DESIGN CALCULATIONS & DRAWINGS

Three Barrel Culvert Replacement

90% SUBMITTAL FOR REVIEW

VERN FREEMAN DIVERSION CONVEYANCE SYSTEM

Ventura County, California

Gannett Fleming Project No. 067376

For

Northwest Hydraulic Consultants

August 14, 2023

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DESIGN OVERVIEW

INTRODUCTION

Provided herein are the 90% design criteria for the replacement of the three barrel culvert (TBC) road crossing over the Vern Freeman (Freeman) Diversion Conveyance System, owned and operated by the United Water Conservation District (United), in Ventura County, California. This report includes design information for the project that has been updated, revised, or progressed since the draft 30% design submittal (Ref. 1). The TBC redesign described herein includes changes and requirements outlined in reviews (Refs. 2 & 3) from the Ventura County Watershed Protection District (VCWPD).

REFERENCES

1. *Design Criteria & Drawings, Three Barrel Culvert, Draft 30% Submittal for Review, Vern Freeman Diversion Conveyance System, Ventura County, California, Gannett Fleming Project No. 067376*, prepared by Gannett Fleming, dated October 13, 2020.
2. *Permit 2021-019 – Project 8D103, Review of 30% Plans – Replacement of Santa Clara River Levee Penetration, Zone 2 – Freeman Diversion – Ventura County, California*, prepared by VCWPD, dated April 29, 2021.
3. *Permit 2021-019 – Project 8D103, Review of 30% Plans – Replacement of Santa Clara River Levee Penetration, Zone 2 – Freeman Diversion – Ventura County, California*, prepared by VCWPD, dated August 10, 2021.
4. *Design Drawings for Inverted Siphon Replacement at the Vern Freeman Diversion Conveyance System, Ventura County, CA*, (AFC Drawings), prepared by Gannett Fleming, dated June 10, 2022.
5. *VFD Three Barrel Culvert and Inverted Siphon Design – Alternatives Analysis Technical Memorandum*, prepared by NHC, dated July 16, 2020.
6. *United 3BC*, Email Correspondence from Ed Wallace (NHC) to Jennifer Allen (Gannett Fleming), prepared by NHC, dated November 6, 2022.
7. *Preliminary Geotechnical Evaluation – 30% Design, Vern Freeman Diversion System: Three Barrel Culvert & Inverted Siphon, United Water Conservation District Saticoy Facility, Ventura County, California, Project No. 67376*, prepared by Gannett Fleming, dated September 30, 2020.
8. *Lower River System, Main Supply Line, Culvert, STA 40+80.40 to STA 41+85.40*, As-Built Drawing (M-100-15), prepared by United, dated March 19, 1962.
9. *Canal Pothole Drawing – United Water Company*, Hand-Drawn Drawing Titled "United Pothole Drawing 404 Sec 4A", prepared by United, unknown date.
10. *RE: TBC 30% Design*, Email Correspondence from Bryce Cruey (NHC) to Jennifer Allen (Gannett Fleming), prepared by NHC, dated September 25, 2020.
11. *Sediment Transport and Deposition Assessment of the Freeman Diversion Conveyance System, Phase 1: Existing System Performance. Project 6000088,* prepared by NHC, dated January 7, 2015.
12. *Sediment Transport and Deposition Assessment of the Freeman Diversion Conveyance System, Phase 2: Evaluate Alternatives. Project 6000785*, prepared by NHC, dated September 1, 2016.
13. *EM 1110-2-2902 Conduits, Pipes, and Culverts Associated with Dams and Levee Systems, Engineer Manual,* prepared by USACE, dated December 31, 2020.

14. *Santa Clara River Levee and Appurtenances Foundation Investigation Record Drawing As-Constructed Cont. No. 59-159 Rev. Date 28 Jun 61,* prepared by USACE – Office of the District Engineer, Los Angeles CA.
15. *Lower River System Main Supply Line Culvert STA 40+80.40 to STA 41+85.40 Drawing C-2, M-100-15*, prepared by United, dated August 3, 1954 (marked As-Built 20 Feb 1956 and marked revised with Note 8 19 Mar 1962).
16. *HEC-RAS River Analysis System Version 5.07*, prepared by USACE – Hydrologic Engineering Center, 2016.

PROJECT BACKGROUND

Freeman Diversion Conveyance System hydraulics were analyzed by Northwest Hydraulic Consultants (NHC) in Phase 1 of a sediment transport assessment, and flow capacity restrictions were identified at United's existing TBC, inverted siphon facilities, and other facilities. The present design flow for the canal is 375 cubic feet per second (cfs), but United's goal is to increase the system-wide capacity to 750 cfs, thereby requiring improvements to the TBC and inverted siphon. Potential modifications to the TBC were identified in the Phase 2 evaluation of alternatives for improvement of the conveyance system (Ref. 12), and design is being completed in this project. NHC has been leading the hydraulic design and topographic survey efforts for the project, with Gannett Fleming being retained to perform geotechnical, structural, and civil design (Ref. 1). Construction plans for the inverted siphon replacement were completed in June 2022 (Ref. 4); the focus of this design report hereafter is for the replacement of the TBC facility.

In July 2020, NHC performed an alternatives analysis in conjunction with a hydraulic analysis of the redesign options for both the TBC and inverted siphon facilities (Ref. 5). Based on their analyses and discussions with United and Gannett Fleming, NHC proposed the most viable option for the TBC redesign as replacing the existing three barrel (round) culverts with two higher-capacity box culverts, including replacement of the existing flow control gates, as required by VCWPD.

In October 2020, NHC and Gannett Fleming prepared a Draft 30% Design Submittal package for United to provide to VCWPD (Ref. 1). VCWPD completed two sets of preliminary reviews between April and August 2021, in which they presented several considerations to United for revising the TBC's replacement to meet standards set by both Ventura County and the United States Army Corps of Engineers (USACE), particularly due to the Freeman system located within the USACE-regulated Santa Clara River levee system (Refs. 2 & 3). While re-evaluating the TBC's replacement, United elected to continue moving forward with the inverted siphon's replacement, thereby separating the construction plans for each facility within this project.

In November 2022, NHC completed their initial re-evaluation of their hydraulic design for the TBC and re-engaged Gannett Fleming to provide structural and civil design of the new design (Ref. 6). As part of the revised TBC project, NHC and Gannett Fleming coordinated with United to ensure existing features, utilities, and facilities around the TBC are accommodated for or are accordingly improved with the new system.

SUMMARY OF WORK

Gannett Fleming performed a preliminary geotechnical evaluation of the project site in September 2020 (Ref. 7; see **Attachment B**). Subsequent discussions with NHC and United regarding site conditions and desired redesign options established the geotechnical and structural design criteria for the TBC replacement.

Gannett Fleming coordinated with NHC and United to further incorporate the revised hydraulic design and VCWPD and USACE requirements/standards (Refs. 2 & 3). Through these coordination efforts and revised redesign considerations, Gannett Fleming prepared the design criteria outlined below for use in preparing the 90% revised design drawings (see **Attachment A**) and design calculations (see **Attachment D**) for the replacement of the TBC.

EXISTING CONDITIONS

Channel System

Flows in the existing channel are regulated at the Freeman Diversion, approximately 1.5 miles upstream of the TBC. The diversion is a concrete river intake equipped with a trash rack, closure slide gates, a fish screen, and channel-regulating slides gates. The diversion structure can isolate the channel system from the river during flood events. Flows in the diversion channel pass through a headworks and piping system and are diverted using two sets of slide gates through a desilting basin upstream of the TBC site. The desilting basin removes sediment from the flows to reduce maintenance and improve percolation in groundwater recharge basins located downstream of the TBC site. Flows are returned to the channel from the desilting basin through a set of slide gates. The existing channel is a rock-lined trapezoidal channel with a top width of approximately 60 feet and a depth of approximately 12 feet at the road crossing. The rock lining in the area immediately adjacent to the crossing is grouted in place. The existing TBC culverts are equipped with flap gates on the upstream end that are each secured in the open position by cables extending to a box (presumably containing a cable reel) mounted on concrete pedestals behind the headwall (**Figure 1**). The gates are not shown on as-built drawings for the structure dated 1956 (Ref. 15), but there is a note added in 1962 stating that the structure was modified in 1959 in conjunction with construction of the federal flood control levee. The gates are noted to be "…normally open [and] held by cables; manual release effects closure during floods".

United has no record of operation of the gates since installation.

Figure 1: Looking West at Existing Channel and Road Crossing

Flood Control Levee

The TBC site is located at the upstream end of the federal flood control levee, and the TBC culverts are a penetration through the levee to carry diversion flows from upstream of the levee into the area protected by the levee downstream. The levee was constructed by the USACE and is maintained by VCWPD. The levee location and top of levee profile are shown on a March 1959 foundation investigation drawing sheet for the Santa Clara River Levee (USACE, 1959). **Figure 2** shows the levee alignment in relation to the TBC crossing.

Figure 2: Levee Alignment and TBC Crossing (*Photo from Google Earth, 2019*)

Cross-Channel Flood/Drainage Ditch System

The road crossing of the channel serves the agricultural land to the south and provides access to a road along the south side of the channel upstream of the TBC site. This road is used by VCWPD to access a depression or linear basin (noted as a flood/drainage ditch in the design drawings) south of the channel that collects drainage from the steep northern slopes of South Mountain, as well as potential flood flows/runoff. Drainage flows pass through the basin and enter a 36-inch-diameter concrete pipe at a headwall near the intersection of the levee and road crossing of the channel with adjacent higher ground. This flow intersects with drainage flows from the agricultural property collected in a rectangular concrete cross-channel. Flow then passes through a series of rectangular concrete open channel segments and underground concrete pipes. The tops of the concrete channel segments are approximately at elevation 156[1] feet, the design top of the levee height. No documentation for the design of the drainage system was available for review. **Figure 3** shows the general configuration of the cross-channel drainage system.

[1] Unless otherwise stated, all elevations used in this report refer to the National Geodetic Vertical Datum of 1929 (NGVD29).

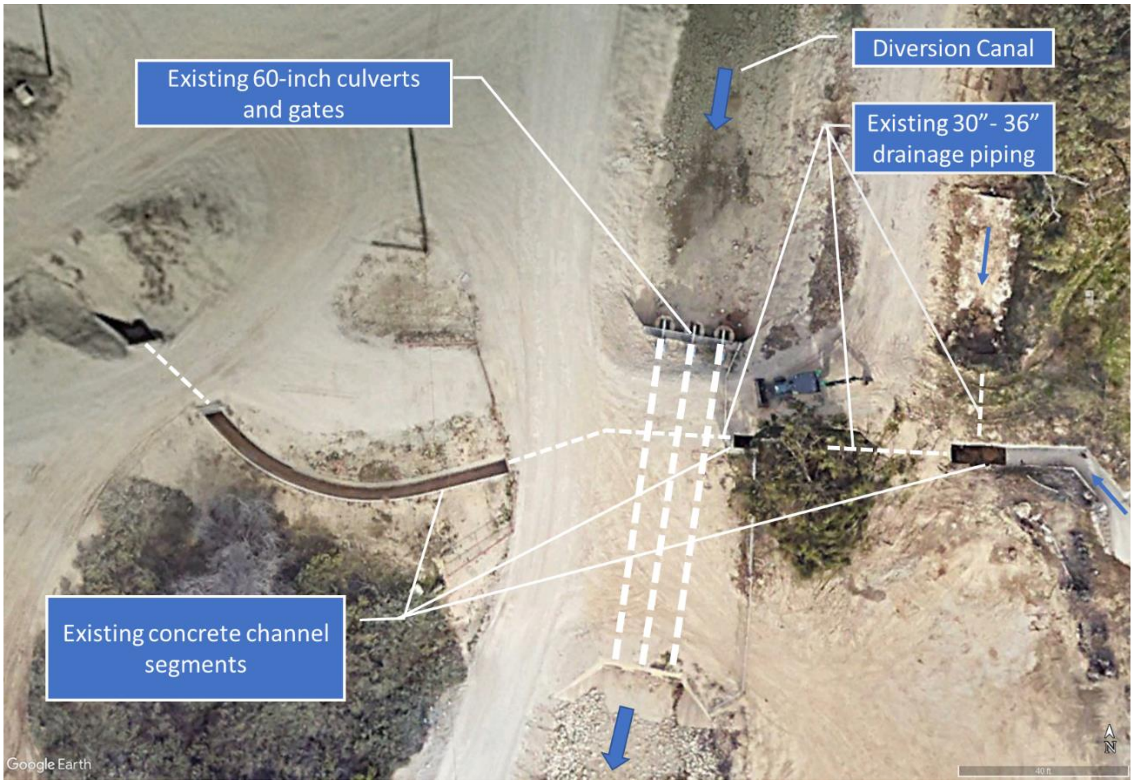

Figure 3: Cross-Channel Drainage System at TBC Site (*Photo from Google Earth, 2019*)

Other Utilities

Previous as-built drawings (Ref. 8) show two 18-inch-diameter gas lines crossing perpendicular to and underneath the existing TBC crossing. Based on correspondences with United (Refs. 9 & 10; see **Attachment C**), Gannett Fleming understands that the gas transmission has been relocated to a 22-inch-diameter pipeline that travels beneath the channel away from the existing TBC crossing. It is assumed that the 18-inch-diameter gas lines have been abandoned in place, but this will need to be verified by the contractor prior to construction.

An existing water line was also identified crossing perpendicular to the TBC. The water line will need to be verified by the contractor prior to construction and protected in place or relocated as appropriate.

DESIGN DESCRIPTION

Gannett Fleming is providing geotechnical, structural, and civil layout design services for the TBC and cross-channel pipeline. Any geotechnical or structural components that follow established design standards or guidelines (e.g., Caltrans standard plans, AWWA HDPE pipe design) will be confirmed to align with project needs and requirements. All other design services and design of other components (e.g., the steel gates) will be performed by others. Regrading of intersecting or approach roadways will be approved by United and conform with the grades on the design drawings (see **Attachment A**). Location, layout, and geometry of the new structures will generally match the existing structures to the extent practicable.

The project will involve the replacement of the three existing 5-foot-diameter "barrel" culverts with two higher-capacity cast-in-place (CIP) reinforced concrete box (RCB) culverts—also referred to as a double-box CIP RCB culvert—with each box having 14-foot spans and 7-foot heights. The new RCB culverts will be approximately 76 feet long, with an inlet invert elevation of 144.10 feet, a downgrade slope of 0.0006%, and an outlet invert elevation of 144.05 feet. Both the inlet and outlet end of the new RCB culverts will comprise new CIP reinforced concrete headwalls and wingwalls, with the footings also serving as the new aprons for transitioning between channel and culvert inverts. Per USACE 1110-2-2902 (Ref. 13), both inlet and outlet headwalls will accommodate new stainless steel slide gate systems (e.g., gate, frame, stem, actuator) for each culvert span (four gates in total). Both transitions between existing channel and new aprons will comprise a section of concreted rock slope protection (RSP) to better conform to existing channel dimensions and to mitigate seepage and scour potential.

Designs and layouts, as well as material and construction specifications and appropriate design codes and standards, for the TBC replacement are depicted in the design drawings (see **Attachment A**).

Cross-Channel Flood/Drainage Ditch System

An existing cross-channel system runs perpendicularly above the TBC and comprises open concrete channel structures and a 36-inch-diameter concrete pipe system. The existing 36-inch-diameter concrete pipe will be removed and replaced with two 30-inch-diameter, approximately 110-foot-long HDPE (high density polyethylene) pipes to accommodate the new TBC sizes. The invert elevations of the new pipes will be slightly higher than the existing concrete pipe; however, the HDPE pipes will maintain a slope that complements the flow of the existing system. The cross-channel pipeline will include new CIP reinforced concrete inlet and outlet structures that will replace the two existing transition structures to the open concrete channel. Some channel and roadway regrading will be required to provide sufficient soil cover for the new pipe system.

An extension of the new HDPE pipe system will be included to connect the new inlet structure of the cross-channel system with an existing Ventura County flood/drainage ditch that runs adjacent to the Freeman system. This HDPE pipe system extension, comprising two 30-inch-diameter and approximately 20-foot-long pipes, will involve a new CIP reinforced concrete headwall at the flood/drainage ditch end.

DESIGN CRITERIA AND LOADS

The following design criteria focuses on the geotechnical, structural, and civil aspects of the TBC replacement. NHC's hydraulic design and modeling design criteria are also described below. Applicable design codes, guides, and manuals are provided in the Design Procedure section of this report.

Hydraulic Design

Because the TBC road crossing penetrates the levee, USACE guidance (Ref. 13) applies for design of all the conduits. The most significant design requirements from USACE standards include:

- Precast reinforced concrete boxes are not permitted due to potential joint leakage.

- Conduit backfill must be designed to prevent soil loss around the conduits with differential water surfaces on the water and landsides of the levee.
- Gates must be provided for flood closure:
  - Gates are required for the upstream and downstream sides of the levee crossing.
  - USACE closure requirements provide for one active and one passive gate for fast rising flood scenarios and allow two active gates for slow rising flood scenarios; in the project's case, passive gates are not feasible because channel flow is into the protected area rather than out of it (as is typically the case for interior drainage applications).

The hydraulic design criteria for the TBC replacement are summarized as follows:

- Design Flow = 750 cfs
- Desired Velocity = similar to channel velocities to minimize entrance and exit losses
- Flow Control = two sets of gates to comply with USACE requirements for flood closure

United wishes to maximize capacity and minimize head loss at the crossing to accommodate the design flow and any physical or operational changes that may occur in the channel system in the future. Hydraulic design for replacement of the drainage system is intended to provide the same hydraulic capacity as the existing system. Several different pipe types are allowed by the USACE guidance (Ref. 13), with an emphasis on pipe design to prevent leakage and potential soil piping or loss due to seepage paths along the conduits.

Civil Design

Gannett Fleming's civil design and layout, including grading and work limits, are depicted in the design drawings in **Attachment A**. A maximum slope of 5% is determined for grading work along and adjacent to the road crossing over the TBC site to accommodate agricultural vehicle traffic, per discussions with NHC and United.

Geotechnical Design

For geotechnical and seismic design parameters, see **Attachment B** for Gannett Fleming's geotechnical evaluation (Ref. 7).

Structural Design

The following design criteria were used to guide the structural designs of the new culvert, headwalls, and wingwalls:

- Soil Unit Weight = 120 pcf
- Reinforced Concrete Unit Weight = 150 pcf
- Portable Reinforced Concrete Barrier = 900 pounds per linear foot (plf)
- Stainless Steel Slide Gate System = 10,000 pounds (maximum for one gate system)
- Vehicle Vertical Surcharge = 250 psf
- Load Factors per USACE EM 1110-2-2104, Table 3-1 (see Design Procedure section below for additional USACE engineering manual usage).

DESIGN PROCEDURE

Cast-in-Place Reinforced Concrete Box Culverts

The new double box CIP RCB culverts to be used for the TBC crossing redesign are to follow the 2022 Caltrans Standard Plans and Specifications for a CIP RCB double box culvert, as referenced in the design drawings. To accommodate the slide gate systems, modifications to the inner and outer wall thicknesses of the double box culvert were made and are noted in the design drawings. Though these modifications differ from the Caltrans Standard Plans, the intent of the standard design is maintained. Per requirements of VCWPD, adherence of applicable design guidelines and engineering manuals (EMs) set forth by USACE, including EM 1110-2-2100, EM 1110-2-2104, and EM 1110-2-2902, have also been confirmed or accommodated for, such as with modifications to reinforcing steel to meet USACE minimum reinforcement requirements.

Reinforced Concrete Retaining Headwalls and Wingwalls

Due to existing and anticipated soil conditions, slide gate system installation, and channel layout, the new proposed CIP reinforced concrete headwalls and wingwalls are designed the same for both the inlet and outlet end of the culvert. These new walls are to serve as standard retaining walls and will be designed in accordance with guidelines and codes set forth by the 2019 California Building Code (CBC) and ACI 318-19 from the American Concrete Institute (ACI). Per requirements of VCWPD, the new retaining walls are also designed in accordance with applicable USACE EMs, including EM 1110-2-2100, EM 1110-2-2104, and EM 1110-2-2902. The reinforced concrete footings of the new headwalls and wingwalls are also serving as the new aprons for the inlet and outlet ends of the culvert; the top of footings align with the invert elevations of the culvert and channel.

External stability of the new headwalls and wingwalls is provided by additional reinforcing tie-ins/connections with the new culvert. The headwalls comprise a stepped thickness design, with the thicker section aligned with the culvert's height to provide sufficient thickness/depth for reinforcing tie-ins/connections and embedment for the slide gate system anchor bolts. The footings for the new headwalls span the invert and side slopes of the channel to form the apron that serves as a transition for flow between the open channel and culvert. The wingwalls are designed to tie into the sloped extensions of the headwall footings. A typical keyway is also included on each toe end of the new footings as a "cutoff wall" for supplementing the concreted RSP's mitigation of seepage and scour potential at the transitions between existing open channel and new apron.

The thinner, upper section of the headwalls above the top of culvert are designed as a retaining wall, accounting for previously mentioned loads and spanning across the full width of the culvert. The thicker, lower section of the headwalls is not intended as a retaining wall and is instead meant to serve as a stable connection point for the overall culvert system, tying together the culvert, wingwalls, footings/aprons, and slide gate systems. The slide gate system will be bolted along the vertical culvert walls, fully attached to the entire height of the headwalls whose external stability is also tied with the culvert, wingwalls, and footings. The new headwalls have been designed with assumed accommodations for common slide gate systems similar to that needed for this project. Actual fabrication, installation, and layout design of the systems will adhere to the selected manufacturer requirements and instructions. The contractor will provide submittals of the selected gate systems to undergo engineering review in accordance with the design drawings (see **Attachment A**).

The cantilevered stem wingwalls, by contrast, are designed as full retaining walls with shared footings with the headwalls.

Portable concrete barriers are included along roadway edges near the new headwalls and wingwalls to provide protection to the new structures, as well as guide vehicle traffic along the adjacent roadways. The portable barriers will still allow access to the slide gate systems, as needed.

Cross-Channel Flood/Drainage Ditch System

A new CIP reinforced concrete headwall has also been included in the modifications to the existing Ventura County flood/drainage ditch. The new headwall will connect to the new inlet structure of the existing cross-channel system via the new HDPE pipes. This new headwall will be constructed per the 2022 Caltrans Standard Plans and Specifications for a CIP headwall with a pipe connection. Previously mentioned USACE EM guidelines have been confirmed or accommodated for with adjustments in the construction drawings for this additional headwall design.

The new CIP reinforced concrete inlet and outlet structures of the cross-channel system are in mostly box-shaped layouts and are designed to retain the surrounding soil and possible traffic loads. Both cross-channel structures are externally stabilized as they are buried on all applicable sides, with each structure's walls sharing the same footing, as well as the new walls and footings tying into the existing cross-channel's walls and inverts, respectively. The cross-channel inlet structure has two of its walls requiring pipe penetration detailing, while the cross-channel outlet structure only has one of its walls requiring pipe penetration detailing. The cross-channel inlet structure will also require new concrete fill to be placed in the invert, extending beyond the structure into the existing cross-channel to attain an invert elevation of 152.7 feet to maintain the flow into the HDPE pipes.

High Density Polyethylene (HDPE) Pipe Systems

The new HDPE pipe size was selected based on the increased size of the new culvert and the need to maintain gravity flow from the inlet and outlet structures of the cross-channel system, including the extension from the flood/drainage ditch. Comprising two 30-inch-diameter, smooth-surfaced (interior and exterior) HDPE pipes, the new pipe system is designed per guidelines established by the M55 Manual by the American Water Works Association (AWWA). Using AWWA M55's "Design Window", the 30-inch-diameter HDPE pipe design follows standardized parameters and properties with a standard dimension ratio (SDR) of 17.0, as shown in the design drawings. Additional considerations and requirements outlined by USACE EM 1110-2-2902 for pipe systems are confirmed or accommodated for in the AWWA-based design.

Controlled Low Strength Material (CLSM) Backfill

Per USACE EM 1110-2-2902, controlled low strength material (CLSM) is to be used for structural backfill around the HDPE pipes, as outlined in the design drawings. CLSM will provide a more durable backfill that also acts as an internal seepage filter for the HDPE pipes. Additionally, due to the proposed project layouts, the use of CLSM allows for more efficient placement of the HDPE pipes over the culvert crossing, as it is difficult to perform proper compaction of soil backfill around the span of the culvert.

Hydraulic Design

Several design options were originally considered, including a simple bridge and a precast arch culvert. However, the levee penetration requirements make these options infeasible. A series of culvert sizes were tested in an updated HEC-RAS (RAS) model (Ref. 16) of the channel system. The RAS model was derived from the model used to assess the channel conveyance system capacity and make recommendations for improvements (Ref. 12), modified to include proposed improvements at the inverted siphon structure approximately 0.2 miles downstream of the TBC site, and construction of a proposed flow split structure and new canal segment bypassing the Grand Canal and Saticoy Ponds approximately 0.4 miles downstream. These improvements are necessary to achieve the desired 750-cfs future flow capacity and are thus consistent with design flows for the TBC site.

A double-box RCB culvert is a hydraulically efficient option for replacement of the three 60-inch-diameter pipes. Double-box RCB culvert sizes between 10 feet wide by 6 feet high and 14 feet wide by 8 feet high were simulated in the RAS model. A constraint on the vertical height of the RCB is the need to pass the HDPE drainage pipes over the top of the RCB culverts. Heights greater than 7 feet appear to be impractical for this reason.

The selected double-box RCB culvert size has a span of 14 feet wide by 7 feet high, with headwalls at the upstream and downstream ends. United requested that the existing length of the RCB crossing be maintained. The twin RCB structure will be slightly wider than the existing canal bed, requiring angled wingwalls to avoid slope grading that would encroach on existing roads. The closure gates will be fabricated stainless steel slide gates and will be installed on the headwalls. The gates will have manual operators, with provisions for portable electric operators to also be used. Based on coordination with potential gate manufacturers, the selected gate size is also the maximum that can practically be manufactured with the design operating head at the elevation of the adjacent road (elevation 156 feet). This elevation also approximately corresponds to the USACE design top of levee elevation.

RAS modeling results for various options are included in **Attachment D**. The computed design water levels and velocities for the selected option are shown in **Table 1**. Culvert hydraulic computations use entrance and exit loss coefficients of 0.5, and contraction and expansion loss coefficients of 0.3 and 0.5, respectively. Friction in the culverts is represented by a Manning's n-value of 0.02.

Table 1. Hydraulic Characteristics of Double-Box 14' x 7' RCB Culvert

Flow (cfs)	Upstream Water Surface Elevation (feet)	Downstream Water Surface Elevation (feet)	Culvert Velocity (feet/second)	Upstream Channel Velocity (feet/second)
750	151.40	151.23	3.8	3.7
500	149.89	149.94	3.1	3.5
375	149.12	149.15	2.7	3.3

The conveyance of the proposed cross-channel drainage system replacement was designed to be equal to or greater than the existing concrete pipes using an n-value of 0.013 for existing concrete pipes and 0.011 for new HDPE pipes. The new drainage system will operate at a higher elevation to clear the proposed RCB culverts. Inlet and outlet losses will be reduced compared to the existing system by eliminating one segment of rectangular channel. The capacity of the two new HDPE pipes is estimated to be 39.4 cfs (flowing full). Computations for the cross-channel drainage system capacity are also included in **Attachment D**.

LIMITATIONS

This report has been prepared for only the replacement and redesign of the TBC facility described herein. Information related to the replacement of the inverted siphon facility associated with the Freeman Diversion Conveyance System is exclusive to this report's conclusions and recommendations.

This report has been prepared for the sole use of NHC, United, and their respective agents, specifically for design of the proposed improvements at the project site referenced on the cover of this report. The conclusions and recommendations contained in this report are based upon the information obtained from the references listed above. Gannett Fleming is not responsible for the data presented by others.

The information provided in this report is valid as of the date shown on the cover page for the designs described herein. Structural issues may arise that were not apparent at the time of this design (e.g., changes in design geometries, soil design parameters, loadings, etc.). Accordingly, the information provided in this report may be invalidated, wholly or partially, by changes outside of Gannett Fleming's control. Should changes occur that might affect the design presented herein, Gannett Fleming should be notified to evaluate the validity of this report to those changes.

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ATTACHMENT A – 90% DESIGN DRAWINGS

The reduced size (11" x 17") plans provided herein are intended as reference documents.

UNITED WATER CONSERVATION DISTRICT

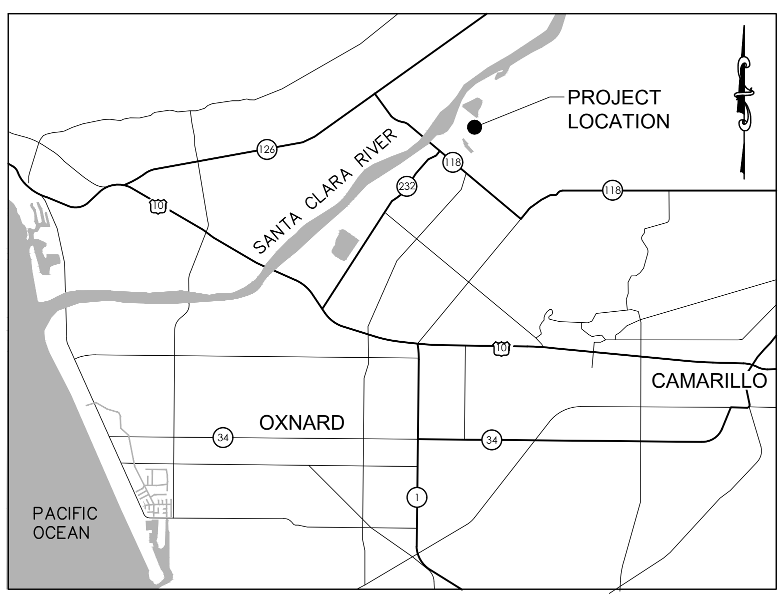
90% DESIGN DRAWINGS FOR

THREE BARREL CULVERT REPLACMENT

AT THE

VERN FREEMAN DIVERSION CONVEYANCE SYSTEM VENTURA COUNTY, CA

LOCATION MAP
NOT TO SCALE



AREA MAP
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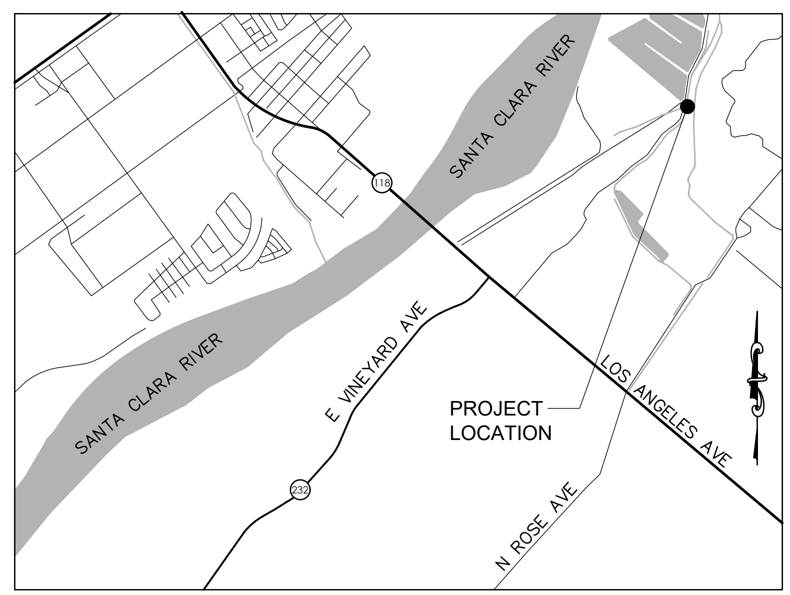

VICINITY MAP
NOT TO SCALE

SHEET INDEX

SHT NO.	DWG NO.	TITLE
1	G1	TITLE SHEET
2	G2	NOTES
3	G3	ABBREVIATIONS
4	C1	DEMOLITION PLAN
5	C2	NEW CULVERT PLAN
6	C3	NEW CULVERT PROFILE & SECTIONS
7	S1	PARTIAL PLAN & SECTION
8	S2	SECTIONS
9	S3	DETAILS
10	S4	SECTIONS

REFERENCES

1. LOWER RIVER SYSTEM, MAIN SUPPLY LINE, CULVERT, STA 40+80.40 TO STA 41+85.40, (AS-BUILT DWG, M-100-15), PREPARED BY UNITED WATER CONSERVATION DISTRICT, REV 2, DATED 3/19/1962.
2. TBC 30% DESIGN, EMAIL CORRESPONDENCE WITH "UNITED POTHOLE DRAWING 404 SEC 4A" HAND DRAWING FROM BRYCE CRUEY (NHC) TO JENNIFER ALLEN (GANNETT FLEMING), PREPARED BY NHC, DATED 9/25/2020.

