
Tensile stress in anchor	$f_t = f_{tb} + f_{ta} =$ **24.6** ksi
Nominal tensile stress of anchor (Table J3.2)	$F_{nt} = 0.75 \times f_{uta} =$ **43.5** ksi
Nominal shear stress of anchor (Table J3.2)	$F_{nv} = 0.45 \times f_{uta} =$ **26.1** ksi
Tensile stress of anchor	$\phi F_{nt} = \phi \times F_{nt} =$ **32.6** ksi
Modified tensile stress (Eqn J3-3a)	$\phi F'_{nt} = \min(\phi \times (1.3 \times F_{nt} - (F_{nt} / (\phi \times F_{nv})) \times f_v), \phi F_{nt}) =$ **32.6** ksi

PASS - Combined shear and bending resistance of rods exceed tensile stress in rods

Tension weld

Tension flange weld leg length	$t_{wf} =$ **0.3125** in
Effective flange weld width	$l_{Tweld,eff} = l_{Tweld,eff_ud} =$ **2** in
Tensile load per inch	$R_{wf} = T_{rod} / l_{Tweld,eff} =$ **0.8** kips/in
Electrode classification number	$F_{EXX} =$ **70.0** ksi
Design weld stress	$\phi F_{nw} = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5}) =$ **47.250**ksi
Design strength of weld per in	$\phi R_{nf} = \phi F_{nw} \times t_{wf} / \sqrt{(2)} =$ **10.4** kips/in

PASS - Available strength of weld exceeds force in tension weld

Local stress on flange	$f_{T,local} = (T_{rod}) / (l_{Tweld,eff} \times t_f) =$ **4.246** ksi
Column flange allowable stress	$F_{yCol} / 1.67 =$ **20.958** ksi

PASS - Local column capacity exceeds local column stress

Shear weld

Shear web weld leg length	$t_{ww} =$ **0.3125** in
Shear web weld force per in	$R_{wl} = F_v / (2 \times (d - 2 \times t_f)) =$ **0.199** kips/in
Electrode classification number	$F_{EXX} =$ **70.0** ksi
Design weld stress	$\phi F_{nw} = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(0deg))^{1.5}) =$ **31.500**ksi
Design strength of weld per in	$\phi R_{nl} = \phi F_{nw} \times t_{ww} / \sqrt{(2)} =$ **7.0** kips/in

PASS - Available strength of shear weld exceeds force in shear weld

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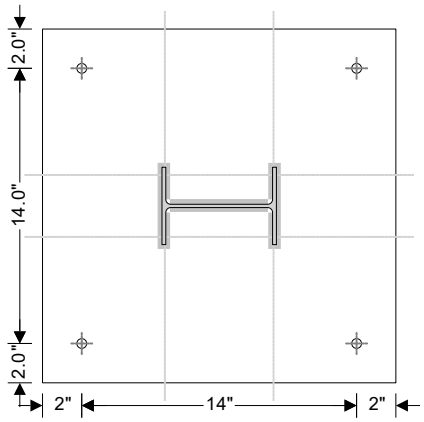
STAINLESS STEEL EMBED PLATE DESIGN

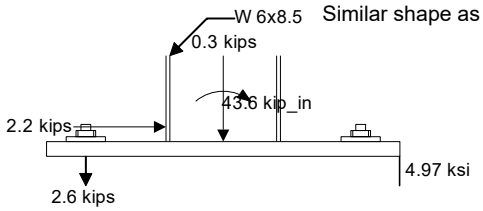
Column base plate design in accordance with AISC Steel Design Guide 1 and AISC 360-10

Tedds calculation version 2.1.02



Plan on baseplate

Bolt diameter - 0.5"
Bolt embedment - 6.0"
Flange/base weld - 0.3"
Web/base weld - 0.3"



Elevation on baseplate

Design forces and moments

Axial force	P_u = **0.3** kips (Compression)
Bending moment	M_u = **43.6** kip_in
Shear force	F_v = **2.2** kips
Eccentricity	e = ABS(M_u / P_u) = **145.200** in
Anchor bolt to center of plate	f = N/2 - e_1 = **7.000** in

Column details

Column section	W 6x8.5
Depth	d = **5.830** in
Breadth	b_f = **3.940** in
Flange thickness	t_f = **0.195** in
Web thickness	t_w = **0.170** in

Baseplate details

Depth	N = **18.000** in	
Breadth	B = **18.000** in	
Thickness	t_p = **0.750** in	
Design strength	F_y = **42.0** ksi	Stainless Steel

Foundation geometry

Member thickness	h_a = **18.000** in
Dist center of baseplate to left edge foundation	x_{ce1} = **18.000** in
Dist center of baseplate to right edge foundation	x_{ce2} = **18.000** in
Dist center of baseplate to bot edge foundation	y_{ce1} = **18.000** in
Dist center of baseplate to top edge foundation	y_{ce2} = **18.000** in

Holding down bolt and anchor plate details

Total number of bolts	N_{bolt} = **4**
Bolt diameter	d_o = **0.500** in
Bolt spacing	s_{bolt} = **14.000** in

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Edge distance	e_1 = **2.000** in	
Minimum tensile strength, base plate	F_y = **42** ksi	Stainless Steel
Minimum tensile strength, column	F_{yCol} = **35** ksi	6061-T6 Aluminum
Compressive strength of concrete	f'_c = **4.5** ksi	

Strength reduction factors

Compression	ϕ_c = **0.65**
Flexure	ϕ_b = **0.90**
Weld shear	ϕ_v = **0.75**

Plate cantilever dimensions

Area of base plate	$A_1 = B \times N$ = **324.000** in^2
Maximum area of supporting surface	$A_2 = (N + 2 \times l_{min}) \times (B + 2 \times l_{min})$ = **1296.000** in^2
Nominal strength of concrete under base plate	$P_p = 0.85 \times f'_c \times A_1 \times \min(\sqrt{(A_2 / A_1)}, 2)$ = **2478.6** kips
Plate bending coefficient X	$X = (4 \times d \times b_f) / (d + b_f)^2 \times \min(1.0, P_u / (P_p \times \phi_c))$ = **0.00**
Plate bending coefficient λ	$\lambda_l = \min(1, 2 \times \sqrt{(X)} / (1 + \sqrt{(1 - X)}))$ = **0.01**
Bending line cantilever distance m	$m = (N - 0.95 \times d) / 2$ = **6.231** in
Bending line cantilever distance n	$n = (B - 0.8 \times b_f) / 2$ = **7.424** in
Yield line theory cantilever distance n'	$n' = \sqrt{(d \times b_f)} / 4$ = **1.198** in
Maximum bending line cantilever	$l = \max(m, n, \lambda_l \times n')$ = **7.424** in

Check eccentricity

Maximum bearing stress	$f_{p,max} = 0.85 \times f'_c \times \phi_c \times \min(\sqrt{(A_2 / A_1)}, 2)$ = **4.97** ksi
Maximum bearing pressure	$q_{max} = f_{p,max} \times B$ = **89.51** kips/in
Critical eccentricity	$e_{crit} = N / 2 - P_u / (2 \times q_{max})$ = **8.998** in

e > e_{crit} so loads cannot be resisted by bearing alone. Therefore consider as a large moment

Plate dimensions adequate as $(f + N/2)^2 >= (2 \times P_u \times (e + f))/q_{max}$ and a real solution for bearing length exists

Bearing length - quadratic solution 1	$Y_1 = (f + N/2) + \sqrt{((f + N/2)^2 - (2 \times P_u \times (e + f))/q_{max})}$ = **31.968** in
Bearing length - quadratic solution 2	$Y_2 = (f + N/2) - \sqrt{((f + N/2)^2 - (2 \times P_u \times (e + f))/q_{max})}$ = **0.032** in
Bearing length	$Y = \min(Y_1, Y_2)$ = **0.032** in
Tension force in bolts	$T_u = q_{max} \times Y - P_u$ = **2.6** kips
Max tension in single bolt	$T_{rod} = T_u / (N_{bolt}/2)$ = **1.3** kips

Base plate yielding limit at bearing interface

Required plate thickness	$t_{p,req} = \sqrt{((4 \times f_{p,max} \times Y \times (l - Y/2))/(\phi_b \times F_y))}$ = **0.353** in

PASS - Thickness of plate exceeds required thickness

Base plate yielding limit at tension interface

Distance from bolt CL to plate bending lines	$x = \text{abs}(m - e_1)$ = **4.231** in
Plate thickness required	$t_{p,req} = 2.11 \times \sqrt{((T_u \times x)/(B \times F_y))}$ = **0.252** in

PASS - Thickness of plate exceeds required thickness

Combined tension and bending in anchor bolts

Gross cross sectional area of anchor	$A_{bolt} = \pi \times d_a^2 / 4$ = **0.196** in^2
Thickness of washer	t_{wash} = **0.250** in
Shear stress in anchor	$f_v = F_v / (N_{boltV} \times A_{bolt})$ = **2.75** ksi
Lever arm	$z = t_p + t_{wash} / 2$ = **0.875** in
Bending in anchor	$M_l = F_v \times z / N_{boltV}$ = **0.04** kips_ft

Plastic modulus of anchor	$Z_{bolt} = d_a^3 / 6 =$ **0.021** in^3
Stress in anchor due to bending	$f_{tb} = M_l / Z_{bolt} =$ **22.7** ksi
Axial stress in anchor	$f_{ta} = T_{rod} / A_{bolt} =$ **6.5** ksi
Tensile stress in anchor	$f_t = f_{tb} + f_{ta} =$ **29.2** ksi
Nominal tensile stress of anchor (Table J3.2)	$F_{nt} = 0.75 \times f_{uta} =$ **43.5** ksi
Nominal shear stress of anchor (Table J3.2)	$F_{nv} = 0.45 \times f_{uta} =$ **26.1** ksi
Tensile stress of anchor	$\phi F_{nt} = \phi \times F_{nt} =$ **32.6** ksi
Modified tensile stress (Eqn J3-3a)	$\phi F'_{nt} = \min(\phi \times (1.3 \times F_{nt} - (F_{nt} / (\phi \times F_{nv})) \times f_v), \phi F_{nt}) =$ **32.6** ksi

PASS - Combined shear and bending resistance of rods exceed tensile stress in rods

Tension weld

Tension flange weld leg length	$t_{wf} =$ **0.3125** in
Effective flange weld width	$l_{Tweld,eff} = l_{Tweld,eff_ud} =$ **2** in
Tensile load per inch	$R_{wf} = T_{rod} / l_{Tweld,eff} =$ **0.6** kips/in
Electrode classification number	$F_{EXX} =$ **70.0** ksi
Design weld stress	$\phi F_{nw} = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5}) =$ **47.250**ksi
Design strength of weld per in	$\phi R_{nf} = \phi F_{nw} \times t_{wf} / \sqrt{(2)} =$ **10.4** kips/in

PASS - Available strength of weld exceeds force in tension weld

Local stress on flange	$f_{T,local} = (T_{rod}) / (l_{Tweld,eff} \times t_f) =$ **3.278** ksi
Column flange allowable stress	$F_{yCol} / 1.67 =$ **20.958** ksi

PASS - Local column capacity exceeds local column stress

Shear weld

Shear web weld leg length	$t_{ww} =$ **0.3125** in
Shear web weld force per in	$R_{wl} = F_v / (2 \times (d - 2 \times t_f)) =$ **0.199** kips/in
Electrode classification number	$F_{EXX} =$ **70.0** ksi
Design weld stress	$\phi F_{nw} = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(0deg))^{1.5}) =$ **31.500**ksi
Design strength of weld per in	$\phi R_{nl} = \phi F_{nw} \times t_{ww} / \sqrt{(2)} =$ **7.0** kips/in

PASS - Available strength of shear weld exceeds force in shear weld

STAINLESS STEEL EMBED PLATE DESIGN

Anchor bolt design in accordance with ACI318-05

Tedds calculation version 2.1.02

Anchor bolt geometry

Anchor bolt	Cast-in headed stud anchor
Diameter of anchor bolt	$d_a =$ **0.5** in
Number of bolts in x direction	$N_{boltx} =$ **2**
Number of bolts in y direction	$N_{bolty} =$ **2**
Total number of bolts	$n_{total} = (N_{boltx} \times 2) + (N_{bolty} - 2) \times 2 =$ **4**
Total number of bolts in tension	$n_{tens} = (N_{boltN} \times 2) + (N_{bolty} - 2) =$ **2**
Spacing of bolts in x direction	$s_{boltx} =$ **14** in
Spacing of bolts in y direction	$s_{bolty} =$ **14** in
Effective cross-sectional area of anchor	$A_{se} = \pi \times d_a^2 / 4 =$ **0.196** in^2
Embedded depth of each anchor bolt	$h_{ef} =$ **6** in

Material details

Minimum yield strength of steel	$f_{ya} =$ **36** ksi

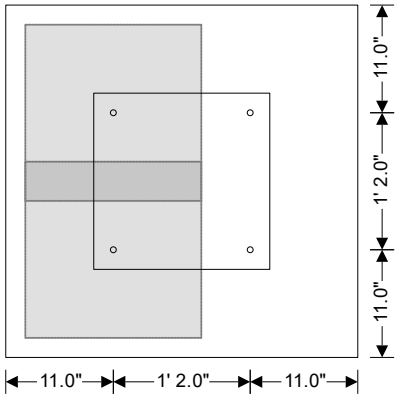
Nominal tensile strength of steel	f_{uta} = **58** ksi
Compressive strength of concrete	f'_c = **4.5** ksi
Strength reduction factors	
Tension of steel element	$\phi_{t,s}$ = **0.75**
Shear of steel element	$\phi_{v,s}$ = **0.65**
Concrete tension	$\phi_{t,c}$ = **0.65**
Concrete shear	$\phi_{v,c}$ = **0.70**
Concrete tension for pullout	$\phi_{t,cB}$ = **0.70**
Concrete shear for pryout	$\phi_{v,cB}$ = **0.70**

Steel strength of anchor in tension (D.5.1)

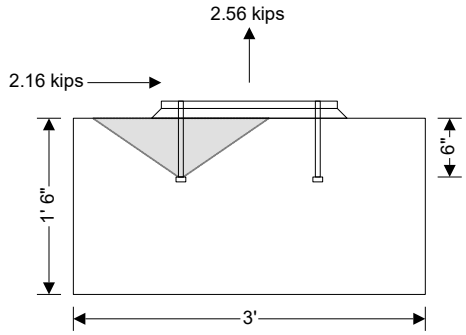
Nominal strength of anchor in tension	$N_{sa} = A_{se} \times f_{uta}$ = **11.39** kips
Steel strength of anchor in tension	$\phi N_{sa} = \phi_{t,s} \times N_{sa}$ = **8.54** kips

PASS - Steel strength of anchor exceeds max tension in single bolt

Check concrete breakout strength of anchor bolt in tension (D.5.2)



Plan on foundation



Section A-A

Concrete breakout - tension

Coeff for basic breakout strength in tension	k_c = **24**
Breakout strength for single anchor in tension	$N_b = k_c \times \sqrt{(f'_c \times 1\ psi)} \times h_{ef}^{1.5} \times 1\ in^{0.5}$ = **23.66** kips
Projected area for groups of anchors	A_{Nc} = **576** in^2
Projected area of a single anchor	$A_{Nco} = 9 \times h_{ef}^2$ = **324** in^2
Min dist center of anchor to edge of concrete	$c_{a,min}$ = **11** in
Mod factor for groups loaded eccentrically	$\psi_{ec,N} = \min(1 / (1 + ((2 \times e'_N) / (3 \times h_{ef}))), 1)$ = **1.000**
Modification factor for edge effects	$\psi_{ed,N}$ = 1.0 = **1.000**
Modification factor for no cracking at service loads	$\psi_{c,N}$ = **1.000**
Modification factor for cracked concrete	$\psi_{cp,N}$ = **1.000**
Nominal concrete breakout strength	$N_{cbg} = A_{Nc} / A_{Nco} \times \psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N} \times N_b$ = **42.07** kips
Concrete breakout strength	$\phi N_{cbg} = \phi_{t,c} \times N_{cbg}$ = **27.34** kips

PASS - Breakout strength exceeds tension in bolts

Pullout strength (D.5.3)

Net bearing area of the head of anchor	A_{brg} = **1** in^2
Mod factor for no cracking at service loads	$\psi_{c,P}$ = **1.000**
Pullout strength for single anchor	$N_p = 8 \times A_{brg} \times f'_c$ = **36.00** kips

Nominal pullout strength of single anchor $N_{pn} = \psi_{c,P} \times N_p =$ **36.00** kips

Pullout strength of single anchor $\phi N_{pn} = \phi_{t,cB} \times N_{pn} =$ **25.20** kips

PASS - Pullout strength of single anchor exceeds maximum axial force in single bolt

Side face blowout strength (D.5.4)

As h_{ef} <= 2.5 × min(c_{a1}, c_{a2}) the edge distance is considered to be far from an edge and blowout strength need not be considered

Steel strength of anchor in shear (D.6.1)

Built-up grout pads are used so nominal strength will be multiplied by 0.8 (D.6.1.3)

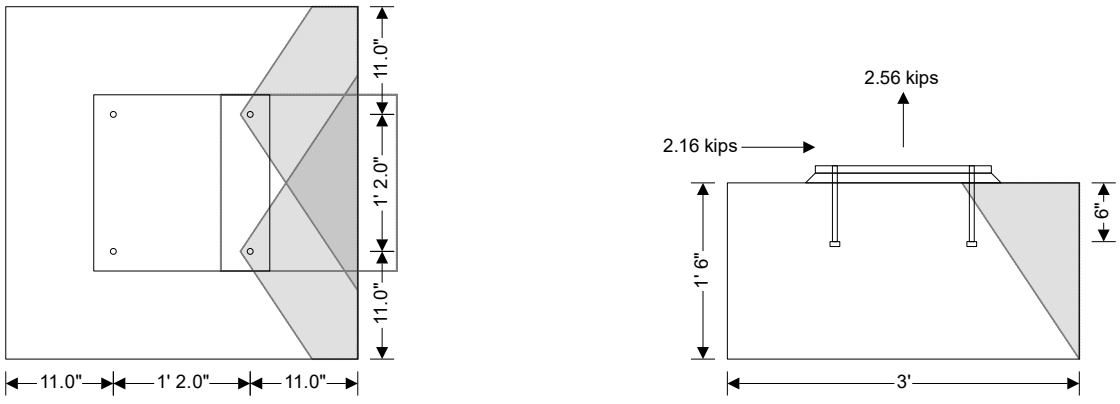
Effective number of anchors in shear $N_{boltV} =$ **4**

Nom strength of anchor in shear $V_{sa} = 0.8 \times N_{boltV} \times A_{se} \times f_{uta} =$ **36.44** kips

Steel strength of anchor in shear $\phi V_{sa} = \phi_{v,s} \times V_{sa} =$ **23.69** kips

PASS - Steel strength of anchor exceeds shear in bolts

Concrete breakout strength in shear perpendicular to edge - Case 2. All shear resisted by rear bolts (D.6.3)



Plan on foundation

Section A-A

Concrete breakout - shear

The anchors are influenced by three or more edges where any edge distance is less than 1.5c_{a1} so value of c_{a1} is limited to c'_{a1} (D.6.2.4).

Bolt offset for limiting shear $x_{V,r} =$ **13.00** in

Limiting edge distance $c'_{a1} =$ **12** in

Applied shear $V_{app} = V =$ **2.16** kips

Edge distance x for shear near corner $c_{a1} =$ **25** in

Edge distance y for shear near corner $c_{a2} = \min(y_{ce1}, y_{ce2}) - (((N_{bolty} - 1)/2) \times s_{bolty}) =$ **11** in

Load bearing length of anchor $l_e = \min(h_{ef}, 8 \times d_a) =$ **4** in

Basic concrete breakout strength $V_b = 7 \times (l_e / d_a)^{0.2} \times \sqrt{(d_a)} \times \sqrt{(f'_c \times 1psi)} \times (c'_{a1})^{1.5} =$ **20.92** kips

Projected area of a single anchor $A_{Vco} = 4.5 \times c'^2_{a1} =$ **648** in^2

Projected area of a group of anchors $A_{Vc} =$ **648** in^2

Mod factor for edge effect $\psi_{ed,V} = 0.7 + 0.3 \times c_{a2} / (1.5 \times c'_{a1}) =$ **0.883**

Eccentricity of loading $e'_V =$ **0** in

Modification factor of eccentric loading $\psi_{ec,V} = \min(1, 1 / (1 + ((2 \times e'_V) / (3 \times c'_{a1})))) =$ **1.000**

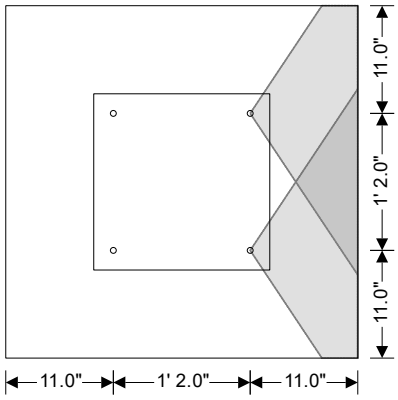
Modification factor for cracking $\psi_{c,V} =$ **1.000**

Nominal concrete break out strength in shear $V_{cbg} = A_{Vc} / A_{Vco} \times \psi_{ec,V} \times \psi_{ed,V} \times \psi_{c,V} \times V_b =$ **18.48** kips

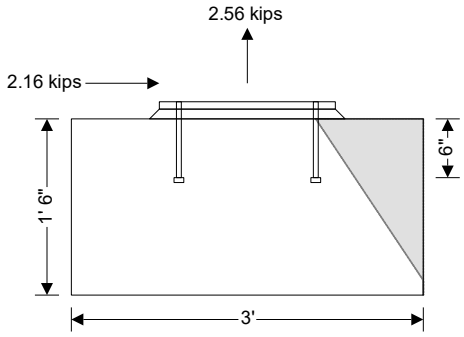
Concrete break out strength in shear	$\phi V_{cbg} = \phi_{v,c} \times V_{cbg} =$ **12.94** kips

PASS - Shear breakout perpendicular to edge strength exceeds shear in bolts

Concrete breakout strength in shear - Case 1. Half of shear resisted by front bolts (D.6.3)



Plan on foundation



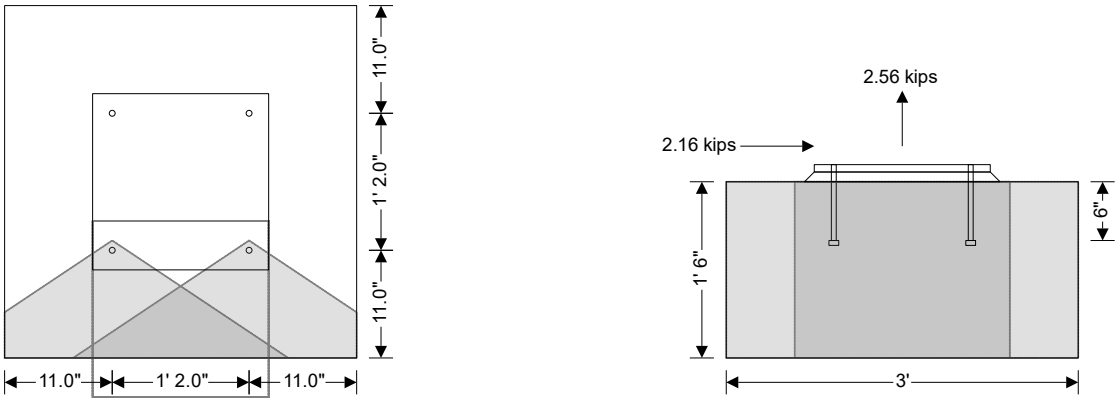
Section A-A

Concrete breakout - shear

Applied shear	$V_{app} = V / 2 =$ **1.08** kips
Edge distance x for shear near corner	$c_{a1} =$ **11** in
Edge distance y for shear near corner	$c_{a2} = \min(y_{ce1}, y_{ce2}) - (((N_{bolty} - 1)/2) \times s_{bolty}) =$ **11** in
Load bearing length of anchor	$l_e = \min(h_{ef}, 8 \times d_a) =$ **4** in
Basic concrete breakout strength	$V_b = 7 \times (l_e / d_a)^{0.2} \times \sqrt{(d_a)} \times \sqrt{(f'_c \times 1psi)} \times (c_{a1})^{1.5} =$ **18.36** kips
Projected area of a single anchor	$A_{Vco} = 4.5 \times c_{a1}^2 =$ **544.5** in^2
Projected area of a group of anchors	$A_{Vc} =$ **594** in^2
Mod factor for edge effect	$\psi_{ed,V} = 0.7 + 0.3 \times c_{a2} / (1.5 \times c_{a1}) =$ **0.900**
Eccentricity of loading	$e'_V =$ **0** in
Modification factor of eccentric loading	$\psi_{ec,V} = \min(1, 1 / (1 + ((2 \times e'_V) / (3 \times c_{a1})))) =$ **1.000**
Modification factor for cracking	$\psi_{c,V} =$ **1.000**
Nominal concrete break out strength in shear	$V_{cbg} = A_{Vc} / A_{Vco} \times \psi_{ec,V} \times \psi_{ed,V} \times \psi_{c,V} \times V_b =$ **18.03** kips
Concrete break out strength in shear	$\phi V_{cbg} = \phi_{v,c} \times V_{cbg} =$ **12.62** kips

PASS - Shear breakout perpendicular to edge strength exceeds shear in bolts

Concrete breakout strength in shear parallel to edge - Case 2. All shear resisted by rear bolts (parallel to edge - Case 2. All shear resisted by rear bolts)



Concrete breakout - side shear

The anchors are influenced by three or more edges where any edge distance is less than $1.5c_{a1,p}$ so value of $c_{a1,p}$ is limited to $c'_{a1,p}$

Bolt offset for limiting shear	$y_{V,r,p}$ = **13.00** in
Limiting edge distance	$c'_{a1,p}$ = **12** in
Applied shear	V_{app} = V = **2.16** kips
Edge distance x for shear near corner	$c_{a1,p}$ = **25** in
Edge distance y for shear near corner	$c_{a2,p} = \min(x_{ce1}, x_{ce2}) - (((N_{boltx} - 1)/2) \times s_{boltx})$ = **11** in
Load bearing length of anchor	$l_e = \min(h_{ef}, 8 \times d_a)$ = **4** in
Basic concrete breakout strength	$V_{b,p} = 7 \times (l_e / d_a)^{0.2} \times \sqrt{(d_a)} \times \sqrt{(f'_c \times 1psi)} \times (c'_{a1,p})^{1.5}$ = **20.92** kips
Projected area of a single anchor	$A_{Vco,p} = 4.5 \times c'^2_{a1,p}$ = **648** in^2
Projected area of a group of anchors	$A_{Vc,p}$ = **648** in^2
Mod factor for edge effect	$\psi_{ed,V,p}$ = **1.000**
Eccentricity of loading	$e'_{V,p}$ = **0** in
Modification factor of eccentric loading	$\psi_{ec,V,p} = \min(1, 1 / (1 + ((2 \times e'_{V,p}) / (3 \times c'_{a1,p}))))$ = **1.000**
Modification factor for cracking	$\psi_{c,V}$ = **1.000**
Modification factor for edge distance	$\psi_{h,V,p}$ = 1.0 = **1.000**
Nominal concrete break out strength in shear	$V_{cbg,p} = 2 \times A_{Vc,p} / A_{Vco,p} \times \psi_{ec,V,p} \times \psi_{ed,V,p} \times \psi_{c,V} \times V_{b,p}$ = **41.84** kips
Concrete break out strength in shear	$\phi V_{cbg,p} = \phi_{v,c} \times V_{cbg,p}$ = **29.29** kips

PASS - Shear breakout strength parallel to edge exceeds shear in bolts

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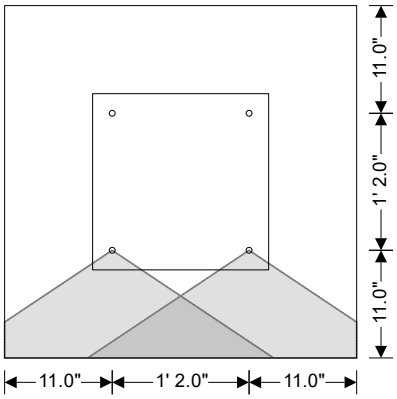
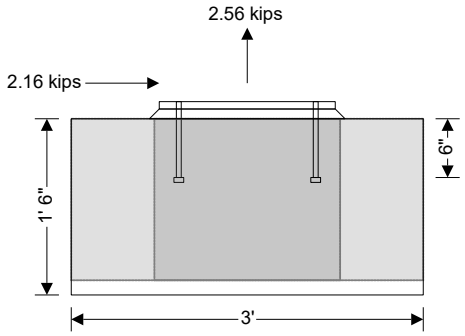
	Section: Stop Log Design				Sheet no./rev.: 8	
	Calc. by: FN	Date: 2/14/2021	Chk'd by: MBE	Date: 2/15/2021	App'd by	Date

Concrete breakout strength in shear parallel to edge - Case 1. Half of shear resisted by front bolts (parallel to edge - Case 1. Half of shear resisted by front bolts)



Plan on foundation



Section A-A

Concrete breakout - side shear

Applied shear	$V_{app} = V / 2 =$ **1.08** kips
Edge distance x for shear near corner	$c_{a1,p} =$ **11** in
Edge distance y for shear near corner	$c_{a2,p} = \min(x_{ce1}, x_{ce2}) - (((N_{boltx} - 1)/2) \times s_{boltx}) =$ **11** in
Load bearing length of anchor	$l_e = \min(h_{ef}, 8 \times d_a) =$ **4** in
Basic concrete breakout strength	$V_{b,p} = 7 \times (l_e / d_a)^{0.2} \times \sqrt{(d_a)} \times \sqrt{(f'_c \times 1psi)} \times (c_{a1,p})^{1.5} =$ **18.36** kips
Projected area of a single anchor	$A_{Vco,p} = 4.5 \times c_{a1,p}^2 =$ **544.5** in^2
Projected area of a group of anchors	$A_{Vc,p} =$ **594** in^2
Mod factor for edge effect	$\psi_{ed,V,p} =$ **1.000**
Eccentricity of loading	$e'_{V,p} =$ **0** in
Modification factor of eccentric loading	$\psi_{ec,V,p} = \min(1, 1 / (1 + ((2 \times e'_{V,p}) / (3 \times c_{a1,p})))) =$ **1.000**
Modification factor for cracking	$\psi_{c,V} =$ **1.000**
Modification factor for edge distance	$\psi_{h,V,p} = 1.0 =$ **1.000**
Nominal concrete break out strength in shear	$V_{cbg,p} = 2 \times A_{Vc,p} / A_{Vco,p} \times \psi_{ec,V,p} \times \psi_{ed,V,p} \times \psi_{c,V} \times V_{b,p} =$ **40.06** kips
Concrete break out strength in shear	$\phi V_{cbg,p} = \phi_{v,c} \times V_{cbg,p} =$ **28.04** kips

PASS - Shear breakout strength parallel to edge exceeds shear in bolts

Pryout strength of anchor in shear (D.6.3)

Coefficient of pryout strength	$k_{cp} =$ **2.0**
Nominal pryout strength of anchor in shear	$V_{cpg} = k_{cp} \times N_{cbg} =$ **84.13** kips
Pryout strength of anchor in shear	$\phi V_{cpg} = \phi_{v,cB} \times V_{cpg} =$ **58.89** kips

PASS - Pryout strength of anchor exceeds shear in bolts

Interaction of tensile and shear forces

Critical design strength in tension	$\phi N_n = \phi N_{sa} =$ **8.54** kips
Critical applied tensile force	$N_{ua} = N_{max,s} =$ **1.28** kips
	$N_{ua} / \phi N_n =$ **0.15**
Critical design strength in shear	$\phi V_n = \phi V_{cbg,r} =$ **12.94** kips
Critical applied shear force	$V_{ua} = abs(V) =$ **2.16** kips
	$V_{ua} / \phi V_n =$ **0.167**

Full strength in both shear and tension are permitted

PASS - Applied tension and shear are less than tension and shear capacities

Stop Log Storage Container Volume Calculation:

The longest plank size is 10'-7". The height of each plank is assumed 6 inches and depth of plank is 4 inches. For storage and handling purposes we shall add 2 more inches on eaither side of each dimensions.

Volume of each plank

$$Vol_{plank} := (10\cdot ft + 7in + 2\cdot 2\cdot in)(6\cdot in + 2\cdot 2\cdot in)\cdot(4\cdot in + 2\cdot 2\cdot in) = 6.065\cdot ft^3$$

There are 5 bays with 4 planks in each bay. Total plank volume,

$$V_{total_plank} := Vol_{plank}\cdot 4\cdot 5 = 121.296\cdot ft^3$$

Assuming the box for plank is 12 ft. long and 4 ft deep and 5 ft tall, this will allow 5 layers of 4 planks to be stacked. There will be additional space at the bottom for a cart type system with casters that can be rolled out.

Volume of the proposed box,

$$V_{storage_plank} := 12\cdot ft\cdot 4\cdot ft\cdot 5\cdot ft = 240\cdot ft^3$$

The removavble post has a base plate which is 14 inches square. The height of each post is slightly taller than 2 ft. 2 in. We shall also add 2 inches on either side of each dimension for handling purposes.

Volume of each post,

$$V_{post} := (14\cdot in + 2\cdot 2\cdot in)\cdot(14\cdot in + 2\cdot 2\cdot in)\cdot(2\cdot ft + 2\cdot in + 2\cdot 2\cdot in) = 5.625\cdot ft^3$$

There are 4 posts for each stop log alignment. Total volume of all posts,

$$V_{total_posts} := 4\cdot V_{post} = 22.5\cdot ft^3$$

There are 4 posts for each stop log alignment. Posts shall be stored in seperate box. We shall assume that the post storage box is 4 ft. long, 2 ft. deep and 5 ft. high. This will allow storage of two layers of posts vertically with each layer having two posts.

Volume of the proposed storage box for posts,

$$v_{storage_post} := 4\cdot ft\cdot 2\cdot ft\cdot 5\cdot ft = 40\cdot ft^3$$

Stoplog - Roadway Gap Calculation:

Width of clear roadway is 50 ft. 4 in. As per the drawings there shall be five equal spans of stoplogs. The calculation for gap between bottom most stoplog and roadway shall be calculated assuming that four of the stoplog spans are 10 ft. in length and the fifth span is 10ft. 4in. wide.

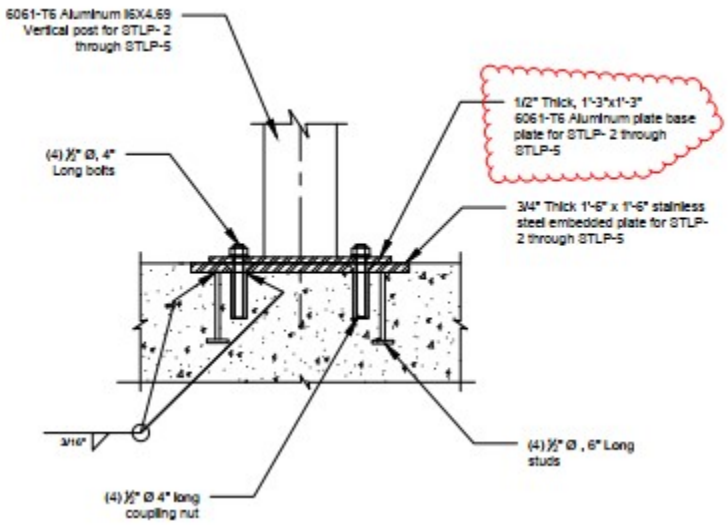
Length of stoplog span,

$$L_{stoplog} := 10 \cdot ft + 4 \cdot in$$

Slope of roadway, $Slope_{roadway} := \frac{1}{100} = 1\ \%$

Gap due to roadway slope along the stoplog span,

$$Gap_{roadway_slope} := L_{stoplog} \cdot Slope_{roadway} = 1.24\ in$$



Gap due to height of base plate at the bottom of the removable post,

$$Gap_{base_plate} := 0.5 \cdot in$$

Maximum total gap in between the bottom most stoplog and the roadway is,

$$Gap_{total} := Gap_{roadway_slope} + Gap_{base_plate} = 1.74\ in$$

Appendix C.4: Wall Raise

The purpose of this calculation is to check the stability of existing floodwall at the intersection of proposed East Levee Stoplog structure and the existing wall for a 2-ft rise in wall height from existing height for usual flood (no flood) condition.

Following assumptions were made for this design calculations:

Loading Condition: Usual- No Flood Condition:

- Average Grade EL. 3755.0 feet on River Side (R/S) and EL. 3756.0 on Land Side (L/S.)
- Top of wall EL. 3758.0. (Approx. 3 ft free board above grade level-R/S)
- The following stability criteria is followed to check the stability of the floodwall:

  1) Location of Resultant: Within the base. (100% base in compression)
  2) Minimum Sliding F.S.: 1.5
  3) Minimum Floatation F.S.: 1.3

REFERENCES

USACE EM 1110-2-2100 - Stabilitry Analysis Of Concrete Structure.

USACE EM 1110 -2 -2502- Retaining and Flood Walls.

USACE - Hurricane and Storm Damage Risk Reduction System Design Guidelines, Rev.June 2012.

ASCE -7-10 Minimum Design Loads for Buildings and Other Structures.

Design Parameters:

Soil Parameters:

Density of water: $\gamma_w := 64.0pcf$

Unit Weight of Saturated Soil: $\gamma_s := 120pcf$

Unit weight of buoyant soil: $\gamma_{s.buoy.} := \gamma_s - \gamma_w = 56 \cdot pcf$

Unit Weight of Reinforced Concrete: $\gamma_c := 150 \cdot pcf$

Angle of Internal Friction: $\phi := 18.0deg$

Cohesion of Soil $C := 100psf$

Allowable Bearing Capacity of Soil(assumed): $\sigma := 1000psf$

Coefficient of Friction with Concrete: $f := \tan(\phi) = 0.325$ (From Geotechnical data)

T - Wall Design Parameters.

Top of Flood Wall EL.