PROJECT CONTACTS

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ENGINEER OF RECORD
GANNETT FLEMING INC.
JENNIFER S. ALLEN
PHONE: (510) 768-0180

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90% NOT FOR CONSTRUCTION

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REV	BY	DATE	DESCRIPTION

TITLE SHEET
THREE BARREL CULVERT
VERN FREEMAN DIVERSION CONVEYANCE SYSTEM
SANTA PAULA VENTURA COUNTY CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER JENNIFER S. ALLEN CIVIL STATE OF CALIFORNIA
NOT FOR CONSTRUCTION

DATE: 08/14/23
SCALE: AS SHOWN
DESIGNED BY: RC/SMU/WLM
DRAFTED BY: P. BARBER
CHECKED BY: JSA/TRS
JOB NO.: 067376
FILE: 067376 001.dwg

G1
SHEET 1 OF 10

PLOT DATE: Monday, August 14, 2023 TIME: 6:33:21 PM BY: CONRAD, RYAN CTB: ---- TAB: G1
FILE: C:\Users\rconrad\OneDrive - Gannett Fleming Inc\504-CADD-067376-NHC_FreemanDiv\067376 001.dwg

GENERAL NOTES

1. ALL WORK SHALL BE IN ACCORDANCE WITH THE CONTRACT DOCUMENTS.
2. THE NOTES PROVIDED ON THESE DRAWINGS DO NOT REPRESENT A COMPLETE DESCRIPTION OF THE WORK TO BE PERFORMED AND ARE INTENDED TO COMPLEMENT THE SPECIFICATIONS AND PLANS. TECHNICAL SPECIFICATIONS AND PLANS ASSOCIATED WITH THIS WORK ARE AS FOLLOWS:
   A. CALTRANS 2022 STANDARD PLANS, OR MOST RECENT UPDATE:
   - PLAN A62E: EXCAVATION AND BACKFILL CAST-IN-PLACE REINFORCED CONCRETE BOX AND ARCH CULVERTS
   - PLANS A63A AND A63B: PORTABLE CONCRETE BARRIER (TYPE 60K)
   - PLAN B0-3: BRIDGE DETAILS
   - PLAN D81: CAST-IN-PLACE REINFORCED CONCRETE DOUBLE BOX CULVERT
   - PLAN D89A: PIPE CULVERT HEADWALLS STRAIGHT AND "L"
   - REVISED PLAN D82: CAST-IN-PLACE REINFORCED CONCRETE BOX CULVERT MISCELLANEOUS DETAILS

   B. CALTRANS 2022 STANDARD SPECIFICATIONS, OR MOST RECENT UPDATE:
   - SECTION 19-3: STRUCTURE EXCAVATION AND BACKFILL
   - SECTION 51: CONCRETE STRUCTURES
   - SECTION 68-4: EDGE DRAINS
   - SECTION 68-7: GEOCOMPOSITE DRAIN SYSTEMS
   - SECTION 72-2: ROCK SLOPE PROTECTION
   - SECTION 72-3: CONCRETED-ROCK SLOPE PROTECTION
   - SECTION 83-1: GENERAL (FOR RAILINGS AND BARRIERS)
   - SECTION 83-3: CONCRETE BARRIERS
3. IN CASE OF CONFLICT BETWEEN THE CONSTRUCTION DOCUMENTS AND CALTRANS STANDARD SPECIFICATIONS AND PLANS, NOTIFY THE OWNER AND THE ENGINEER.
4. THE CONTRACTOR IS RESPONSIBLE FOR ALL CONSTRUCTION MEANS AND METHODS.
5. LOCATIONS AND DIMENSIONS OF EXISTING STRUCTURES AND FEATURES HAVE NOT BEEN VERIFIED. VERIFY ALL CONTROLLING DIMENSIONS OF NEW AND EXISTING FEATURES PRIOR TO ORDERING OR FABRICATING MATERIAL OR CONSTRUCTING PROPOSED IMPROVEMENTS. REPORT ANY DISCREPANCIES TO THE ENGINEER PRIOR TO PROCEEDING WITH CONSTRUCTION OF THE FEATURE IN QUESTION.
6. PRIOR TO THE START OF CONSTRUCTION, LOCATE ALL EXISTING AND UNDERGROUND UTILITIES IN AND AROUND THE AREAS OF NEW CONSTRUCTION. VERIFY THAT THE PROPOSED CONSTRUCTION DOES NOT CONFLICT WITH EXISTING OR PROPOSED UTILITIES OR THAT APPROPRIATE MEANS ARE PROVIDED FOR REROUTING, SUPPORTING, PROTECTING, OR OTHERWISE INCORPORATING THE UTILITIES INTO THE CONSTRUCTION.
7. NOTIFY THE OWNER AND ENGINEER WHERE A CONFLICT OR DISCREPANCY OCCURS BETWEEN THESE DRAWINGS AND ANY OTHER PORTION OF THE CONTRACT DOCUMENTS OR EXISTING FIELD CONDITIONS.
8. PRODUCTS REFERENCED IN THE DRAWINGS AND SPECIFICATIONS SHALL BE CONSTRUCTED, INSTALLED, AND APPLIED IN ACCORDANCE WITH THE MANUFACTURER'S WRITTEN RECOMMENDATIONS UNLESS OTHERWISE NOTED.
9. DO NOT SCALE DRAWINGS. CONTACT THE ENGINEER FOR ANY DIMENSIONS OR SPECIFIC DETAIL NOT SHOWN.
10. THE CONTRACTOR SHALL MAINTAIN RECORDS SUITABLE FOR DEVELOPING "AS-BUILT" DRAWINGS THROUGHOUT THE COURSE OF CONSTRUCTION, INCLUDING, BUT NOT LIMITED TO, THE LOCATIONS AND GRADES OF ALL UNDERGROUND AND SURFACE IMPROVEMENTS. THESE RECORDS SHALL BE MARKED IN RED (INCLUDE), GREEN (REMOVE), BLUE (COMMENTS/DIRECTIONS) STANDARD FORMAT. THESE RECORDS SHALL BE DELIVERED TO THE OWNER PRIOR TO THE ACCEPTANCE OF WORK.

DESIGN BASIS AND LOADING

1. THE DESIGNS DEPICTED IN THESE DRAWINGS ARE BASED ON INFORMATION PROVIDED IN THE FOLLOWING REFERENCES:
   A. TOPOGRAPHIC BASE MAP, PROVIDED BY STANTEC, DATED JUNE 11, 2020.
   B. PRELIMINARY GEOTECHNICAL MEMORANDUM, PROVIDED BY GANNETT FLEMING, DATED SEPTEMBER 30, 2020.
2. DESIGN IS IN ACCORDANCE WITH THE FOLLOWING CODES AND STANDARDS:
   A. ACI 318-19, BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE
   B. 2019 CALIFORNIA BUILDING CODE
   C. USACE ENGINEER MANUALS:
   - EM 1110-2-2100: STABILITY ANALYSIS OF CONCRETE STRUCTURES, DATED 12/1/2005.
   - EM 1110-2-2104: STRENGTH DESIGN FOR REINFORCED CONCRETE HYDRAULIC STRUCTURES, DATED 11/30/2016.
   - EM 1110-2-2902: CONDUITS, PIPES, AND CULVERTS ASSOCIATED WITH DAMS AND LEVEE SYSTEMS, DATED 12/31/2020.

   D. AWWA MANUAL M55, PE PIPE – DESIGN AND INSTALLATION, FIRST EDITION
   E. CALTRANS 2022 STANDARD PLANS AND REVISED STANDARD PLANS
3. GEOTECHNICAL DESIGN PARAMETERS PER GEOTECHNICAL MEMORANDUM (SEE DESIGN BASIS AND LOADING NOTE 1B).
4. DESIGN LOADS:
   - DEAD LOADS :
     - REINFORCED CONCRETE = 150 PCF
     - PORTABLE CONCRETE BARRIER = 900 PLF
     - STAINLESS STEEL SLIDE GATE SYSTEM = 10,000 POUNDS
   - LIVE LOADS :
     - VEHICLE TRAFFIC SURCHARGE = 250 PSF
   - LATERAL EARTH PRESSURES :
     - SOIL UNIT WEIGHT = 120 PCF
5. SEISMIC DESIGN PARAMETERS:
   - SDS = 1.292g
   - PEAK GROUND ACCELERATION = 0.943g
   - SITE CLASS = D (STIFF SOIL)
   - RISK CATEGORY = II
6. STRUCTURES HAVE BEEN DESIGNED FOR OPERATIONAL LOADS ON THE COMPLETED STRUCTURES ONLY. PROTECT AND STABILIZE STRUCTURES AS NECESSARY DURING CONSTRUCTION AND UNTIL DESIGN STRENGTHS ARE ACHIEVED.

SURVEY

1. HORIZONTAL COORDINATE SYSTEM: NAD83 CALIFORNIA STATE PLANE, ZONE 5.
2. VERTICAL DATUM: NGVD29.
3. ALL UNITS SHOWN IN ARE IN U.S. SURVEY FEET.
4. CONTRACTOR TO PROVIDE FIELD SURVEYING FOR PROJECT LAYOUT AND CONTROL.
5. AERIAL PHOTOGRAPHS ARE FROM GOOGLE EARTH AND ARE SOLELY FOR ILLUSTRATIVE PURPOSES. IMAGERY REFERENCES PROVIDED ON RELEVANT SHEETS.

DEWATERING

1. THE CANAL SYSTEM IN THE PROJECT VICINITY WILL BE DEWATERED BY THE OWNER FOR THE DURATION OF THE WORK.
2. GROUNDWATER AND/OR SURFACE WATER MAY BE ENCOUNTERED DURING EXCAVATION AND SUBGRADE PREPARATION. THE CONTRACTOR IS RESPONSIBLE FOR DEWATERING AS NECESSARY TO MAINTAIN STABLE AND CLEAN EXCAVATIONS. DIRECT DISCHARGE OF AFOREMENTIONED GROUNDWATER/SURFACE WATER INTO STREAMBED IS NOT PERMISSIBLE. FINAL DISCHARGE POINTS FOR ALL DEWATERING SHALL BE APPROVED BY UNITED WATER CONSERVATION DISTRICT.
3. ALL PERMANENT IMPROVEMENTS SHALL BE CONSTRUCTED IN THE DRY.
4. CONTRACTOR SHALL PROVIDE A DEWATERING PLAN PRIOR TO CONSTRUCTION FOR REVIEW AND APPROVAL BY THE ENGINEER.

EXCAVATION

1. NOTIFY UNDERGROUND SERVICE ALERT (USA SOUTH) TO IDENTIFY THE LOCATION OF EXISTING UTILITIES AT LEAST TWO WORKING DAYS PRIOR TO ANY EXCAVATION WORK: (800)-422-4133 OR WWW.DIGALERT.ORG.
2. LOCATIONS AND LAYOUTS OF EXISTING UTILITIES, BOTH ACTIVE AND ABANDONED, ARE BASED ON PROVIDED TOPOGRAPHY AND REFERENCES ON SHEET G1. FIELD VERIFICATION IS REQUIRED PRIOR TO PROJECT CONSTRUCTION.
3. THE CONTRACTOR IS RESPONSIBLE FOR STABILITY AND SHORING OF TEMPORARY CUT SLOPES AND TRENCHES, AND SHALL CONFORM TO THE REQUIREMENTS OF CAL-OSHA.
4. EXCAVATIONS SHALL BE KEPT CLEAN AND DRY.

DEMOLITION

1. PROTECT EXISTING FEATURES THAT ARE TO REMAIN IN PLACE FROM DAMAGE UNLESS OTHERWISE NOTED.

FOUNDATION

1. CONTRACTOR SHALL BE PREPARED FOR POSSIBLE GRAVEL, COBBLES AND/OR ROCKFILL SUBGRADE CONDITIONS.
2. THE SUBGRADE SURFACE SHALL COMPRISE FIRM, NON-YIELDING MATERIALS. SHOULD OVER-EXCAVATION BE NEEDED TO REACH FIRM MATERIAL, BACKFILL SUBGRADE UP TO DESIGN GRADE

EARTHWORK

1. REFER TO CALTRANS 2022 STANDARD PLAN A62E FOR EARTHWORK RELATED TO THE CULVERTS. UNLESS OTHERWISE NOTED, ALL EARTHWORK SHALL ADHERE TO SECTION 19-3 OF THE CALTRANS 2022 STANDARD SPECIFICATIONS.
2. PREPARE DESIGNATED FILL AREAS BY GRUBBING AND STRIPPING VEGETATION, REMOVING DEBRIS, AND SCARIFYING TO A MINIMUM DEPTH OF 8 INCHES PRIOR TO MATERIAL PLACEMENT.
3. UNLESS OTHERWISE NOTED, COMPACT FILL MATERIAL IN 8-INCH LOOSE LIFTS AND COMPACT TO AT LEAST 95% RELATIVE COMPACTION (RC) AT A MOISTURE CONTENT AT LEAST 2% OVER OPTIMUM PER ASTM D1557 AND D6938.
4. ONSITE FILL MATERIAL SHALL BE FREE OF ORGANIC MATERIAL (LESS THAN 3% BY VOLUME) AND SHOULD NOT CONTAIN ANY PARTICLES GREATER THAN 3" IN DIAMETER.
5. IMPORTED FILL MATERIAL SHALL BE FREE OF ORGANICS (LESS THAN 3% BY VOLUME), DEBRIS, HAVE AT LEAST 20% FINES AND NO PARTICLES GREATER THAN 3" IN DIAMETER PER ASTM D6913, AND HAVE A PLASTICITY INDEX OF 12 OR LESS PER ASTM D4318.
6. PERFORM GRADING TO THE LINES AND GRADES SHOWN. FINISHED SURFACES SHALL PROVIDE POSITIVE SURFACE DRAINAGE TO PREVENT PONDING.
7. SEE HDPE PIPE NOTES, THIS SHEET.

TEMPORARY ENVIRONMENTAL/EROSION CONTROL

1. CONTAIN SURFACE RUNOFF AND CEMENTITIOUS MATERIAL DURING CONSTRUCTION TO PREVENT CONTAMINATION OF GROUND AND SURFACE WATERS.
2. MAINTAIN THE SITE AND ADJACENT PROPERTY IN A CLEAN, SAFE, AND USABLE CONDITION. ALL SPOILS OF SOIL, ROCK, OR CONSTRUCTION DEBRIS SHALL BE PROMPTLY REMOVED.
3. IMPLEMENT EROSION AND SEDIMENT CONTROL PLANS AND BEST MANAGEMENT PRACTICES AS REQUIRED BY LOCAL AND STATE JURISDICTIONS.

DRAINAGE

1. WEEP HOLES, OR DRAINAGE PIPES, SHALL BE PER BRIDGE DETAIL 3-1 OF PLAN B0-3 OF THE CALTRANS 2022 STANDARD PLANS, OAE.
2. DRAINAGE PIPES AND FILTER FABRIC SHALL COMPLY WITH SECTION 68-7 OF THE CALTRANS 2022 STANDARD SPECIFICATIONS. DRAINAGE PIPES SHALL ALSO COMPLY WITH SECTION 68-4.02B OF THE CALTRANS 2022 STANDARD SPECIFICATIONS.
3. PERVIOUS BACKFILL MATERIAL SHALL COMPLY WITH SECTION 19-3.02D OF THE CALTRANS 2022 STANDARD SPECIFICATIONS FOR GRAVEL OR CRUSHED GRAVEL.
4. TERMINATE PERVIOUS BACKFILL BEHIND WINGWALLS WHEN 3'-0" DEPTH IS UNABLE TO BE MET PER BRIDGE DETAIL 3-1 OF PLAN B0-3 OF THE CALTRANS 2022 STANDARD PLANS.

CONCRETE

1. UNLESS OTHERWISE NOTED, ALL CONCRETE STRUCTURES SHALL BE CAST-IN-PLACE.
2. CONCRETE STRENGTH AND MIX REQUIREMENTS:
   - MINIMUM 28-DAY COMPRESSIVE STRENGTH (f'c) = 4,500 PSI
   - WATER/CEMENT RATIO = 0.45 (±.03)
   - MAXIMUM AGGREGATE SIZE = 1"
   - AIR ENTRAINMENT = 4.5% MIN (SEE ACI 318-19 TABLE 19.3.3.1 FOR SMALLER AGGREGATE SIZE REQUIREMENTS)
   - CEMENT = ASTM C150 TYPE II
   - EXPOSURE CLASSES:
     - FREEZING AND THAWING = F0
     - SULFATE = S0
     - PERMEABILITY = W2
     - CORROSION = C1
3. FORMS SHALL BE PROVIDED TO ACHIEVE LINES, GRADES, AND GEOMETRY OF CONCRETE STRUCTURES AS INDICATED ON THESE DRAWINGS.
4. EXPOSED CONCRETE EDGES SHALL HAVE A ¾" CHAMFER.
5. CONCRETE SURFACES SHALL BE CLASS 1 SURFACE FINISH FOR ALL HEADWALLS AND WINGWALLS PER CALTRANS 2022 STANDARD SPECIFICATION SECTION 51-1.03F(3).
6. REINFORCING STEEL:
   - DEFORMED REBAR = ASTM A615, GRADE 60 (fy = 60 KSI)
7. REINFORCEMENT SPACING SHOWN IS CENTER TO CENTER OF BARS. REINFORCEMENT FOR CULVERTS AND DRAINAGE/FLOOD DITCH HEADWALL NOT SHOWN FOR CLARITY IN SECTIONS AND DETAILS.
8. UNLESS OTHERWISE NOTED, MAINTAIN 3" OF COVERAGE TO THE FACE OF REBAR.
9. MINIMUM LAP LENGTHS SHALL CONFORM TO TABLE 1, THIS SHEET.
10. STEEL SHALL BE KEPT CLEAN AND FREE OF RUST SCALES.
11. REINFORCING BARS SHALL BE PLACED IN LENGTHS AS LONG AS POSSIBLE. ALL REINFORCING STEEL SHALL BE COLD BENT.
12. REINFORCING AND INSERTS SHALL BE RIGIDLY HELD IN PLACE PRIOR TO CONCRETE PLACEMENT.
13. INSPECTION AND TESTING REQUIREMENTS FOR FIRST BATCH PRODUCED EACH DAY SHALL MEET THE FOLLOWING:
    - TEMPERATURE PER ASTM C172
    - AIR CONTENT PER ASTM C231
    - SLUMP PER ASTM C143
    - CONCRETE COMPRESSIVE STRENGTH PER CALIFORNIA TEST 529 & 533 FOR FIVE TEST CYLINDERS PER EVERY 300 CUBIC YARDS (1 AT 7 DAYS, 1 AT 14 DAYS, 2 AT 28 DAYS, AND ONE HOLD)

ROCK SLOPE PROTECTION

1. RSP SHALL BE CONCRETED AND COMPLY WITH THE ROCK GRADING AND FABRIC REQUIREMENTS SHOWN IN SECTION 72-3.02C OF THE CALTRANS 2022 STANDARD SPECIFICATIONS FOR CLASS III ROCK.
2. RSP SHALL CONFORM TO THE REQUIREMENTS OF CALTRANS METHOD B PLACEMENT PER SECTION 72-3.03C OF THE CALTRANS 2022 STANDARD SPECIFICATIONS.
3. ROCK AND CONCRETE MATERIAL MUST MEET THE REQUIREMENTS IN SECTION 72-3.02 OF THE CALTRANS 2022 STANDARD SPECIFICATIONS FOR CLASS III ROCK.
4. RSP SHALL BE TESTED AND IN ACCORDANCE WITH SECTION 72-3 OF THE 2022 CALTRANS STANDARD SPECIFICATIONS FOR CLASS III ROCK.
5. ROCK SHALL BE ANGULAR. ROUNDED ROCK AND COBBLES ARE NOT ACCEPTABLE.
6. CONCRETE SHALL HAVE A SLUMP OF 3 TO 4 INCHES
7. MINIMUM RSP LAYER THICKNESS = 2'-0".
8. THE AREA COVERED WITH RSP SHALL BE CLEARED OF LOOSE SOIL AND DEBRIS. ALL EXCESS EXCAVATED MATERIAL SHALL BE DISPOSED OF IN UNITED WATER CONSERVATION DISTRICT'S DESIGNATED AREAS NEAR THE SITE OR OFF-HAULED TO AN ACCEPTABLE WASTE DISPOSAL FACILITY AS DETERMINED BY UNITED WATER CONSERVATION DISTRICT.

HDPE PIPE

1. PIPE MATERIAL = SOLID HIGH DENSITY POLYETHYLENE (HDPE) PIPE, SDR 17, WITH SMOOTH INTERIOR AND EXTERIOR SURFACES.
2. HDPE PIPE SHALL BE DESIGNED, MANUFACTURED, AND INSTALLED PER GUIDELINES IN AWWA M55. PIPE SHALL BE IN ACCORDANCE WITH THE DESIGN WINDOW OF AWWA M55 CHAPTER 5.
3. PIPE SHALL BE PRODUCED PER AWWA M55 CHAPTER 1, IN ACCORDANCE WITH ANSI/AWWA C906, MEETING REQUIREMENTS IN ASTM D3350.
4. PIPE SHALL BE INSTALLED PER AWWA M55 CHAPTER 8, IN ACCORDANCE WITH ANSI/AWWA C906, MEETING REQUIREMENTS IN ASTM D2321. IN ADDITION TO ASTM D2321, OVERSIGHT OF PIPE INSTALLATION SHALL ADHERE TO SECTIONS 5.8 AND 5.9 OF USACE EM 1110-2-2902.
5. PIPE SIZING SHALL CONFORM TO AWWA M55 CHAPTER 3, TABLE 3-1, FOR A 30" OD.
6. PIPE SEGMENTS SHALL BE BUTT FUSION WELDED PER AWWA M55 CHAPTER 6, IN ACCORDANCE WITH ANSI/AWWA C906, MEETING REQUIREMENTS IN ASTM F2620 (ASTM D2657 SPECIFIES ASTM F2620 FOR HDPE PIPES).
7. INSTALL PIPE TO THE LINES AND GRADES SHOWN ON CIVIL SHEETS. COLD BEND THE PIPE WHERE NECESSARY:
   - MINIMUM BEND RADIUS = 67.5 FEET, OR PER MANUFACTURER RECOMMENDATIONS
8. MINIMUM SOIL COVER OVER PIPE = 3 FEET
9. CONNECTION OF PIPE AND CONC HEADWALLS SHALL COMPRISE A RUBBER MANHOLE STOP RING, MANUFACTURED BY NORTHTOWN PIPE PROTECTION PRODUCTS, OAE, AND SHALL MEET REQUIREMENTS IN ASTM C923. INSTALL PER MANUFACTURER'S RECOMMENDATIONS.
10. CULVERT INSTALLATION MUST BE COMPLETED PRIOR TO HDPE PIPE INSTALLATION. CULVERT INSTALLATION SHALL FOLLOW CALTRANS STANDARD PLAN A62E. HDPE PIPE INSTALLATION SHALL FOLLOW THE TRENCHING METHOD PER AWWA M55 CHAPTER 8.
11. CLSM SHALL BE USED AS STRUCTURAL BACKFILL AROUND THE HDPE PIPES, AS FOLLOWS:
    - CLSM PROPERTIES SHALL ADHERE TO SECTION 5.5.18.1, INCLUDING TABLE 5-2, OF USACE EM 1110-2-2902.
    - CLSM SHALL BE USED FOR BEDDING AND INITIAL BACKFILL AROUND THE HDPE PIPES, PER AWWA M55 CHAPTER 8, PRIOR TO FINAL BACKFILL. CLSM SHALL BE PLACED A MINIMUM OF 12 INCHES ABOVE THE PIPE CROWN AND TO THE SIDES OF THE PIPE SPRINGLINE. TEMPORARY FORMWORK OR A WEIGHTED ANCHOR SYSTEM, PER SECTION 5.5.18.1 OF USACE EM 1110-2-2902, OAE, SHALL BE USED DURING CLSM BACKFILL AROUND THE PIPES TO PREVENT FLOTATION AND UNTIL BACKFILL ABOVE THE CULVERT CAN BE COMPLETED.
    - THE ROOF OF THE CULVERT MUST BE CAST AND FULLY CURED TO SERVE AS THE FOUNDATION OF THE HDPE PIPES AND CLSM BACKFILL PRIOR TO PIPE INSTALLATION.

INSPECTION AND OBSERVATION

1. CONTRACTOR SHALL PROVIDE QUALITY CONTROL, MATERIALS TESTING AND SPECIAL INSPECTION RELATED TO THE PROPOSED WORK. CONTRACTOR SHALL PERFORM AND/OR RETAIN THE SERVICES OF A CERTIFIED TESTING LABORATORY TO PERFORM ALL QUALITY CONTROL TESTS OF THE PROPOSED WORK. ONLY THE CERTIFIED TESTS BY THE TESTING LABORATORY CAN BE USED TO VERIFY COMPLIANCE TO THE PROJECT DOCUMENTS.
2. CONSTRUCTION OBSERVATION BY THE INSPECTOR, APPROVED BY THE OWNER, IS REQUIRED AT THE FOLLOWING STAGES OF CONSTRUCTION:
   - SITE LAYOUT
   - COMPLETION OF EXCAVATION/APPROVAL OF FOUNDATION (ENGINEER HOLD POINT)
   - PLACEMENT OF FORM WORK
   - PLACEMENT OF REINFORCING STEEL (ENGINEER HOLD POINT)
   - PLACEMENT OF CONCRETE
   - PLACEMENT OF SLIDE GATE
   - INSTALLATION OF HDPE PIPES
   - PLACEMENT AND COMPACTION OF FILL MATERIALS (ENGINEER HOLD POINT)
3. NOTIFY THE INSPECTOR/ENGINEER AT LEAST 48 HOURS BEFORE INSPECTION OR OBSERVATION IS NEEDED.
4. CONTRACTOR SHALL SUBMIT PROPOSED MATERIALS AND PRODUCTS CALLED FOR IN THE PLANS FOR REVIEW AND APPROVAL OF THE ENGINEER: DEWATERING PLAN, CONCRETE, REINFORCING STEEL, RSP, IMPORTED BACKFILL, PERVIOUS BACKFILL, CLSM, HDPE PIPES, SLIDE GATE ANCHOR BOLTS, ETC.

TABLE 1 - TYPICAL DEVELOPMENT/LAP LENGTHS [A]

BAR SIZE	TOP BARS [B]		OTHER BARS		STD HOOKS (90° OR 180°)
	DEV LENGTH Ld (IN)	CLASS B LAP SPLICE (IN)	DEV LENGTH Ld (IN)	CLASS B LAP SPLICE (IN)	DEV LENGTH Ldh (IN)
#4	24	31	15	24	6
#5	30	38	18	30	8
#6	35	46	22	35	10
#7	51	67	40	51	11
#8	59	76	45	59	13
#9	66	86	51	66	15

FOOTNOTES:

A. BASED ON ACI 318-19 SECTIONS 25.4.2 – 25.4.3, WITH f'c = 4,500 PSI AND fy = 60,000 PSI. STD HOOK DIAMETERS AND EXTENSIONS FOLLOW STD DETAILS FROM CONCRETE REINFORCING STEEL INSTITUTE (CRSI).

B. TOP BARS SHALL BE DEFINED AS ANY HORIZONTAL BAR PLACED SUCH THAT MORE THAN 12 INCHES OF FRESH CONCRETE IS CAST IN THE MEMBER BELOW THE BAR IN ANY SINGLE POUR. HORIZONTAL WALL BARS ARE CONSIDERED TOP BARS.