$EL_{TOW} := 3758.0ft$

Grade Elevation Riverside R/S (avg):

$EL_{grade.Rs} := 3755.0ft$

Grade ELevation Landside L/S (avg):

$EL_{grade.Ls} := 3756.0ft$

Elevation of Top of Footing:

$EL_{top.base} := 3754.0ft$

Elevation of the Bottom of the Footing:

$EL_{bot.base} := 3753.0ft$

Height of Soil from bottom of footing to exposed grade R/S:

$H_{soil.Rs} := EL_{grade.Rs} - EL_{bot.base} = 2 \cdot ft$

Height of Soil from bottom of footing to the exposed grade L/S:

$H_{soil.Ls} := EL_{grade.Ls} - EL_{bot.base} = 3 \cdot ft$

Height of water above Base R/S:

$H_{water.Rs.} := \left(EL_{bot.base} - EL_{bot.base}\right) = 0 \cdot ft$

Height of soil above Base R/S:

$H_{soil.Rs.base} := EL_{grade.Rs} - EL_{top.base} = 1 \cdot ft$

Height of Soil, L/S above base

$H_{soil.Ls.base} := EL_{grade.Ls} - EL_{top.base} = 2 \cdot ft$

Height of struc. from bottom of the Footing:

$H_{struc.} := EL_{TOW} - EL_{bot.base} = 5 \cdot ft$

Stem Thickness top:

$T_{top} := 1.0ft$

Stem thickness bottom:

$T_{bottom} := 1.0ft$

Stem Height :

$H_{stem} := EL_{TOW} - EL_{top.base} = 4 \cdot ft$

Base Thickness:

$T_{base} := \left(EL_{top.base} - EL_{bot.base}\right) = 1 \cdot ft$

Base Length:

$L_{base} := 4.0ft$

Length of Toe:

$L_{toe} := 1.0ft$

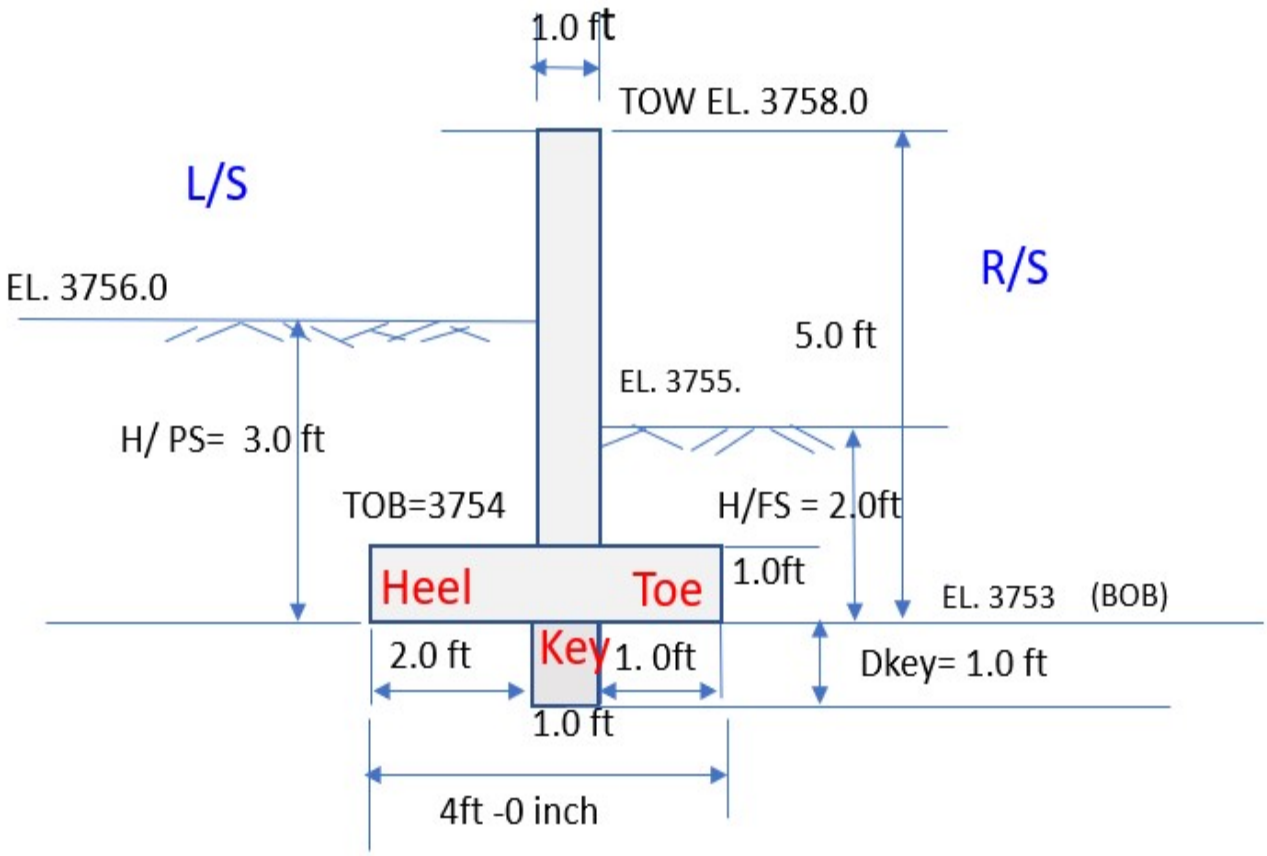
Length of Heel:

$L_{heel} := L_{base} - L_{toe} - T_{bottom} = 2 \cdot ft$

Soil Coefficients: (Ref: Geotechnical data for the Site)

***** Since this analysis is for Usual Condition, at Rest Soil Pressure Coefficient value is required only.

At Rest Earth Pressure Coefficient: $K_o := 0.69$



Flood Wall Stability – No Flood (Usual)

Load Calculations:

Structural Weight :

Area of Stem: $A_{stem} := 0.5\left(T_{top} + T_{bottom}\right)\cdot H_{stem} = 4\cdot ft^2$

Weight of Stem: $W_{stem} := A_{stem}\cdot\gamma_c = 0.6\cdot\frac{kip}{ft}$

Weight of Base Toe: $W_{toe} := L_{toe}\cdot T_{base}\cdot\gamma_c = 0.15\cdot\frac{kip}{ft}$

Weight of Base Heel: $W_{heel} := L_{heel}\cdot T_{base}\cdot\gamma_c = 0.3\cdot\frac{kip}{ft}$

Weight of Base Slab: $W_{base} := L_{base}\cdot T_{base}\cdot\gamma_c = 0.6\cdot\frac{kip}{ft}$

Addition of a friction/ shear key to the structure:

Key Dimension: Squared Section; $T_{key} := 1.0ft$ $B_{key} := 1.0ft$ $D_{key} := 1.0ft$

Dead weight of the key: $W_{key} := 0.5\left(T_{key} + B_{key}\right)\cdot D_{key}\cdot\gamma_c = 0.15\cdot\frac{kip}{ft}$

Dead Weight of the Structure: $W_{struc} := W_{stem} + W_{base} + W_{key} = 1.35\cdot\frac{kip}{ft}$

Moments About the Heel End due to weight of the structure.

Moments from Structure weight:

Resisting Moment From Stem: $M_{R.stem} := W_{stem}\cdot\left(L_{toe} + 0.5\cdot T_{bottom}\right)\cdot 1ft = 0.9\cdot kip\cdot ft$

Resisting Moment From Base: $M_{R.base} := W_{base}\cdot\left(L_{base}\cdot 0.5\right)\cdot 1ft = 1.2\cdot kip\cdot ft$

Resisting Moment from shear key: $M_{R.key} := W_{key}\cdot\left(B_{key}\cdot 0.5 + L_{toe}\right)\cdot 1ft = 0.225\cdot kip\cdot ft$

$$M_{R.struct.} := M_{R.stem} + M_{R.base} + M_{R.key} = 2.325\cdot kip\cdot ft$$

Stability Calculation for T-Wall Structure:

Location of Center of Gravity of T- Wall:

Cg. of Stem Weight from toe end: $L_{stem} := 0.5 \cdot (T_{bottom}) + L_{toe} = 1.5 \cdot ft$

Cg. of Base Weight from heel end: $L_{bs} := 0.5 \cdot (L_{base}) = 2 \cdot ft$

Location of C.G of the Wall from Toe end: $C_G := \frac{(W_{base} \cdot L_{bs} + W_{stem} \cdot L_{stem} + W_{key} \cdot L_{stem})}{W_{base} + W_{stem} + W_{key}} = 1.722 \cdot ft$

1.0 Wind Load on Structure.

$V_{wind} := 130mph$ $I := 1.15$

Wind Pressure as per ASCE 26-5

$$q_z := \max\left[0.00256 \cdot I \left(\frac{V_{wind}}{mph}\right)^2 psf, 50psf\right] = 50 \cdot psf$$

Wind Load as per HSDRRS-design guide, minim.of as obtained from above equation or 50 PSF

1. Wind From R/S only, (No flood conditionl)

$F_{wind.Rs.} := q_z \cdot (EL_{TOW} - EL_{grade.Rs}) = 0.15 \cdot \frac{kip}{ft}$

$L_{wind.Rs} := 0.5(EL_{TOW} - EL_{grade.Rs}) + (H_{soil.Rs}) = 3.5 \cdot ft$

$M_{r.wind.Rs} := F_{wind.Rs.} \cdot L_{wind.Rs} \cdot 1ft = 0.525 \cdot kip \cdot ft$

2.0 Lateral Earth Pressure On Wall

Lateral Earth Pressure from R/S (No flood Condition):

Horizonatl Earth Force at Bottom of Base R/S: $F_{soil.Rs} := 0.5 \cdot K_o \cdot \gamma_s \cdot \left(H_{soil.Rs} + D_{key}\right)^2 = 0.373 \cdot \frac{kip}{ft}$

Moment Arm for Lateral Earth Force from R/S: $L_{soil.Rs.} := \frac{1}{3}\left(H_{soil.Rs} + D_{key}\right) - 1 \cdot ft = 0 \cdot ft$

Moment from R/S Lateral Earth Pressure: $M_{r.soil.Rs.} := F_{soil.Rs} \cdot L_{soil.Rs.} \cdot 1ft = 0 \cdot kip \cdot ft$

Weight of soil on top of Toe R/S: $W_{soil.toe} := \left(EL_{grade.Rs} - EL_{top.base}\right) \cdot \gamma_s \cdot L_{toe} = 0.12 \cdot \frac{kip}{ft}$

Moment Arm due to weight of soil on heel: $L_{w.heel} := \left(0.5 L_{toe}\right) = 0.5 \cdot ft$

Resisting Moment due to Weight of soil on Heel: $M_{r.soil.heel.Rs} := W_{soil.toe} \cdot L_{w.heel} \cdot (1ft) = 0.06 \cdot kip \cdot ft$

Lateral Earth Pressure from Land Side (no flood condition):

Horizonatl Earth Force at Bottom of Base L/S: $F_{soil.Ls.} := 0.5 \cdot K_o \cdot \left(\gamma_s\right) \cdot \left(H_{soil.Ls} + D_{key}\right)^2 = 0.662 \cdot \frac{kip}{ft}$

Moment Arm for Horizonatl Earth force L/S: $L_{soil.Ls.} := \frac{1}{3}\left(H_{soil.Ls} + D_{key}\right) - 1 \cdot ft = 0.333 \cdot ft$

Moment from L/S Lateral Earth Pressure (overturning): $M_{o.soil.Ls.} := F_{soil.Ls.} \cdot L_{soil.Ls.} \cdot 1ft = 0.221 \cdot kip \cdot ft$

Weight of Soil on top of Heel (L/S): $W_{soil.heel} := \left(\gamma_s\right) \cdot \left(EL_{grade.Ls} - EL_{top.base}\right) \cdot L_{heel} = 0.48 \cdot \frac{kip}{ft}$

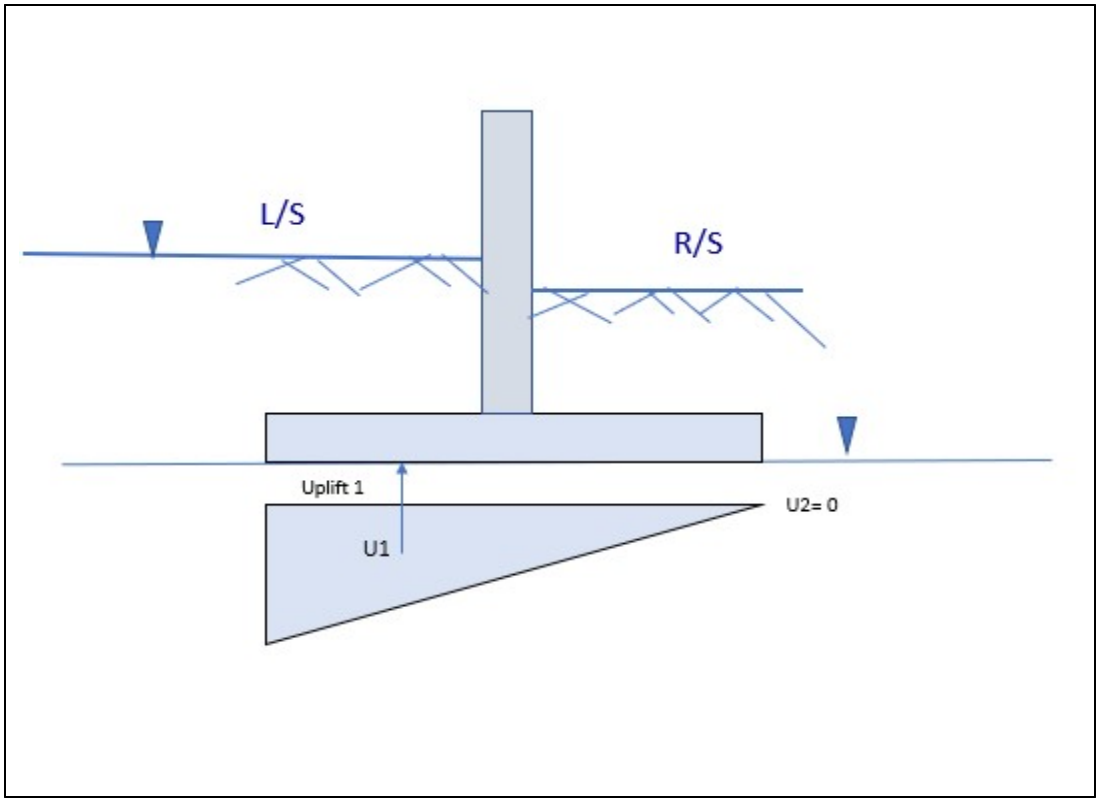
Moment Arm for weight of soil on heel (L/S): $L_{w.heel.} := \left(0.5 \cdot L_{heel} + T_{bottom} + L_{toe}\right) = 3 \cdot ft$

Moment due to weight of Soil on Toe slab F/S (resisting moment): $M_{r.soil.toe.Ls} := W_{soil.heel} L_{w.heel.} \cdot 1ft = 1.44 \cdot kip \cdot ft$

Uplift Loads (No flood, assuming ground water pore pressure on soil):

Uplift Pressure at the Land Side: $P_{uplift.Ls} := (\gamma_w) \cdot H_{soil.Ls} = 192 \cdot psf$

Uplift Pressure at the River Side: $P_{uplift.Rs} := (\gamma_w) \cdot (H_{water.Rs.}) = 0 \cdot psf$



Calculation of Uplift Loads:

$$V_{uplift.Rs} := P_{uplift.Rs} \cdot L_{base} = 0 \cdot \frac{kip}{ft}$$

$$V_{uplift.Ls} := 0.5 \cdot L_{base} \cdot (P_{uplift.Ls} - P_{uplift.Rs}) = 0.384 \cdot \frac{kip}{ft}$$

Sum of Uplift Load: $V_{uplift} := V_{uplift.Rs} + V_{uplift.Ls} = 0.384 \cdot \frac{kip}{ft}$

Vertical Resultant Force:

Net Vertical Resultant Force (Downward):

$$V_{net} := \left(W_{soil.toe} + W_{soil.heel} + W_{struc} - V_{uplift}\right) \cdot 1ft = 1.57 \cdot kip$$

Moment from Uplift Pressure:

Moment from Uplift Pressure:

$$M_{o.uplift} := \left(V_{uplift.Ls} \cdot \frac{2L_{base}}{3}\right) \cdot 1ft = 1.02 \cdot kip \cdot ft$$

Sum of Overturning Moments:

$$M_{o.sum} := \left(M_{o.soil.Ls.} + M_{o.uplift}\right) = 1.24 \cdot kip \cdot ft$$

Sum of Resisting Moments:

$$M_{R.sum} := \begin{pmatrix} M_{R.struct.} + M_{r.soil.toe.Ls} \cdots \\ + M_{r.soil.Rs.} + M_{r.soil.heel.Rs} + M_{r.wind.Rs} \end{pmatrix} = 4.35 \cdot kip \cdot ft$$

Sum of Lateral loads from L/S (No flood):

$$F_{lateral.Ls.} := F_{soil.Ls.} = 0.662 \cdot \frac{kip}{ft}$$

Sum of Lateral load from R/S (No flood)

$$F_{lateral.Rs.} := F_{wind.Rs.} + F_{soil.Rs} = 0.523 \cdot \frac{kip}{ft}$$

Net Lateral Force (No water on F/S):

$$P_{net.lat.} := F_{lateral.Ls.} - F_{lateral.Rs.} = 0.14 \cdot \frac{kip}{ft}$$

Check for Base in Compression

Reference Table: 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

Location of Resultant Force:

Kern Length: $\text{Kern} := \frac{L_{base}}{3} = 1.333 \cdot \text{ft}$

Balance Moment: $M_{balance} := \left(M_{R.sum} - M_{o.sum}\right) = 3.105 \cdot \text{kip} \cdot \text{ft}$

Resultant Location: $x_R := \frac{\left|M_{balance}\right|}{V_{net}} = 1.98 \cdot \text{ft}$

Eccentricity: $e_x := \frac{L_{base}}{2} - x_R = 0.02 \cdot \text{ft}$

$$\text{Check_Resultant_Location} := \begin{cases} \text{"Resultant Within the Kern"} & \text{if } \left|e_x\right| \le \frac{\text{Kern}}{2} \\ \text{"Resultant Outside the Kern but Within the Base"} & \text{if } \frac{\text{Kern}}{2} < \left|e_x\right| < \frac{L_{base}}{2} \\ \text{"FAILED"} & \text{otherwise} \end{cases}$$

Check_Resultant_Location = "Resultant Within the Kern"

The Base is 100 % in Compression

Check for Sliding (Stability):

Ref. Table: EM -1110-2-2502, Table 4-1.

Table 4-1
Retaining Wall Stability Criteria

Case No.	Loading Condition	Sliding Factor of Safety, FS	Shear Strength Test Required		Overturning Criteria Minimum Base Area in Compression		Minimum Bearing Capacity Safety Factor
			Soil Foundation	Rock Foundation[3]	Soil Foundation	Rock Foundation	
R1	Usual	1.5	(Q &/or S)[2,1]	Direct shear	100%[4]	75%[4]	3.0
R2	Unusual	1.33	(Q &/or S)[2,1]	Direct shear	75%[4]	50%[4]	2.0
R3	Earthquake	1.1	(Q)	Direct shear	Resultant within base	Resultant within base	>1.0

Sum of Horizontal Load on the River Side:

$$F_{Rs} := \left(F_{soil.Rs} + F_{wind.Rs.}\right)\cdot 1ft = 0.52\cdot kip$$

Sum of Horizontal Load on the Land Side:

$$F_{Ls} := \left(F_{soil.Ls.}\right)\cdot 1ft = 0.66\cdot kip$$

Friction Resistance Force: $F_R := V_{net}\cdot f = 0.51\cdot kip$

$$C_{ohesion} := C\cdot L_{base}\cdot 1ft = 0.4\cdot kip$$

$$FS_{sliding} := \frac{\left(F_R + F_{Rs}\right)}{F_{Ls}} = 1.56$$

$$Sliding_Factor_of_Safety_Check := \begin{cases} \text{"OK, adequate safety factor for this load case"} & \text{if } FS_{sliding} \geq 1.5 \\ \text{"FAILED"} & \text{otherwise} \end{cases}$$

Sliding_Factor_of_Safety_Check = "OK, adequate safety factor for this load case"

Check for Floatation (Stability):

Downward Vertical Force:

$$V_{down} := W_{struc} + W_{soil.heel} + W_{soil.toe} = 1.95 \cdot \frac{kip}{ft}$$

Upward Vertical Force:

$$V_{up} := V_{uplift} = 0.384 \cdot \frac{kip}{ft}$$

$$FS_{floatation} := \frac{V_{down}}{V_{up}} = 5.08$$

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

$$Check_Floatation_Stability := \begin{cases} \text{"OK"} & \text{if } FS_{floatation} \geq 1.3 \\ \text{"Fail"} & \text{otherwise} \end{cases}$$

Check_Floatation_Stability = "OK"

Check for Bearing Pressure Check:

Vertical DL force per / linear ft of floodwall section:

For Bearing pressure check, it is assumed that the backfill soil on top of the heel will be at dry condition. Therefore,

Weight of Dry Soil on Top of the Toe:

$$W_{soil.toe.} := W_{soil.toe} = 0.12 \cdot \frac{kip}{ft}$$

Weight of Dry Soil on Top of the Heel:

$$W_{soil.heel.} := W_{soil.heel} = 0.48 \cdot \frac{kip}{ft}$$

Total DL force:

$$V_{DL} := W_{struc} + W_{soil.toe.} + W_{soil.heel.} = 1.95 \cdot \frac{kip}{ft}$$

Average Bearing Pressure:

$$Bearing_{pressure} := \frac{V_{DL}}{L_{base}} = 0.49 \cdot ksf$$

Max. Pressure at Toe:

$$B_{max.toe} := 2 \cdot \frac{V_{DL}}{3 \cdot x_R} = 0.656 \cdot ksf$$

** allowable pressure is 1.0 ksf. (assumed)

The purpose of this calculation is to check the stability of existing floodwall, which is to be raised by 2 ft along Sunland East Levee at the intersection of existing concrete wall and the proposed East Levee Stoplog structure.

Following assumptions were made for this design calculations:

- Loading Condition: Unsual-Flood Condition
- Average Grade EL. 3755 feet on River Side (R/S) and EL. 3756 for Land Side (L/S).
- The flood water is assumed at EL +3757.0 ' at River side.
- Top of wall EL. 3758.0. (1 ft free board above flood level)
- The following stability criteria is followed to check the stability of the floodwall:

1) Location of Resultant: Within the base. 75% Base is in Compression
2) Minimum Sliding F.S.: 1.333
3) Minimum Floatation F.S.: 1.2

REFERENCES:

USACE EM 1110-2-2100 - Stabilitry Analysis Of Concrete Structure.

USACE EM 1110 -2 -2502- Retaining and Flood Walls.

USACE - Hurricane and Storm Damage Risk Reduction System Design Guidelines, Rev.June 2012.

ASCE -7-10 Minimum Design Loads for Buildings and Other Structures.

Design Parameters:

Soil Parameters:

Density of water: $\gamma_w := 64.0pcf$

Unit Weight of Saturated Soil: $\gamma_s := 115pcf$

Buoyant Unit Weight of Soil : $\gamma_{s.buoy.} := \gamma_s - \gamma_w = 51 \cdot pcf$

Unit Weight of Reinforced Concrete: $\gamma_c := 150 \cdot pcf$

Angle of Internal Friction: $\phi := 18.0deg$

Cohesion of Soil $C := 100psf$

Allowable Bearing Capacity of Soil (assumed): $\sigma := 1000psf$

Coefficient of Friction with Concrete: $f := \tan(\phi) = 0.325$ (From Geotechnical Data)

T - Wall Design Parameters.

Top of Flood Wall EL. $EL_{TOW} := 3758.0ft$

Design Flood Elevation (DFE): $EL_{DFE} := 3757.0ft$

Grade Elevation Riverside R/S (avg): $EL_{grade.Rs} := 3755.0ft$

Grade ELevation Landside L/S (avg): $EL_{grade.Ls} := 3756.0ft$

Elevation of Top of Base: $EL_{top.base} := 3754.0ft$

Elevation of the Bottom of Base: $EL_{bot.base} := 3753.0ft$

Elevation Bottom of Key: $EL_{bot.key} := 3752.0ft$

Height of Soil from bottom of footing below the Exposed grade R/S: $H_{soil.Rs} := EL_{grade.Rs} - EL_{bot.base} = 2 \cdot ft$

Height of Soil from bottom of footing to below the Exposed grade L/S: $H_{soil.Ls} := EL_{grade.Ls} - EL_{bot.base} = 3 \cdot ft$

Height of Soil above Base R/S: $H_{soil.base.Rs} := EL_{grade.Rs} - EL_{top.base} = 1 \cdot ft$

Height of Soil above Base L/S: $H_{soil.base.Ls} := EL_{grade.Ls} - EL_{top.base} = 2 \cdot ft$

Height of Structure from bottom of the base: $H_{struc.} := EL_{TOW} - EL_{bot.base} = 5 \cdot ft$

Avg. Height of Flood Water Above the Ground Surface (R/S): $H_{water} := EL_{DFE} - EL_{grade.Rs} = 2 \cdot ft$

T -Wall Section:

Stem Thickness top: $T_{top} := 1.0ft$

Stem thickness bottom: $T_{bottom} := 1.0ft$

Stem height : $H_{stem} := EL_{TOW} - EL_{top.base} = 4 \cdot ft$

Base Thickness: $T_{base} := \left(EL_{top.base} - EL_{bot.base}\right) = 1 \cdot ft$

Base Length: $L_{base} := 4.0ft$

Length of Toe: $L_{toe} := 2.0ft$

Length of Heel: $L_{heel} := L_{base} - L_{toe} - T_{bottom} = 1 \cdot ft$

Soil Coefficients:

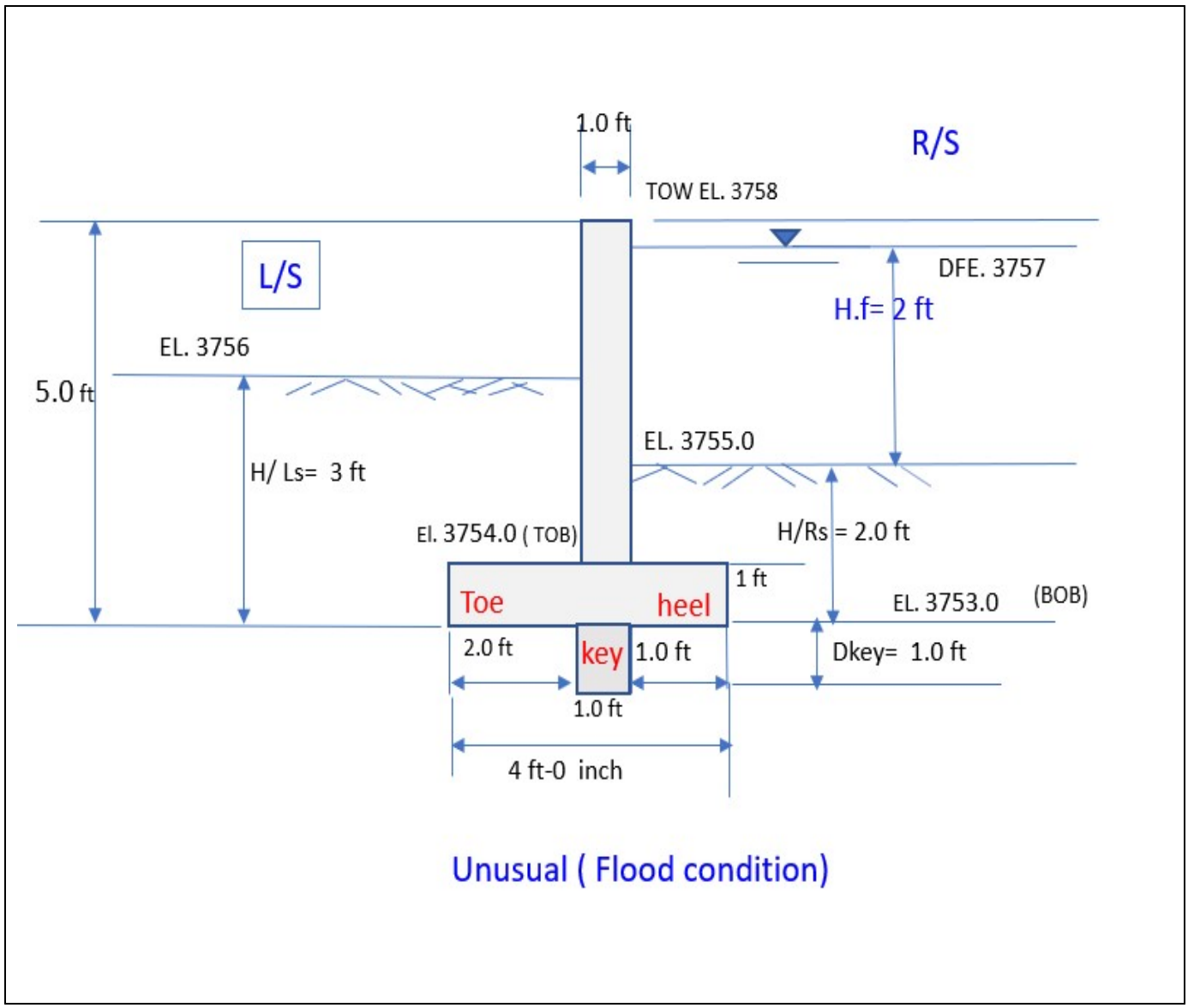
At Rest Earth Pressure Coefficient: $K_o := 0.69$

Active Earth Pressure Coefficient: $K_a := 0.53$ (From geotechnical data)

Passive Earth Pressure Coefficient: $K_p := 1.89$

Actual Kp value is assumed in-between Ko, and Kp, $K_{p.actual} := 1.5 \cdot K_o = 1.035$

*** Assumed Kp, actual,
Much higher force is required to mobilize the full potential Positive Earth Pressure)


1.0 ft
R/S
TOW EL. 3758
L/S
DFE. 3757
H.f= 2 ft
EL. 3756
5.0 ft
H/ Ls= 3 ft
EL. 3755.0
El. 3754.0 (TOB)
H/Rs = 2.0 ft
1 ft
Toe
heel
EL. 3753.0
(BOB)
2.0 ft
key
1.0 ft
Dkey= 1.0 ft
1.0 ft
4 ft-0 inch
Unusual (Flood condition)

Load Calculations:

Structural Dead Weight:

Area of Stem: $A_{stem} := 0.5\left(T_{top} + T_{bottom}\right)\cdot H_{stem} = 4\cdot ft^2$

Weight of Stem: $W_{stem} := A_{stem}\cdot\gamma_c = 0.6\cdot\frac{kip}{ft}$

Weight of Footing Toe: $W_{toe} := L_{toe}\cdot T_{base}\cdot\gamma_c = 0.3\cdot\frac{kip}{ft}$

Weight of Footing Heel: $W_{heel} := L_{heel}\cdot T_{base}\cdot\gamma_c = 0.15\cdot\frac{kip}{ft}$

Weight of Base of Footing: $W_{base} := L_{base}\cdot T_{base}\cdot\gamma_c = 0.6\cdot\frac{kip}{ft}$

Addition of a friction/ shear key to the structure:

Key Dimension: Squared Section; Width of key: $B_{key} := 1.00ft$ $D_{key} := 1.0ft$

Dead weight of the key: $W_{key} := \left(B_{key}\right)\cdot D_{key}\cdot\gamma_c = 0.15\cdot\frac{kip}{ft}$

Dead Weight of the Structure: $W_{struc} := W_{stem} + W_{base} + W_{key} = 1.35\cdot\frac{kip}{ft}$

Location of Center of Gravity of T- Wall:

Cg. of Stem Weight from Toe end: $L_{stem} := 0.5\cdot\left(T_{bottom}\right) + L_{toe} = 2.5\cdot ft$

Cg. of Base Weight from Toe end: $L_{bs} := 0.5\cdot\left(L_{base}\right) = 2\cdot ft$

Cg. of key from Toe End: $L_{key} := 0.5B_{key} + L_{toe} = 2.5\cdot ft$

Location of C.G of the Wall from Toe end: $C_G := \frac{\left(W_{base}\cdot L_{bs} + W_{stem}\cdot L_{stem} + W_{key}\cdot L_{stem}\right)}{W_{base} + W_{stem} + W_{key}} = 2.278\cdot ft$

Stability Calculation for the T-Wall Structure:

Moments about the Toe end due to weight of the structure.

Resisting Moments from Structure Self Weight:

Resisting Moment From Stem: $M_{R.stem} := W_{stem} \cdot (L_{stem}) \cdot 1ft = 1.5 \cdot kip \cdot ft$

Resisting Moment From Base: $M_{R.base} := W_{base} \cdot (L_{bs}) \cdot 1ft = 1.2 \cdot kip \cdot ft$

Resisting Moment from shear key: $M_{R.key} := W_{key} \cdot (L_{key}) \cdot 1ft = 0.375 \cdot kip \cdot ft$

Total Resisting Moment from the structure: $M_{R.struct} := M_{R.stem} + M_{R.base} + M_{R.key} = 3.075 \cdot kip \cdot ft$

Loads on floodwall:

1.0 Wind Load on Wall:

Velocity of Wind:

$V_{wind} := 130mph$ $I := 1.15$

Wind Pressure as per ASCE 7-16, Section 26-5

$$q_z := \max\left[0.00256 \cdot I \left(\frac{V_{wind}}{mph}\right)^2 psf, 50psf\right] = 50 \cdot psf$$

Wind load as per HSDRRS -Design guide is as obtained from ASCE-7-16, Section-26.6 but 50 psf minimum. Wind Load as per HSDRRS= 50 PSF

I) . Wind From L/S only, (Flood condition)

$$F_{wind.Ls.} := q_z \cdot \left(EL_{TOW} - EL_{grade.Ls}\right) = 0.1 \cdot \frac{kip}{ft}$$

$$L_{wind.Ls} := 0.5\left(EL_{TOW} - EL_{grade.Ls}\right) + \left(H_{soil.Ls}\right) = 4 \cdot ft$$

Moment due to wind from L/S:

$$M_{r.wind.Ls} := F_{wind.Ls.} \cdot L_{wind.Ls} \cdot 1ft = 0.4 \cdot kip \cdot ft$$

2.0 Lateral Earth Pressure from River Side (Flood condition):

Horizonatl Earth Force at Bottom of Base R/S:

$$F_{soil.Rs.} := 0.5 \cdot K_o \cdot \gamma_{s.buoy.} \cdot \left(H_{soil.Rs} + D_{key}\right)^2 = 0.158 \cdot \frac{kip}{ft}$$

Moment Arm for Horizonatl Earth force R/S:

$$L_{soil.Rs.} := \frac{1}{3}\left(H_{soil.Rs} + D_{key}\right) - 1ft = 0 \cdot ft$$

Moment from R/S Lateral Earth Pressure (overturning):

$$M_{o.soil.Rs.} := F_{soil.Rs.} \cdot L_{soil.Rs.} \cdot 1ft = 0 \cdot kip \cdot ft$$

Weight of Soil on top of Heel (R/S):

$$W_{soil.heel} := \left(\gamma_{s.buoy.}\right) \cdot \left(EL_{grade.Rs} - EL_{top.base}\right) \cdot L_{heel} = 0.051 \cdot \frac{kip}{ft}$$

Moment Arm for weight of soil on top of Heel (R/S):

$$L_{w.heel.} := 0.5 \cdot \left(L_{heel}\right) + T_{bottom} + L_{toe} = 3.5 \cdot ft$$

Moment due to weight of Soil on Heel slab R/S :

$$M_{r.soil.heel} := W_{soil.heel} L_{w.heel.} \cdot 1ft = 0.179 \cdot kip \cdot ft$$

Lateral Earth Pressure from Land Side (flood Condition):

Horizonatl Earth Force at Bottom of Base L/S:

$$F_{soil.Ls} := 0.5 \cdot K_{p.actual} \cdot \gamma_s \cdot \left(H_{soil.Ls} + D_{key}\right)^2 = 0.952 \cdot \frac{kip}{ft}$$

Moment Arm for Horizontal Earth force from P/S:

$$L_{soil.Ls.} := \frac{1}{3}\left(H_{soil.Ls} + D_{key}\right) - 1ft = 0.333 \cdot ft$$

Resisting Moment from L/S Lateral Earth Pressure:

$$M_{r.soil.Ls.} := F_{soil.Ls} \cdot L_{soil.Ls.} \cdot 1ft = 0.317 \cdot kip \cdot ft$$

Due to weight of soil on top of toe:

$$W_{soil.toe} := \left(EL_{grade.Ls} - EL_{top.base}\right) \cdot \gamma_s \cdot L_{toe} = 0.46 \cdot \frac{kip}{ft}$$

Resisting Moment due to Weight of soil on Toe: $M_{r.toe.soil.} := W_{soil.toe} \cdot 0.5 \cdot L_{toe} \cdot 1ft = 0.46 \cdot kip \cdot ft$

3.0 Hydrostatic load due to Floodwater:

Water to Design Flood Elevation. *Unusual Condition*

Hydrostatic Force on River Side: $F_{hyd.} := 0.5 \cdot \gamma_w \cdot \left(EL_{DFE} - EL_{bot.key}\right)^2 = 0.8 \cdot \frac{kip}{ft}$

Moment Arm for Design flood: $L_{hyd.} := \frac{EL_{DFE} - EL_{bot.base}}{3} = 1.333 \cdot ft$

Moment Due to design flood: $M_{o.hyd.} := F_{hyd.} \cdot L_{hyd.} \cdot 1ft = 1.067 \cdot kip \cdot ft$

Weight of flood water sitting on Heel: $W_{water.heel} := \left(\gamma_w\right) \cdot \left(EL_{DFE} - EL_{top.base}\right) \cdot L_{heel} = 0.192 \cdot \frac{kip}{ft}$

Moment arm for design flood water on Heel: $L_{w.hyd.} := 0.5\left(L_{heel}\right) + T_{bottom} + L_{toe} = 3.5 \cdot ft$

Resisting Moment due to weight of flood water sitting on heel: $M_{r.hyd.heel} := L_{w.hyd.} \cdot W_{water.heel} \cdot 1ft = 0.672 \cdot kip \cdot ft$

Water surcharge load due to hydrostatic: $P_{surch.} := 0.5 K_a \cdot \gamma_w \cdot H_{water} = 0.034 \cdot ksf$

Force due to water surcharge: $F_{sur.} := P_{surch.} \cdot H_{soil.Rs} = 0.068 \cdot \frac{kip}{ft}$

Moment arm for hyd. water surcharge: $L_{hyd.sur.} := \frac{H_{soil.Rs}}{2} = 1 \cdot ft$

Moment due to water surcharge: $M_{o.sur.} := \left(F_{sur.} \cdot L_{hyd.sur.}\right) \cdot 1ft = 0.068 \cdot kip \cdot ft$

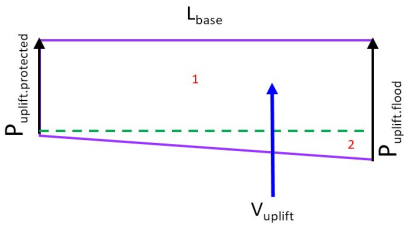
4.0 Uplift :

Uplift Pressure at Land Side:

$$P_{uplift.Ls.} := (\gamma_w)\cdot H_{soil.Ls} = 192\cdot psf$$

Uplift Pressure at the River Side:

$$P_{uplift.Rs} := (\gamma_w)\cdot (H_{soil.Rs} + H_{water}) = 256\cdot psf$$



Calculation of Uplift Loads

$$V_{uplift1.} := P_{uplift.Ls.}\cdot L_{base} = 0.768\cdot \frac{kip}{ft}$$

$$V_{uplift2.} := 0.5\cdot L_{base}\cdot (P_{uplift.Ls.} - P_{uplift.Rs}) = -0.128\cdot \frac{kip}{ft}$$

Sum of Uplift Load:

$$V_{uplift.} := V_{uplift1.} + V_{uplift2.} = 0.64\cdot \frac{kip}{ft}$$

Net Vertical Force:

$$V_{net.} := (W_{water.heel} + W_{soil.toe} + W_{soil.heel} + W_{struc} - V_{uplift.})\cdot 1ft = 1.41\cdot kip$$

Moment about toe end from Uplift Pressure :

Moment from Uplift Pressure:

$$M_{o.uplift.} := \left[V_{uplift1.}\cdot \frac{L_{base}}{2} + V_{uplift2.}\cdot \left(\frac{2L_{base}}{3}\right)\right]\cdot 1ft = 1.19\cdot kip\cdot ft$$

Sum of Overturning Moments:

$$M_{o.sum.} := \left(M_{o.hyd.} + M_{o.soil.Rs.} + M_{o.uplift.} + M_{o.sur.}\right) = 2.33 \cdot kip \cdot ft$$

Sum of Resisting Moments :

$$M_{R.sum} := \begin{pmatrix} M_{R.struct} + M_{r.soil.heel} \cdots \\ + M_{r.soil.Ls.} + M_{r.hyd.heel} + M_{r.wind.Ls} + M_{r.toe.soil.} \end{pmatrix} = 5.10 \cdot kip \cdot ft$$

Sum of Lateral loads from R/S (DFE water on R/S):

$$F_{lateral.Rs.} := F_{soil.Rs.} + F_{hyd.} + F_{sur.} = 1.026 \cdot \frac{kip}{ft}$$

Sum of Lateral load from L/S (water to DFE on F/S)

$$F_{lat.Ls.} := F_{soil.Ls} + F_{wind.Ls.} = 1.052 \cdot \frac{kip}{ft}$$

Net Lateral Force (Water to DFE):

$$F_{net.lat.} := F_{lateral.Rs.} - F_{lat.Ls.} = -0.026 \cdot \frac{kip}{ft}$$

Check for Base Compression

Location of Resultant Force:

Kern Length:

$$Kern := \frac{L_{base}}{3} = 1.333 \cdot ft$$

Balance Moment:

$$M_{balance} := \left(M_{R.sum} - M_{o.sum.}\right) = 2.774 \cdot kip \cdot ft$$