0 1"
REFERENCE SCALE

90% NOT FOR CONSTRUCTION

GANNETT FLEMING
2251 Douglas Blvd., Ste. 200
Roseville, CA 95661
(916) 677-4800
www.GANNETTFLEMING.com

REV	BY	DATE	DESCRIPTION

NOTES
THREE BARREL CULVERT
VERN FREEMAN DIVERSION CONVEYANCE SYSTEM
SANTA PAULA VENTURA COUNTY CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
JENNIFER S. ALL
No. C71396
CIVIL
STATE OF CALIFORNIA
NOT FOR CONSTRUCTION

DATE: 08/14/23
SCALE: AS SHOWN
DESIGNED BY: RC/SMU/WLM
DRAFTED BY: P. BARBER
CHECKED BY: JSA/TRS
JOB NO.: 067376
FILE: 067376 002.dwg

G2
SHEET 2 OF 10

PLOT DATE: Monday, August 14, 2023 TIME: 6:33:29 PM BY: CONRAD, RYAN CTB: SAGE.CTB TAB: G2
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LEGEND & ABBREVIATIONS

(E)	3500	MAJOR CONTOUR
(E)		MINOR CONTOUR
(N)	3500	MAJOR CONTOUR
(N)		MINOR CONTOUR
APPROX		APPROXIMATE
BOF		BOTTOM OF FOOTING
BOW		BOTTOM OF WALL
CIP		CAST-IN-PLACE
CJ		CONSTRUCTION JOINT
℄		CENTER LINE
CLR		CLEAR
CLSM		CONTROLLED LOW STRENGTH MATERIAL
CONC		CONCRETE, CONCRETED
D/S		DOWNSTREAM
DIA, ø		DIAMETER OR PIPE DIAMETER
DEMO		DEMOLISH
DWG		DRAWING
(E)		EXISTING FEATURE
EF		EACH FACE
EG		EXISTING GRADE
ELEC		ELECTRIC
ELEV		ELEVATION
FG		FINISH GRADE
FTG		FOOTING
FV		FIELD VERIFY
G		GAS LINE
GM		GAS METER
GV		GAS VALVE
HORIZ		HORIZONTAL
ID		INSIDE DIAMETER
IE		INVERT ELEVATION
INV		INVERT
LF		LINEAR FEET
MAX		MAXIMUM
MIN		MINIMUM
(N)		NEW FEATURE
OAE		OR APPROVED EQUIVALENT
OC		ON CENTER
OD		OUTSIDE DIAMETER
OHW		OVERHEAD WIRES
PIP		PROTECT-IN-PLACE
RC		RELATIVE COMPACTION
RCB		REINFORCED CONCRETE BOX
REINF		REINFORCEMENT
RSP		ROCK SLOPE PROTECTION
S		SLOPE
SCH		SCHEDULE
SDR		STANDARD DIMENSION RATIO
SPEC		SPECIFICATION
SS		STAINLESS STEEL
STD		STANDARD
TOF		TOP OF FOOTING
TOT		TOTAL
TOW		TOP OF WALL
TYP		TYPICAL
U/S		UPSTREAM
W		WATER LINE
WV		WATER VALVE

FLOW PATH

CONTROL POINT

COORDINATE POINT

DEMOLITION

269 A 272

SECTION OR DETAIL IDENTIFICATION

NUMBER OF DRAWING WHERE SECTION IS SHOWN

NUMBER OF DRAWING WHERE SECTION IS CUT

0 1"
REFERENCE SCALE

90% NOT FOR CONSTRUCTION

GANNETT FLEMING
2251 Douglas Blvd., Ste. 200
Roseville, CA 95661
(916) 677-4800
www.GANNETTFLEMING.com

REV	BY	DATE	DESCRIPTION

ABBREVIATIONS

THREE BARREL CULVERT
VERN FREEMAN DIVERSION CONVEYANCE SYSTEM

SANTA PAULA VENTURA COUNTY CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER JENNIFER S. ALLEN CIVIL STATE OF CALIFORNIA
NOT FOR CONSTRUCTION

DATE:	08/14/23
SCALE:	AS SHOWN
DESIGNED BY:	RC/SMU/WLM
DRAFTED BY:	P. BARBER
CHECKED BY:	JSA/TRS
JOB NO.:	067376
FILE:	067376 002-1.dwg

G3
SHEET 3 OF 10

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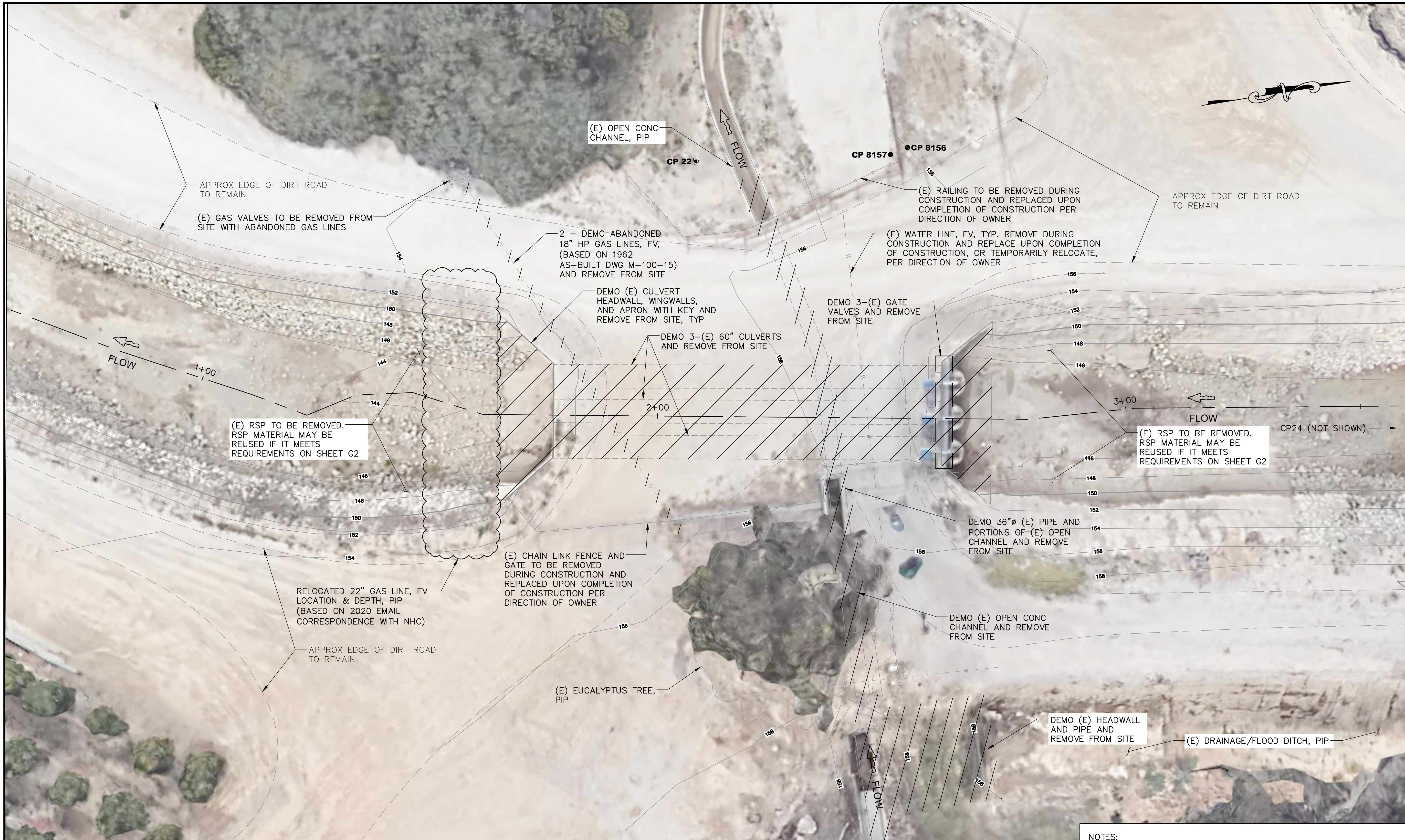

MAP DATA: GOOGLE EARTH, © 2023
IMAGERY: © 2023 AIRBUS, MAXAR TECHNOLOGIES, U.S. GEOLOGICAL SURVEY

CONTROL POINT TABLE

POINT #	NORTHING	EASTING	ELEVATION	DESCRIPTION
CP22	1927196.12	6222444.73	155.04	FND 1/2" IP W/PP
CP8157	1927237.96	6222449.98	156.28	FND 2" BD CNTR PUNCH
CP8156	1927241.17	6222449.07	156.28	FND 2" BD 490+45
CP24	1927599.33	6222590.77	154.68	SET CP ON WEIR

NOTES:

1. NOT ALL EXISTING FEATURES ARE SHOWN. AERIAL PHOTOGRAPHY IS FOR VISUAL REFERENCE ONLY.
2. SEE REFERENCES ON G1 FOR ADDITIONAL INFORMATION.
3. 22" HIGH PRESSURE GAS LINE UNDER THE CANAL NOT SHOWN. CONTRACTOR TO VERIFY LOCATION OF ALL UTILITIES.
4. UTILITIES SHOWN ARE FOR IDENTIFICATION PURPOSES ONLY. LOCATIONS TO BE FIELD VERIFIED. COORDINATION TO MAINTAIN, BYPASS, OR TEMPORARILY HALT UTILITY SERVICE SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR.

0 1"
REFERENCE SCALE

PLAN
SCALE: 1" = 10'

90% NOT FOR CONSTRUCTION

REV	BY	DATE	DESCRIPTION

DEMOLITION PLAN
THREE BARREL CULVERT
VERN FREEMAN DIVERSION CONVEYANCE SYSTEM
SANTA PAULA VENTURA COUNTY CALIFORNIA

NOT FOR CONSTRUCTION

DATE:	08/14/23
SCALE:	AS SHOWN
DESIGNED BY:	RC/SMU/WLM
DRAFTED BY:	P. BARBER
CHECKED BY:	JSA/TRS
JOB NO.:	067376
FILE:	067376 003.dwg

C1
SHEET 4 OF 10

PLOT DATE: Monday, August 14, 2023 TIME: 6:33:39 PM BY: CONRAD, RYAN CTB: ---- TAB: C1
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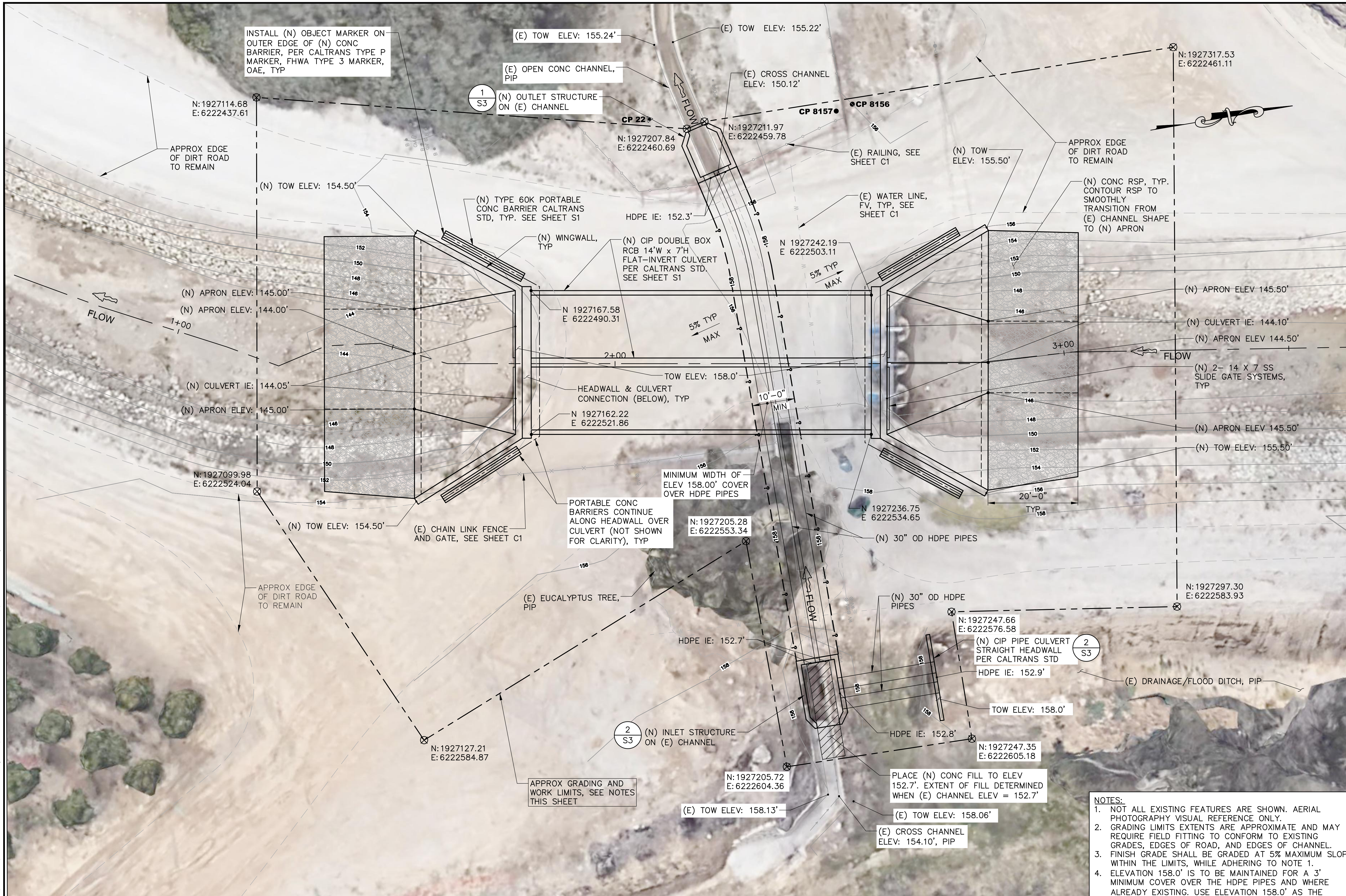

NOTES:
1. NOT ALL EXISTING FEATURES ARE SHOWN. AERIAL PHOTOGRAPHY VISUAL REFERENCE ONLY.
2. GRADING LIMITS EXTENTS ARE APPROXIMATE AND MAY REQUIRE FIELD FITTING TO CONFORM TO EXISTING GRADES, EDGES OF ROAD, AND EDGES OF CHANNEL.
3. FINISH GRADE SHALL BE GRADED AT 5% MAXIMUM SLOPE WITHIN THE LIMITS, WHILE ADHERING TO NOTE 1.
4. ELEVATION 158.0' IS TO BE MAINTAINED FOR A 3' MINIMUM COVER OVER THE HDPE PIPES AND WHERE ALREADY EXISTING. USE ELEVATION 158.0' AS THE STARTING POINT WITHIN THE GRADING LIMITS.

0 1"
REFERENCE SCALE

MAP DATA: GOOGLE EARTH, © 2023
IMAGERY: © 2023 AIRBUS, MAXAR TECHNOLOGIES, U.S. GEOLOGICAL SURVEY

PLAN
SCALE: 1" = 10'

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REV	BY	DATE	DESCRIPTION

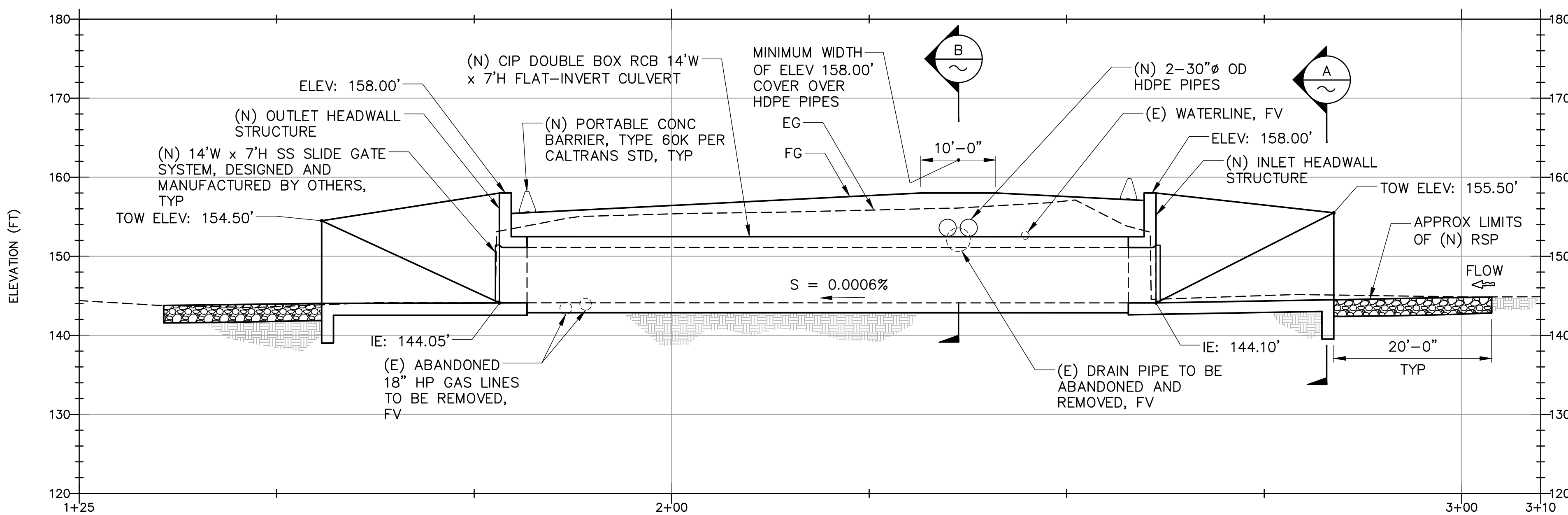
NEW CULVERT PLAN
THREE BARREL CULVERT
VERN FREEMAN DIVERSION CONVEYANCE SYSTEM
SANTA PAULA VENTURA COUNTY CALIFORNIA

NOT FOR CONSTRUCTION

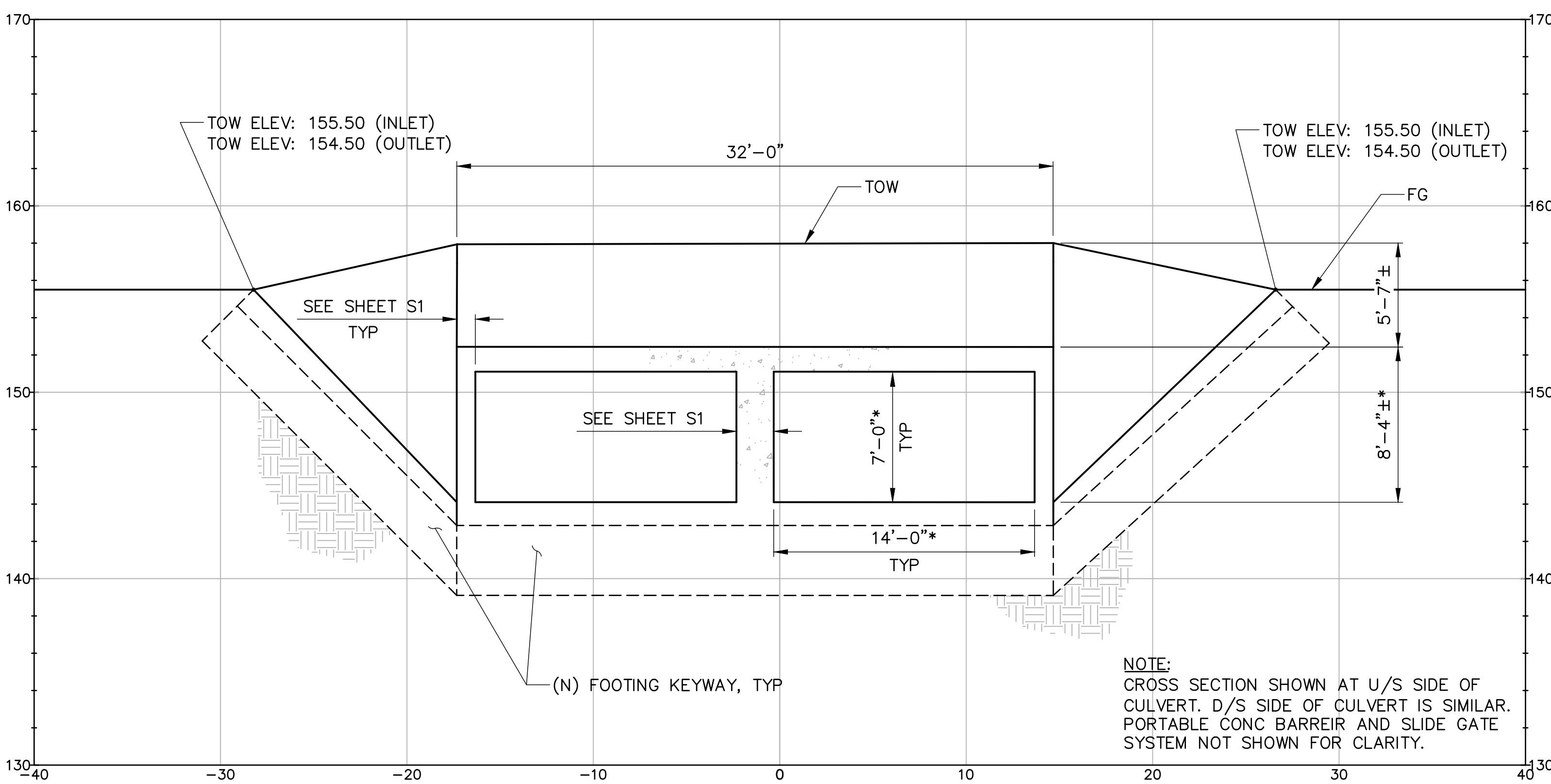
DATE:	08/14/23
SCALE:	AS SHOWN
DESIGNED BY:	RC/SMU/WLM
DRAFTED BY:	P. BARBER
CHECKED BY:	JSA/TRS
JOB NO.:	067376
FILE:	067376 004.dwg

C2
SHEET 5 OF 10

PLOT DATE: Monday, August 14, 2023 TIME: 6:33:44 PM BY: CONRAD, RYAN CTB: SAGE.CTB TAB: C2
FILE: C:\Users\rconrad\OneDrive - Gannett Fleming Inc\504-CADD-067376-NHC_FreemanDiv\067376 004.dwg



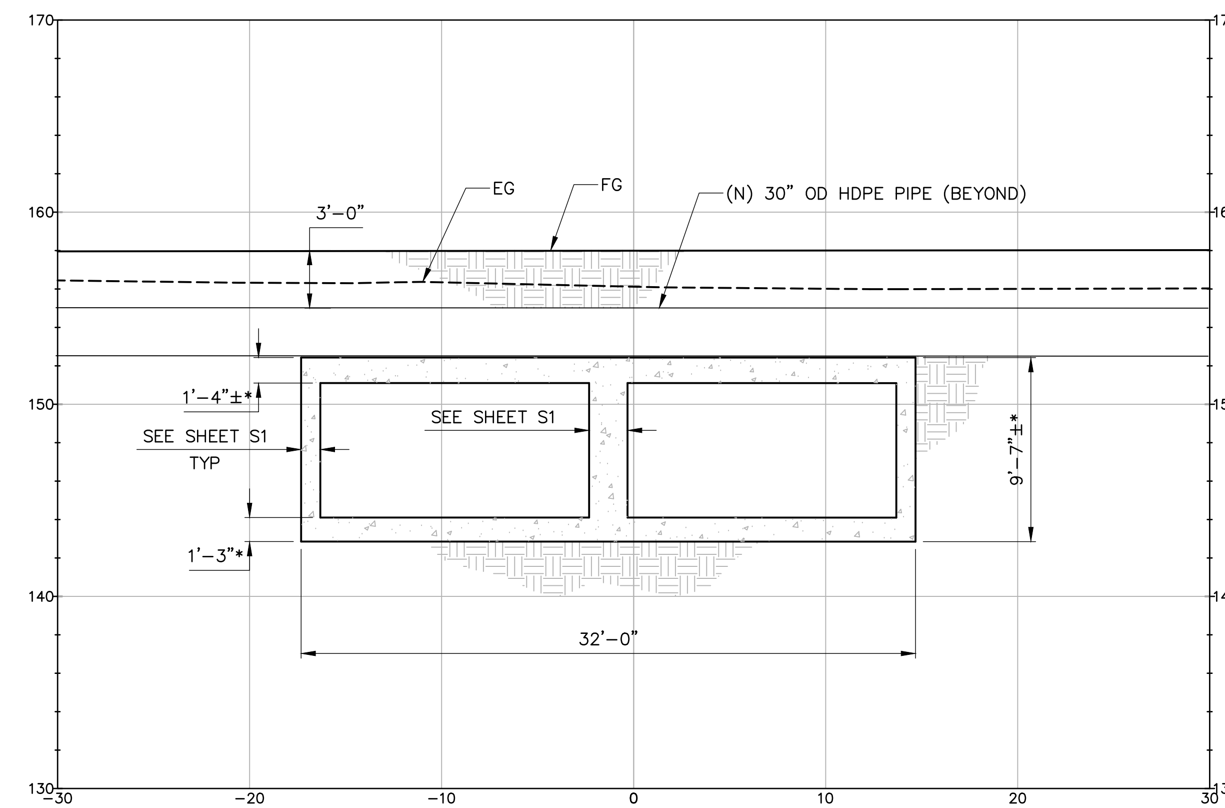
PROFILE
SCALE: 1" = 10'



* = PER CALTRANS STD

CIP CULVERT AT INLET/OUTLET STRUCTURE

A SECTION
SCALE: 1" = 5'



* = PER CALTRANS STD

CIP CULVERT AT PIPE CROSSING

B SECTION
SCALE: 1" = 5'

0 1"
REFERENCE SCALE

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REV	BY	DATE	DESCRIPTION

NEW CULVERT PROFILE & SECTIONS
THREE BARREL CULVERT
VERN FREEMAN DIVERSION CONVEYANCE SYSTEM
SANTA PAULA VENTURA COUNTY CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER JENNIFER S. ALLEN CIVIL STATE OF CALIFORNIA
NOT FOR CONSTRUCTION

DATE:	08/14/23
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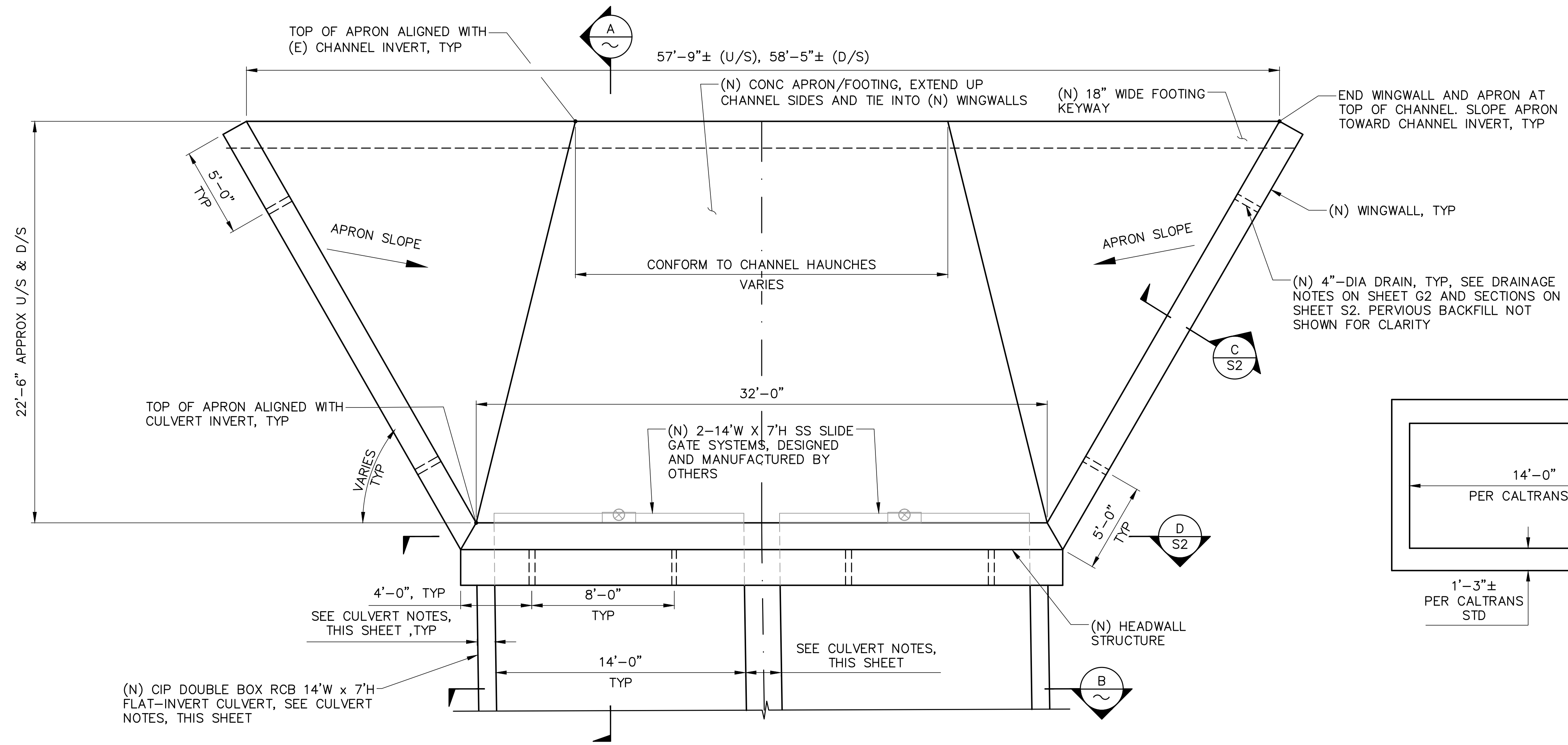
FILE:	067376 005.dwg

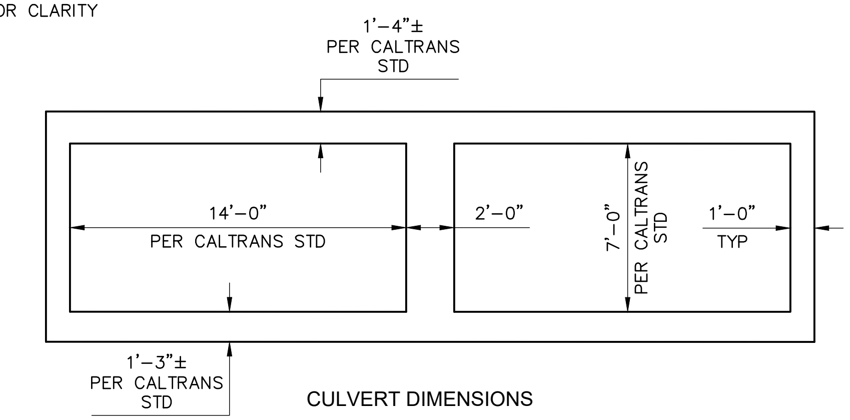
C3
SHEET 6 OF 10

PLOT DATE: Monday, August 14, 2023 TIME: 6:33:59 PM BY: CONRAD, RYAN CTB: SAGE.CTB TAB: C3
FILE: C:\Users\rconrad\OneDrive - Gannett Fleming Inc\504-CADD-067376-NHC_FreemanDiv\067376 005.dwg



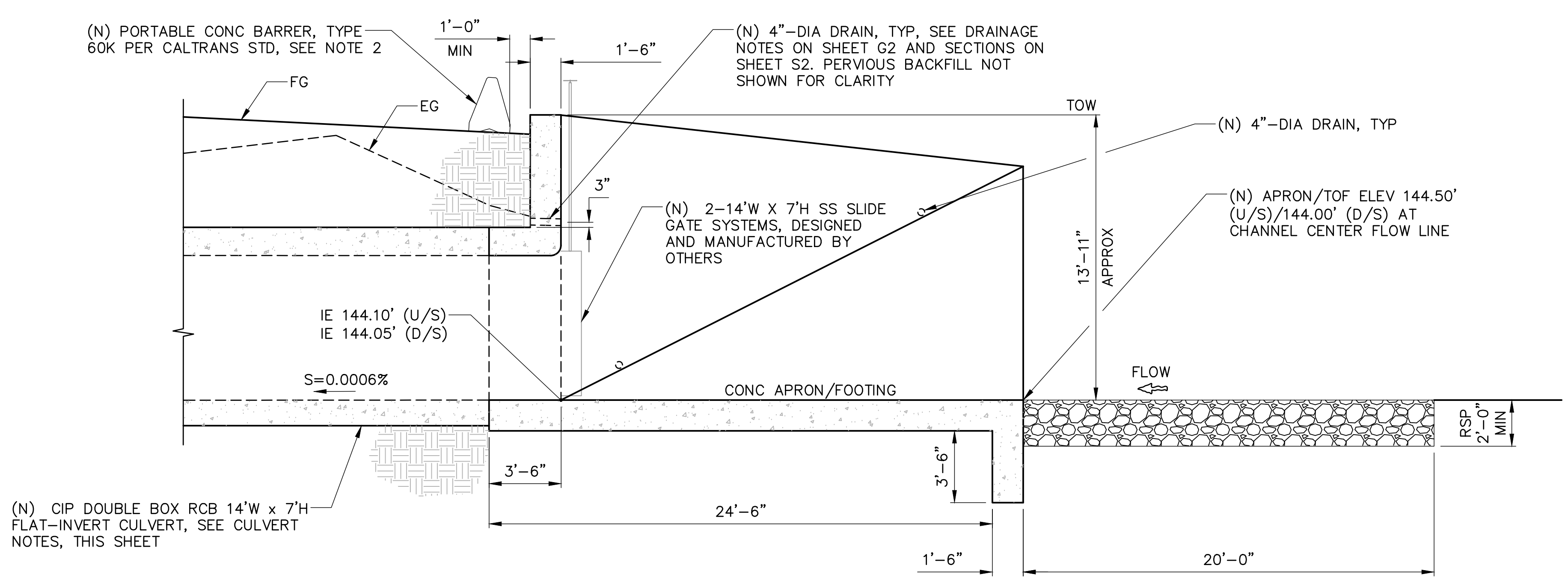
INLET/OUTLET STRUCTURE

PARTIAL PLAN
SCALE: 1" = 4'



CULVERT DIMENSIONS

B SECTION
SCALE: 1" = 4'



INLET/OUTLET STRUCTURE

A SECTION
SCALE: 1" = 4'

CULVERT NOTES:

1. CIP DOUBLE BOX RCB FLAT-INVERT CULVERT SHALL BE PER CALTRANS STD PLAN D81 AND REVISED D82, OAE, EXCEPT AS NOTED BELOW.
2. INSTALL CULVERT PER CALTRANS STD SPECIFICATION SECTION 51, OAE.
3. WALL THICKNESSES ARE INCREASED FROM THE CALTRANS STD PLANS TO ACCOMMODATE SLIDE GATE SYSTEM ANCHOR BOLTS:
   - CENTER/INNER WALL THICKNESS IS 24", WITH CENTERLINE OF WALL TO EDGE OF WALL BEING 12" THICK INSTEAD OF 4". "Bm" DIMENSION FROM STD PLAN D81 SHALL BE 7'-4" INSTEAD OF 6'-8".
   - OUTER WALL THICKNESS (T2) IS 12" INSTEAD OF 10". "B" DIMENSION FROM STD PLAN D81 SHALL BE 5'-6" INSTEAD OF 5'-4".
4. TO ACCOMMODATE USACE REQUIREMENTS FOR TEMPERATURE AND SHRINKAGE REINF (PER EM 1110-2-2104, SECTION 2.9), THE FOLLOWING CULVERT REINF NEEDS TO BE MODIFIED FROM CALTRANS STD PLAN D81:
   - FOR THE CENTER/INNER WALL, REINF SHALL BE #6 INSTEAD OF #4, INCLUDING FOR "h" BARS. SPACING SHALL REMAIN THE SAME.
   - FOR THE OUTER WALLS, THE #4 @ 12" MAX REINF SHALL BE #5 @ 12".
   - FOR THE ROOF, THE #4 @ 12" MAX REINF SHALL BE #5 @ 12". THE "i" BARS SHALL BE #5 INSTEAD OF #4 WITH THE QUANTITY REMAINING THE SAME.
   - FOR THE INVERT, THE #4 @ 12" MAX REINF SHALL BE #5 @ 12".
5. ALL CULVERT REINF SHALL HAVE 3" COVER.