Resultant Location:

$$x_R := \frac{\left|M_{balance}\right|}{V_{net.}} = 1.96 \cdot ft$$

Eccentricity:

$$e_x := \frac{L_{base}}{2} - x_R = 0.04 \cdot ft$$

$$Check_Resultant_Location := \begin{pmatrix} \text{"Resultant Within the Kern"} & \text{if } \left|e_x\right| \le \frac{Kern}{2} \\ \text{"Resultant Outside the Kern but Within the Base"} & \text{if } \frac{Kern}{2} < \left|e_x\right| < \frac{L_{base}}{2} \\ \text{"FAILED"} & \text{otherwise} \end{pmatrix}$$

Check_Resultant_Location = "Resultant Within the Kern"

More than 75% of Base is in Compression

Check for Sliding (Stability)

Ref : EM-1110-2-2502 Table 4-1 Factor of safety (sliding) :

Table 4-1

Retaining Wall Stability Criteria

Case No.	Loading Condition	Sliding Factor of Safety, FS	Shear Strength Test Required		Overturning Criteria Minimum Base Area in Compression		Minimum Bearing Capacity Safety Factor
			Soil Foundation	Rock Foundation[3]	Soil Foundation	Rock Foundation	
R1	Usual	1.5	(Q &/or S)[2,1]	Direct shear	100%[4]	75%[4]	3.0
R2	Unusual	1.33	(Q &/or S)[2,1]	Direct shear	75%[4]	50%[4]	2.0
R3	Earthquake	1.1	(Q)	Direct shear	Resultant within base	Resultant within base	>1.0

Sliding Safety Factor Check:

Sum of Horizontal Load from the Flood Side (R/S):

$$F_{Rs.} := \left(F_{soil.Rs.} + F_{hyd.} + F_{sur.}\right)\cdot 1ft = 1.03\cdot kip$$

Sum of Horizontal Load on the Protected Side (L/S):

$$F_{Ls.} := \left(F_{soil.Ls} + F_{wind.Ls.}\right)\cdot 1ft = 1.052\cdot kip$$

Friction Resistance Force: $F_R := V_{net.}\cdot f = 0.46\cdot kip$

Ref : ' FS' sliding: EM 11102-2-2502, Table: (4-1)

$$C_{ohesion} := C\cdot L_{base}\cdot 1ft = 0.4\cdot kip$$

$$FS_{sliding} := \frac{\left(F_R + F_{Ls.}\right)}{F_{Rs.}} = 1.47$$

$$Sliding_Factor_of_Safety_Check := \begin{cases} \text{"OK, adequate safety factor for this load case"} & \text{if } FS_{sliding} \geq 1.33 \\ \text{"FAILED"} & \text{otherwise} \end{cases}$$

Sliding_Factor_of_Safety_Check = "OK, adequate safety factor for this load case"

Floatation Stability Check:

Downward Vertical Force:

$$V_{down} := W_{struc} + W_{water.heel} + W_{soil.heel} + W_{soil.toe} = 2.053 \cdot \frac{kip}{ft}$$

Ref : EM-1110-2-2100 Table 3:4

Upward Vertical Force:

$$V_{up} := V_{uplift.} = 0.64 \cdot \frac{kip}{ft}$$

$$FS_{floatation} := \frac{V_{down}}{V_{up}} = 3.21$$

$$Check_Floatation_Stability := \begin{cases} \text{"OK"} & \text{if } FS_{floatation} \geq 1.2 \\ \text{"Fail"} & \text{otherwise} \end{cases}$$

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

Check_Floatation_Stability = "OK"

Bearing Pressure Check:

Vertical DL force per / linear ft of floodwall section:

For Bearing pressure check, it is assumed that the backfill soil on top of the heel will be at dry condition. Therefore,

Weight of Dry Soil on Top of the Toe: $W_{soil.toe.} := (\gamma_s) \cdot H_{soil.Ls} \cdot L_{toe} = 0.69 \cdot \frac{kip}{ft}$

Weight of Dry Soil on Top of the Heel: $W_{soil.heel.} := W_{soil.heel} = 0.051 \cdot \frac{kip}{ft}$

Total DL Force: $V_{DL} := W_{struc} + W_{soil.toe.} + W_{soil.heel.} = 2.09 \cdot \frac{kip}{ft}$

Average Bearing Pressure per linear ft of wall Section:

Average Bearing Pressure under base: $Bearing_{pressure} := \frac{V_{DL}}{L_{base}} = 0.52 \cdot ksf$

Maximum Pressure at Toe: $q := 2 \cdot \frac{V_{DL}}{3 \cdot x_R} = 0.71 \cdot ksf$

**** Bearing Pressure is Checked against assumed value of 1.0ksf

$$Check_Bearing_Pressure := \begin{cases} \text{"OK, Bearing Pressure is within Allowable"} & \text{if } Bearing_{pressure} \le \sigma \\ \text{"FAILED"} & \text{otherwise} \end{cases}$$

Check_Bearing_Pressure = "OK, Bearing Pressure is within Allowable"

APPENDIX D

Appendix D – Subsurface Utility Exploration

2019

Sunland Park Levee Utility Report

PREPARED BY FRANK X. SPENCER & ASSOCIATES

JOHN R. BARNES

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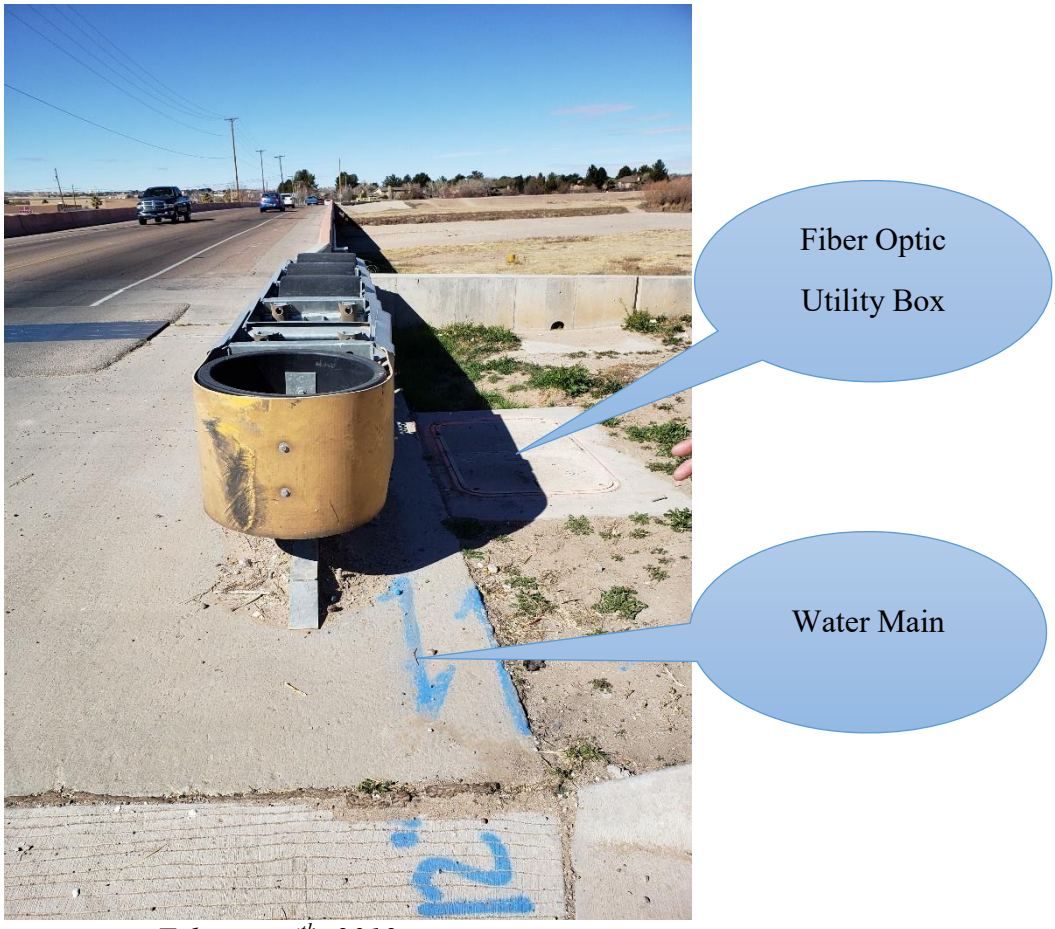
SUE at Country Club Rd. (North Side)

SUE Level D & C

- 12-Inch Water Main: El Paso Water
    - FXSA located and gathered survey data for utilities marked by Texas 811 on the northside of Country Club Road bridge, as well as on the east and west levee areas to identify the approximate location of a 12-inch water main crossing beneath the Rio Grande River. Based on the proposed depth as shown on the as-built drawings provided by El Paso Water and the proposed level of depth for the proposed structures as shown on Arcadis design drawings, it has been determined that the existing water main is at a depth that will not interfere with the design and nondestructive exposure will not be necessary. (See attached as-built draws)



February 4th, 2019

- Fiber Optic: City of El Paso

    - The fiber optic was found to be running alongside the northside of Country Club Road bridge, there are two existing fiber utility boxes on the east and west levee areas. FXSA made two attempts to contact the City of El Paso to request access to the two fiber utility boxes to gather depth information. On February 20, 2019 FXSA contacted the City of El Paso ((915) 212-0151) and was directed by City of El Paso employee to submit request via email (linespots@elpasotexas.gov). This email was viewed by several City employee due to attached read receipt sent with the requested email. On February 21, 2019 an additional attempt was made via phone ((915) 212-0151) to follow-up on submitted email request with no answer. No response from the City of El Paso was received.

February 4th, 2019

SUE Level B & A

- Based on the proposed depth as shown on the as-built drawings provided by El Paso Water and the proposed level of depth for the proposed structures as shown on Arcadis design drawings, it has been determined that the existing water main is at a depth that will not interfere with the design and nondestructive exposure will not be necessary.

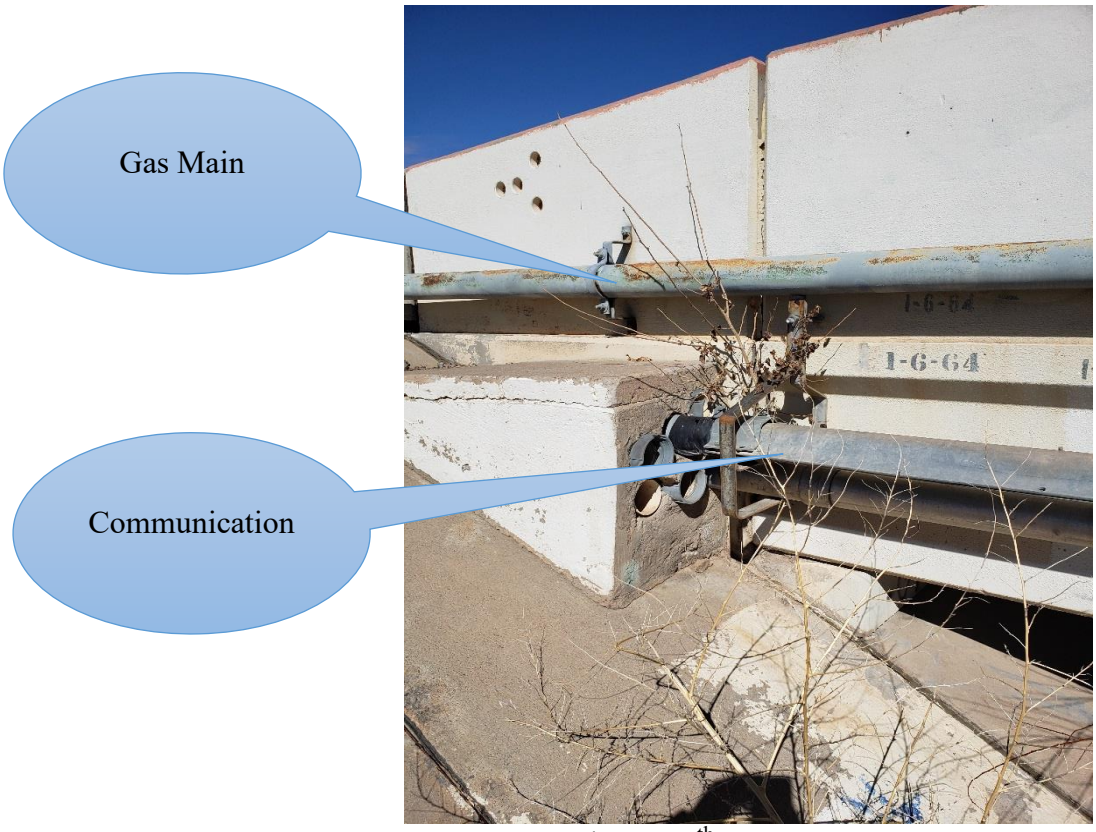
SUE at Country Club Rd. (South Side)

SUE Level D & C

- 4-Inch Gas Main: Texas ONE Gas
    - FXSA located and gathered survey data for the utilities marked as a result Texas 811 request. The 4-inch Gas main is approximately located parallel to and on the southside of Country Club Road Bridge. The exact location and depth of the gas main on the east and west levee areas cannot be determined without performing a SUE (Level A) survey. The approximate pipe size, material and location of this utility are confirmed by a GIS map provided by Texas ONE Gas. (See attached GIS map)

February 4th, 2019

- Underground Communication: AT&T
    - FXSA located and gathered survey data for the utilities marked as a result Texas 811 request. The two 4-inch conduit communication lines are located alongside the bridge structure on the southside of Country Club Road Bridge. The location and depth of the communication lines cannot be determined without performing a SUE (Level A) survey for the areas near the east and west levee. There was no response to requests made by FXSA for an as-built and/or proposed facilities drawing from AT&T.



February 4th, 2019

SUE Level B & A

- The exact location and depth of the gas main on the east and west levee areas cannot be determined without performing a SUE (Level A) survey. The approximate pipe size, material and location of this utility are confirmed by a GIS map provided by Texas ONE Gas. (See attached GIS map) The gas line could be located on dirt areas only- concrete paving cannot be pothole per Tri-State. The line appears to be attached to the bridge within grey pipeline although cannot ascertain this. If utility line goes down the levee, this will pose a problem as there is no access.

- The location and depth of the communication lines cannot be determined without performing a SUE (Level A) survey for the areas near the east and west levee. AT&T communication line runs into comm. The concrete/paved area cannot be saw-cut/potholed as per Tri-State. Potholing in these areas can cause damage to bridge deck slab and traffic attenuator as per Tri-State.

SUE at Montoya Lateral- South of Country Club East Levee

SUE Level D

SUE Level C

SUE Level B

SUE Level A

- No underground utilities.

SUE at Canutillo Lateral- North of Country Club and South of Borderland West Levee

SUE Level D

SUE Level C

SUE Level B

SUE Level A

- No underground utilities.

AS-BUILT DRAWINGS

Exhibit A – County Club Road Improvements

Water Distribution Plan Views for County Club Road

STA. 12+00 TO 24+00

Provided by El Paso Water

Exhibit B – County Club Road Improvements

Water Distribution Plan Views for County Club Road

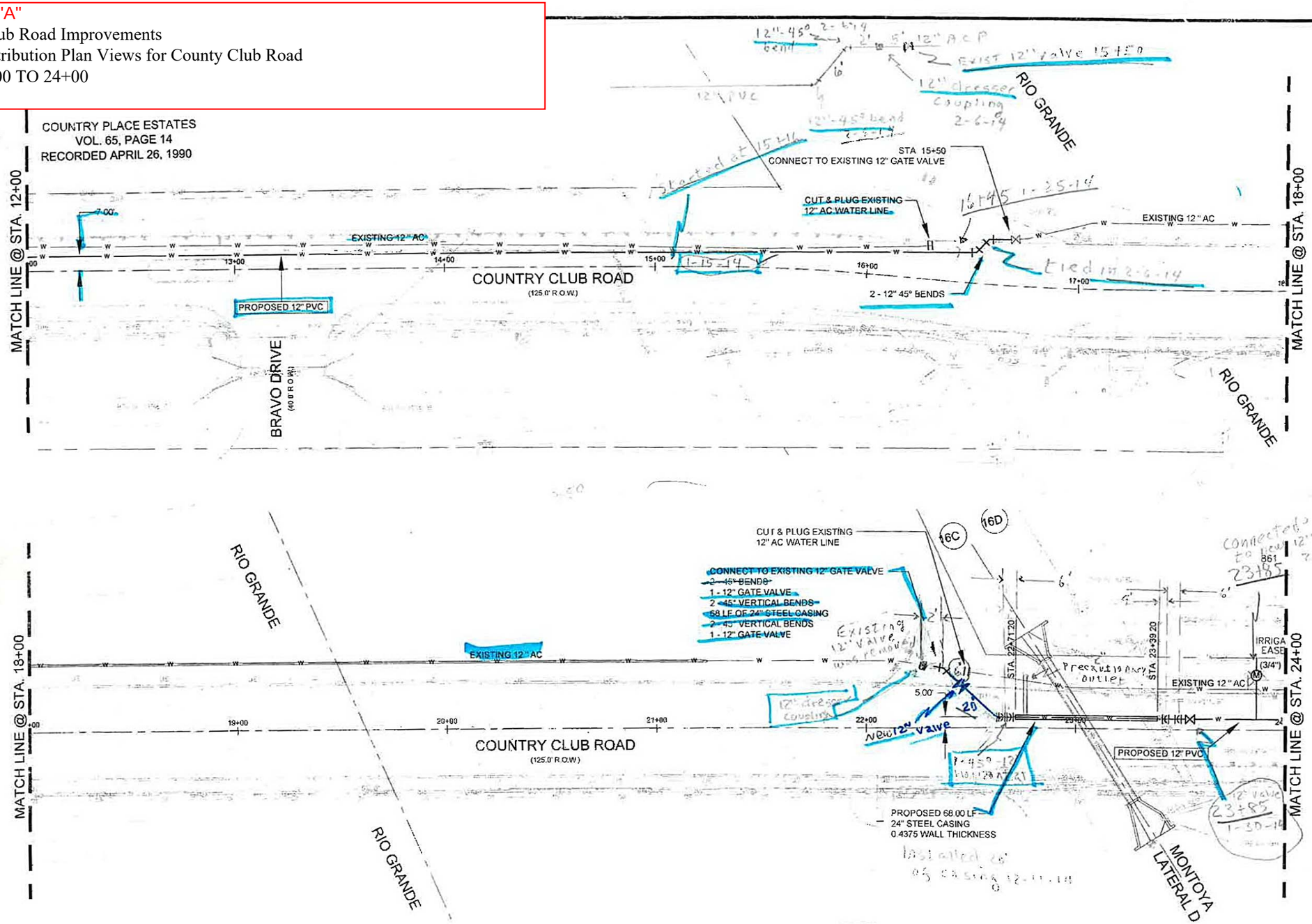
STA. 24+00 TO 36+00

Provided by El Paso Water

Exhibit C – ONE Gas GIS Map

Provided by Texas ONE Gas

EXHIBIT "A"
County Club Road Improvements
Water Distribution Plan Views for County Club Road
STA. 12+00 TO 24+00



GRAPHIC SCALE
1 inch = 50 ft

LEGEND

R.O.W. LINE
CENTER LINE
LOT LINE

PROPOSED IMPROVEMENTS
MANHOLE
SEWER LINE
SEWER SERVICE RECONNECTION
WATER LINE
FIRE HYDRANT
WATER METER
RECONNECTION LOCATION

EXISTING IMPROVEMENTS
MANHOLE / EXISTING SEWER LINE
SEWER LINE TO BE ABANDONED
WATER LINE TO BE ABANDONED
WATER LINE TO BE REMOVED
FIRE HYDRANT
EXISTING WATER METER
WATER VALVE
B-2 APPROXIMATE BORE LOCATION
POWER POLE
UNDERGROUND ELECTRIC
TELEPHONE MANHOLE
GAS LINES
SIGN POLE
POWER POLE W/ GUY WIRE
OVERHEAD ELECTRIC LINES
FENCES

REV	DESCRIPTION	DATE	BY
1	AMENDMENT NO. 3	05/13/13	ESSCO
2	DESIGN CHANGE NO.1	07/30/13	ESSCO
3	DESIGN CHANGE NO.2	12/03/13	ESSCO

ITEM	CODE	DESCRIPTION	UNIT	QUANTITY
5678	0185 C	WATER MAIN (PVC)(C-900)(12 IN)	LF	655
5678	9085 A	CASING (STEEL)(24 INCH)(OPEN TRENCH)	LF	70
5678	4085	WATER SVC REPLACE AND RECONNECT (3/4")	EA	1
5678	0185 C	ABANDON EXISTING 12" WATER LINE	LF	587
5678	1885	TRENCH EXCAVATION PROTECTION	LF	655

CITY OF EL PASO
ENGINEERING &
CONSTRUCTION MANAGEMENT

EL PASO WATER UTILITIES
PUBLIC SERVICE BOARD

ESSCO
1000 Newman
El Paso, Texas 79912
tel (915) 533-1102
fax (915) 533-1103

ENGINEER'S SEAL

STATE OF TEXAS
ROBERT CONCHA JR.
89814
REGISTERED
PROFESSIONAL ENGINEER
12/3/13

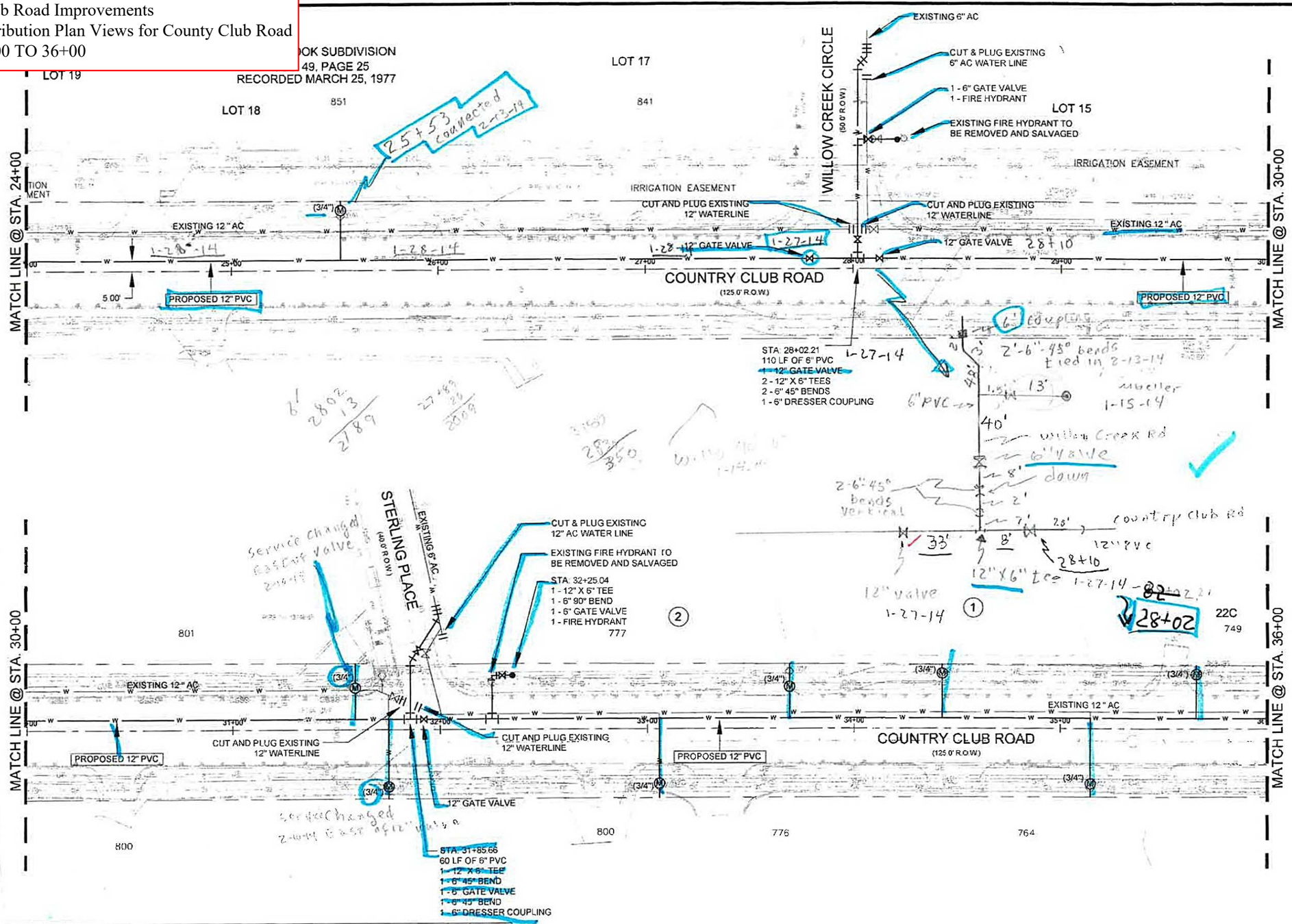
COUNTRY CLUB ROAD
IMPROVEMENTS

WATER DISTRIBUTION
PLAN VIEWS FOR COUNTRY CLUB ROAD
STA: 12+00 TO 24+00

SHEET 2 OF 9

FED RD DIV NO	FEDERAL AID PROJECT NO	SHEET NO
6		
STATE	DISTRICT	COUNTY
TEXAS	ELP	EP
CONTROL	SECTION	JOB
0924	08	

EXHIBIT "B"
County Club Road Improvements
Water Distribution Plan Views for County Club Road
STA. 24+00 TO 36+00



CODE	DESCRIPTION	UNIT	QUANTITY
0185 A	WATER MAIN (PVC)(C-900)(6-IN)	LF	221
0185 C	WATER MAIN (PVC)(C-900)(12 IN)	LF	1200
5085	WATER SVC REPLACE AND RECONNECT (3/4")	EA	8
5285	FIRE HYDRANT	EA	2
5485	SALVAGE EXISTING FIRE HYDRANT	EA	2
0985 A	ABANDON EXISTING 6" WATER LINE	LF	221
0985 C	ABANDON EXISTING 12" WATER LINE	LF	1200
1885	TRENCH EXCAVATION PROTECTION	LF	1421

GRAPHIC SCALE
1 inch = 50 ft

LEGEND

R.O.W. LINE
CENTER LINE
LOT LINE

PROPOSED IMPROVEMENTS
MANHOLE
SEWER LINE
SEWER SERVICE RECONNECTION
WATER LINE
FIRE HYDRANT
WATER METER
RECONNECTION LOCATION

EXISTING IMPROVEMENTS
MANHOLE / EXISTING SEWER LINE
SEWER LINE TO BE ABANDONED
WATER LINE TO BE ABANDONED
WATER LINE TO BE REMOVED
FIRE HYDRANT
EXISTING WATER METER
WATER VALVE
B-2 APPROXIMATE BORE LOCATION
POWER POLE
UNDERGROUND ELECTRIC
TELEPHONE MANHOLE
GAS LINES
SIGN POLE
POWER POLE W/ GUY WIRE
OVERHEAD ELECTRIC LINES
FENCES

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CITY OF EL PASO
ENGINEERING &
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fax (915) 533-1103

ENGINEER'S SEAL

STATE OF TEXAS
ROBERT CONCHA JR.
89814
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PROFESSIONAL ENGINEER
12/3/13

COUNTRY CLUB ROAD
IMPROVEMENTS

WATER DISTRIBUTION
PLAN VIEWS FOR COUNTRY CLUB ROAD
STA: 24+00 TO 36+00

SHEET 3 OF 9

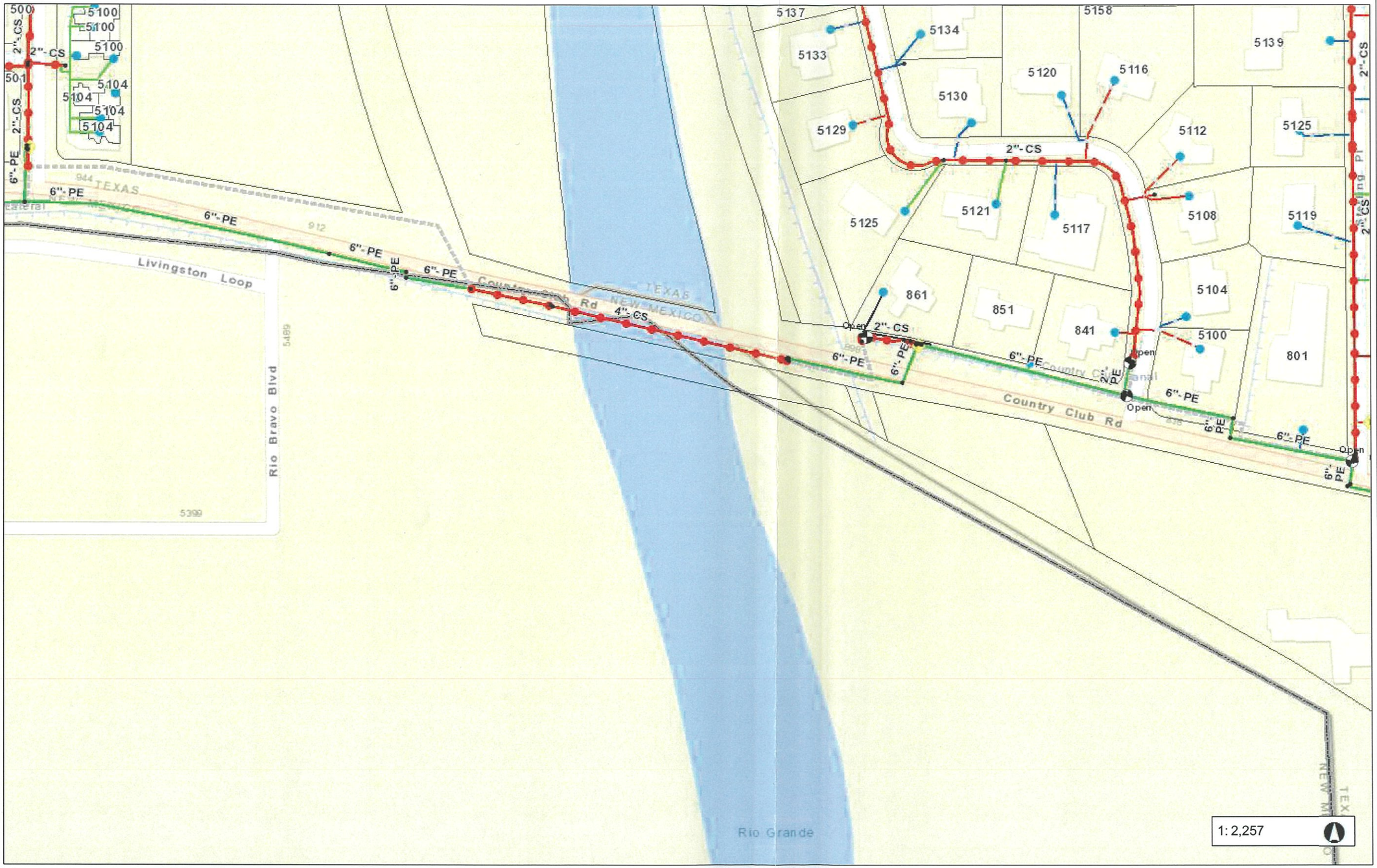
FED RD DIV NO	FEDERAL AID PROJECT NO		SHEET NO
6			
STATE	DISTRICT	COUNTY	
TEXAS	ELP	EL PASO	
CONTROL	SECTION	JOB	HIGHWAY NO
0924	06	417	CS COUNTRY CLUB

WATER DISTRIBUTION PLAN VIEWS FOR COUNTRY CLUB ROAD STA. 24+00 TO 36+00

EXHIBIT "C"
ONE GAS GIS Map

ONE Gas

ONE Gas Map



Legend

- Designer Drip
- Designer Test Point
- Designer Anode
- Designer Rectifier
- Designer Bond Junction
- Designer Ground Bed
- Designer Cable
  - Bond Wire
  - Rectifier Cable
- Designer Rural Tap
- Designer Gas Pipe Casing
  - Casing
  - Insertion
- Designer Meter Setting
- Designer Non-Controllable Fitting
  - Coupling
  - Ell
  - End Cap
  - Expansion Joint
  - Flange
  - Reducer
  - Reinforcing Sleeve
  - Screw
  - Tee
  - Transition
  - Purge Point
  - Threaded O-ring
  - Unknown
- Designer Gas Valve
- Designer Location Notes
- Designer Controllable Fitting
  - Short Stop
  - Three-Way Tee
- Designer Regulator Station
- Designer Town Border Station
- Designer Excess Flow Valve
- Designer Utility Easement
- Designer Abandon Service Line
  - to be Removed (Contains Gas)
  - to be Abandoned (Contains Gas)
  - Removed (No Longer Contains Gas
  - Abandoned (No Longer Contains G
- Designer Service Line
  - Status Not Available
  - New Design - Status Pending
  - Proposed
  - Approved
  - Under Construction
  - Operating
- Designer Abandon Mains
  - to be Removed (Contains Gas)
  - to be Abandoned (Contains Gas)
  - Removed (No Longer Contains Gas
  - Abandoned (No Longer Contains G
- Designer Mains
  - Status Not Available
  - New Design - Status Pending
  - Proposed
  - Approved
  - Under Construction
  - Operating
- Designer Work Order Polygon
  - Status Not Available
  - New Design - Status Pending
  - Proposed
  - Approved
  - Under Construction
  - Operating
- Service Point
- Non-Controllable Gas Fitting
  - Insulated Coupling
  - Other
- Gas Valve
  - Closed
  - Open
- Regulator Station
- Town Border Station
- Rural Tap

376.2 0 188.08 376.2 Feet

WGS_1984_Web_Mercator_Auxiliary_Sphere

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Notes

AT&T COMMUNICATION LINE
PHOTOS OF AREA

Exhibit A – Manhole West Levee Area

Exhibit B – Manhole East Levee Area

Exhibit C – County Club Road Southside, (The two 4-inch conduit communication lines are located alongside the bridge structure on the southside of Country Club Road Bridge). Looking East to West.

Exhibit D – County Club Road Southside, (The two 4-inch conduit communication lines are located alongside the bridge structure on the southside of Country Club Road Bridge). Looking East to West.

Exhibit E– County Club Road Southside, (The two 4-inch conduit communication lines are located alongside the bridge structure on the southside of Country Club Road Bridge).

Exhibit F – County Club Road Southside, (The two 4-inch conduit communication lines are located alongside the bridge structure on the southside of Country Club Road Bridge). Looking West to East.

Exhibit G – County Club Road GIS Map

EXHIBIT "A"
February 4th, 2019
Manhole West Levee Area

EXHIBIT "B"
February 4th, 2019
Manhole East Levee Area

EXHIBIT "C"
February 4th, 2019
County Club Road Southside, (The two 4-inch conduit communication lines are located alongside the bridge structure on the southside of Country Club Road Bridge). Looking East to West.

EXHIBIT "D"
February 4th, 2019
County Club Road Southside, (The two 4-inch conduit communication lines are located alongside the bridge structure on the southside of Country Club Road Bridge). Looking East to West.

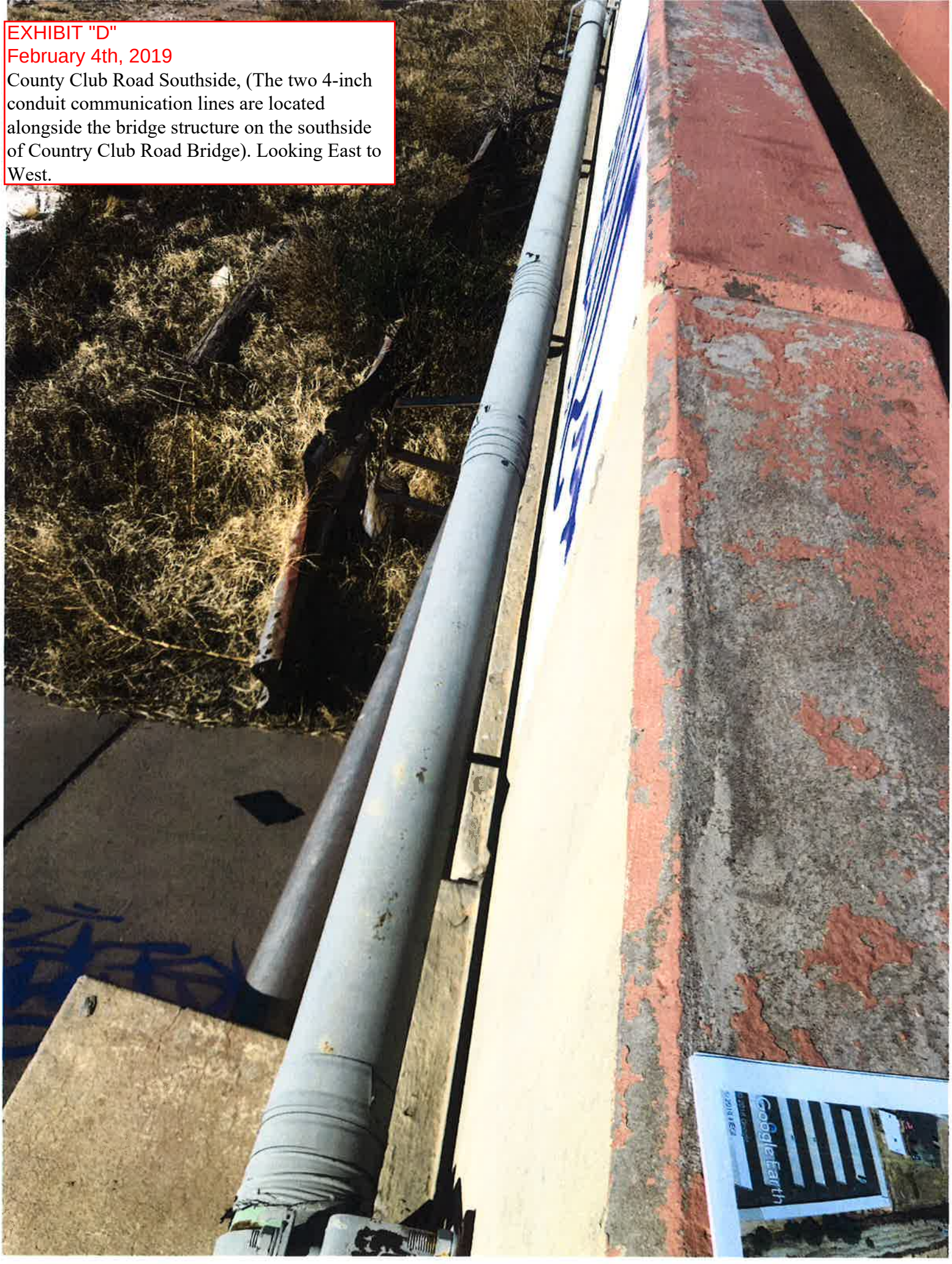


EXHIBIT "E"
February 4th, 2019
County Club Road Southside, (The two 4-inch conduit communication lines are located alongside the bridge structure on the southside of Country Club Road Bridge)