NOTES:

1. PARTIAL PLAN AND CROSS SECTION SHOWN AT U/S SIDE OF CULVERT. D/S SIDE OF CULVERT IS SIMILAR. REINF NOT SHOWN FOR CLARITY.
2. PORTABLE CONCRETE BARRIER NOT SHOWN IN PARTIAL PLAN FOR CLARITY. PORTABLE CONCRETE BARRIER SHALL BE TYPE 60K PER CALTRANS STD PLANS A63A AND A63B, OAE. INSTALL PER CALTRANS STD SPECIFICATION SECTIONS 83-1 AND 83-3, OAE.
3. ALL DIMENSIONS TO DRAINS ARE TO CENTER OF PIPES.

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REV	BY	DATE	DESCRIPTION

PARTIAL PLAN & SECTION
THREE BARREL CULVERT
VERN FREEMAN DIVERSION CONVEYANCE SYSTEM
SANTA PAULA VENTURA COUNTY CALIFORNIA

NOT FOR CONSTRUCTION

DATE:	08/14/23
SCALE:	AS SHOWN
DESIGNED BY:	RC/SMU/WLM
DRAFTED BY:	P. BARBER
CHECKED BY:	JSA/TRS
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FILE:	067376 007.dwg

S1

0 1"
REFERENCE SCALE

PLOT DATE: Monday, August 14, 2023 TIME: 6:34:02 PM BY: CONRAD, RYAN CTB: SAGE.CTB TAB: S1
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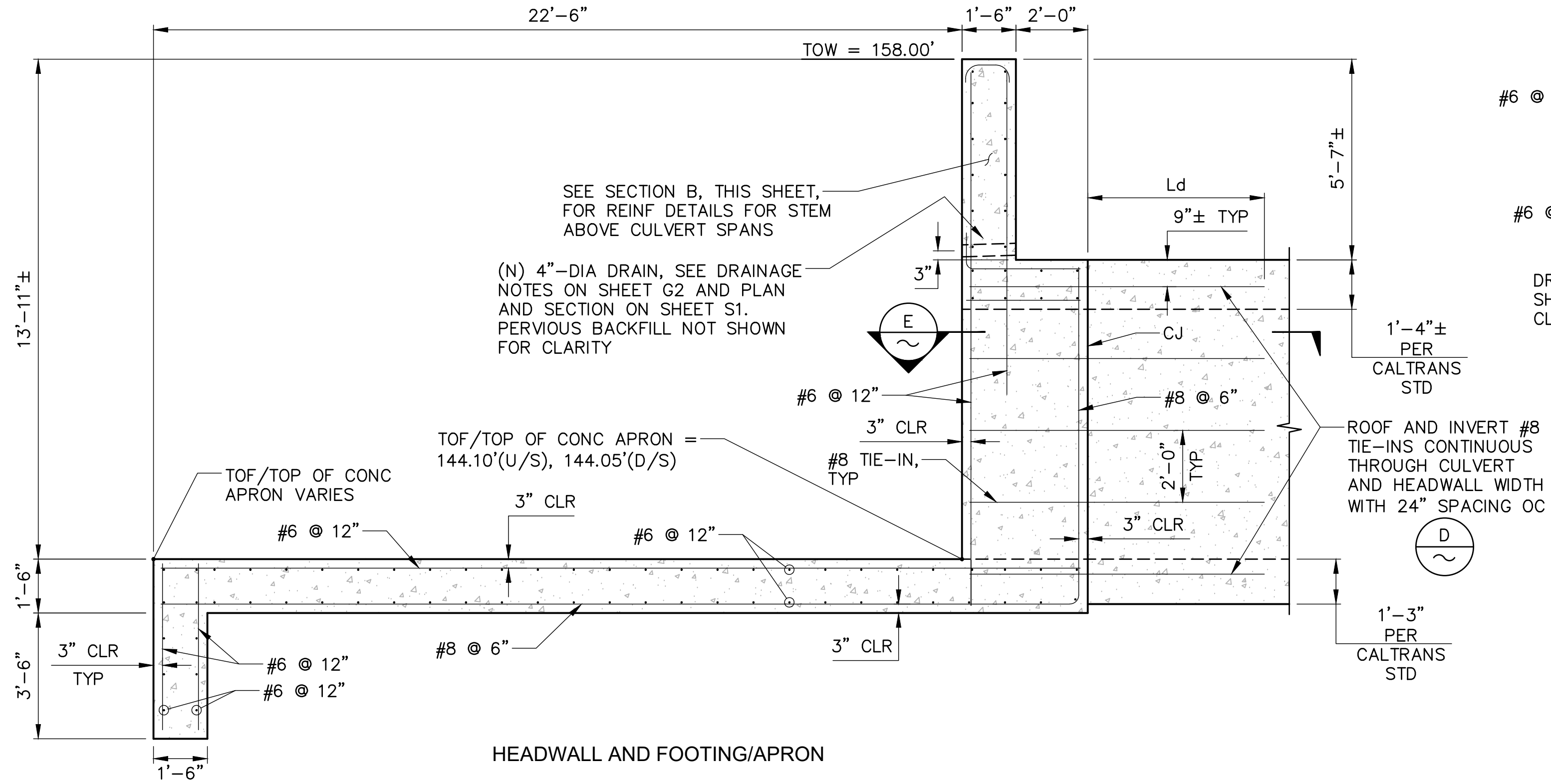
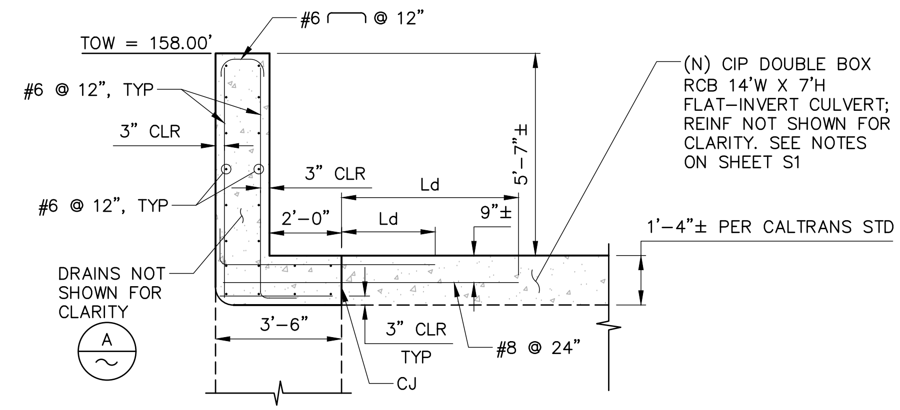


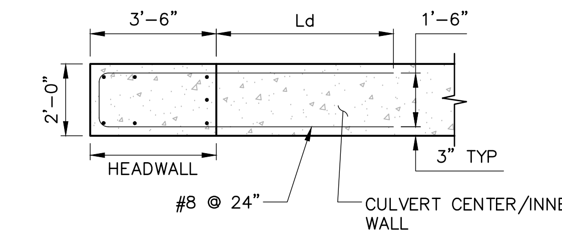
HEADWALL AND FOOTING/APRON

A SECTION
SCALE: 3/8" = 1'-0"



HEADWALL SECTION OVER CULVERT SPAN

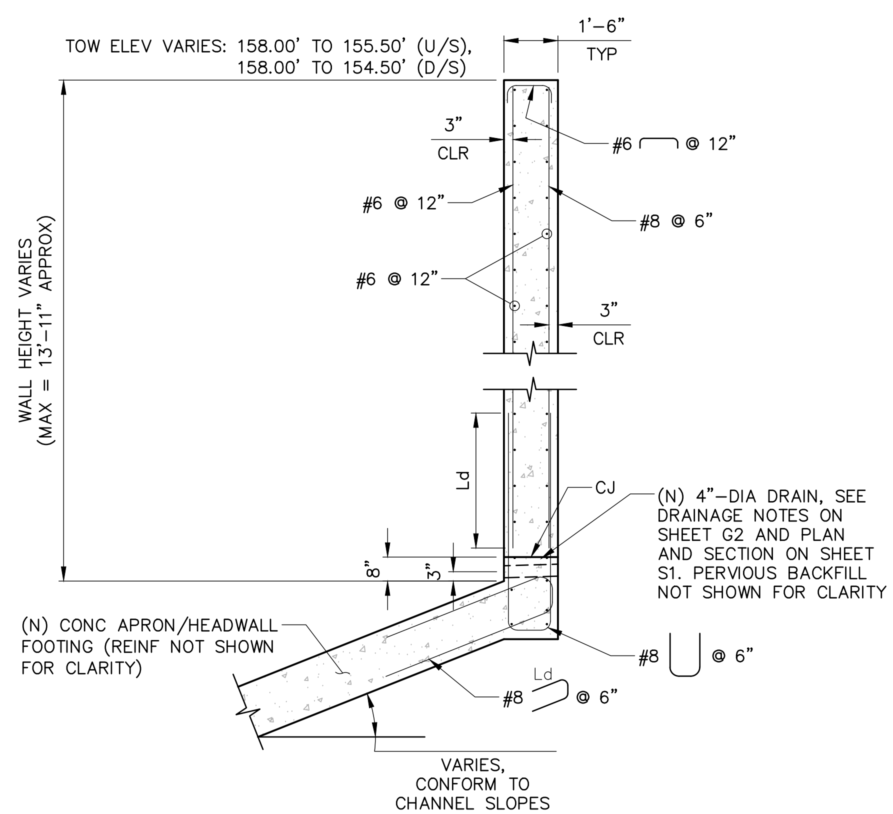
B SECTION
SCALE: 3/8" = 1'-0"



NOTE:
SECTION SHOWN FOR HEADWALL TIE-IN AT CENTER/INNER CULVERT WALL. TIE-IN AT EDGE/SIDE CULVERT WALLS WILL COMPRISE OF ONE CENTERED #8 BAR WITH THE STD HOOK ORIENTED VERTICALLY UPWARD, PARALLEL WITH THE HEADWALL FRONT FACE.

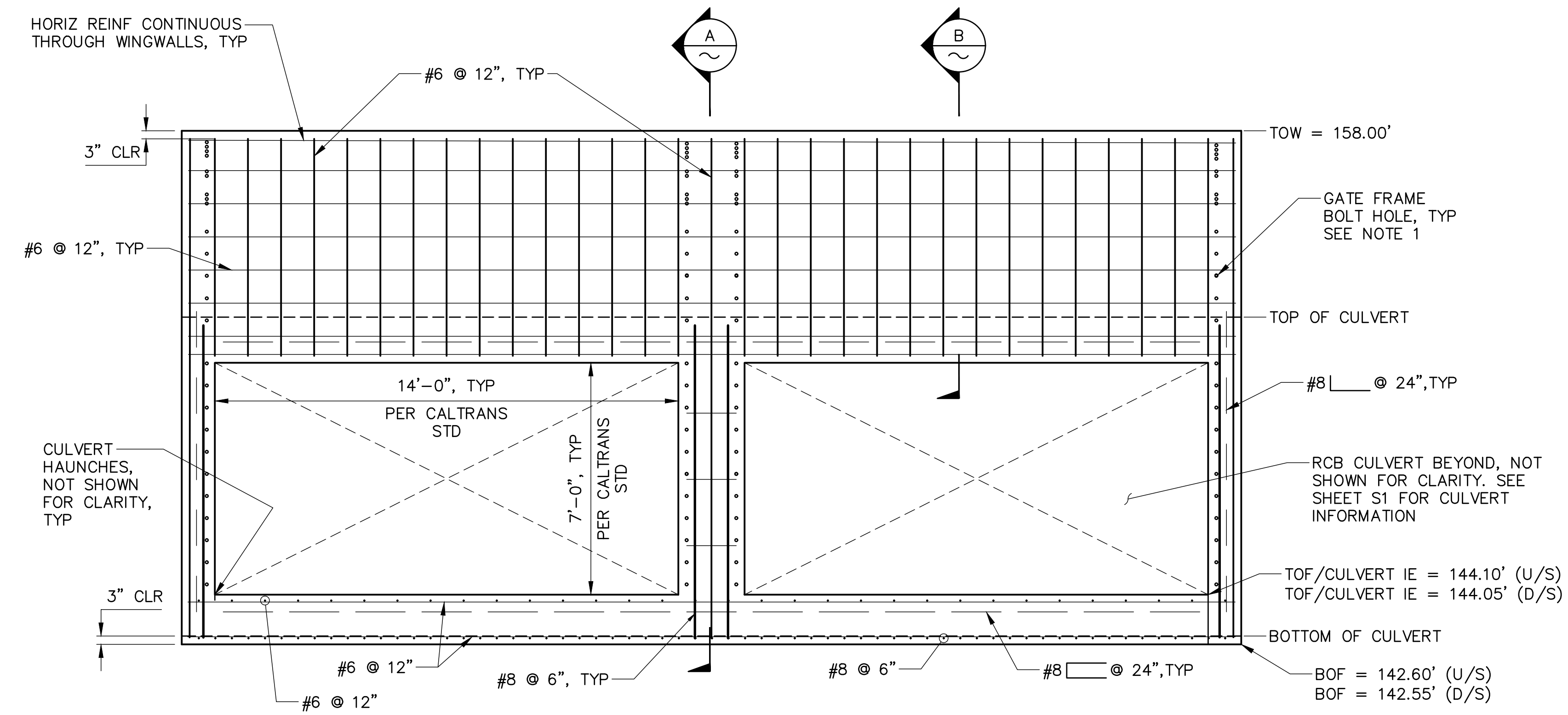
HEADWALL TIE-IN AT CULVERT WALL

E SECTION
SCALE: 3/8" = 1'-0"



WINGWALL & APRON

S1 C S2 SECTION
SCALE: 3/8" = 1'-0"



ELEVATION AT HEADWALL

S1 D S2 SECTION
SCALE: 3/8" = 1'-0"

NOTES:
1. BOLT LOCATIONS SHOWN FOR GATE MOUNTING ARE ESTIMATED. CONTRACTOR TO VERIFY ACTUAL LOCATIONS DO NOT CONFLICT WITH REBAR. CONTRACTOR SHALL PROVIDE SUBMITTAL VERIFYING THAT BOLT MOUNTING FOR GATES DOES NOT CONFLICT WITH HEADWALL REBAR.
2. ORIENTATION OF SECTIONS A, B, AND C SHOWN AT D/S SIDE OF CULVERT. REVERSE ORIENTATION FOR U/S SIDE.
3. DRAINS NOT SHOWN FOR CLARITY IN SECTION D.
4. ALL DIMENSIONS TO DRAINS ARE TO CENTER OF PIPES.
5. SEE TABLE 1, SHEET G2, FOR Ld VALUES.

0 1"
REFERENCE SCALE

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SECTIONS
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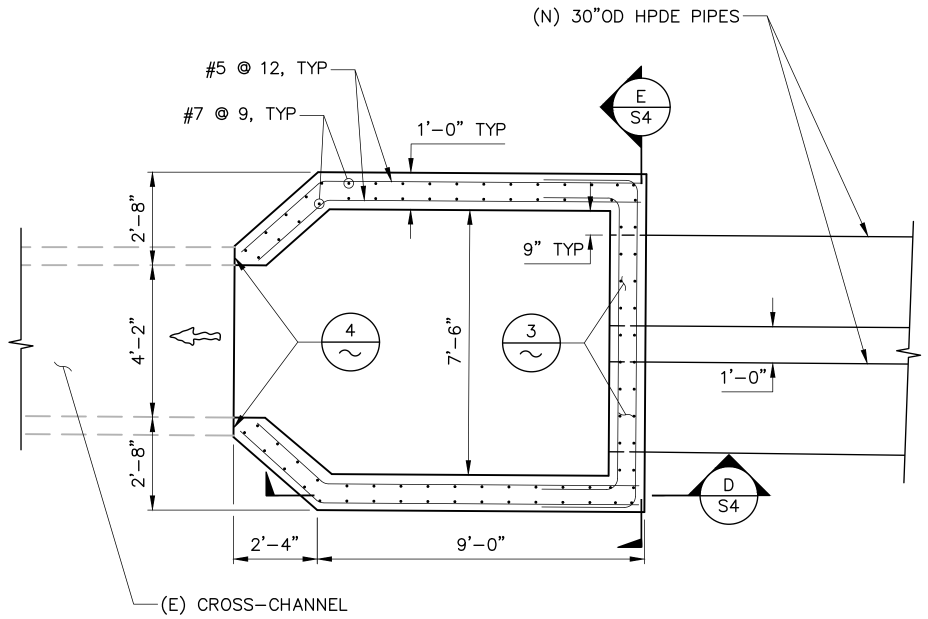
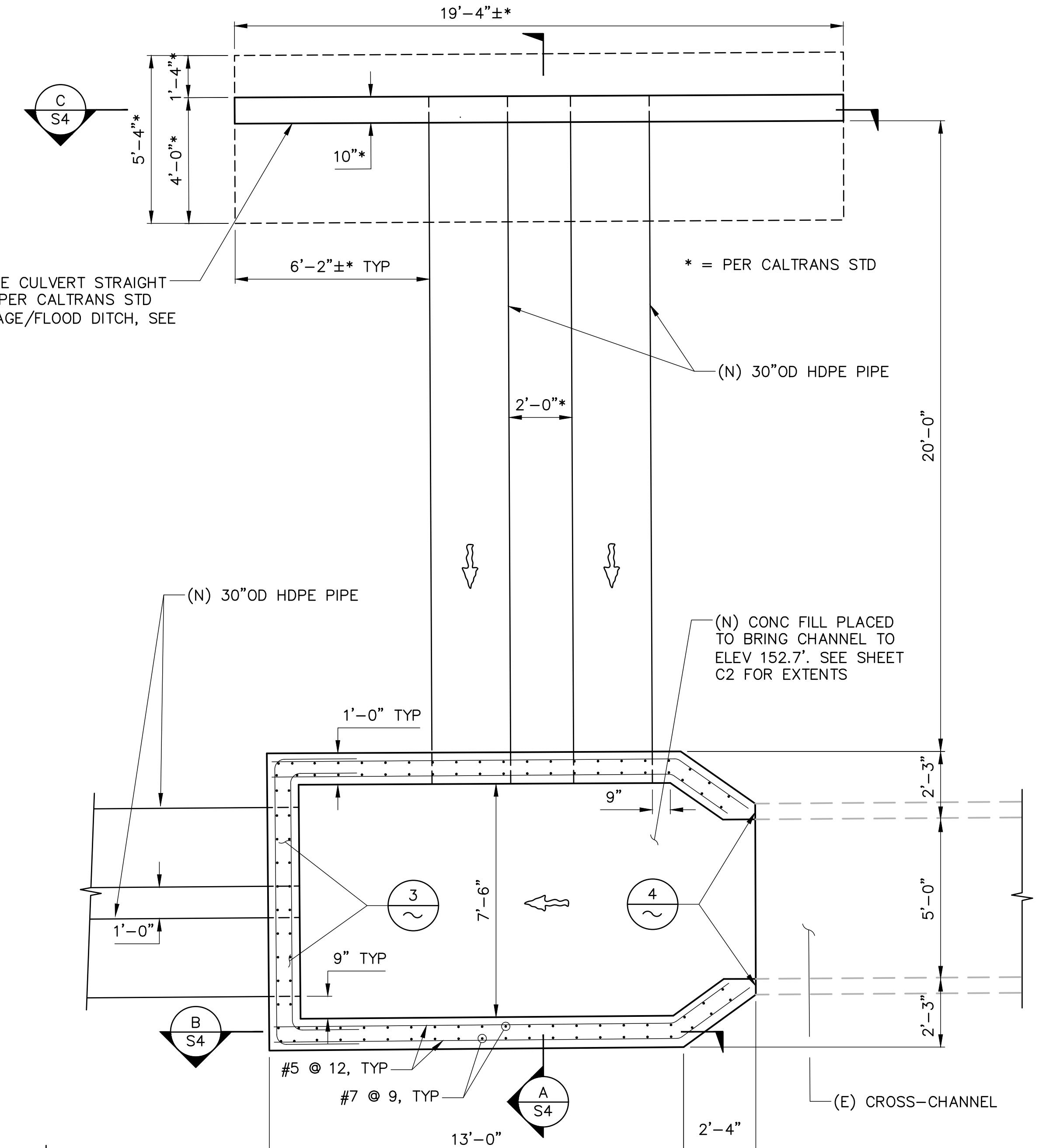
S2
SHEET 8 OF 10

PLOT DATE: Monday, August 14, 2023 TIME: 6:34:06 PM BY: CONRAD, RYAN CTB: SAGE.CTB TAB: S2
FILE: C:\Users\rconrad\OneDrive - Gannett Fleming Inc\504-CADD-067376-NHC_FreemanDiv\067376 008.dwg



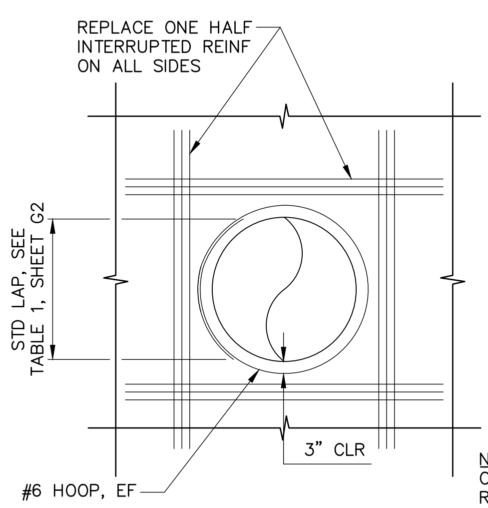
PARTIAL PLAN - CROSS CHANNEL OUTLET

C2 1 / S3 DETAIL
SCALE: 3/8" = 1'-0"



PARTIAL PLAN - CROSS CHANNEL INLET

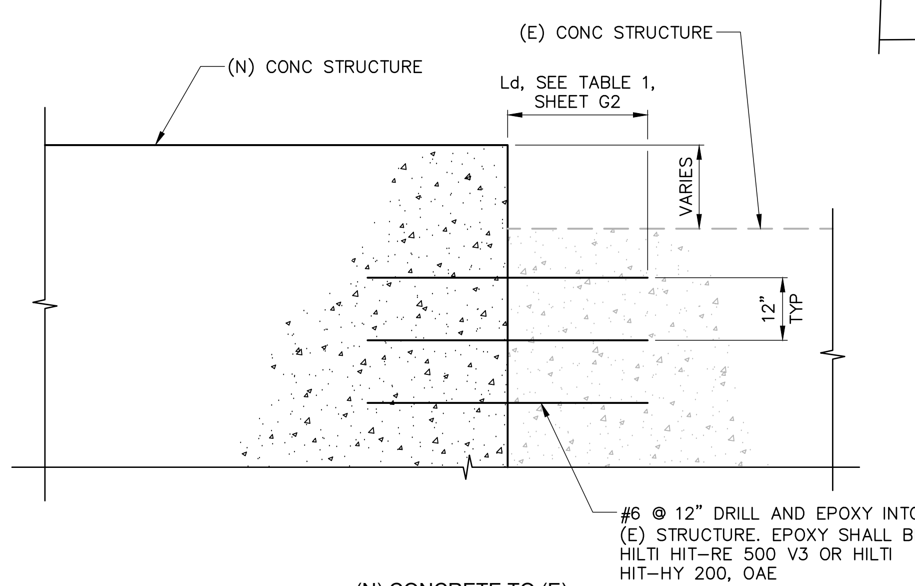
C2 2 / S3 DETAIL
SCALE: 3/8" = 1'-0



NOTE:
ONLY ADDITIONAL PENETRATION REINF SHOWN. TYPICAL REINF AND RUBBER MANHOLE STOP RING NOT SHOWN FOR CLARITY.