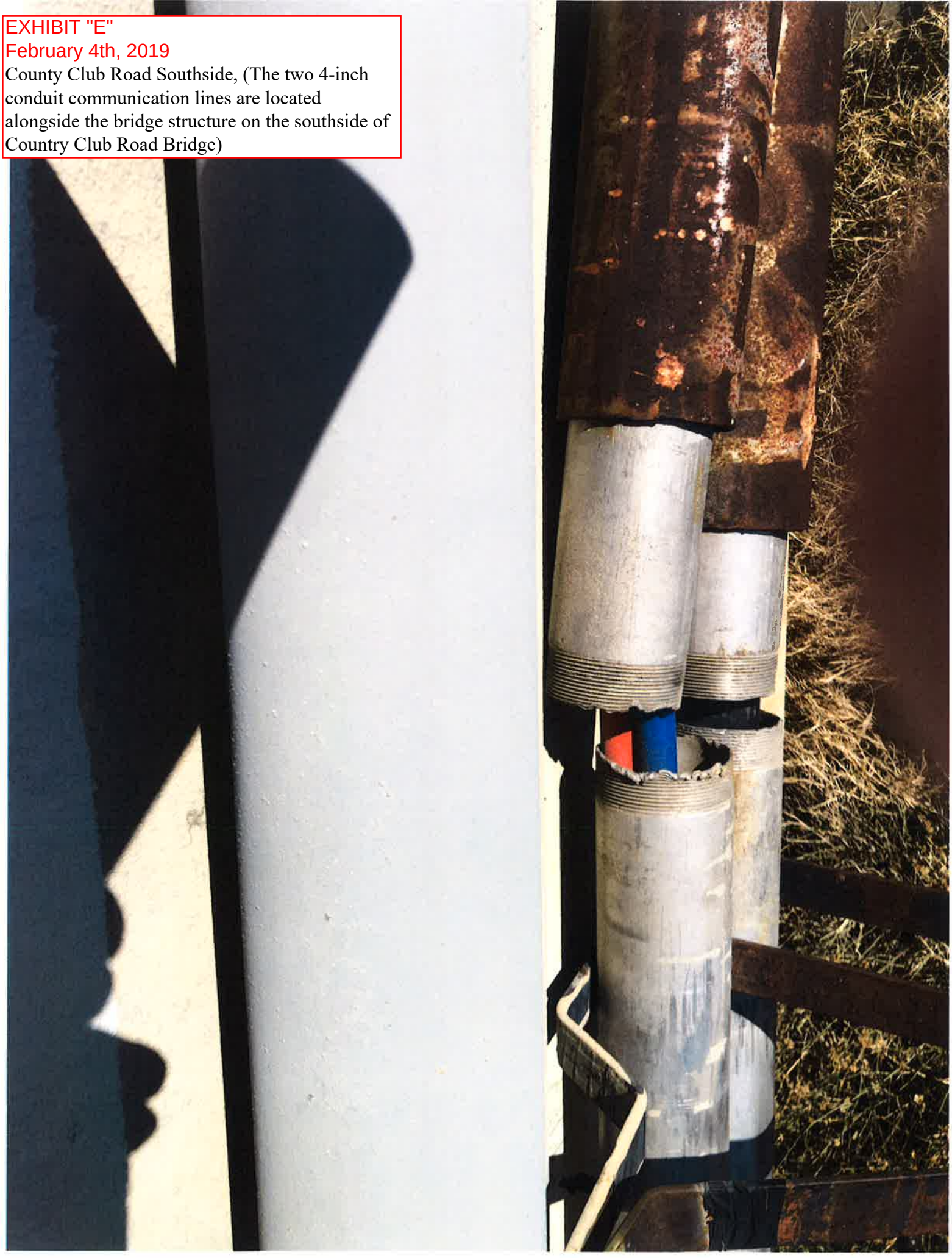
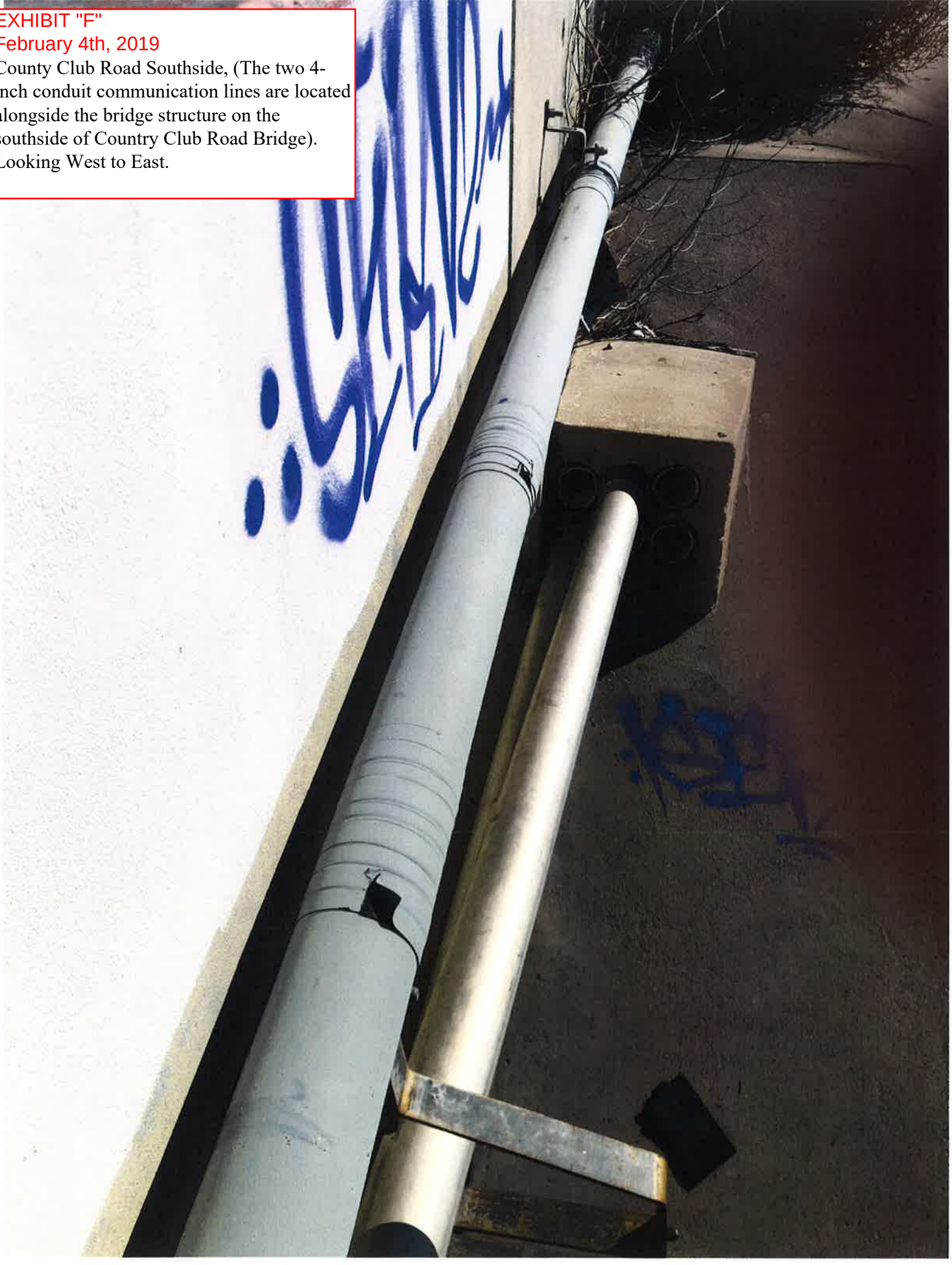
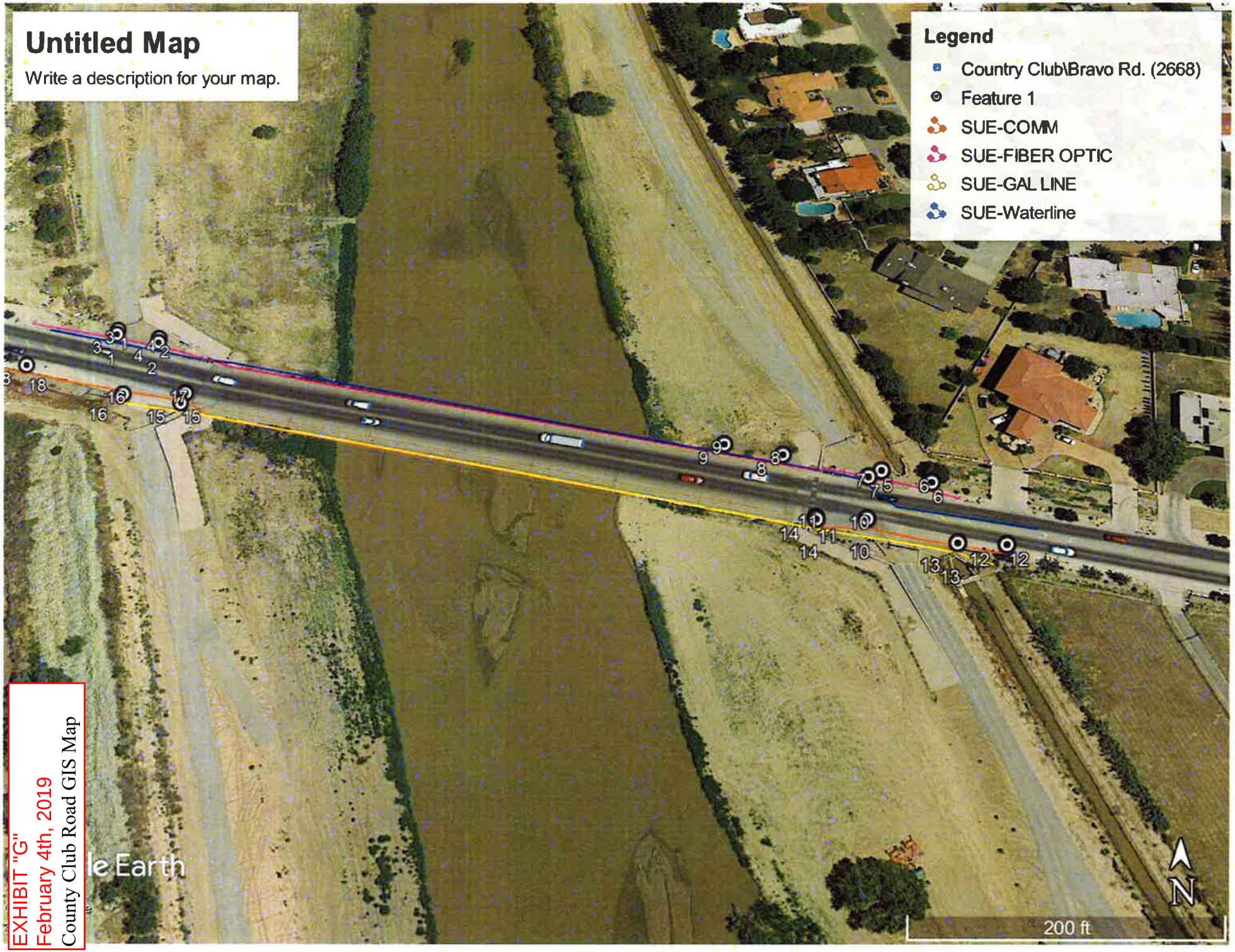


EXHIBIT "F"
February 4th, 2019
County Club Road Southside, (The two 4-inch conduit communication lines are located alongside the bridge structure on the southside of Country Club Road Bridge). Looking West to East.




Untitled Map
Write a description for your map.
Legend
Country Club\Bravo Rd. (2668)
Feature 1
SUE-COMM
SUE-FIBER OPTIC
SUE-GAL LINE
SUE-Waterline
EXHIBIT "G"
February 4th, 2019
County Club Road GIS Map
N
200 ft

CITY OF EL PASO (Fiber Optic)
PHOTOS OF AREA

Exhibit A – County Club Road Northside of bridge & East Levee Area (Looking East)

Exhibit B – County Club Road Northside of bridge & East Levee Area (Fiber Utility Boxes). Looking West.

Exhibit C – County Club Road Northside of bridge (Fiber Optic was found to be running alongside the northside of Country Club Road bridge) looking from East to West.

Exhibit D – County Club Road Northside & West Levee Area (Looking from West to East)

Exhibit E– County Club Road Northside of bridge & West Levee Area (Fiber Utility Boxes), (Looking East).

Exhibit F – County Club Road Northside of bridge (Fiber Optic was found to be running alongside the northside of Country Club Road bridge) looking from West to East.

Exhibit G – County Club Road Northside of bridge on West Levee Area. (Looking West).

EXHIBIT "A"
February 4th, 2019
County Club Road Northside of bridge
& East Levee Area (Looking East)

EXHIBIT "B"
February 4th, 2019
County Club Road Northside of bridge &
East Levee Area (Fiber Utility Boxes).
Looking West.

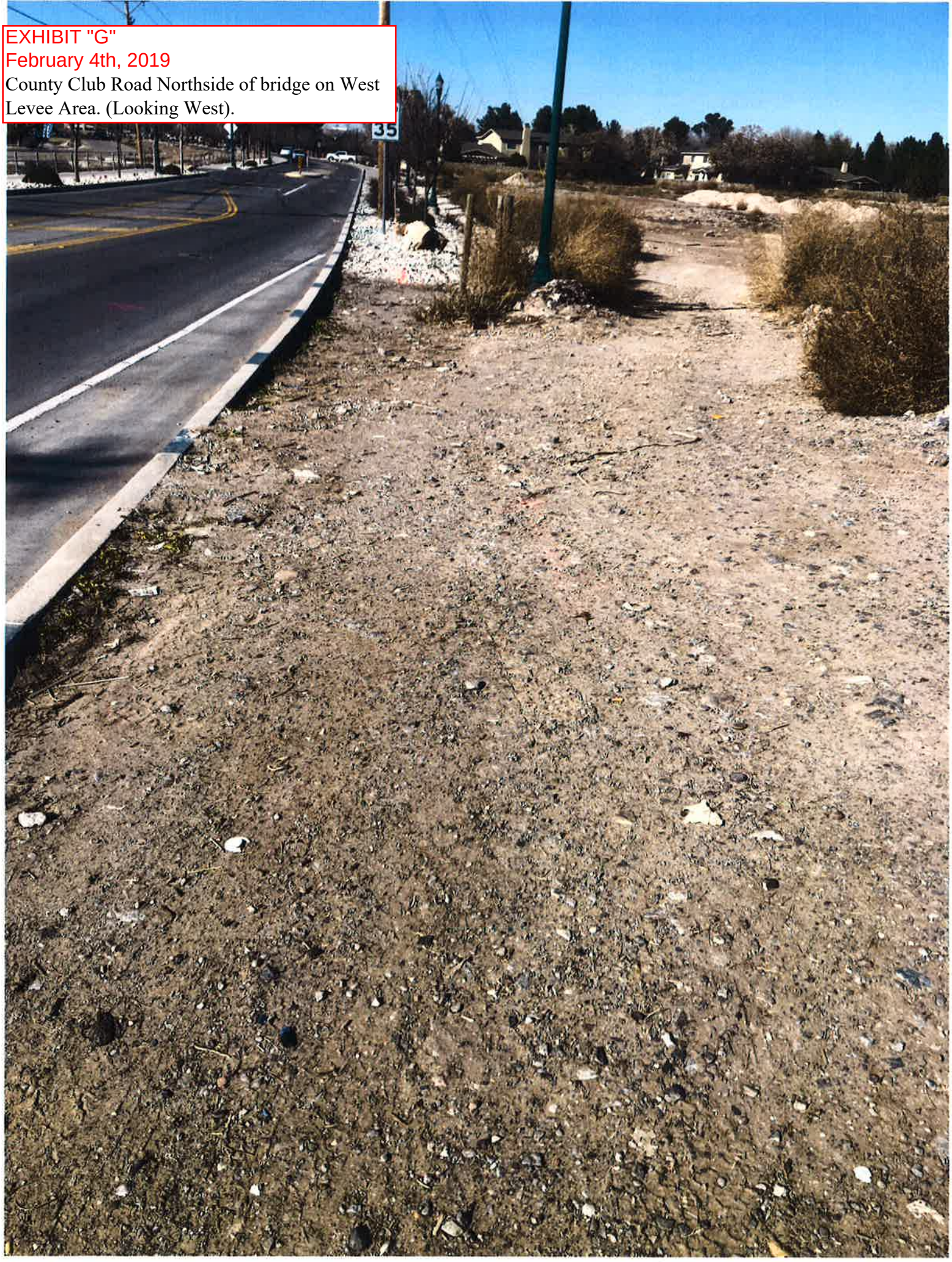
EXHIBIT "C"
February 4th, 2019
County Club Road Northside of bridge (Fiber Optic was found to be running alongside the northside of Country Club Road bridge) looking from East to West.

EXHIBIT "D"
February 4th, 2019
County Club Road Northside & West Levee Area (Looking from West to East)

EXHIBIT "E"
February 4th, 2019
County Club Road Northside of bridge & West Levee Area (Fiber Utility Boxes), (Looking East).

EXHIBIT "F"
February 4th, 2019
County Club Road Northside of bridge (Fiber Optic was found to be running alongside the northside of Country Club Road bridge) looking from West to East.

EXHIBIT "G"
February 4th, 2019
County Club Road Northside of bridge on West Levee Area. (Looking West).



TEXAS ONE GAS (Gas Main)
PHOTOS OF AREA

Exhibit A – County Club Road Southside & East Levee Area (The 4-inch Gas main is approximately located parallel to and on the southside of Country Club Road Bridge). Looking East to West.

Exhibit B – County Club Road Southside & East Levee Area. Looking East

Exhibit C – County Club Road Southside & East Levee Area (The 4-inch Gas main is approximately located parallel to and on the southside of Country Club Road Bridge).

Exhibit D – County Club Road Southside & West Levee Area (The 4-inch Gas main is approximately located parallel to and on the southside of Country Club Road Bridge). Looking West to East.

Exhibit E– County Club Road Southside & West Levee Area. Looking West

Exhibit F – County Club Road Southside & West Levee Area.

EXHIBIT "A"
February 4th, 2019
County Club Road Southside & East Levee Area (The 4-inch Gas main is approximately located parallel to and on the southside of Country Club Road Bridge). Looking East to West

EXHIBIT "B"
February 4th, 2019
County Club Road Southside & East
Levee Area. Looking East

EXHIBIT "C"
February 4th, 2019
County Club Road Southside & East Levee Area (The 4-inch Gas main is approximately located parallel to and on the southside of Country Club Road Bridge).

EXHIBIT "D"
February 4th, 2019
County Club Road Southside & West Levee Area (The 4-inch Gas main is approximately located parallel to and on the southside of Country Club Road Bridge). Looking West to East.

EXHIBIT "E"
February 4th, 2019
County Club Road Southside & West Levee Area. Looking West

EXHIBIT "F"
February 4th, 2019
County Club Road Southside & West Levee Area.

EL PASO WATER (Water Main)
PHOTOS OF AREA

Exhibit A – County Club Road Northside & East Levee Area (approximate location of a 12-inch water main crossing beneath the Rio Grande River). Looking East to West

Exhibit B – County Club Road Northside & East Levee Area (approximate location of a 12-inch water main crossing beneath the Rio Grande River). Looking East to West

Exhibit C – County Club Road Northside & East Levee Area (approximate location of a 12-inch water main crossing beneath the Rio Grande River). Looking East to West

EXHIBIT "A"
February 4th, 2019
County Club Road Northside & East Levee Area (approximate location of a 12-inch water main crossing beneath the Rio Grande River). Looking East to West

EXHIBIT "B"
February 4th, 2019
County Club Road Northside & East Levee Area (approximate location of a 12-inch water main crossing beneath the Rio Grande River). Looking East to West

EXHIBIT "C"
February 4th, 2019
County Club Road Northside & East Levee Area
(approximate location of a 12-inch water main crossing beneath the Rio Grande River). Looking East to West

COMMUNICATION

Exhibit A – FXSA to AT&T

Exhibit B – FXSA to City of El Paso

Exhibit C – Tri-State Electric, Ltd. to FXSA

Exhibit D – Tri-State Electric, Ltd. to FXSA

Exhibit E– Arcadis to FXSA

Exhibit F – 811 Ticket (12/3/18)

Exhibit G – 811 Ticket (12/3/18)

Exhibit H – 811 Ticket (12/3/18)

Exhibit I – 811 Ticket (12/3/18)

Exhibit J – 811 Ticket (12/20/18)

EXHIBIT "A"
FXSA to AT&T

James Gardenhire

From: Raymond M. Salazar
Sent: Tuesday, March 5, 2019 8:22 AM
To: MCKOWN, DIANA
Cc: John R. Barnes
Subject: Utility Records Search (AT&T Communication Lines)

Tracking:

Recipient	Read
MCKOWN, DIANA	
John R. Barnes	Read: 3/5/2019 10:32 AM

Good Morning Mrs. McKown,

Our phone conversation got cut off for some reason. I wanted to know if I can stop by your office today to try and obtain any PDF drawings of any AT&T lines that could be in possible conflict with a project we are working on. I have printed maps and a KMZ file for your viewing. Any help would be greatly appreciated. Please give me a call at your earliest convenience.