HDPE PIPE PENETRATION

3 / ~ DETAIL
NOT TO SCALE



(N) CONCRETE TO (E) CONCRETE CONNECTION

4 / ~ DETAIL
NOT TO SCALE

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0 1"
REFERENCE SCALE

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DETAILS
THREE BARREL CULVERT
VERN FREEMAN DIVERSION CONVEYANCE SYSTEM
SANTA PAULA VENTURA COUNTY CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER JENNIFER S. ALLEN CIVIL STATE OF CALIFORNIA
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DATE:	08/14/23
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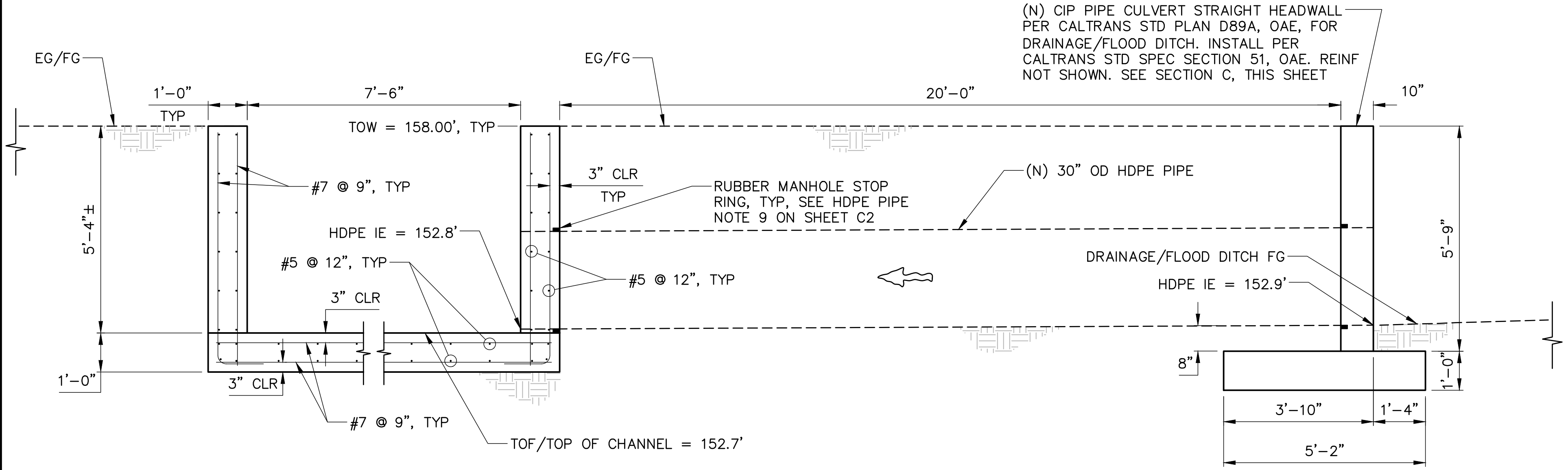
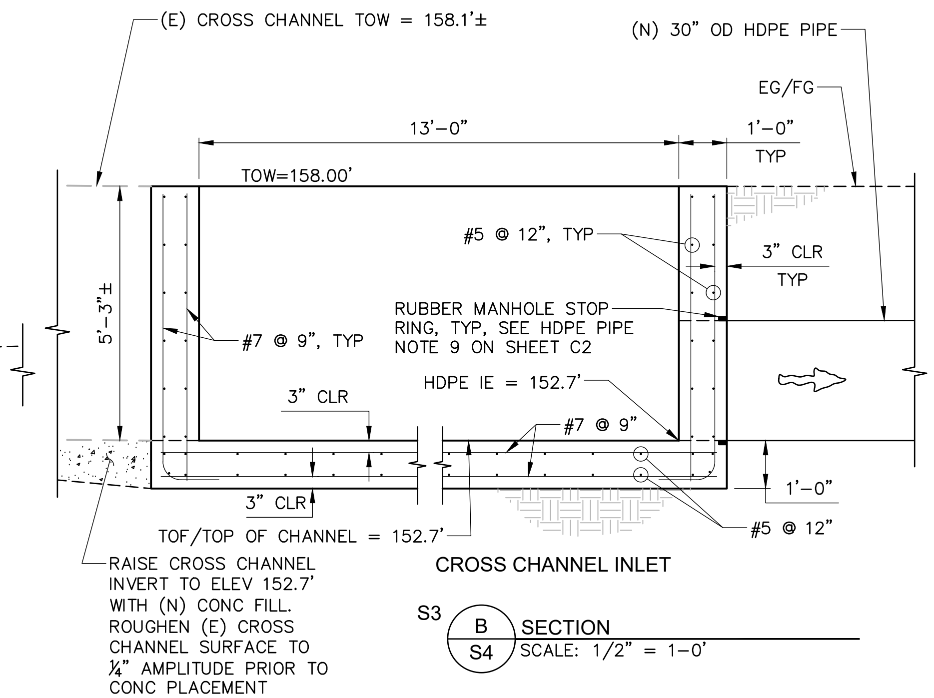
S3
SHEET 9 OF 10



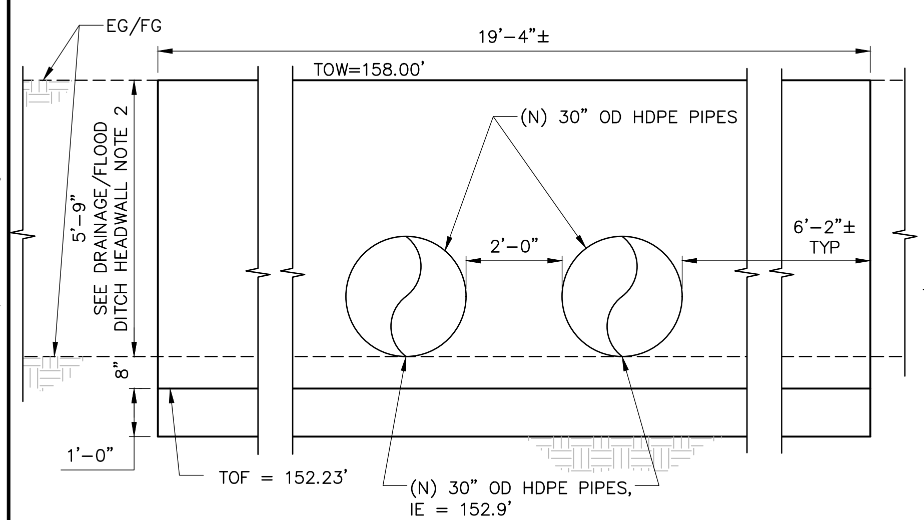
DRAINAGE/FLOOD DITCH INTO CROSS CHANNEL INLET

S3 A S4 SECTION
SCALE: 1/2" = 1'-0"



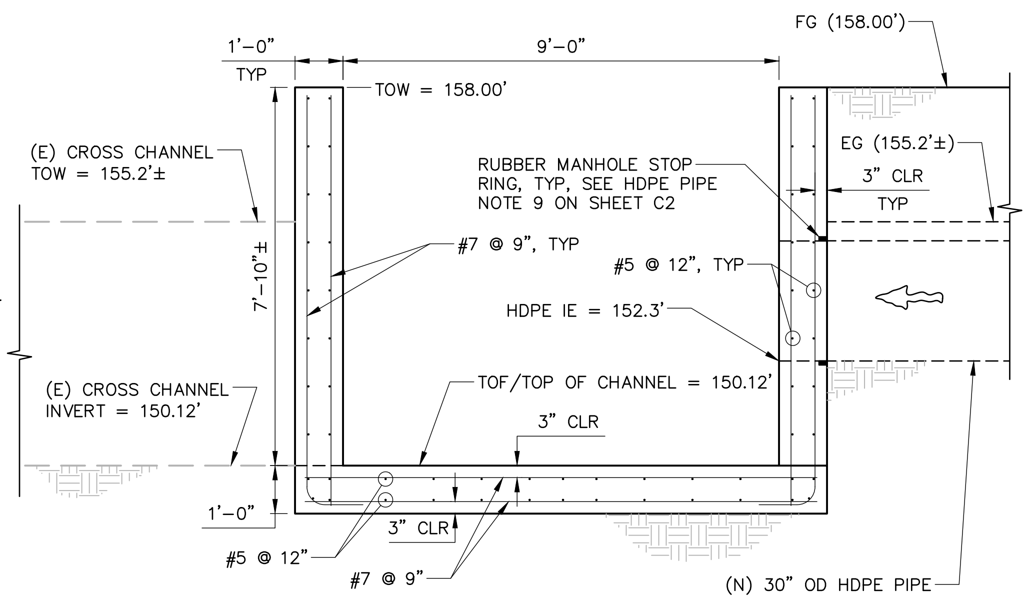
CROSS CHANNEL INLET

S3 B S4 SECTION
SCALE: 1/2" = 1-0'



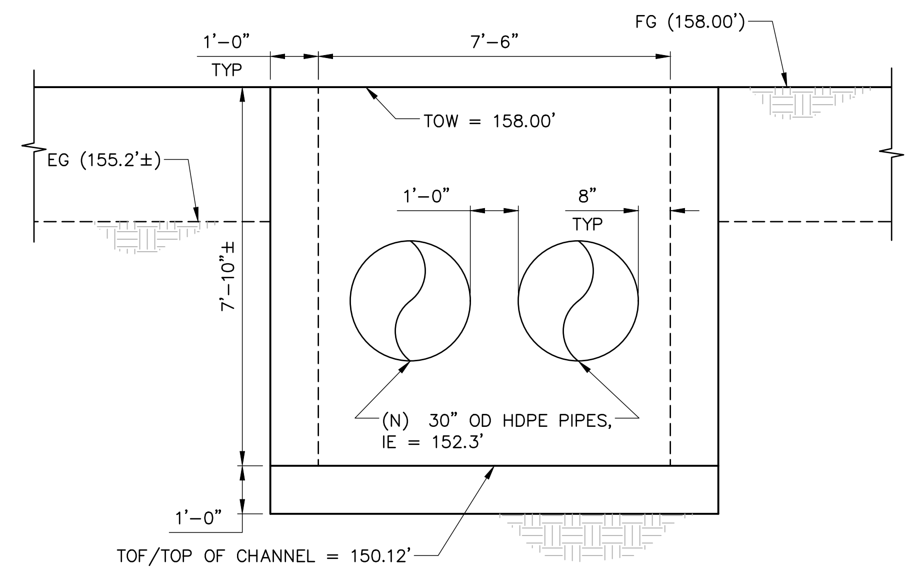
DRAINAGE/FLOOD DITCH HEADWALL

S3 C S4 SECTION
SCALE: 1/2" = 1'-0"



CROSS CHANNEL OUTLET

S3 D S4 SECTION
SCALE: 1/2" = 1'-0"



NOTE:
REINF AND RUBBER MANHOLE STOP RING NOT SHOWN FOR CLARITY.

CROSS CHANNEL OUTLET

S3 E S4 SECTION
SCALE: 1/2" = 1'-0"

DRAINAGE/FLOOD DITCH HEADWALL NOTES:

1. REINF AND RUBBER MANHOLE STOP RING NOT SHOWN FOR CLARITY. HEADWALL SHALL BE PER CALTRANS STANDARD PLAN D89A, EXCEPT AS FOLLOWS:
   - TO ACCOMMODATE USACE REQUIREMENTS FOR TEMPERATURE AND SHRINKAGE REINF (PER EM 1110-2-2104, SECTION 2.9), THE FOLLOWING HEADWALL REINF NEEDS TO BE MODIFIED FROM CALTRANS STANDARD PLAN D89A:
   - FOR THE FOOTING, THE #4 @ 12" REINF SHALL BE #5 @ 12".
   - ALL REINF SHALL HAVE 3" COVER.
2. USING CALTRANS STANDARD PLAN D89A, USE INFORMATION FOR AN "H" DIMENSION OF 5'-11".

NOTES:

1. REINF DISCONTINUES WHERE HDPE PIPE PENETRATION OCCURS. ADDITIONAL REINF FOR HDPE PIPE PENETRATION NOT SHOWN FOR CLARITY, SEE DETAIL 3 ON SHEET S3.
2. DIAGONAL SECTIONS THAT TIE NEW WALLS INTO EXISTING WALLS ARE NOT SHOWN. EXTEND REINF FROM STRAIGHT WALLS AND CONFORM TO WALL DIMENSIONS, AS SHOWN ON SHEET S3.

0 1"
REFERENCE SCALE

PLOT DATE: Monday, August 14, 2023 TIME: 6:34:17 PM BY: CONRAD, RYAN CTB: SAGE.CTB TAB: S4
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VERN FREEMAN DIVERSION CONVEYANCE SYSTEM
SANTA PAULA VENTURA COUNTY CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER JENNIFER S. ALLEN CIVIL STATE OF CALIFORNIA
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FILE:	067376 0010.dwg

S4

SHEET 10 OF 10

ATTACHMENT B – GEOTECHNICAL EVALUATION MEMORANDUM

Excellence Delivered **As Promised**

TECHNICAL MEMORANDUM

To: Ed Wallace
Northwest Hydraulic Consultants, Inc.
200 S. Los Robles Avenue, Suite 405
Pasadena, California 91101

From: Alma Luna, PE
Jerry S. Pascoe, PE, GE

Date: September 30, 2020

Re: **Preliminary Geotechnical Evaluation - 30% Design**
Vern Freeman Diversion System: Three Barrel Culvert & Inverted Siphon
United Water Conservation District Saticoy Facility
Ventura County, California
Project No. 67376

In accordance with our agreement with Northwest Hydraulic Consultants, Inc. (Ref. 1), Gannett Fleming is pleased to present this technical memorandum summarizing the results of our preliminary geotechnical evaluation for two locations along the Vern Freeman Diversion (VFD) conveyance and recharge system project in Ventura County, California. The approximate coordinates (WGS84) for the two components of the project are presented in Table 1 and their approximate locations are shown on the attached Vicinity Map, Figure 1.

Project Component	Latitude	Longitude
Three Barrel Culvert	34.2828	-119.1228
Inverted Siphon	34.2803	-119.1251

Northwest Hydraulic Consultants, Inc. (NHC), as the prime consultant, is leading the hydraulic and civil design aspect of the project. Gannett Fleming is providing geotechnical and structural consultation and design services for this effort. Gannett Fleming participated in an onsite meeting with NHC and the owner, United Water Conservation District (United), on May 27, 2020 to discuss the project goals and observe the existing conditions.

PROJECT BACKGROUND

The Three Barrel Culvert (TBC) and the Inverted Siphon (Siphon) are part of United's Saticoy Spreading Grounds site, a facility that recharges the groundwater table through percolation. Specifically, these two structures are road crossings over the open-channel VFD conveyance system. NHC prepared a report in 2016 (Ref. 2) in which the need to replace the TBC and Siphon structures was identified to increase the capacity of the canal. The 2016 report identified the design discharge goal of 750 cfs as the basis of the design in order to improve the sediment management of the system.

The design of the TBC is constrained by an existing canal and 36-inch pipe that crosses perpendicular to the conveyance canal and by a set of flap gates that are to remain operational per the Ventura County Watershed Protection District. Based on these restrictions, the possible alternatives under consideration by NHC (Ref. 3) include:

- Modifying the existing structure to accommodate an additional culvert pipe with a flap gate.
- Replacing the existing structure with a box culvert that can accommodate the necessary flap gate(s).

The scenarios analyzed by NHC indicate changes to the existing channel may include increasing the top of bank or widening the channel to provide the necessary freeboard conditions. The extent of these modifications would need to be evaluated further in future project phase(s).

At the Siphon location, the recommended improvement would include the removal of the existing structure and the construction/installation of a new bridge. The bridges under consideration are a CON/SPAN® O-Series precast system or a CONTECH steel clear span bridge. Either of these alternatives will likely require slight modifications to the existing channel.

SCOPE OF SERVICES

Our scope of services, designated as Tasks 1a in our agreement (Ref. 1), include the following:

- Review existing geotechnical reports prepared by others and containing subsurface data from areas in the vicinity of the proposed improvements.
- Review published topographic, geologic, and fault documents to obtain generalized geotechnical data relevant to the site.
- Develop preliminary geotechnical design parameters and recommendations for 30% development of plans.
- Prepare this preliminary technical memorandum summarizing relevant information from existing geotechnical reports and published data. This memorandum also includes preliminary geotechnical design and considerations for the project earthwork, foundations, and seismic considerations, as applicable.

This scope of our services did not include field subsurface exploration or laboratory testing. Environmental assessment for the presence of hazardous or toxic materials, detailed inspections of the existing conveyance system and evaluation of the system's flow capacity were also beyond our scope of services.

SITE CONDITIONS

Reviewed Documents

We reviewed four previous geotechnical investigations performed at or near the site by other consultants. The four reports are listed in the references below and were provided to us by NHC.

We also received a topographic survey by Stantec for the project site and dated June 11, 2020. The following is a summary of relevant geotechnical information from the reports reviewed.

Proposed Shop Building and Water Tank, Saticoy Facility (Ref. 4 and 5)

In 2012 and 2015, Earth Systems Southern California (ESSC), performed a geotechnical engineering study (Ref 4.) and an update (Ref. 5) for a shop building and water tank at the Saticoy facility. Two borings were drilled at the site of the shop building and water tank to depths between 16½ and 51½ feet below ground surface.

Boring 1 was drilled approximately 200 feet north of the Siphon site and approximately 1,000 ft south of the TBC site. The materials encountered in this boring generally consisted of 8½ feet of loose, olive brown to yellow brown, silty sand (SM). The silty sand was underlain by medium dense to dense, coarse to medium, well graded gravel (GW) with silty sand and cobbles. The gravel extended to the full depth of the boring at 50½ feet below ground surface. Boring 2, just to the east of Boring 1, also encountered about 8 ft of loose silty sand (SM) underlain by the well grade gravel (GW). Groundwater was not encountered in either of the two borings. ESSC concluded in the 2012 report that fault rupture hazard and liquefaction potential were low. However, in the 2015 report, ESSC used a historic high groundwater level of 10 feet and the applicable ground acceleration at the time in a new analysis. In the updated analysis, they indicated that there is a liquefaction potential at the site and the anticipated total settlement would be about 1.1 inches with about ½-inch of differential across the proposed structure.

Noble Basin and Saticoy Spreading Grounds Improvements (Ref. 6)

In 1993, Geotechnical Consultants, Inc. (GCI) performed a geotechnical investigation for the UWCD Noble Basin and Saticoy Spreading Ground Improvements projects. The site is located along Los Angeles Avenue, southeast of Saticoy, Ventura County and it extended to the western portion of the VFD system. Subsurface conditions were explored by drilling nine hollow stem auger borings to depths between 17½ and 41 feet below the ground surface.

Boring DH-1 is the closest boring to the project site from this report and it was drilled approximately 1,500 and 2,500 feet southwest of the Siphon and TBC, respectively. The materials encountered in this boring generally consisted of 10 feet of artificial fill, underlain by alluvium (Qal) soils that extend to the maximum depth explored of 41 feet. The artificial fill is described as very dense, damp, fine-grained silty sand with cobbles and gravel. The top five feet of the alluvium soils is described as very dense, damp, poorly graded sand with scattered fine gravel. Below the poorly graded sand, the alluvium soils become gravelly with cobbles and coarse to very coarse rounded gravel. Groundwater was encountered near the bottom of the hole (40 feet below ground surface), at the time of drilling. Laboratory tests performed indicate the soils have approximate dry densities between 98 to 108 pounds per cubic foot and moisture contents between 5 and 11 percent.

Except for borings DH-7 and DH-9, the other borings also encountered artificial fill, which varied in thickness from about 2½ to 10 feet. All borings that encountered alluvium deposits were generally consistent with the boring DH-1 description. Soft, dark brown silt lenses were observed in borings DH-2 and DH-3 and loose sands were encountered at about 5½ feet in boring DH-6. Groundwater depths in other borings varied between 21 to 24 feet below grade.

In general, GCI concluded that the Oak Ridge fault is located about 1 mile from the site and strong ground shaking during an earthquake should be expected. Loose sands encountered in boring DH-6 may become susceptible to liquefaction when saturated. This potentially liquefiable layer was not encountered in DH-1.

Construction of Four Well Pads, Saticoy Groundwater Storage Management Program (Ref. 7)

In 2002, Padre Associates, Inc. (PAS), performed a geotechnical study for the construction of four groundwater well drilling pads within the Saticoy Spreading ground facility. Their study involved reviewing previous geotechnical reports, collecting near-surface bulk samples for laboratory testing, and providing a report summarizing their findings, conclusions and recommendations for the geotechnical aspects of the well pads. Based on their review, PAS concluded that materials underlying the spreading grounds facility are fairly uniform and consist primarily of dense, coarse grained sand, gravel and cobbles.

Other Published Data

The site is approximately 500 feet southeast of the Oak Ridge fault, approximately one-mile northwest of the Wright Road fault and about 3.5 miles southeast of the Ventura fault (Ref. 8).

The site is not located within an Alquist-Priolo Earthquake Fault Zone (Ref. 9). The site is located within a zone identified as being prone to earthquake-induced liquefaction (Ref. 10).

A geologic map of the Ventura 7.5' Quadrangle (Ref. 11), indicates the site is underlain by latest Holocene alluvial fan deposits (Qhfy). The descriptions accompanying the map described this unit as composed of moderately to poorly sorted, and moderately to poorly bedded sandy clay with some gravel.

Site Reconnaissance

A site reconnaissance was performed on May 27, 2020 to observe the existing conditions and to note if any obvious geotechnical concerns are evident. The VFD conveyance canal was unwatered at the time of our site visit.

The canal is unlined and has a trapezoidal shape with 1.7H:1V to 2H:1V side slopes. The side slopes of the canal are lined with riprap, although the area immediately upstream of the TBC crossing consists of grouted riprap. The invert of the canal is generally bare earth, although there is some riprap and concrete fragments upstream and downstream of each crossing. The soils exposed in the canal and adjacent roadways consists of brown silty fine to medium sand. We did not observe indications of instabilities in the canal sideslopes nor excessive erosion or scour. Additionally, the Siphon and TBC structures and surrounding improvements appeared to be performing satisfactorily, with no visible indications of excessive settlement or other geotechnical hazards, such as cracking, buckling, or distortion.

CONCLUSIONS

Based on our review of existing data and observations from our site reconnaissance, it is our opinion from a geotechnical standpoint that the site is suitable for the proposed alternatives being considered to increase flow capacity at the facility, provided the recommendations presented below are considered in the design and construction of the project.

Based on the existing data, the general subsurface profile at the location of the proposed Siphon and TBC is expected to consist of about 8 to 10 feet of silty sand, underlain by well graded gravel with a dense to very dense consistency. The two borings drilled closest to the project site (Ref. 4) indicate the silty sand layer is relatively loose. However, the boring further to the west and within the embankment limits, suggest the silty sand layer is very dense fill material. Assuming the embankment at the Siphon and TBC sites was constructed in a similar manner as the embankment further west, it is anticipated the underlying materials consists of silty sand fill with a dense to very dense consistency. This is consistent with our observations of the existing channel slopes that indicate they appear generally stable and not showing signs of excessive erosion or scour. The alluvial soils below the embankment fill are expected to be dense to very dense, gravelly sand with some cobbles.

Groundwater was not encountered in the two borings closest to the project site (Ref. 4). Groundwater was encountered in three of the eight borings further to the west (Ref. 6) at depths between 21 and 40 feet below the ground surface. However, these boring were completed prior to the groundwater recharging system being placed in operation, so the groundwater is expected to be encountered at much shallower depths if and when new construction commences.

RECOMMENDATIONS

Based on the anticipated subsurface soil profile as discussed above, we have provided preliminary geotechnical design parameters for the development and evaluation of alternatives for the Grand Canal Headworks Improvements.

Site and Subgrade Preparation

Subgrade preparation efforts may vary based on the option selected to increase the capacity of the existing structures. Preliminary recommendations are as follows.

After dewatering the Canal, the areas of the proposed improvements should be cleared of any elements of the existing system that will not be part of the improved structure. This may include pipes, gates, electric actuators, concrete structures and foundation elements. Concreted riprap may be stockpiled at an appropriate location for reuse.

Any excavations into the existing levee will most likely encounter dense to very dense silty sands. These materials, classified as type C (Cal-OSHA), should be sloped back at an inclination of 1½ to 1 (horizontal to vertical) or temporarily shored if excavated at steeper inclination. The final determination of temporary excavation inclinations and requirements for, and design of temporary shoring, is the responsibility of the Contractor.

The subgrade surface should be reasonably free of loose soil, standing water or mud. Prior to placement of fill or rebar for the foundation system, the subgrade should be moisture conditioned and compacted to the requirements for engineered fill described below. The subgrade should be firm and unyielding, and any soft soils should be excavated to expose firm, non-yielding materials.

If the option selected involves the complete removal of the existing structure and partial removal and replacement of the levee, additional geotechnical review and analysis of the levee may be required.

Fill Materials and Placement

On-site materials are anticipated to be suitable for use as engineered fill behind and around the structures. Any soils with organic contents greater than about 3 percent by volume should not be used. Fill materials placed at the site should not contain particles greater than 3 inches in diameter. Any imported soils should be free of organics, debris, and oversize particles. It should be predominantly granular, with at least 20 percent fines and a plasticity index of 12 or less.

All engineered fill should be spread and compacted in lifts not exceeding 8 inches in uncompacted thickness. The engineered fill should be moisture conditioned to at least 2 percent over optimum and compacted to at least 95 percent relative compaction based on ASTM D-1557. Any areas of the service road damaged by the construction activities, should be repaired to match preconstruction conditions.

Existing Utilities

It is anticipated that overhead wires and possibly underground utilities cross through the project site. The contractor must coordinate with the site representative and the necessary utility owners for proper removal and/or relocation of any existing utilities that may be affected the proposed improvements. Any utility trenches should be backfilled with engineered fill as described above. A shading layer extending at least 6 inches below small diameters pipes and 6 to 12 inches above should be placed prior to placement of engineered fill. The shading layer should consist of granular materials carefully placed and tamped around the utility lines.

Allowable Bearing Pressures

A new structure for the TBC or additions to the existing structure may be supported on a shallow foundation system bearing on the dense to very dense, gravelly sand alluvium.

For preliminary design, an allowable bearing pressure of 3,000 pounds per square foot due to dead plus live loads and 4,000 pounds per square foot for all loads including wind and seismic, may be assumed. At a minimum, footings for the TBC structure should be at least 12 inches embedded into the lowest adjacent grade, such that there is at least 5 feet of horizontal cover. Spread footings may also be used for the bridge at the Siphon site provided they are at least 48 inches wide and 24 inches deep below the lowest adjacent grade.

If a slab-type structure is planned, a modulus of subgrade reaction of 100 psi/in is applicable for subgrade soils that are submerged.

Total long-term settlements of foundations designed per the above criteria are expected to be less than ½-inch. Due to the presence of granular soils, the settlements are expected to occur relatively immediately upon loading. Differential settlements are expected to be less than ½ of the total settlements. If additions are planned to the existing structure(s), it is recommended to dowel into the existing foundation to minimize differential movements.

Friction Coefficient

Lateral load resistance for the structures may be developed in friction between the foundation bottom and the supporting grade. A friction coefficient of 0.4 is considered applicable[1].

Lateral Design Parameters

Retaining walls should be evaluated to resist lateral earth pressures and any additional lateral loads caused by surcharge on the adjoining ground surface. Assuming the wall backfill consists of engineered fill with similar characteristics to the existing levee fill, the following earth pressures can be assumed for initial evaluation.

- Active earth pressures (level backslope): 35 pcf above water, 17 pcf below water.
- Passive earth pressures[1]: 285 pcf above water, 120 pcf below water.

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[1] The values presented for passive and frictional resistance can be used in combination and include factors of safety of at least 1.5 to reduce the potential for lateral movement.

- At-rest earth pressures (level backslope): 50 pcf above water, 25 pcf below water.

When using the "below water" values for active or at-rest pressures, lateral loads due to hydrostatic pressure must also be applied. The upper 1 foot should be neglected for passive resistance unless a scour resistant surface (e.g., concreted riprap or slab) is present.

Seismic Design Parameters

Based on the data reviewed and the 2019 California Building Code, the following parameters should be used for preliminary seismic design:

NAME	VALUE
Site Class	D
S_S	1.938
S_1	0.726
S_{MS}	1.938
S_{M1}	**
S_{DS}	1.292
S_{D1}	**
PGA	0.858
PGA_M	0.943

** See ASCE 7-16, §11.4.8, Exception 2.

LIMITATIONS

This Preliminary Geotechnical Evaluation Memorandum has been prepared for the sole use of NHC and United Water Conservation District, and is specific to the conditions at the Freeman Diversion Conveyance System as discussed above. The opinions, conclusions, and recommendations contained in this letter are based upon the information obtained from our review of the existing data, site reconnaissance, experience, and engineering judgment, and have been formulated in accordance with generally accepted geotechnical practices that existed at the time this letter was prepared; no other warranty is expressed or implied. In addition, the recommendations presented in this letter are based solely on surface exposures at the site. No subsurface investigation was completed by Gannett Fleming; actual conditions may vary. If conditions encountered in the field differ from those described in this letter, we should be consulted to determine if changes to the conclusions presented herein or supplemental recommendations are required.

The opinions presented in this letter are valid as of the date of this letter for the site being evaluated. Changes in the condition of a site can occur with the passage of time, whether due to natural processes or the works of man. In addition, changes in applicable standard of practice can occur, whether from legislation or the broadening of knowledge. Accordingly, the opinions presented in this letter may be invalidated, wholly or partially, by changes outside of Gannett Fleming's control. In any case, this letter should not be relied upon after a period of three years without prior review and approval by Gannett Fleming. This document may not be reproduced for any other reason than pertains to the project for which it was prepared.

REFERENCES

(Listed in order as they were referenced in the text)

1. *Agreement for Subconsultant Services, Subcontract No. 5005686-1,* between Northwest Hydraulic Consultants, Inc. (NHC) and Gannet Fleming, Inc., fully executed on May 18, 2020.
2. *Sediment Transport and Deposition Assessment of Freeman Diversion Conveyance System, Phase 2: Evaluate Alternatives, Final Report,* prepared by NHC, dated September 1, 2016.
3. *Technical Memorandum: VFD Three Barrel Culvert and Inverted Siphon Design – Alternative Analysis*, prepared by NHC, dated July 16, 2020.
4. *Geotechnical Engineering Report for United Water Conservation District, Proposed Shop Building and Water Tank, Saticoy Facility, Ventura County, California*, prepared by Earth Systems Southern California, dated May 23, 2012.
5. *Update of Geotechnical Engineering Report for United Water Conservation District, Saticoy Facility, Saticoy Area of Ventura County, California*, prepared by Earth Systems Southern California, dated May 28, 2015.
6. *Geotechnical Investigation for UWCD Noble Basin and Saticoy Spreading Grounds Improvement,* prepared by Geotechnical Consultants, Inc., dated December 1, 1993.
7. *Geotechnical Study for Construction of Four Well Pads, Saticoy Groundwater Storage Management Project*, prepared by Padre Associates, Inc., dated September 17, 2002.
8. *Quaternary fault and fold database for the United States,* accessed July 9, 2019, from USGS web site: http//earthquake.usgs.gov/hazards/qfaults/., U.S. Geological Survey and California Geological Survey, 2006.
9. *Earthquake Fault Zones (Alquist-Priolo Earthquake Fault Zoning Act)*, by California Division of Mines and Geology, 1978.
10. *Seismic Hazard Zones, Saticoy 7.5-minute Quadrangle*, by Department of Conservation, California Geological Survey, 2003
11. *Geologic Map of the Saticoy 7.5-minute Quadrangle, Ventura County, California: A Digital Database,* by Siang S. Tan, Terry A. Jones, and Kevin B. Clahan, Department of Conservation, California Geological Survey, 2003.

Gannett Fleming

VICINITY MAP Vern Freeman Diversion System Three Barrel Culvert & Inverted Siphon VENTURA COUNTY, CALIFORNIA				FIGURE 1 1 OF 1
Date: SEP 2020	By: A. LUNA	Scale: as shown	Project No: 67376	

PLOT DATE: Tuesday, September 29, 2020 TIME: 8:01:09 PM BY: LUNA, ALMA TAB: SITE PLAN
FILE: C:\Users\aluna\Desktop\Freeman Diversion\504-CADD\8-Figures\65853 Vicinity Map Fig.1.dwg

ATTACHMENT C – REFERENCE INFORMATION REGARDING EXISTING GAS LINE

Allen, Jennifer S.

From: Bryce Cruey <BCruey@nhcweb.com>
Sent: Friday, September 25, 2020 12:13 PM
To: Allen, Jennifer S.
Cc: Conrad, Ryan; Ed Wallace
Subject: RE: TBC 30% Design

Jen,

Just to follow up on our phone conversation and to have this issue resolved in email, I summarized the resolution below.

According to Craig at United, the 18" gas lines that are shown in the "As-built" drawings were replaced sometime in recent years by So.Cal. Gas. The new gas line is the 22" line that is shown in the figure Craig sent and it crosses the channel somewhere downstream or upstream of the culverts and is 8 or more feet below the channel invert. Based on the proposed inverts, there should be no issues at all with the high pressure 22" gas line.

Regarding the removal of the abandoned 18" lines (if they still exist) should be included in the removals sheet or summary for the contractor. There would be a utility locate before digging here, Craig will let us know fast if the 18" lines don't exist anymore and it will only be a small correction.

Have a great weekend!

Bryce Cruey, P.E., C.F.M.
northwest hydraulic consultants
2600 Capitol Ave, Ste 140 | Sacramento, CA 95816 | United States
I am currently working remotely due to COVID-19 and can be reached on my cell phone (612) 418-0565
bcruey@nhcweb.com www.nhcweb.com

From: Allen, Jennifer S. <jeallen@GFNET.com>
Sent: Friday, September 25, 2020 11:46 AM
To: Bryce Cruey <BCruey@nhcweb.com>
Subject: RE: TBC 30% Design

I can chat now, or sometime after lunch.
-Jen

From: Bryce Cruey <BCruey@nhcweb.com>
Sent: Friday, September 25, 2020 10:53 AM
To: Allen, Jennifer S. <jeallen@GFNET.com>
Subject: RE: TBC 30% Design

Jen,

Let me know if you have a minute to chat. I ca call you via teams and update you as to the conversation that I just had with Craig.

Bryce

From: Allen, Jennifer S. <jeallen@GFNET.com>
Sent: Friday, September 25, 2020 10:31 AM
To: Bryce Cruey <BCruey@nhcweb.com>
Cc: Conrad, Ryan <rconrad@gfnet.com>
Subject: RE: TBC 30% Design

Thanks Bryce.

From: Bryce Cruey <BCruey@nhcweb.com>
Sent: Friday, September 25, 2020 10:28 AM
To: Allen, Jennifer S. <jeallen@GFNET.com>
Cc: Conrad, Ryan <rconrad@gfnet.com>
Subject: RE: TBC 30% Design

Jen,

I have not heard back yet. I just pinged him again.

Bryce

From: Allen, Jennifer S. <jeallen@GFNET.com>
Sent: Wednesday, September 23, 2020 2:23 PM
To: Bryce Cruey <BCruey@nhcweb.com>
Cc: Conrad, Ryan <rconrad@gfnet.com>
Subject: RE: TBC 30% Design

Hi Bryce,

Were you able to get any other information from Craig regarding the correct gas line? Or did you want me to respond directly back to Craig?

Thanks,
Jen

From: Allen, Jennifer S.
Sent: Monday, September 21, 2020 5:56 PM
To: Bryce Cruey <BCruey@nhcweb.com>
Cc: Conrad, Ryan <rconrad@gfnet.com>
Subject: FW: TBC 30% Design

Hey Bryce,

I don't think this is the same gas line. The drawing is showing a 22" line under the canal. We are talking about 2 – 18" lines going under the siphon.

I think we are going to need further clarification still.
Thanks,
Jen

From: Craig Morgan <craigm@unitedwater.org>
Sent: Monday, September 21, 2020 4:55 PM
To: 'Bryce Cruey' <BCruey@nhcweb.com>
Cc: Allen, Jennifer S. <jeallen@GFNET.com>
Subject: RE: TBC 30% Design

Bryce,

Attached is what we have on the plans for the So Cal Gas Company's lines. They are well below the invert of the canal and even a little deeper than what is shown as there were issues with the boring.

Best Regards,

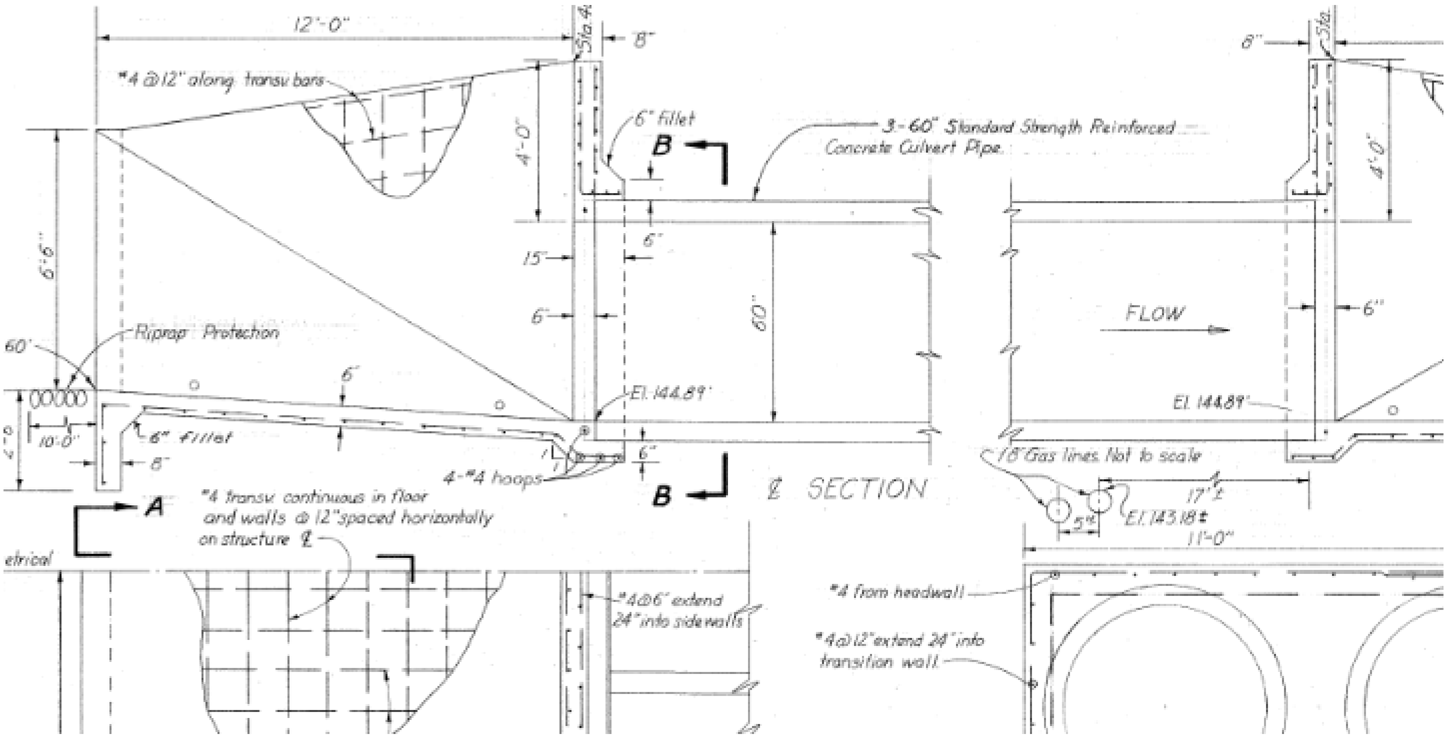
Craig Morgan, P.E. | Senior Engineer
United Water Conservation District
Main (805) 525-4431 • Direct (805) 695-3743

From: Bryce Cruey <BCruey@nhcweb.com>
Sent: Monday, September 21, 2020 4:17 PM
To: Craig Morgan <craigm@unitedwater.org>
Cc: Allen, Jennifer S. <jeallen@GFNET.com>
Subject: TBC 30% Design

[EXTERNAL]
Craig,

I was talking with Jen over at Gannett Flemming about the 30% design for TBC. There were a few questions that come up based on potential utility conflicts. As shown in the drawing below there are two 18" gas lines that are shown crossing the channel at the TBC perpendicular at an elevation of 143.18. We are proposing to put the new culverts at 144.03 which would not be an issue necessarily, but the as-built elevations did differ with the most recent survey data on the culvert inverts by about 0.5'. That would be a little too close for comfort with utility conflicts. However, the field notes indicate that the lines might actually be crossing over the top of the existing culverts. Additionally, water lines were observed crossing the channel at the TBC. Long story short is that we will need to get a utility locate on these as soon as possible, but certainly, before we have bid ready plans. Can you let me know if United has any information on these utilities? Do we know who owns them?

Secondly, is there any word on the Inverted Siphon?

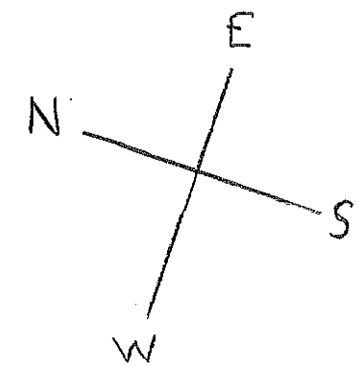


Thanks.

Bryce Cruey, P.E., C.F.M.
northwest hydraulic consultants
2600 Capitol Ave, Ste 140 | Sacramento, CA 95816 | United States
Tel: (916) 371-7400, Ext. 1121
bcruey@nhcweb.com www.nhcweb.com

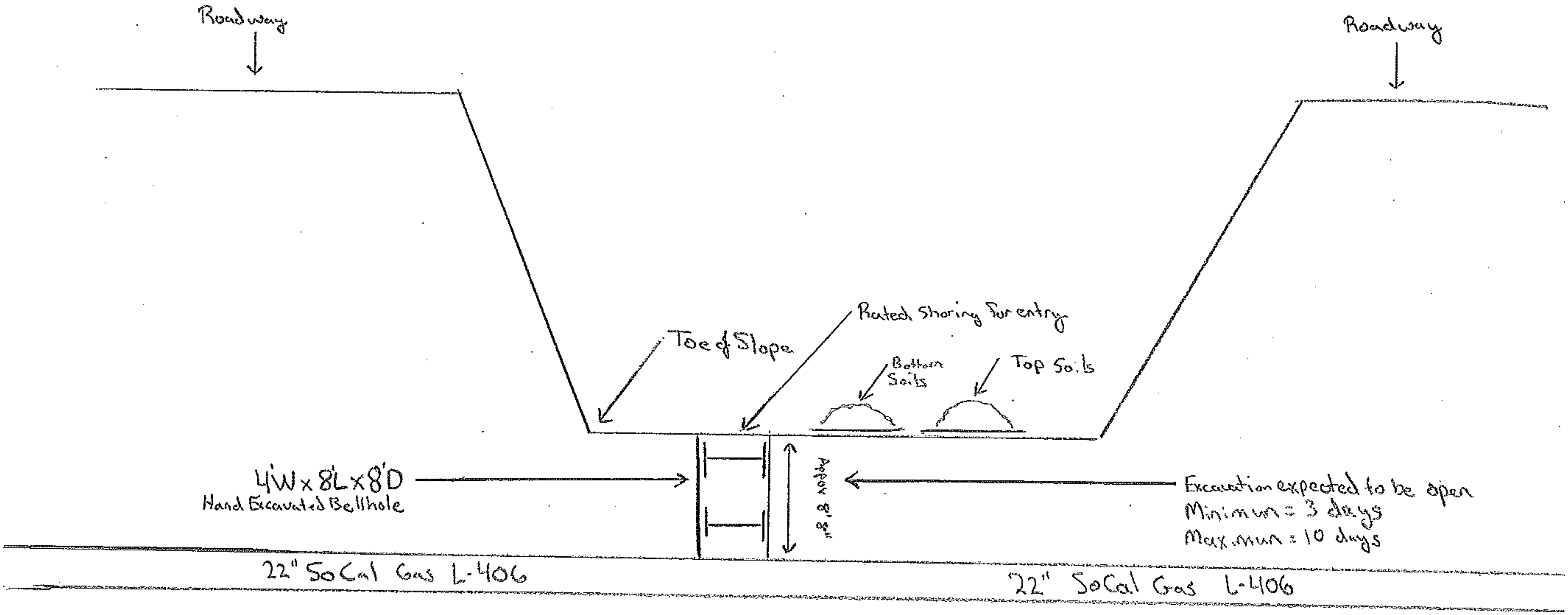
Canal Pothole Drawing - United Water Company

Work presumed to be done under dry conditions



Soil/Bedding Removal/Backfill

1. Place plastic down for separation
2. Remove appox 4' of soils and place on plastic
3. Remove appox 4' of bottom soils on additional plastic
4. Replace and compact bottom layer of native material after completion
5. Replace and compact top layer of native material after completion



ATTACHMENT D – DESIGN CALCULATIONS

ATTACHMENT D
90 Percent Hydraulic Design Notes - Selected Alternative

Jul-23
USACE 1110-2-2902 requires 2 gate closure for pipes > 36 in
For fast rise in flood waters, 1 active and one passive required; for slow rise 2 active acceptable
Passive gates not feasible with canal function
Use active closure gates both sides - 2 fabricated slide gates
USACE 1110-2-2902 3.3.3.2.4 does not allow use of precast RCB - RCB must be CIP; sloped side walls may be requested (see 4.17.4)

Box Culverts

Options Considered

Several RAS plans run with new rating curve for Inv Siphon based on A3_lin750_5-10-22 L&5gates lower from system model
and 0530 at Inv Siphon sta 17651

2 RCBs at 3BC				750 cfs	sta 17651		500 cfs			
	Span	Rise	US Inv	US WSE	Channel V	Culvert V	US WSE	Channel V	Culvert V	Notes
3BCR1	12	6	144.1	151.72	3.58	5.21	150.05	2.96	3.47	US WSE 1.6' above crown at750 cfs, approx a
3BCR2	14	6	144.1	151.58	3.67	4.46	150.02	2.99	2.98	US WSE 1.5' above crown at 750 cfs, approx a
3BCR3	14	8	144.1	151.39	3.82	3.7	150.02	3.02	2.77	Not sealed
3BCR4	10	6	144.1	151.99	3.4	6.25	150.15	2.89	4.17	US WSE 2' above crown at 750 cfs, approx at
3BCR5	12	8	144.1	151.45	3.77	4.34	150.05	3	3.24	Not sealed
3BCR6	14	7	144.1	151.43	3.78	3.83	150.02	3.02	2.77	WSE 0.41' above crown 750 cfs, not sealed at
3BCR8	14	6	144.1	151.58	3.67	4.46	150.02	2.99	2.98	WSE 1.6' above crown 750 cfs, approx at crov
3BCR9	14	6.5	144.1	151.5	3.75	4.12	150.01	3.02	2.77	WSE 1.5' above crown 750 cfs, not sealed at !

Selected Alternative

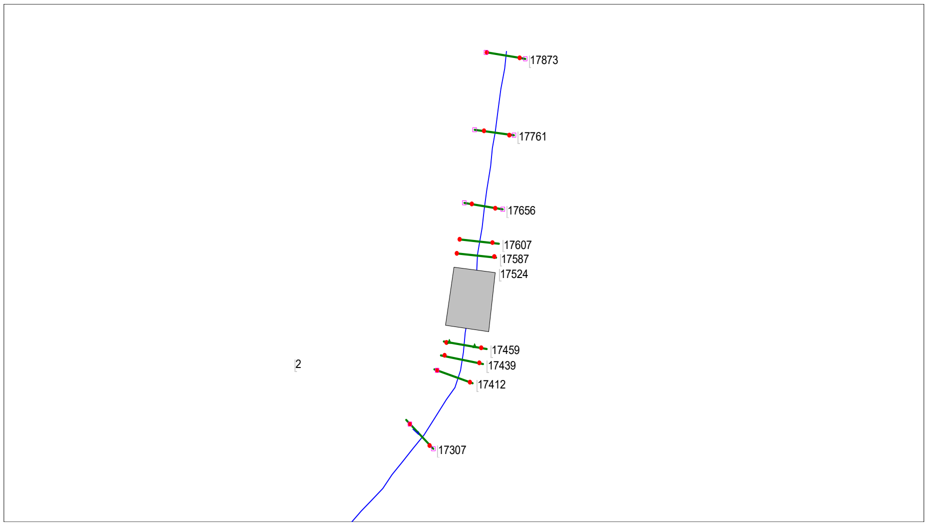
	W	H	Entrance Loss Coeff	Exit Loss Coeff	n Culvert
Selected culvert size	14	7	0.5	0.5	0.02

Represented in HEC-RAS Plan SiphonR3_3BCR6.1

Represents channel and headwall gepmetry from 90% draft plans
Cross sections developed from terrain based on field survey by Stantec May 2020
Note: results for Plan 3BCR6.1 differ slightky from 3BCR6 due to more
accurate representation of 90 percent channel geometry and wingwalls

ATTACHMENT D
90 Percent Hydraulic Design Notes - Selected Alternative

Jul-23



HEC-RAS Cross Section Layout: TBC Culvert at Sta 17524 - 83' long

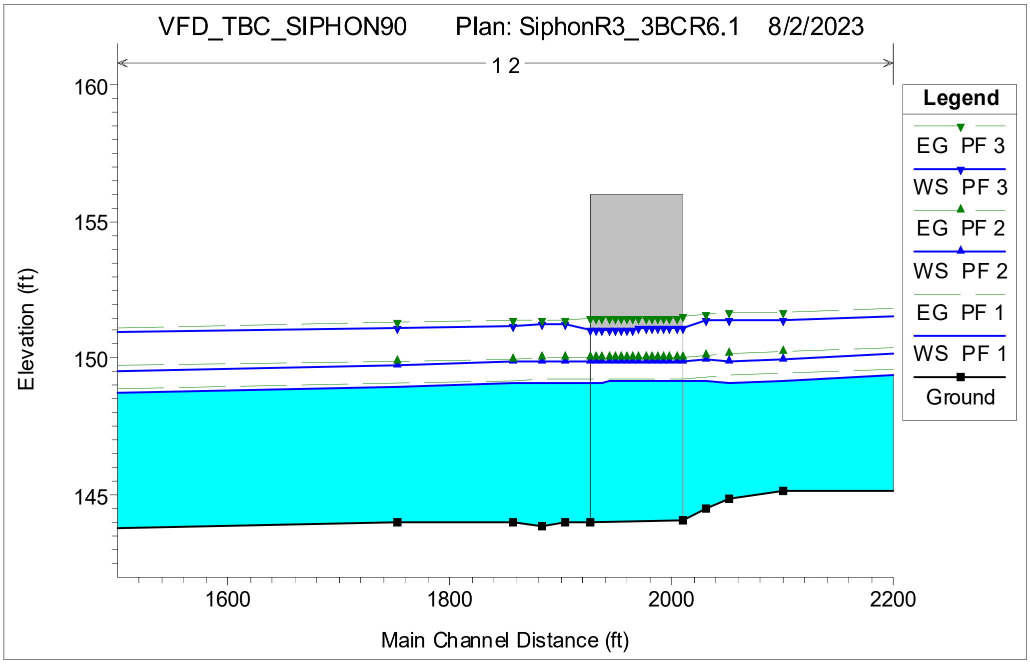
Plan 3BCR6.1
RAS Results

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
2	17873	PF 1	375	144.11	149.64	146.81	149.74	0.000503	2.56	146.25	39.45	0.23
2	17873	PF 2	500	144.11	150.39	147.21	150.51	0.000526	2.83	176.83	42.39	0.24
2	17873	PF 3	750	144.11	151.76	147.89	151.92	0.000514	3.14	238.87	47.76	0.25
2	17761	PF 1	375	145.12	149.4	148.01	149.64	0.001632	4.03	96.9	35.5	0.42
2	17761	PF 2	500	145.12	150.15	148.4	150.41	0.001408	4.18	125.01	38.82	0.4
2	17761	PF 3	750	145.12	151.56	149.05	151.83	0.001075	4.28	184.02	45.09	0.36
2	17656	PF 1	375	145.15	149.18	147.8	149.45	0.001849	4.15	90.55	32.5	0.43
2	17656	PF 2	500	145.15	149.96	148.21	150.25	0.001531	4.3	117.11	35.58	0.41
2	17656	PF 3	750	145.15	151.4	148.91	151.7	0.001139	4.45	172.22	41.17	0.37
2	17607	PF 1	375	144.88	149.11		149.36	0.001692	4	93.72	32.4	0.41
2	17607	PF 2	500	144.88	149.9		150.17	0.00147	4.15	120.62	35.34	0.4
2	17607	PF 3	750	144.88	151.36		151.64	0.001146	4.26	176.16	40.74	0.36
2	17587	PF 1	375	144.5	149.14	147.04	149.31	0.000961	3.27	114.84	35.03	0.32
2	17587	PF 2	500	144.5	149.94	147.44	150.12	0.000903	3.48	143.78	37.87	0.31
2	17587	PF 3	750	144.5	151.39	148.15	151.6	0.000779	3.7	202.63	43.08	0.3
2	17524		Culvert									
2	17459	PF 1	375	144	149.11		149.23	0.000624	2.76	135.64	38.59	0.26
2	17459	PF 2	500	144	149.88		150.02	0.000607	3.01	166.27	41.69	0.26
2	17459	PF 3	750	144	151.22		151.4	0.000534	3.39	221.08	47.08	0.26
2	17439	PF 1	375	143.86	149.13		149.21	0.000434	2.38	157.38	42.58	0.22
2	17439	PF 2	500	143.86	149.9		150	0.000439	2.61	191.24	45.21	0.22
2	17439	PF 3	750	143.86	151.24		151.38	0.000436	2.94	255.11	49.8	0.23
2	17412	PF 1	375	144	149.09	146.63	149.2	0.000566	2.59	144.63	41.99	0.25
2	17412	PF 2	500	144	149.87	146.99	149.99	0.000551	2.81	178.11	44.79	0.25
2	17412	PF 3	750	144	151.21	147.6	151.36	0.000518	3.1	241.62	49.44	0.25
2	17307	PF 1	375	144	148.97	146.88	149.12	0.000901	3.12	120.15	37.31	0.31
2	17307	PF 2	500	144	149.74	147.27	149.91	0.00085	3.33	150.05	40.25	0.3
2	17307	PF 3	750	144	151.09	147.94	151.29	0.000757	3.61	207.5	44.51	0.3

ATTACHMENT D
90 Percent Hydraulic Design Notes - Selected Alternative

Jul-23

2	16917 PF 1	375	143.65	148.66	146.54	148.79	0.000784	2.91	128.72	40.37	0.29
2	16917 PF 2	500	143.65	149.46	146.9	149.6	0.000718	3.08	162.17	43.5	0.28
2	16917 PF 3	750	143.65	150.85	147.53	151.02	0.000628	3.31	226.41	48.95	0.27
2	16776 PF 1	375	143.57	148.57	146.28	148.69	0.000664	2.77	135.4	40.53	0.27
2	16776 PF 2	500	143.57	149.37	146.64	149.51	0.000621	2.96	169.15	43.37	0.26
2	16776 PF 3	750	143.57	150.77	147.26	150.93	0.00056	3.22	233.26	48.31	0.26
2	16472 PF 1	375	143.38	148.37	145.88	148.49	0.000611	2.73	137.51	39.41	0.26
2	16472 PF 2	500	143.38	149.19	146.29	149.32	0.000583	2.93	170.74	42.13	0.26
2	16472 PF 3	750	143.38	150.6	146.94	150.76	0.000539	3.21	233.62	46.85	0.25
2	16395 PF 1	375	143.33	148.38	144.92	148.44	0.000227	1.96	191.45	44.21	0.16
2	16395 PF 2	500	143.33	149.2	145.24	149.27	0.000233	2.22	225.72	46.18	0.17
2	16395 PF 3	750	143.33	150.61	145.83	150.72	0.000241	2.63	284.97	49.59	0.18
2	16384 PF 1	375	143.33	148.29	145.04	148.42	0.000358	2.87	130.85	19.63	0.23
2	16384 PF 2	500	143.33	149.05	145.41	149.24	0.000555	3.47	144.17	13.16	0.26
2	16384 PF 3	750	143.33	150.23	146.06	150.62	0.001517	5.03	148.98		0.34



Hydraulic Profile Through Culvert Crossing

ATTACHMENT D
90 Percent Hydraulic Design Notes - Selected Alternative

Jul-23

Cross Drainage Piping

	Proposed vs existing capacity			
	hdpe - 30 in OD		concrete - 36 in ID	
D=	26.28	" ID	36	" ID
D=	2.19	ft	3	ft
A=	3.77	sf	7.0686	
P=	6.88	ft	9.4248	
R=	0.55	ft	0.75	
n=	0.011	hdpe	0.013	concrete

Conveyance
K=Q/Sf= 1.486/n*R^0.666*A

K=	340.6316	per 30"
	681.2632	for 2- 30"
K=	667.1134	for 36" concrete

Capacity of 36" pipe - segment over 3-60" pipes		Inlet El	150.74	
		Outlet El	150.12	
HY-8 result	39 cfs @	Headwater	153.96	inlet submerged 0.2 ft

Capacity of 2-30" HDPE

Replace pipe and concrete channel with 2-30" OD HDPE pipes (1.765 in wall)

2-26" ID Inv in	152.7	ft		
2-26" ID inv out	152.3	ft	152.2	bottom OD
Length	113	ft		
Slope	0.00354			
Vf	5.41	ft/s	uniform flow	
Qf	39.9	cfs	2 pipes	

		Inlet El	152.7	
		Outlet El	152.3	
HY-8 result	39 cfs@	Headwater	154.98	WSE below crown 0.2 ft
		Grd at Inlet	158	Freeboard ~3 ft
Inv at inlet from south			154.1	2-30" should not backwater inlet
TOW at inlet			158	

Assume full flow in culvert connection to linear basin (~backwatered by Segment 2)

Inlet El	152.9			
Outlet El	152.8			
V=	4.0 ft/s			
Outlet Head Loss C=1.0	HL	0.25	ft	
Inlet Head Loss C=0.5	HL	0.12	ft	
Use Headwater		155.52	ft	Freeboard 2.5 ft
HY8 result		155.21	ft	

WINGWALL DESIGN (14 FT)

Calculation is based on user defined combination values

Tedds calculation version 2.9.11

Overall design summary provided by Tedds software does not account for overall structural system. Design explanations provided in results.

Analysis summary

~~**Design summary**~~

~~Overall design utilisation~~ ~~3.279~~

~~Overall design status~~ ~~Fail~~

By inspection, wingwall is stabilized against sliding due to tie-in with concrete apron.

Description	Unit	Capacity	Applied	F o S	Result
~~Sliding stability~~	~~plf~~	~~3740~~	~~12263~~	~~0.305~~	~~FAIL~~
Overturning stability	lb_ft/ft	138198	94688	1.460	PASS
Bearing pressure	psf	4000	1120	3.571	PASS

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Stem p0 rear face - Flexural reinforcement	in²/ft	1.571	1.254	0.798	PASS
Stem p0 - Shear resistance	lb/ft	13175	11083	0.841	PASS
Base bottom face - Flexural reinforcement	in²/ft	1.571	1.512	0.962	PASS
Base - Shear resistance	lb/ft	14582	5460	0.374	PASS
Transverse stem reinforcement	in²/ft	0.884	0.389	0.440	PASS
Transverse base reinforcement	in²/ft	0.884	0.389	0.440	PASS

Retaining wall details

Stem type Cantilever

Stem height h_{stem} = **14** ft

Conservative rounding of wingwall height at tallest section for design purposes.

Stem thickness t_{stem} = **18** in

Angle to rear face of stem α = **90** deg

Stem density γ_{stem} = **150** pcf

Toe length l_{toe} = **22.5** ft

Aligning with extent of concrete apron.

Base thickness t_{base} = **18** in

t_base = h_base

Base density γ_{base} = **150** pcf

Height of retained soil h_{ret} = **14** ft

Angle of soil surface β = **0** deg

Conservative retained soil parameters for tallest section of wingwall for design purposes.

Depth of cover d_{cover} = **0** ft

Retained soil properties

Soil type Dense fine or silty sand

Moist density γ_{mr} = **120** pcf

Saturated density γ_{sr} = **120** pcf

Based on Sept. 2020 Geotechnical Memorandum by Gannett Fleming (Geotech Memo).

Base soil properties

Soil type Dense fine or silty sand

Soil density γ_b = **120** pcf

Gross allowable bearing pressure q_{allow_gross} = **4000** psf

Based on Geotech Memo for seismic loading plus all other loads.

Seismic details

Horizontal seismic acceleration factor K_h = **0.472**

Vertical seismic acceleration factor K_v = **0**

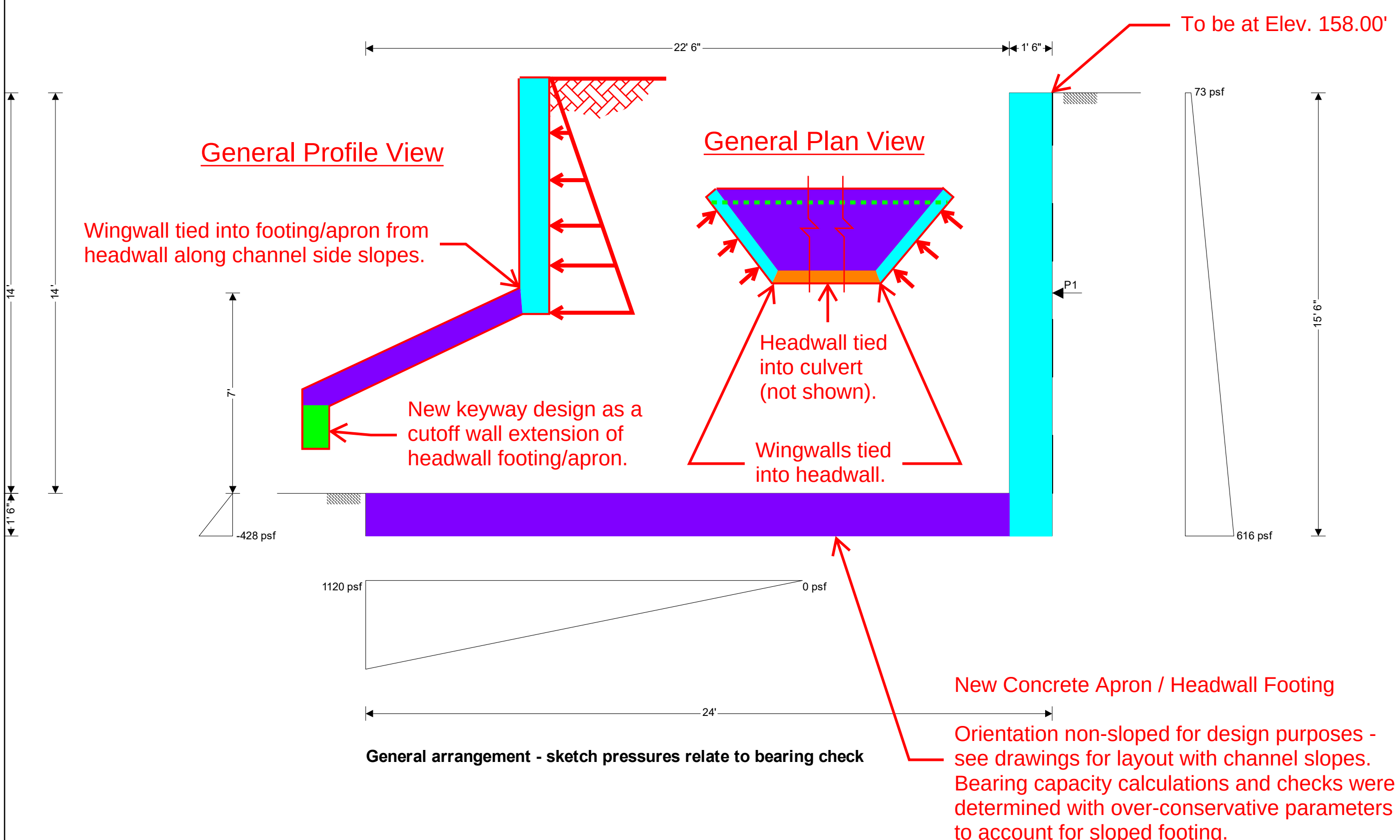
Kh = 0.5*(PGAm) = 0.5*(0.943), from Geotech Memo; Kv = 0 (conservative assumption)

Seismic acceleration angle $\theta = \text{atan}(K_h / (1 - K_v))$ = **25.267** deg

Loading details

Live surcharge load — $Surcharge_L$ = **250** psf

Horizontal line load at 7 ft — P_{D1} = **263** plf ← Type 60K Concrete Barrier Load = 900 plf x Ka



General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Base length — $l_{base} = l_{toe} + t_{stem} = \mathbf{24}$ ft

Moist soil height — $h_{moist} = h_{soil} = \mathbf{14}$ ft

- Distance to horizontal seismic component — $x_{seismic\_h} = 0.6 \times (h_{soil} + h_{base}) = \mathbf{9.3}$ ft

Length of surcharge load — $l_{sur} = l_{heel} = \mathbf{0}$ ft

- Distance to vertical component — $x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{24}$ ft

Effective height of wall — $h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{15.5}$ ft

- Distance to horizontal component — $x_{sur\_h} = h_{eff} / 2 = \mathbf{7.75}$ ft

Area of wall stem — $A_{stem} = h_{stem} \times t_{stem} = \mathbf{21}$ ft^2

- Distance to vertical component — $x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{23.25}$ ft

Area of wall base — $A_{base} = l_{base} \times t_{base} = \mathbf{36}$ ft^2

- Distance to vertical component — $x_{base} = l_{base} / 2 = \mathbf{12}$ ft

Soil coefficients

Coefficient of friction to back of wall — K_{fr} = **0.400**

Coefficient of friction to front of wall — K_{fb} = **0.400** ← Based on Geotech Memo.