Thank You,

Frank X. Spencer & Associates, Inc.
Consulting Civil Engineers & Surveyors

Raymond M. Salazar
Associate Engineer
1130 Montana Ave., El Paso, Texas 79902
Ph: (915) 533.4600 • F: (915) 533.4673
rsalazar@fxsa.com

www.fxsa.com
TBPLS #100490-00 | 100490-01 •TBPE # F-3584

EXHIBIT "**B**"
FXSA to City of El Paso

James Gardenhire

From: Raymond M. Salazar
Sent: Wednesday, February 20, 2019 3:55 PM
To: linespots@elpasotexas.gov
Subject: Existing Utility Markings
Attachments: 65017-ITS-POE_Survey_12-10-18.kmz; kmz_65017-ITS-POE_1-31-19.kmz

Good Afternoon,

I would like to receive any As-Built drawings or CAD Files you may have relating to any existing utilities that are in the areas shown on attached KMZ files. There are several areas outlined in blue, green, and red. If none are available, paint markings would be acceptable. Please let me know if you have any questions or need more information.

Thank You,

Frank X. Spencer & Associates, Inc.
Consulting Civil Engineers & Surveyors

Raymond M. Salazar
Associate Engineer

1130 Montana Ave., El Paso, Texas 79902
Ph: (915) 533.4600 • F: (915) 533.4673
rsalazar@fxsa.com

www.fxsa.com
TBPLS #100490-00 | 100490-01 • TBPE # F-3584

EXHIBIT "C"
Tri-State Electric, Ltd. to FXSA

Good afternoon John,

As per your conversation with Manny, attached are the photos discussed.

A few notes below:

- The City of El Paso fiber optic line should be not be a problem to locate/expose in both directions as it marked on dirt areas and running towards the Type 1 ground box shown by the guardrail crash cushion. Additionally, the line is attached to the bridge deck within GRC conduit.
- The gas line could be located on dirt areas only – concrete paving cannot be potholed per City of El Paso requirements. The line appears to be attached to the bridge (within grey pipeline) although we cannot ascertain this. If utility line goes down the levee, this will pose a problem as there is no access.
- AT&T communication line runs into comm. manhole shown in the photo. The concreted area cannot be saw-cut/potholed. Line also appears to be attached along the bridge deck.
- Water line is marked in close proximity to the bridge approach slab which cannot be potholed or disturbed. Water line also appears to go down the levee on both sides of bridge where there is no access.

Just a reminder that Mr. Carreón can meet with you or your company representative this week on Tuesday 22nd 2019 or Wednesday 23rd 2019.

Hope this information proves useful.

Thank you,

Alex Motta
Tri-State Electric, Ltd.
530 Valley Chile Road
Vinton, TX 79821
(915) 755-8803
(915) 755-7276 Fax
amotta@tseltd.com

EXHIBIT "**D**"
Tri-State Electric, Ltd. to FXSA

Good afternoon John,

Please contact Manuel Carreón who is our Director of Operations in this regard. He may be reached at (915) 726-0889. He visited the site along Country Club Road yesterday and found a lot of complications for us to do the work.

The main concern is there is very limited access for entry of our machines (as Country Club is a two-lane road along the bridge), cattle gates to access the levee are closed as well as concrete restoration involved. Additionally, the City does not allow for us to saw-cut through concrete paving. There is also a crash cushion attenuator that may have to be removed and reassemble to expose a water line underneath.

Mr. Carreon mentioned he could to discuss these issues with you in greater depth.

Thank you,

Alex Motta

EXHIBIT "**E**"
Arcadis to FXSA

Frank,

Regarding the potholes on the West Levee of the C.C.Bridge, it appears that there should not be any utility conflict for the stop logs or the levee improvement (removing upper 2 ft of soil) in that area. Therefore, instead of the potholes, please confirm the utilities with the utility owners or through as built drawings. Please document all your communication and submit a report with your findings.

You can do the same for East Levee, if the same approach applies. We will send you the design for the stop logs on East Levee once complete but most likely will be the same as West Levee.

Thank you,

Ardita Dushi, PE, APMP | Project Manager | ardita.dushi@arcadis.com
12400 Coit Road, Suite 1200, Dallas TX | 75251 | USA
T. +1 972 663 2157 M. +1 214 673 1918

EXHIBIT "F"
811 Ticket (12/3/18)

James Gardenhire

From: tx@occinc.com
Sent: Monday, December 3, 2018 7:51 AM
To: James Gardenhire
Subject: Ticket: 583245443

Lone Star 811

Click the TicketLINK to view the entire ticket, request a 'Relocate', 'Cancel' a request, view attachments, check the utility responses, and provide access to all your other requests.

Ticket Summary

Ticket Number:	583245443	**NOTI-SURVEY**	**Type:** CALL
Work to Begin Date:	12/05/18	**Time:**	8:30 AM
County:	EL PASO	**Place:**	EL PASO
Address / Street:	COUNTRY CLUB RD		
Nearest Intersection:	HUNTERS GLENN ST & COUNTRY CLUB RD		
Type of Work:	SURVEY/DESIGN		
Explosives:	N	**Duration:**	24 HR
Marked in White:	N	**Digging deeper than 16in.?**	NO

Location of Work:

LOCATE 60 FEET NORTH AND SOUTH OF CROSSING. SEE MARKED MAP. DRIVING DIRECTIONS: FROM THE INTERSECTION OF COUNTRY CLUB RD AND HUNTERS GLENN ST. HEAD IN A AN EASTERLY DIRECTION FOR .2 MILES ON COUNTRY CLUB RD TO RIO GRANDE CROSSING THIS BRIDGE IS THE WORK AREA.

Remarks: ALT CONTACT EMAIL: JGAREDNHIRE@FXSA.COM

TESS: 1883714988 Seq: 1701 Rcd: 12/03/18 08:42 WTBD: 12/05/18 08:30

Caller Information

Requested By:	JAMES GARDENHIRE	**Phone:**	(915)533-4600
Company:	FRANK X. SPENCER & ASSOCIATES,	**Fax Phone:**	
Alt. Contact:	JAMES GARDENHIRE	**Phone:**	(915)533-4600
Email Address:	JGardenhire@fxsa.com		
Work Being Done For:	FXSA, INC.		

Members Notified

Your locate request was processed but no Lone Star 811 notification members were notified in this area.

Excavator Responsibilities

* **TEXAS PIPELINE DAMAGE PREVENTION LAWS REQUIRE: REFRESH LOCATE TICKET BEFORE 14 DAYS; CALL 811 ASAP TO REPORT PIPELINE EXCAVATION DAMAGE; SUBMIT ELECTRONIC PIPELINE DAMAGE REPORT TO TX RRC WITHIN 10 DAYS.**
* This ticket number serves as proof of your request.
* It is the excavator's responsibility to determine which utilities have, or have not, responded prior to beginning excavation. If any utility has not responded by 12/05/18 08:30AM you are required to contact the call center and make a 2nd request.
* Some water, sewer and storage facility owners may not be notified about your excavation. If there are any private underground facility operators in the excavation area you should contact them directly.
* Save time by clicking here to file your next locate request.

EXHIBIT "G"
811 Ticket (12/3/18)

James Gardenhire

From: tx@occinc.com
Sent: Monday, December 3, 2018 8:52 AM
To: James Gardenhire
Subject: Ticket: 583245999

Lone Star 811

Click the TicketLINK to view the entire ticket, request a 'Relocate', 'Cancel' a request, view attachments, check the utility responses, and provide access to all your other requests.

Ticket Summary

Ticket Number:	583245999	**NOTI-SURVEY**	**Type:** CALL
Work to Begin Date:	12/05/18	**Time:**	8:45 AM
County:	EL PASO	**Place:**	EL PASO
Address / Street:	RIO GRAND RIVER		
Nearest Intersection:	VIALE LUNGO AVE & STRAHAN RD		
Type of Work:	SURVEY/DESIGN		
Explosives:	N	**Duration:**	24 HR
Marked in White:	N	**Digging deeper than 16in.?**	NO

Location of Work:

LOCATE AN AREA OF 50'X100' MARKED ON MAP. DRIVING DIRECTIONS: FROM VIALE DEL SOL AVE & W BORDERLAND RD HEAD IN A EASTERLY DIRECTION FOR 900 FEET AND TURN RIGHT AND HEAD IN SOUTHERLY DIRECTION FOR .3 MILES TO WORK AREA. SEE MARKED MAP

Remarks: ALT CONTACT EMAIL: JGAREDNHIRE@FXSA.COM

TESS: 1883715483 Seq: 2095 Rcd: 12/03/18 09:00 WTBD: 12/05/18 08:45

Caller Information

Requested By:	JAMES GARDENHIRE	**Phone:**	(915)533-4600
Company:	FRANK X. SPENCER & ASSOCIATES,	**Fax Phone:**	
Alt. Contact:	JAMES GARDENHIRE	**Phone:**	(915)533-4600
Email Address:	JGardenhire@fxsa.com		
Work Being Done For:	FXSA, INC.		

Members Notified

Your locate request was processed but no Lone Star 811 notification members were notified in this area.

Excavator Responsibilities

* **TEXAS PIPELINE DAMAGE PREVENTION LAWS REQUIRE: REFRESH LOCATE TICKET BEFORE 14 DAYS; CALL 811 ASAP TO REPORT PIPELINE EXCAVATION DAMAGE; SUBMIT ELECTRONIC PIPELINE DAMAGE REPORT TO TX RRC WITHIN 10 DAYS.**
* This ticket number serves as proof of your request.
* It is the excavator's responsibility to determine which utilities have, or have not, responded prior to beginning excavation. If any utility has not responded by 12/05/18 08:45AM you are required to contact the call center and make a 2nd request.
* Some water, sewer and storage facility owners may not be notified about your excavation. If there are any private underground facility operators in the excavation area you should contact them directly.
* Save time by clicking here to file your next locate request.

EXHIBIT "**H**"
811 Ticket (12/3/18)

James Gardenhire

From: James Gardenhire
Sent: Thursday, December 13, 2018 8:10 AM
To: Saul Alva
Subject: FW: Ticket: 583245443

Tracking:

Recipient	Delivery	Read
Saul Alva	Delivered: 12/13/2018 8:10 AM	Read: 12/13/2018 8:15 AM

From: tx@occinc.com <tx@occinc.com>
Sent: Monday, December 3, 2018 7:51 AM
To: James Gardenhire <JGardenhire@fxsa.com>
Subject: Ticket: 583245443

Lone Star 811

Click the TicketLINK to view the entire ticket, request a 'Relocate', 'Cancel' a request, view attachments, check the utility responses, and provide access to all your other requests.

Ticket Summary

Ticket Number:	583245443	**NOTI-SURVEY**	**Type:** CALL
Work to Begin Date:	12/05/18	**Time:**	8:30 AM
County:	EL PASO	**Place:**	EL PASO
Address / Street:	COUNTRY CLUB RD		
Nearest Intersection:	HUNTERS GLENN ST & COUNTRY CLUB RD		
Type of Work:	SURVEY/DESIGN		
Explosives:	N	**Duration:**	24 HR
Marked in White:	N	**Digging deeper than 16in.?**	NO

Location of Work:

LOCATE 60 FEET NORTH AND SOUTH OF CROSSING. SEE MARKED MAP. DRIVING DIRECTIONS: FROM THE INTERSECTION OF COUNTRY CLUB RD AND HUNTERS GLENN ST. HEAD IN A AN EASTERLY DIRECTION FOR .2 MILES ON COUNTRY CLUB RD TO RIO GRANDE CROSSING THIS BRIDGE IS THE WORK AREA.

Remarks: ALT CONTACT EMAIL: JGAREDNHIRE@FXSA.COM

TESS: 1883714988 Seq: 1701 Rcd: 12/03/18 08:42 WTBD: 12/05/18 08:30

Caller Information

Requested By:	JAMES GARDENHIRE	**Phone:**	(915)533-4600
Company:	FRANK X. SPENCER & ASSOCIATES,	**Fax Phone:**	
Alt. Contact:	JAMES GARDENHIRE	**Phone:**	(915)533-4600
Email Address:	JGardenhire@fxsa.com		
Work Being Done For:	FXSA, INC.		

Members Notified

Your locate request was processed but no Lone Star 811 notification members were notified in this area.

Excavator Responsibilities

EXHIBIT "I"
811 Ticket (12/3/18)

James Gardenhire

From: James Gardenhire
Sent: Thursday, December 13, 2018 8:09 AM
To: Saul Alva
Subject: FW: Ticket: 583245999

Tracking:

Recipient	Delivery	Read
Saul Alva	Delivered: 12/13/2018 8:10 AM	Read: 12/13/2018 8:14 AM

From: tx@occinc.com <tx@occinc.com>
Sent: Monday, December 3, 2018 8:52 AM
To: James Gardenhire <JGardenhire@fxsa.com>
Subject: Ticket: 583245999

Lone Star 811

Click the TicketLINK to view the entire ticket, request a 'Relocate', 'Cancel' a request, view attachments, check the utility responses, and provide access to all your other requests.

Ticket Summary

Ticket Number:	583245999	**NOTI-SURVEY**	**Type:** CALL
Work to Begin Date:	12/05/18	**Time:**	8:45 AM
County:	EL PASO	**Place:**	EL PASO
Address / Street:	RIO GRAND RIVER		
Nearest Intersection:	VIALE LUNGO AVE & STRAHAN RD		
Type of Work:	SURVEY/DESIGN		
Explosives:	N	**Duration:**	24 HR
Marked in White:	N	**Digging deeper than 16in.?**	NO

Location of Work:

LOCATE AN AREA OF 50'X100' MARKED ON MAP. DRIVING DIRECTIONS: FROM VIALE DEL SOL AVE & W BORDERLAND RD HEAD IN A EASTERLY DIRECTION FOR 900 FEET AND TURN RIGHT AND HEAD IN SOUTHERLY DIRECTION FOR .3 MILES TO WORK AREA. SEE MARKED MAP

Remarks: ALT CONTACT EMAIL: JGAREDNHIRE@FXSA.COM

TESS: 1883715483 Seq: 2095 Rcd: 12/03/18 09:00 WTBD: 12/05/18 08:45

Caller Information

Requested By:	JAMES GARDENHIRE	**Phone:**	(915)533-4600
Company:	FRANK X. SPENCER & ASSOCIATES,	**Fax Phone:**	
Alt. Contact:	JAMES GARDENHIRE	**Phone:**	(915)533-4600
Email Address:	JGardenhire@fxsa.com		
Work Being Done For:	FXSA, INC.		

Members Notified

Your locate request was processed but no Lone Star 811 notification members were notified in this area.

Excavator Responsibilities

EXHIBIT "J"
811 Ticket (12/20/18)

James Gardenhire

From: tx@occinc.com
Sent: Thursday, December 20, 2018 1:30 PM
To: James Gardenhire
Subject: Ticket: 583418353

Lone Star 811

Click the TicketLINK to view the entire ticket, request a 'Relocate', 'Cancel' a request, view attachments, check the utility responses, and provide access to all your other requests.

Ticket Summary

Ticket Number:	583418353	**NOTI-SURVEY**	**Type:** CALL
Work to Begin Date:	12/24/18	**Time:**	1:15 PM
County:	EL PASO	**Place:**	EL PASO
Address / Street:	COUNTRY CLUB RD		
Nearest Intersection:	HUNTERS GLENN ST		
Type of Work:	SURVEY		
Explosives:	N	**Duration:**	24HR
Marked in White:	N	**Digging deeper than 16in.?**	NO

Location of Work:

LOCATE 100 FEET FROM CENTERLINE OF COUNTRY CLUB RD, 255 FEET TO THE EAST TO EDGE OF RIVER 100 FEET TO CENTERLINE OF CENTERLINE. SEE MARKED MAP. DRIVING DIRECTIONS: FROM THE INTERSECTION OF COUNTRY CLUB RD AND HUNTERS GLENN ST. HEAD IN A AN EASTERLY DIRECTION FOR .15 MILES ON COUNTRY CLUB RD TO RIO GRANDE CROSSING THIS BRIDGE IS THE WORK AREA.

Remarks: ALT CONTACT EMAIL: JGAREDNHIRE@FXSA.COM

TESS: 1885492455 Seq: 5158 Rcd: 12/20/18 13:32 WTBD: 12/24/18 13:15

Caller Information

Requested By:	JAMES GARDENHIRE	**Phone:**	(915)533-4600
Company:	FRANK X. SPENCER & ASSOCIATES,	**Fax Phone:**	
Alt. Contact:	JAMES GARDENHIRE	**Phone:**	(915)533-4600
Email Address:	JGardenhire@fxsa.com		
Work Being Done For:	FXSA, INC		

Members Notified

Your locate request was processed but no Lone Star 811 notification members were notified in this area.

Excavator Responsibilities

* **TEXAS PIPELINE DAMAGE PREVENTION LAWS REQUIRE: REFRESH LOCATE TICKET BEFORE 14 DAYS; CALL 811 ASAP TO REPORT PIPELINE EXCAVATION DAMAGE; SUBMIT ELECTRONIC PIPELINE DAMAGE REPORT TO TX RRC WITHIN 10 DAYS.**
* This ticket number serves as proof of your request.
* It is the excavator's responsibility to determine which utilities have, or have not, responded prior to beginning excavation. If any utility has not responded by 12/24/18 01:15PM you are required to contact the call center and make a 2nd request.
* Some water, sewer and storage facility owners may not be notified about your excavation. If there are any private underground facility operators in the excavation area you should contact them directly.
* Save time by clicking here to file your next locate request.

EXHIBIT "K"
811 Ticket (12/20/18)

James Gardenhire

From: tx@occinc.com
Sent: Thursday, December 20, 2018 1:33 PM
To: James Gardenhire
Subject: Ticket: 583418476

Lone Star 811

Click the TicketLINK to view the entire ticket, request a 'Relocate', 'Cancel' a request, view attachments, check the utility responses, and provide access to all your other requests.

Ticket Summary

Ticket Number:	583418476	NOTI-SURVEY	**Type:** CALL
Work to Begin Date:	12/24/18	**Time:**	1:30 PM
County:	EL PASO	**Place:**	EL PASO
Address / Street:	COUNTRY VLUB RD		
Nearest Intersection:	WILLOW CREEK DR & COUNTRY CLUB RD		
Type of Work:	SURVEY		
Explosives:	N	**Duration:**	24HR
Marked in White:	N	**Digging deeper than 16in.?**	NO

Location of Work:

LOCATE A 200' X 200'. SEE MARKED MAP. DRIVING DIRECTIONS: FROM THE INTERSECTION OF COUNTRY CLUB RD AND WILLOW CREEK DRIVE HEAD IN A AN WESTERLY DIRECTION FOR .11 MILES ON COUNTRY CLUB RD TO RIO GRANDE CROSSING THIS BRIDGE IS THE WORK AREA.

Remarks: ALT CONTACT EMAIL: JGAREDNHIRE@FXSA.COM

TESS: 1885492578 Seq: 5253 Rcd: 12/20/18 13:39 WTBD: 12/24/18 13:30

Caller Information

Requested By:	JAMES GARDENHIRE	**Phone:**	(915)533-4600
Company:	FRANK X. SPENCER & ASSOCIATES,	**Fax Phone:**	
Alt. Contact:	JAMES GARDENHIRE	**Phone:**	(915)533-4600
Email Address:	JGardenhire@fxsa.com		
Work Being Done For:	FXSA, INC		

Members Notified

Your locate request was processed but no Lone Star 811 notification members were notified in this area.

Excavator Responsibilities

* **TEXAS PIPELINE DAMAGE PREVENTION LAWS REQUIRE: REFRESH LOCATE TICKET BEFORE 14 DAYS; CALL 811 ASAP TO REPORT PIPELINE EXCAVATION DAMAGE; SUBMIT ELECTRONIC PIPELINE DAMAGE REPORT TO TX RRC WITHIN 10 DAYS.**
* This ticket number serves as proof of your request.
* It is the excavator's responsibility to determine which utilities have, or have not, responded prior to beginning excavation. If any utility has not responded by 12/24/18 01:30PM you are required to contact the call center and make a 2nd request.
* Some water, sewer and storage facility owners may not be notified about your excavation. If there are any private underground facility operators in the excavation area you should contact them directly.
* Save time by clicking here to file your next locate request.

Arcadis U.S., Inc.

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Suite 400

El Paso, Texas 79901

Tel 401 738 3887

www.arcadis.com