Coefficient of friction beneath base — K_{fbb} = **0.400**

Active pressure coefficient — K_A = **0.292** ← Ka = 35 pcf / 120 pcf, from Geotech Memo.

Passive pressure coefficient K_P = **2.375** Kp = 285 pcf / 120 pcf, from Geotech Memo.

Using Mononobe-Okabe theory

Active dynamic pressure coefficient K_{AE} = **0.754**

Passive dynamic pressure coefficient K_{PE} = **2.374** Assumed as Kp. Tedds software requires Kp and Kpe to differ, so changed slightly.

User defined combination

Load combination 1 $1 \times$ Dead + $1 \times$ Live + $1 \times$ Lateral earth

Sliding check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem}$ = **3150** plf

Wall base $F_{base} = A_{base} \times \gamma_{base}$ = **5400** plf

Total $F_{total\_v} = F_{stem} + F_{base}$ = **8550** plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff}$ = **1132** plf

Line loads $F_{P\_h} = P_{D1}$ = **263** plf

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2$ = **4209** plf

Total $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h}$ = **5604** plf

By inspection, wingwall is stabilized against sliding due to tie-in with concrete apron.

Check stability against sliding

Base soil resistance $F_{exc\_h} = K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2$ = **321** plf

Base friction $F_{friction} = F_{total\_v} \times K_{fbb}$ = **3420** plf

Resistance to sliding $F_{rest} = F_{exc\_h} + F_{friction}$ = **3741** plf

Factor of safety $FoS_{sl} = F_{rest} / F_{total\_h}$ = **0.668** < 1.5

FAIL - Factor of safety against sliding is inadequate

Overturning check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem}$ = **3150** plf

Wall base $F_{base} = A_{base} \times \gamma_{base}$ = **5400** plf

Total $F_{total\_v} = F_{stem} + F_{base}$ = **8550** plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff}$ = **1132** plf

Line loads $F_{P\_h} = P_{D1}$ = **263** plf

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2$ = **4209** plf

Base soil $F_{exc\_h} = -K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2$ = **-321** plf h_pass = 0 h_base = t_base

Total $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{exc\_h}$ = **5283** plf

Overturning moments on wall

Surcharge load $M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h}$ = **8769** lb_ft/ft

Line loads $M_{P\_OT} = abs(P_{D1}) \times (p_1 + t_{base})$ = **2235** lb_ft/ft p1 = 7 ft

Moist retained soil $M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h}$ = **21747** lb_ft/ft

Total $M_{total\_OT} = M_{sur\_OT} + M_{P\_OT} + M_{moist\_OT}$ = **32752** lb_ft/ft

Restoring moments on wall

Wall stem $M_{stem\_R} = F_{stem} \times x_{stem}$ = **73237** lb_ft/ft

Wall base $M_{base\_R} = F_{base} \times x_{base}$ = **64800** lb_ft/ft

Base soil $M_{exc\_R} = -F_{exc\_h} \times x_{exc\_h} =$ **160** lb_ft/ft

Total $M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{exc\_R} =$ **138198** lb_ft/ft

Check stability against overturning

Factor of safety $FoS_{ot} = M_{total\_R} / M_{total\_OT} =$ **4.22** > 1.5

In conformance with USACE EM 1110-2-2100 (Dec. 2005), Table 3-3, Ordinary Category, Usual Load Condition.

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} =$ **3150** plf

Wall base $F_{base} = A_{base} \times \gamma_{base} =$ **5400** plf

Total $F_{total\_v} = F_{stem} + F_{base} =$ **8550** plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} =$ **1132** plf

Line loads $F_{P\_h} = P_{D1} =$ **263** plf

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2 =$ **4209** plf

Base soil $F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 =$ **-321** plf

Total $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{pass\_h} - F_{total\_v} \times K_{fbb} =$ **1863** plf

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} =$ **73237** lb_ft/ft

Wall base $M_{base} = F_{base} \times x_{base} =$ **64800** lb_ft/ft

Surcharge load $M_{sur} = -F_{sur\_h} \times x_{sur\_h} =$ **-8769** lb_ft/ft

Line loads $M_P = -(P_{D1} \times (p_1 + t_{base})) =$ **-2235** lb_ft/ft

Moist retained soil $M_{moist} = -F_{moist\_h} \times x_{moist\_h} =$ **-21747** lb_ft/ft

Base soil $M_{pass} = -F_{pass\_h} \times x_{pass\_h} =$ **160** lb_ft/ft

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{moist} + M_{pass} =$ **105446** lb_ft/ft

Check bearing pressure

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} =$ **12.333** ft

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 =$ **0.333** ft

Loaded length of base $l_{load} = l_{base} =$ **24** ft

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) =$ **327** psf

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) =$ **386** psf

Allowable bearing capacity $q_{allow} = q_{allow\_gross} =$ **4000** psf

Factor of safety $FoS_{bp} = q_{allow} / \max(q_{toe}, q_{heel}) =$ **10.366**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

User defined combination

Load combination 2 1 × Dead + 1 × Live + 1 × Earthquake + 1 × Lateral earth

~~**Sliding check**~~

~~**Vertical forces on wall**~~

~~Wall stem~~ $\cancel{F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{3150}}$ ~~plf~~

~~Wall base~~ $\cancel{F_{base} = A_{base} \times \gamma_{base} = \mathbf{5400}}$ ~~plf~~

~~Total~~ $\cancel{F_{total\_v} = F_{stem} + F_{base} = \mathbf{8550}}$ ~~plf~~

By inspection, wingwall is stabilized against sliding due to tie-in with concrete apron.

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} = \mathbf{1132}$ plf

Line loads $F_{P\_h} = P_{D1} = \mathbf{263}$ plf

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2 = \mathbf{4209}$ plf

Seismic $F_{seismic\_h} = (K_{AE} - K_A) \times \gamma_{mr} \times (h_{soil} + h_{base})^2 / 2 = \mathbf{6660}$ plf

Total $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{seismic\_h} = \mathbf{12263}$ plf

Check stability against sliding

Base soil resistance $F_{exc\_h} = K_{PE} \times \gamma_b \times (h_{pass} + h_{base})^2 / 2 = \mathbf{320}$ plf

By inspection, wingwall is stabilized against sliding due to tie-in with concrete apron.

Base friction $F_{friction} = F_{total\_v} \times K_{fbb} = \mathbf{3420}$ plf

Resistance to sliding $F_{rest} = F_{exc\_h} + F_{friction} = \mathbf{3740}$ plf

Factor of safety $FoS_{sl} = F_{rest} / F_{total\_h} = \mathbf{0.305} < 1.1$

FAIL - Factor of safety against sliding is inadequate

Overturning check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{3150}$ plf

Wall base $F_{base} = A_{base} \times \gamma_{base} = \mathbf{5400}$ plf

Total $F_{total\_v} = F_{stem} + F_{base} = \mathbf{8550}$ plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} = \mathbf{1132}$ plf

Line loads $F_{P\_h} = P_{D1} = \mathbf{263}$ plf

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2 = \mathbf{4209}$ plf

Base soil $F_{exc\_h} = -K_{PE} \times \gamma_b \times (h_{pass} + h_{base})^2 / 2 = \mathbf{-320}$ plf

Seismic $F_{seismic\_h} = (K_{AE} - K_A) \times \gamma_{mr} \times (h_{soil} + h_{base})^2 / 2 = \mathbf{6660}$ plf

Total $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{exc\_h} + F_{seismic\_h} = \mathbf{11943}$ plf

Overturning moments on wall

Surcharge load $M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h} = \mathbf{8769}$ lb_ft/ft

Line loads $M_{P\_OT} = abs(P_{D1}) \times (p_1 + t_{base}) = \mathbf{2235}$ lb_ft/ft

Moist retained soil $M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h} = \mathbf{21747}$ lb_ft/ft

Seismic $M_{seismic\_OT} = F_{seismic\_h} \times x_{seismic\_h} = \mathbf{61935}$ lb_ft/ft

Total $M_{total\_OT} = M_{sur\_OT} + M_{P\_OT} + M_{moist\_OT} + M_{seismic\_OT} = \mathbf{94688}$ lb_ft/ft

Restoring moments on wall

Wall stem $M_{stem\_R} = F_{stem} \times x_{stem} = \mathbf{73237}$ lb_ft/ft

Wall base $M_{base\_R} = F_{base} \times x_{base} = \mathbf{64800}$ lb_ft/ft

Base soil $M_{exc\_R} = -F_{exc\_h} \times x_{exc\_h} = \mathbf{160}$ lb_ft/ft

Total $M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{exc\_R} = \mathbf{138198}$ lb_ft/ft

Check stability against overturning

Factor of safety $FoS_{ot} = M_{total\_R} / M_{total\_OT} = \mathbf{1.46} > 1.1$

In conformance with USACE EM 1110-2-2100 (Dec. 2005), Table 3-3, Ordinary Category, Extreme Load Condition.

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{3150}$ plf

Wall base $F_{base} = A_{base} \times \gamma_{base} =$ **5400** plf

Total $F_{total\_v} = F_{stem} + F_{base} =$ **8550** plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} =$ **1132** plf

Line loads $F_{P\_h} = P_{D1} =$ **263** plf

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2 =$ **4209** plf

Base soil $F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 =$ **-321** plf

Seismic $F_{seismic\_h} = (K_{AE} - K_A) \times \gamma_{mr} \times (h_{soil} + h_{base})^2 / 2 =$ **6660** plf

Total $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{pass\_h} + F_{seismic\_h} - F_{total\_v} \times K_{fbb} =$ **8523** plf

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} =$ **73237** lb_ft/ft

Wall base $M_{base} = F_{base} \times x_{base} =$ **64800** lb_ft/ft

Surcharge load $M_{sur} = -F_{sur\_h} \times x_{sur\_h} =$ **-8769** lb_ft/ft

Line loads $M_P = -(P_{D1} \times (p_1 + t_{base})) =$ **-2235** lb_ft/ft

Moist retained soil $M_{moist} = -F_{moist\_h} \times x_{moist\_h} =$ **-21747** lb_ft/ft

Base soil $M_{pass} = -F_{pass\_h} \times x_{pass\_h} =$ **160** lb_ft/ft

Seismic $M_{seismic} = -F_{seismic\_h} \times x_{seismic\_h} =$ **-61935** lb_ft/ft

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{moist} + M_{pass} + M_{seismic} =$ **43510** lb_ft/ft

Check bearing pressure

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} =$ **5.089** ft

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 =$ **-6.911** ft

Loaded length of base $l_{load} = 3 \times \bar{x} =$ **15.267** ft

Bearing pressure at toe $q_{toe} = 2 \times F_{total\_v} / l_{load} =$ **1120** psf

Bearing pressure at heel $q_{heel} =$ **0** psf

Allowable bearing capacity $q_{allow} = q_{allow\_gross} =$ **4000** psf

Factor of safety $FoS_{bp} = q_{allow} / \max(q_{toe}, q_{heel}) =$ **3.571**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

WINGWALL DESIGN (14 FT)

Retaining wall design in accordance with ACI 318-19

In conformance with USACE EM 1110-2-2104 (Nov. 2016), Sections 2 and 3. Additional information provided, where applicable.

Tedds calculation version 2.9.11

Concrete details

Compressive strength of concrete $f'_c =$ **4500** psi

Concrete type Normal weight

Reinforcement details

Yield strength of reinforcement $f_y =$ **60000** psi

Modulus of elasticity or reinforcement $E_s =$ **29000000** psi

Compression-controlled strain limit $\varepsilon_{ty} =$ **0.002**

Cover to reinforcement

In conformance with USACE EM 1110-2-2104 (Nov. 2016), Table 2-1, for "formed and screeded surfaces such as stilling basin walls, chute spillway slabs, and channel lining slabs on grade: greater than 12 in. and less than 24 in. thick".

Front face of stem $c_{sf} =$ **3** in

Rear face of stem $c_{sr} =$ **3** in

Top face of base $c_{bt} =$ **3** in

Bottom face of base $c_{bb} =$ **3** in

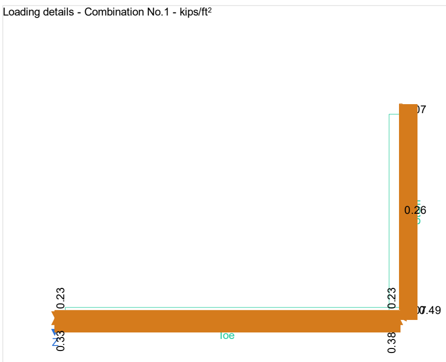
User defined load combinations

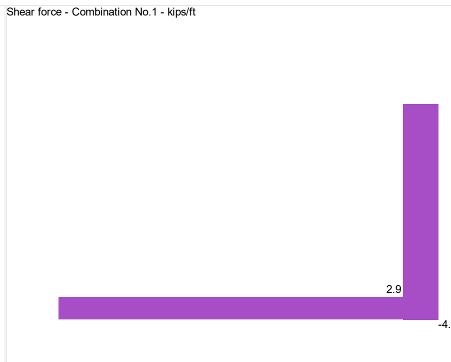
In conformance with USACE EM 1110-2-2104 (Nov. 2016), Table 3-1. Loads with "f" are considered "favorable" conditions by Tedds, but calculations below only use "unfavorable" (non-"f") loads.

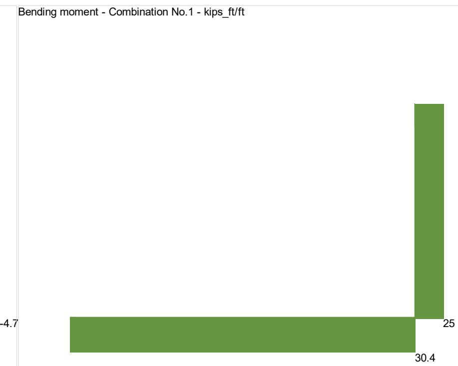
Load combination no.1	2.2D + 1Df + 2.2L + 1Lf + 2.2H + 1Hf
Load combination no.2	2.2D + 1Df + 2.2L + 1Lf + 2.2H + 1Hf
Load combination no.3	1.2D + 1Df + 1L + 1Lf + 1.25E + 1Ef + 1.5H + 1Hf
Load combination no.4	1.2D + 1Df + 1L + 1Lf + 1.25E + 1Ef + 1.5H + 1Hf

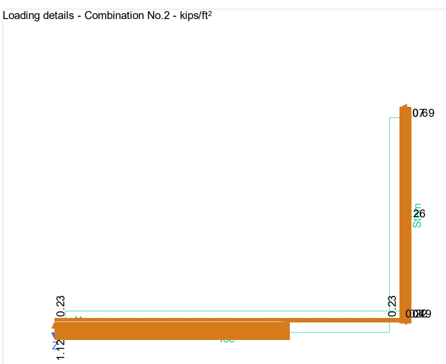
Static Case

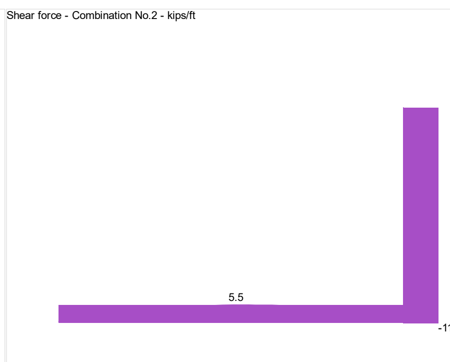
Seismic Case

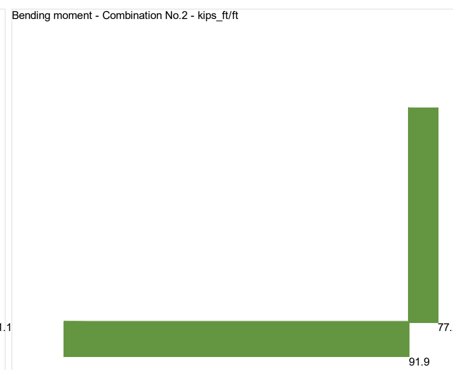


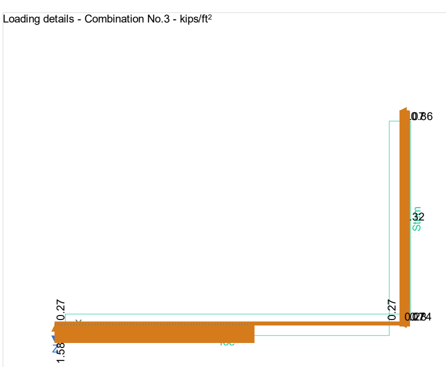


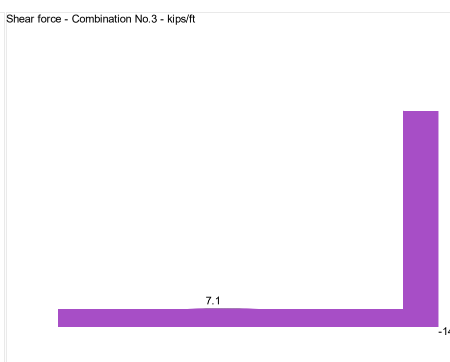


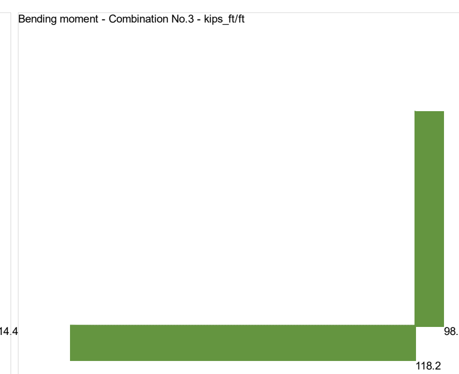


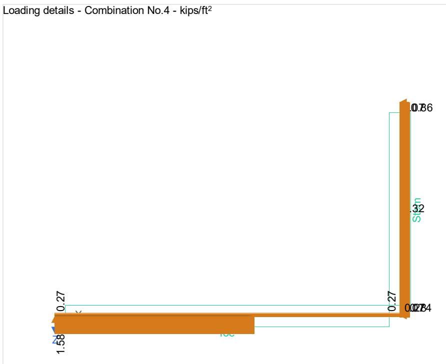


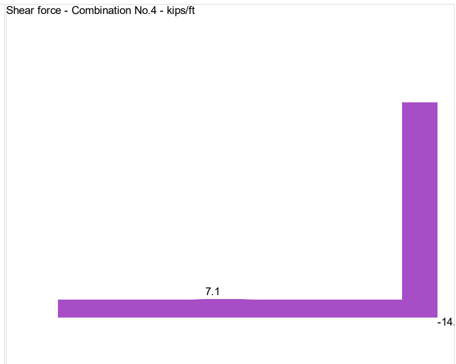


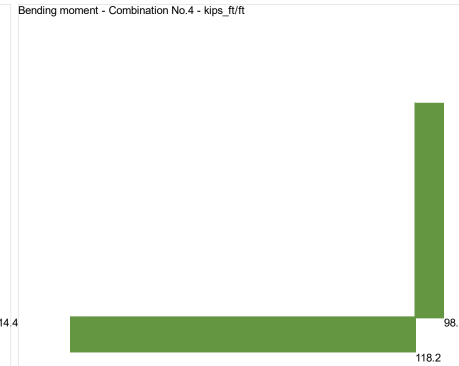












Check stem design at base of stem

Depth of section h = **18** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 M = **77178** lb_ft/ft

Depth of tension reinforcement $d = h - c_{sr} - \phi_{sr} / 2$ = **14.5** in

Compression reinforcement provided No.6 bars @ 12" c/c

Area of compression reinforcement provided $A_{sf.prov} = \pi \times \phi_{sf}^2 / (4 \times s_{sf})$ = **0.442** in²/ft

Tension reinforcement provided No.8 bars @ 6" c/c

Area of tension reinforcement provided $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **1.571** in²/ft

Maximum reinforcement spacing - cl.7.7.2.3 s_{max} = min(18 in, $3 \times h$) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{sr.prov} \times f_y / (0.85 \times f'_c)$ = **2.053** in

Neutral axis factor - cl.22.2.2.4.3 β_1 = min(max(0.85 - 0.05 $\times$ (f'_c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.825**

Depth to neutral axis $c = a / \beta_1$ = **2.489** in

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c$ = **0.014478**

Section is in the tension controlled zone

Strength reduction factor ϕ_f = min(max(0.65 + 0.25 $\times$($\varepsilon_t - \varepsilon_{ty}$) / 0.003, 0.65), 0.9) = **0.9**

Nominal flexural strength $M_n = A_{sr.prov} \times f_y \times (d - a / 2)$ = **105819** lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n$ = **95237** lb_ft/ft

$M / \phi M_n$ = **0.810**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{sr.des}$ = **1.254** in²/ft

Minimum area of reinforcement - cl.7.6.1.1 $A_{sr.min} = 0.0018 \times h$ = **0.389** in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

USACE EM 1110-2-2104 (Nov. 2016), Section 2.9 requires 0.003 instead of 0.0018.
As_min = 0.003*h*(12 in/ft) = 0.648 in^2/ft (both faces)
As_min < As_prov = (1.571+0.442) in^2/ft (both faces)

Rectangular section in shear - Section 22.5

Design shear force V = **11083** lb/ft

Concrete modification factor - cl.19.2.4 λ = **1**

Depth of tension reinforcement d = **14.50** in

Size effect modification factor - cl. 22.5.5.1.3 λ_s = min($\sqrt{2 / (1 + (d / 1 \text{ in}) / 10)}$, 1.0) = **0.904**

Reinforcement ratio $\rho = A_{sr.prov} / d$ = **0.009**

Nominal concrete shear strength - eqn.22.5.5.1 V_c = min($8 \times \lambda_s \times \lambda \times \rho^{1/3}$, $5 \times \lambda$) $\times \sqrt{f'_c \times 1 \text{ psi}} \times d$= **17567** lb/ft

Strength reduction factor ϕ_s = **0.75**

Design concrete shear strength - cl.7.6.3.1 $\phi V_c = \phi_s \times V_c =$ **13175** lb/ft

$V / \phi V_c =$ **0.841**

PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.7.6.1.1 $A_{sx.req} = 0.0018 \times t_{stem} =$ **0.389** in²/ft

Transverse reinforcement provided No.6 bars @ 12" c/c each face

Area of transverse reinforcement provided $A_{sx.prov} = 2 \times \pi \times \phi_{sx}^2 / (4 \times s_{sx}) =$ **0.884** in²/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

As_min = 0.003*h*(12 in/ft) = 0.648 in^2/ft (both faces)
Note that t_stem = h = 18 in.
As_min < As_prov = 0.884 in^2/ft (both faces)

Check base design at toe

Depth of section h = **18** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 M = **91912** lb_ft/ft

Depth of tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2 =$ **14.5** in

Compression reinforcement provided No.6 bars @ 12" c/c

Area of compression reinforcement provided $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) =$ **0.442** in²/ft

Tension reinforcement provided No.8 bars @ 6" c/c

Area of tension reinforcement provided $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) =$ **1.571** in²/ft

Maximum reinforcement spacing - cl.7.7.2.3 $s_{max} = \min(18 \text{ in}, 3 \times h) =$ **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{bb.prov} \times f_y / (0.85 \times f'_c) =$ **2.053** in

Neutral axis factor - cl.22.2.2.4.3 $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) =$ **0.825**

Depth to neutral axis $c = a / \beta_1 =$ **2.489** in

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c =$ **0.014478**

Section is in the tension controlled zone

Strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / 0.003, 0.65), 0.9) =$ **0.9**

Nominal flexural strength $M_n = A_{bb.prov} \times f_y \times (d - a / 2) =$ **105819** lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n =$ **95237** lb_ft/ft

$M / \phi M_n =$ **0.965**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{bb.des} =$ **1.512** in²/ft

Minimum area of reinforcement - cl.7.6.1.1 $A_{bb.min} = 0.0018 \times h =$ **0.389** in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Ab_min = 0.003*h*(12 in/ft) = 0.648 in^2/ft (both faces)
Ab_min < Ab_prov = (1.571+0.442) in^2/ft (both faces)

Rectangular section in shear - Section 22.5

Design shear force V = **5460** lb/ft

Concrete modification factor - cl.19.2.4 $\lambda =$ **1**

Depth of tension reinforcement d = **14.50** in

Size effect modification factor - cl. 22.5.5.1.3 $\lambda_s = \min(\sqrt{2 / (1 + (d / 1 \text{ in}) / 10)}, 1.0) =$ **0.904**

Reinforcement ratio $\rho = A_{bb.prov} / d =$ **0.009**

Nominal concrete shear strength - eqn.22.5.5.1 $V_c = \min(8 \times \lambda \times \rho^{1/3}, 5 \times \lambda) \times \sqrt{f'_c \times 1 \text{ psi}} \times d =$ **19443** lb/ft

Strength reduction factor $\phi_s =$ **0.75**

Design concrete shear strength - cl.7.6.3.1 $\phi V_c = \phi_s \times V_c =$ **14582** lb/ft

$V / \phi V_c =$ **0.374**

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

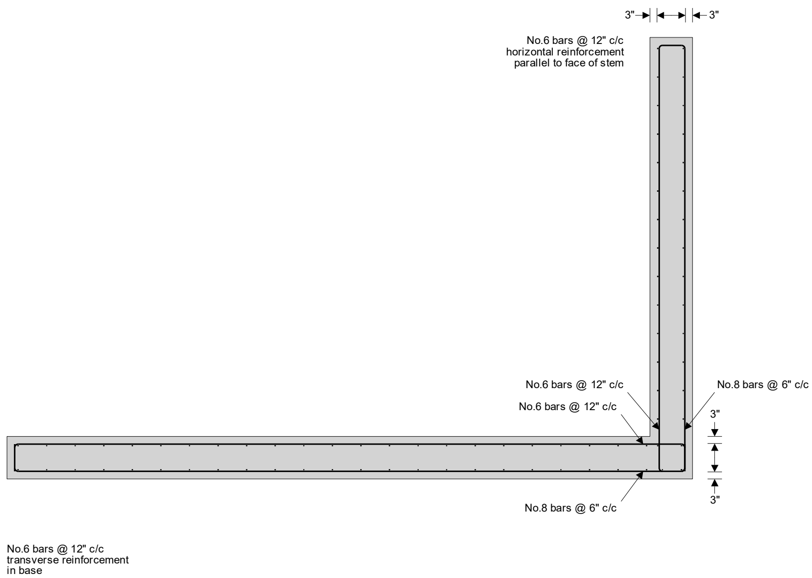
Minimum area of reinforcement - cl.7.6.1.1 $A_{bx.req} = 0.0018 \times t_{base} = \mathbf{0.389}$ in^2/ft

Transverse reinforcement provided No.6 bars @ 12" c/c each face

Area of transverse reinforcement provided $A_{bx.prov} = 2 \times \pi \times \phi_{bx}{}^2 / (4 \times s_{bx}) = \mathbf{0.884}$ in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

Ab_min = 0.003*h*(12 in/ft) = 0.648 in^2/ft (both faces)
Note that t_base = h = 18 in.
Ab_min < Ab_prov = 0.884 in^2/ft (both faces)



Reinforcement details

HEADWALL DESIGN (CULVERT SPANS)

Calculation is based on user defined combination values

Overall design summary provided by Tedds software does not account for overall structural system. Design explanations provided in results.

Tedds calculation version 2.9.11

Analysis summary

~~**Design summary**~~

~~Overall design utilisation~~ ~~0.946~~

~~Overall design status~~ ~~Pass~~

Description	Unit	Capacity	Applied	F o S	Result
~~Sliding stability~~	~~plf~~	~~3162~~	~~2991~~	~~1.057~~	~~FAIL~~
~~Overturning stability~~	~~lb_ft/ft~~	~~18930~~	~~10598~~	~~1.786~~	~~PASS~~
~~Bearing pressure~~	~~psf~~	~~4000~~	~~2615~~	~~1.530~~	~~PASS~~

By inspection, headwall is externally stable due to anchoring/ connection with the roof/top slab of culvert.

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Stem p0 rear face - Flexural reinforcement	in^2/ft	0.442	0.389	0.880	PASS
Stem p0 - Shear resistance	lb/ft	8660	3083	0.356	PASS
Base top face - Flexural reinforcement	in^2/ft	0.442	0.356	0.807	PASS
Base - Shear resistance	lb/ft	8940	3116	0.349	PASS
Transverse stem reinforcement	in^2/ft	0.884	0.389	0.440	PASS
Transverse base reinforcement	in^2/ft	0.884	0.356	0.403	PASS

Retaining wall details

Stem type: Cantilever

Stem height: h_{stem} = **5.625** ft

Stem thickness: t_{stem} = **18** in

Angle to rear face of stem: α = **90** deg

Stem density: γ_{stem} = **150** pcf

Heel length: l_{heel} = **5** ft

Base thickness: t_{base} = **16.5** in

Used only for design purposes with Tedds software. There is no heel or base due to connection with culvert roof.

Base density: γ_{base} = **150** pcf

Height of retained soil: h_{ret} = **5.625** ft

Angle of soil surface: β = **0** deg

Conservative retained soil parameters for design purposes.

Depth of cover: d_{cover} = **0** ft

Retained soil properties

Based on Sept. 2020 Geotechnical Memorandum by Gannett Fleming (Geotech Memo).

Soil type: Dense fine or silty sand

Moist density: γ_{mr} = **120** pcf

Saturated density: γ_{sr} = **120** pcf

~~Base soil properties~~

~~Soil type~~ ~~Dense fine or silty sand~~

~~Soil density~~ ~~γ_b = **120** pcf~~

~~Gross allowable bearing pressure~~ ~~q_{allow_gross} = **4000** psf~~

Not applicable.

Seismic details

Horizontal seismic acceleration factor: K_h = **0.472**

Vertical seismic acceleration factor: K_v = **0**

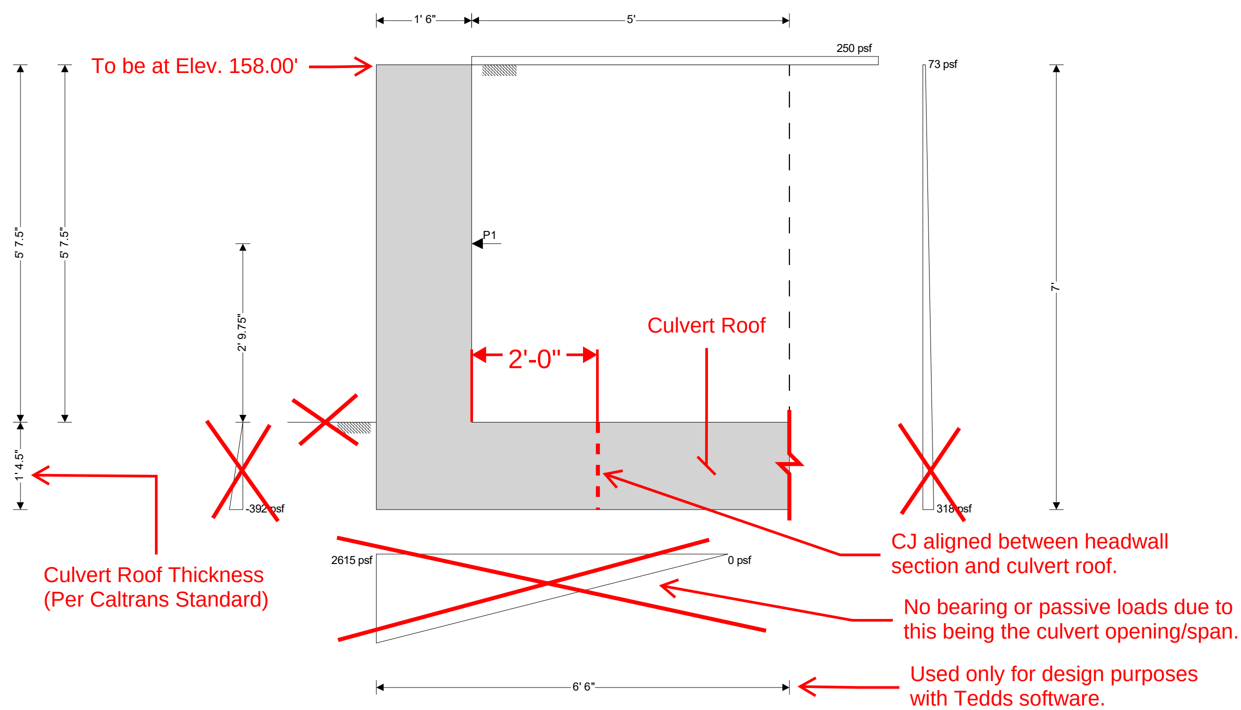
Kh = 0.5*(PGAm) = 0.5*(0.943), from Geotech Memo; Kv = 0 (conservative assumption)

Seismic acceleration angle: $\theta = \mathrm{atan}(K_h / (1 - K_v))$ = **25.267** deg

Loading details

Live surcharge load — Surcharge_L = **250** psf

Horizontal line load at 2.813 ft — P_{D1} = **263** plf ← Type 60K Concrete Barrier Load = 900 plf x Ka



General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Used only for design purposes with Tedds software. Usage of the heel length and overall base length was to avoid errors within Tedds.

Base length — $l_{base} = t_{stem} + l_{heel}$ = **6.5** ft

Moist soil height — $h_{moist} = h_{soil}$ = **5.625** ft

- Distance to horizontal seismic component — $x_{seismic\_h} = 0.6 \times (h_{soil} + h_{base})$ = **4.2** ft

Length of surcharge load — $l_{sur} = l_{heel}$ = **5** ft

- Distance to vertical component — $x_{sur\_v} = l_{base} - l_{heel} / 2$ = **4** ft

Effective height of wall — $h_{eff} = h_{base} + d_{cover} + h_{ret}$ = **7** ft

- Distance to horizontal component — $x_{sur\_h} = h_{eff} / 2$ = **3.5** ft

Area of wall stem — $A_{stem} = h_{stem} \times t_{stem}$ = **8.438** ft^2

- Distance to vertical component — $x_{stem} = l_{toe} + t_{stem} / 2$ = **0.75** ft

Area of wall base — $A_{base} = l_{base} \times t_{base}$ = **8.938** ft^2

- Distance to vertical component — $x_{base} = l_{base} / 2$ = **3.25** ft

Area of moist soil — $A_{moist} = h_{moist} \times l_{heel}$ = **28.125** ft^2

- Distance to vertical component — $x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist}$ = **4** ft
- Distance to horizontal component — $x_{moist\_h} = h_{eff} / 3$ = **2.333** ft

Soil coefficients

Coefficient of friction to back of wall — K_{fr} = **0.400** ← Based on Geotech Memo.

~~Coefficient of friction to front of wall~~ ~~K_{ffb} = **0.400**~~

~~Coefficient of friction beneath base~~ ~~K_{fbb} = **0.400**~~

Active pressure coefficient K_A = **0.292**

Ka = 35 pcf / 120 pcf, from Geotech Memo.

~~Passive pressure coefficient~~ ~~K_P = **2.375**~~

Not applicable.

Using Mononobe-Okabe theory

Active dynamic pressure coefficient K_{AE} = **0.754**

~~Passive dynamic pressure coefficient~~ ~~K_{PE} = **2.374**~~

User defined combination

Load combination 1 — 1 × Dead + 1 × Live + 1 × Lateral earth

Sliding check

Vertical forces on wall

Wall stem — $F_{stem} = A_{stem} \times \gamma_{stem}$ = **1266** plf

Wall base — $F_{base} = A_{base} \times \gamma_{base}$ = **1341** plf

Surcharge load — $F_{sur\_v} = Surcharge_L \times l_{heel}$ = **1250** plf

Moist retained soil — $F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **3375** plf

Total — $F_{total\_v} = F_{stem} + F_{base} + F_{sur\_v} + F_{moist\_v}$ = **7231** plf

Horizontal forces on wall

Surcharge load — $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff}$ = **511** plf

Line loads — $F_{P\_h} = P_{D1}$ = **263** plf

Moist retained soil — $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2$ = **858** plf

Total — $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h}$ = **1632** plf

By inspection, headwall is externally stable due to anchoring/ connection with the roof/top slab of culvert.

Check stability against sliding

Base soil resistance — $F_{exc\_h} = K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2$ = **269** plf

Base friction — $F_{friction} = F_{total\_v} \times K_{fbb}$ = **2893** plf

Resistance to sliding — $F_{rest} = F_{exc\_h} + F_{friction}$ = **3162** plf

Factor of safety — $FoS_{sl} = F_{rest} / F_{total\_h}$ = **1.937** > 1.5

PASS - Factor of safety against sliding is adequate

Overturning check

Vertical forces on wall

Wall stem — $F_{stem} = A_{stem} \times \gamma_{stem}$ = **1266** plf

Wall base — $F_{base} = A_{base} \times \gamma_{base}$ = **1341** plf

Surcharge load — $F_{sur\_v} = Surcharge_L \times l_{heel}$ = **1250** plf

Moist retained soil — $F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **3375** plf

Total — $F_{total\_v} = F_{stem} + F_{base} + F_{sur\_v} + F_{moist\_v}$ = **7231** plf

Horizontal forces on wall

Surcharge load — $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff}$ = **511** plf

Line loads — $F_{P\_h} = P_{D1}$ = **263** plf

Moist retained soil — $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2$ = **858** plf

Base soil — $F_{exc\_h} = -K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2$ = **-269** plf

Total — $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{exc\_h}$ = **1363** plf

Overturning moments on wall

Surcharge load $M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h} =$ **1788** lb_ft/ft

Line loads $M_{P\_OT} = abs(P_{D1}) \times (p_1 + t_{base}) =$ **1101** lb_ft/ft

Moist retained soil $M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h} =$ **2003** lb_ft/ft

Total $M_{total\_OT} = M_{sur\_OT} + M_{P\_OT} + M_{moist\_OT} =$ **4893** lb_ft/ft

Restoring moments on wall

Wall stem $M_{stem\_R} = F_{stem} \times x_{stem} =$ **949** lb_ft/ft

Wall base $M_{base\_R} = F_{base} \times x_{base} =$ **4357** lb_ft/ft

Moist retained soil $M_{moist\_R} = F_{moist\_v} \times x_{moist\_v} =$ **13500** lb_ft/ft

Base soil $M_{exc\_R} = -F_{exc\_h} \times x_{exc\_h} =$ **123** lb_ft/ft

Total $M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{moist\_R} + M_{exc\_R} =$ **18930** lb_ft/ft

Check stability against overturning

Factor of safety $FoS_{ot} = M_{total\_R} / M_{total\_OT} =$ **3.869** > 1.5

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} =$ **1266** plf

Wall base $F_{base} = A_{base} \times \gamma_{base} =$ **1341** plf

Surcharge load $F_{sur\_v} = Surcharge_L \times l_{heel} =$ **1250** plf

Moist retained soil $F_{moist\_v} = A_{moist} \times \gamma_{mr} =$ **3375** plf

Total $F_{total\_v} = F_{stem} + F_{base} + F_{sur\_v} + F_{moist\_v} =$ **7231** plf

By inspection, headwall is externally stable due to anchoring/ connection with the roof/top slab of culvert.

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} =$ **511** plf

Line loads $F_{P\_h} = P_{D1} =$ **263** plf

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2 =$ **858** plf

Base soil $F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 =$ **-269** plf

Total $F_{total\_h} = max(F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{pass\_h} - F_{total\_v} \times K_{fbb}, 0 \text{ plf}) =$ **0** plf

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} =$ **949** lb_ft/ft

Wall base $M_{base} = F_{base} \times x_{base} =$ **4357** lb_ft/ft

Surcharge load $M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} =$ **3211** lb_ft/ft

Line loads $M_P = -(P_{D1} \times (p_1 + t_{base})) =$ **-1101** lb_ft/ft

Moist retained soil $M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} =$ **11497** lb_ft/ft

Base soil $M_{pass} = -F_{pass\_h} \times x_{pass\_h} =$ **123** lb_ft/ft

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{moist} + M_{pass} =$ **19037** lb_ft/ft

Check bearing pressure

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} =$ **2.633** ft

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 =$ **-0.617** ft

Loaded length of base $l_{load} = l_{base} =$ **6.5** ft

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) =$ **1747** psf

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) =$ **478** psf

Allowable bearing capacity $q_{allow} = q_{allow\_gross} = \mathbf{4000}$ psf

Factor of safety $FoS_{bp} = q_{allow} / \max(q_{toe}, q_{heel}) = \mathbf{2.29}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

User defined combination

Load combination 2 $1 \times$ Dead + $1 \times$ Live + $1 \times$ Earthquake + $1 \times$ Lateral earth

Sliding check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{1266}$ plf

Wall base $F_{base} = A_{base} \times \gamma_{base} = \mathbf{1341}$ plf

Surcharge load $F_{sur\_v} = Surcharge_L \times l_{heel} = \mathbf{1250}$ plf

Moist retained soil $F_{moist\_v} = A_{moist} \times \gamma_{mr} = \mathbf{3375}$ plf

Total $F_{total\_v} = F_{stem} + F_{base} + F_{sur\_v} + F_{moist\_v} = \mathbf{7231}$ plf

By inspection, headwall is externally stable due to anchoring/ connection with the roof/top slab of culvert.

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} = \mathbf{511}$ plf

Line loads $F_{P\_h} = P_{D1} = \mathbf{263}$ plf

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2 = \mathbf{858}$ plf

Seismic $F_{seismic\_h} = (K_{AE} - K_A) \times \gamma_{mr} \times (h_{soil} + h_{base})^2 / 2 = \mathbf{1358}$ plf

Total $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{seismic\_h} = \mathbf{2991}$ plf

Check stability against sliding

Base soil resistance $F_{exc\_h} = K_{PE} \times \gamma_b \times (h_{pass} + h_{base})^2 / 2 = \mathbf{269}$ plf

Base friction $F_{friction} = F_{total\_v} \times K_{fbb} = \mathbf{2893}$ plf

Resistance to sliding $F_{rest} = F_{exc\_h} + F_{friction} = \mathbf{3162}$ plf

Factor of safety $FoS_{sl} = F_{rest} / F_{total\_h} = \mathbf{1.057} < 1.1$

FAIL - Factor of safety against sliding is inadequate

Overturning check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{1266}$ plf

Wall base $F_{base} = A_{base} \times \gamma_{base} = \mathbf{1341}$ plf

Surcharge load $F_{sur\_v} = Surcharge_L \times l_{heel} = \mathbf{1250}$ plf

Moist retained soil $F_{moist\_v} = A_{moist} \times \gamma_{mr} = \mathbf{3375}$ plf

Total $F_{total\_v} = F_{stem} + F_{base} + F_{sur\_v} + F_{moist\_v} = \mathbf{7231}$ plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} = \mathbf{511}$ plf

Line loads $F_{P\_h} = P_{D1} = \mathbf{263}$ plf

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2 = \mathbf{858}$ plf

Base soil $F_{exc\_h} = -K_{PE} \times \gamma_b \times (h_{pass} + h_{base})^2 / 2 = \mathbf{-269}$ plf

Seismic $F_{seismic\_h} = (K_{AE} - K_A) \times \gamma_{mr} \times (h_{soil} + h_{base})^2 / 2 = \mathbf{1358}$ plf

Total $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{exc\_h} + F_{seismic\_h} = \mathbf{2721}$ plf

Overturning moments on wall

Surcharge load $M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h} = \mathbf{1788}$ lb_ft/ft

Line loads $M_{P\_OT} = abs(P_{D1}) \times (p_1 + t_{base}) = \mathbf{1101}$ lb_ft/ft

Moist retained soil $M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h} = \mathbf{2003}$ lb_ft/ft

Seismic $M_{seismic\_OT} = F_{seismic\_h} \times x_{seismic\_h} = \mathbf{5705}$ lb_ft/ft

Total $M_{total\_OT} = M_{sur\_OT} + M_{P\_OT} + M_{moist\_OT} + M_{seismic\_OT} = \mathbf{10598}$ lb_ft/ft

Restoring moments on wall

Wall stem $M_{stem\_R} = F_{stem} \times x_{stem} = \mathbf{949}$ lb_ft/ft

Wall base $M_{base\_R} = F_{base} \times x_{base} = \mathbf{4357}$ lb_ft/ft

Moist retained soil $M_{moist\_R} = F_{moist\_v} \times x_{moist\_v} = \mathbf{13500}$ lb_ft/ft

Base soil $M_{exc\_R} = -F_{exc\_h} \times x_{exc\_h} = \mathbf{123}$ lb_ft/ft

Total $M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{moist\_R} + M_{exc\_R} = \mathbf{18930}$ lb_ft/ft

Check stability against overturning

Factor of safety $FoS_{ot} = M_{total\_R} / M_{total\_OT} = \mathbf{1.786} > 1.1$

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{1266}$ plf

Wall base $F_{base} = A_{base} \times \gamma_{base} = \mathbf{1341}$ plf

Surcharge load $F_{sur\_v} = Surcharge_L \times l_{heel} = \mathbf{1250}$ plf

Moist retained soil $F_{moist\_v} = A_{moist} \times \gamma_{mr} = \mathbf{3375}$ plf

Total $F_{total\_v} = F_{stem} + F_{base} + F_{sur\_v} + F_{moist\_v} = \mathbf{7231}$ plf

By inspection, headwall is externally stable due to anchoring/ connection with the roof/top slab of culvert.

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_L \times h_{eff} = \mathbf{511}$ plf

Line loads $F_{P\_h} = P_{D1} = \mathbf{263}$ plf

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times h_{eff}^2 / 2 = \mathbf{858}$ plf

Base soil $F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-269}$ plf

Seismic $F_{seismic\_h} = (K_{AE} - K_A) \times \gamma_{mr} \times (h_{soil} + h_{base})^2 / 2 = \mathbf{1358}$ plf

Total $F_{total\_h} = \max(F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{pass\_h} + F_{seismic\_h} - F_{total\_v} \times K_{fbb}, 0 \text{ plf}) = \mathbf{0}$ plf

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} = \mathbf{949}$ lb_ft/ft

Wall base $M_{base} = F_{base} \times x_{base} = \mathbf{4357}$ lb_ft/ft

Surcharge load $M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} = \mathbf{3211}$ lb_ft/ft

Line loads $M_P = -(P_{D1} \times (p_1 + t_{base})) = \mathbf{-1101}$ lb_ft/ft

Moist retained soil $M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} = \mathbf{11497}$ lb_ft/ft

Base soil $M_{pass} = -F_{pass\_h} \times x_{pass\_h} = \mathbf{123}$ lb_ft/ft

Seismic $M_{seismic} = -F_{seismic\_h} \times x_{seismic\_h} = \mathbf{-5705}$ lb_ft/ft

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{moist} + M_{pass} + M_{seismic} = \mathbf{13332}$ lb_ft/ft

Check bearing pressure

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} = \mathbf{1.844}$ ft

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 = \mathbf{-1.406}$ ft

Loaded length of base $l_{load} = 3 \times \bar{x} = \mathbf{5.531}$ ft

Bearing pressure at toe $q_{toe} = 2 \times F_{total\_v} / l_{load} = \mathbf{2615}$ psf

~~Bearing pressure at heel~~ ~~q_{heel} = **0** psf~~

~~Allowable bearing capacity~~ ~~q_{allow} = q_{allow_gross} = **4000** psf~~

~~Factor of safety~~ ~~FoS_{bp} = q_{allow} / max(q_{toe}, q_{heel}) = **1.53**~~

~~***PASS - Allowable bearing pressure exceeds maximum applied bearing pressure***~~

By inspection, headwall is externally stable due to anchoring/connection with the roof/top slab of culvert.

HEADWALL DESIGN (CULVERT SPANS)

Retaining wall design in accordance with ACI 318-19

In conformance with USACE EM 1110-2-2104 (Nov. 2016), Sections 2 and 3. Additional information provided, where applicable.

Tedds calculation version 2.9.11

Concrete details

Compressive strength of concrete f'_c = **4500** psi

Concrete type Normal weight

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Modulus of elasticity or reinforcement E_s = **29000000** psi

Compression-controlled strain limit ε_{ty} = **0.002**

Cover to reinforcement

In conformance with USACE EM 1110-2-2104 (Nov. 2016), Table 2-1, for "formed and screeded surfaces such as stilling basin walls, chute spillway slabs, and channel lining slabs on grade: greater than 12 in. and less than 24 in. thick".

Front face of stem c_{sf} = **3** in

Rear face of stem c_{sr} = **3** in

Top face of base c_{bt} = **3** in

Bottom face of base c_{bb} = **3** in

User defined load combinations

In conformance with USACE EM 1110-2-2104 (Nov. 2016), Table 3-1. Loads with "f" are considered "favorable" conditions by Tedds, but calculations below only use "unfavorable" (non-"f") loads.

Load combination no.1 2.2D + 1Df + 2.2L + 1Lf + 2.2H + 1Hf

Load combination no.2 2.2D + 1Df + 2.2L + 1Lf + 2.2H + 1Hf

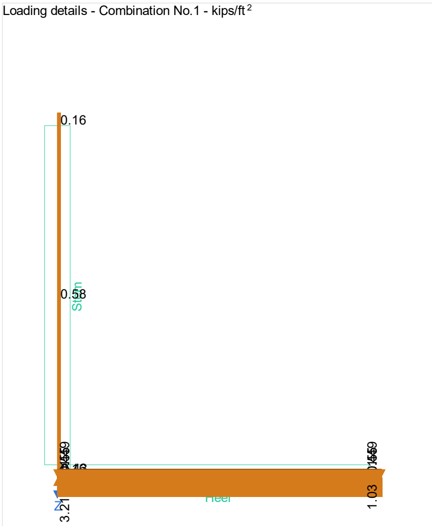
Static Case

Load combination no.3 1.2D + 1Df + 1L + 1Lf + 1.25E + 1Ef + 1.5H + 1Hf

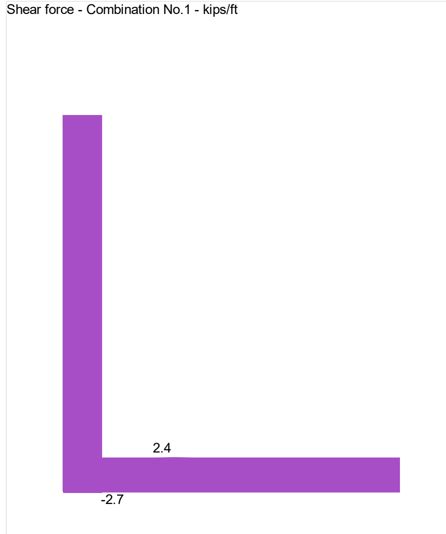
Load combination no.4 1.2D + 1Df + 1L + 1Lf + 1.25E + 1Ef + 1.5H + 1Hf

Seismic Case

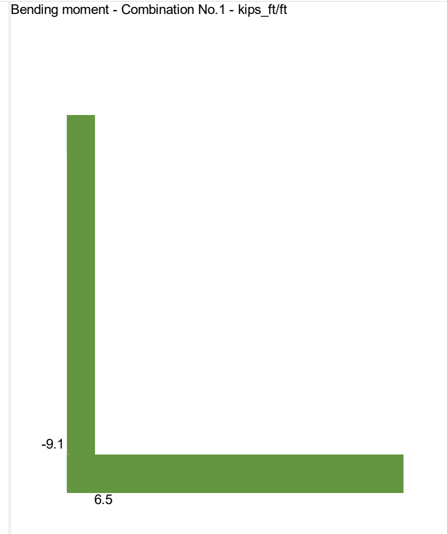
Loading details - Combination No.1 - kips/ft[2]



Shear force - Combination No.1 - kips/ft

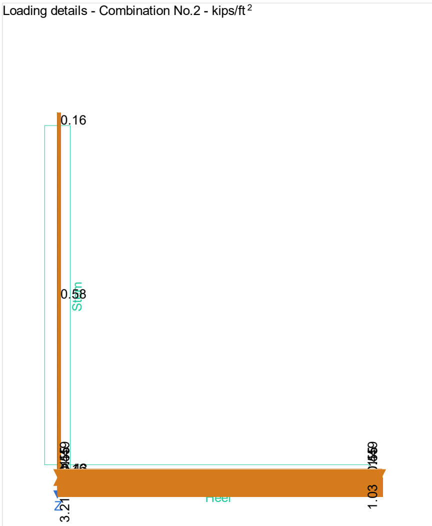


Bending moment - Combination No.1 - kips_ft/ft

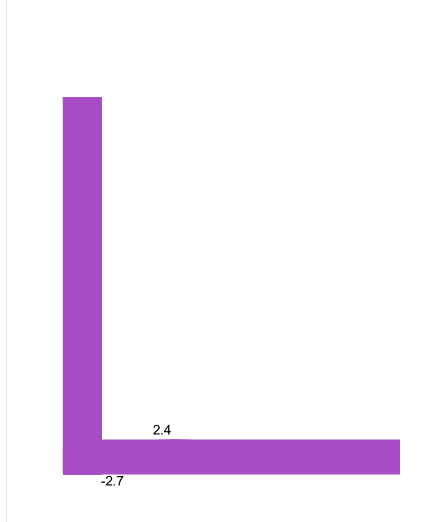


Gannett Fleming 2251 Douglas Boulevard Suite 200 Roseville, CA 95661	Project NHC/United Water - Freeman Diversion - 3BC				Job Ref. 067376	
	Section Headwall Design (Culvert Spans)				Sheet No./Rev. 8 / Rev.4	
	Calc. by RC/SMU	Date 7/11/2023	Chk'd by J. Allen	Date 7/18/2023	App'd by T. Sell	Date 8/11/2023

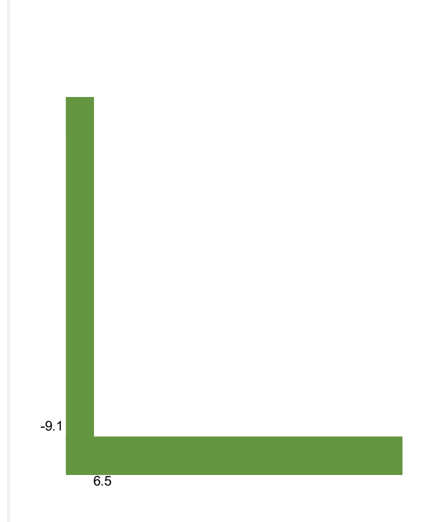
Loading details - Combination No.2 - kips/ft^2



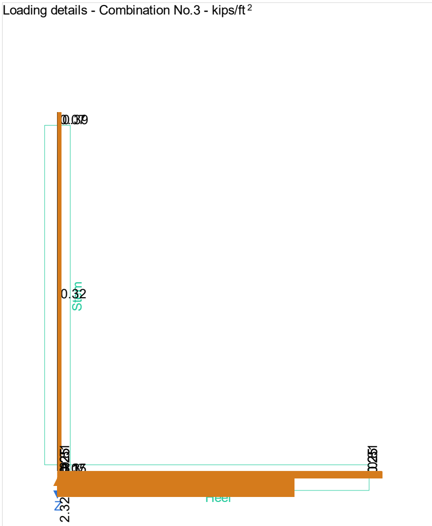
Shear force - Combination No.2 - kips/ft



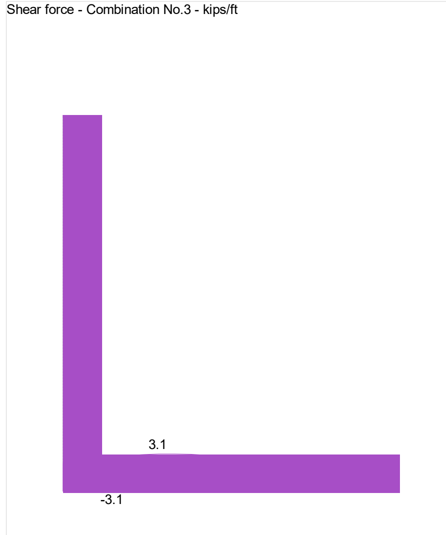
Bending moment - Combination No.2 - kips_ft/ft



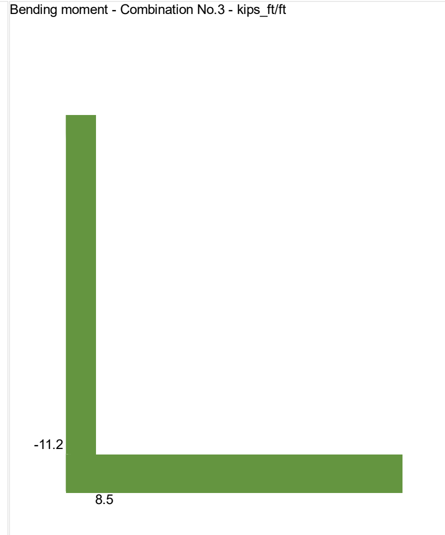
Loading details - Combination No.3 - kips/ft^2



Shear force - Combination No.3 - kips/ft

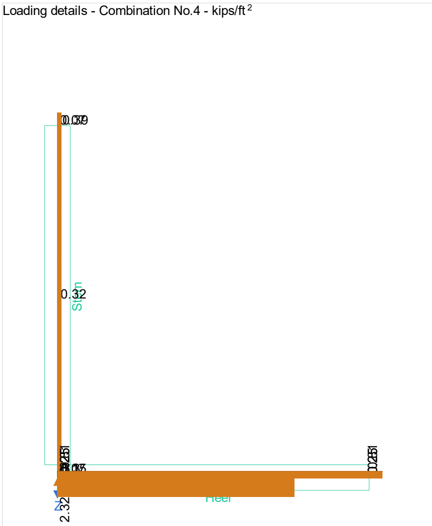


Bending moment - Combination No.3 - kips_ft/ft

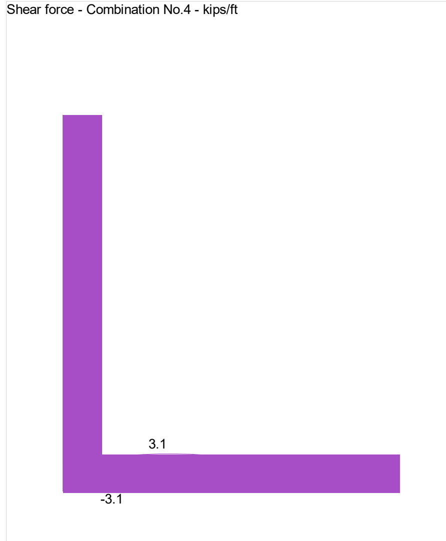


Tekla Tedds Gannett Fleming 2251 Douglas Boulevard Suite 200 Roseville, CA 95661	Project NHC/United Water - Freeman Diversion - 3BC			Job Ref. 067376	
	Section Headwall Design (Culvert Spans)			Sheet No./Rev. 9 / Rev.4	
	Calc. by RC/SMU	Date 7/11/2023	Chk'd by J. Allen	Date 7/18/2023	App'd by T. Sell	Date 8/11/2023

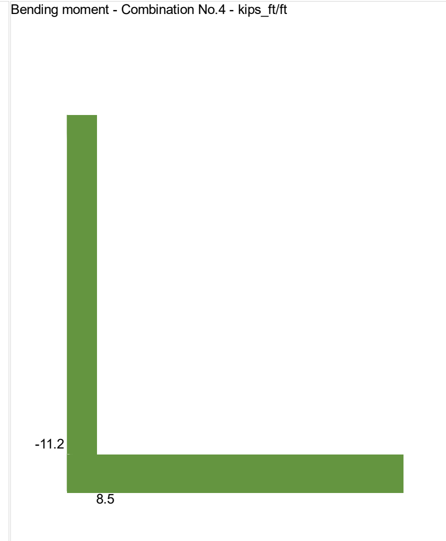
Loading details - Combination No.4 - kips/ft²



Shear force - Combination No.4 - kips/ft



Bending moment - Combination No.4 - kips_ft/ft



Check stem design at base of stem

Depth of section	$h = \mathbf{18}$ in

Rectangular section in flexure - Section 22.3

Design bending moment combination 4	$M = \mathbf{8508}$ lb_ft/ft
Depth of tension reinforcement	$d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{14.625}$ in
Compression reinforcement provided	No.6 bars @ 12" c/c
Area of compression reinforcement provided	$A_{sf.prov} = \pi \times \phi_{sf}^2 / (4 \times s_{sf}) = \mathbf{0.442}$ in²/ft
Tension reinforcement provided	No.6 bars @ 12" c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{0.442}$ in²/ft
Maximum reinforcement spacing - cl.7.7.2.3	$s_{max} = \min(18 \text{ in}, 3 \times h) = \mathbf{18}$ in

PASS - Reinforcement is adequately spaced

Depth of compression block	$a = A_{sr.prov} \times f_y / (0.85 \times f'_c) = \mathbf{0.577}$ in
Neutral axis factor - cl.22.2.2.4.3	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) = \mathbf{0.825}$
Depth to neutral axis	$c = a / \beta_1 = \mathbf{0.7}$ in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c = \mathbf{0.059679}$

Section is in the tension controlled zone

Strength reduction factor	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / 0.003, 0.65), 0.9) = \mathbf{0.9}$
Nominal flexural strength	$M_n = A_{sr.prov} \times f_y \times (d - a / 2) = \mathbf{31668}$ lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n = \mathbf{28501}$ lb_ft/ft
	$M / \phi M_n = \mathbf{0.299}$

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis	$A_{sr.des} = \mathbf{0.13}$ in²/ft
Minimum area of reinforcement - cl.7.6.1.1	$A_{sr.min} = 0.0018 \times h = \mathbf{0.389}$ in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

USACE EM 1110-2-2104 (Nov. 2016), Section 2.9
requires 0.003 instead of 0.0018.
As_min = 0.003*h*(12 in/ft) = 0.648 in^2/ft (both faces)
As_min < As_prov = (0.442+0.442) in^2/ft (both faces)

Design shear force	$V = \mathbf{3083}$ lb/ft
Concrete modification factor - cl.19.2.4	$\lambda = \mathbf{1}$

Depth of tension reinforcement — d = **14.63** in

Size effect modification factor - cl. 22.5.5.1.3 — $\lambda_s = \min(\sqrt{2 / (1 + (d / 1 \text{ in}) / 10)}, 1.0)$ = **0.901**

Reinforcement ratio — $\rho = A_{sr.prov} / d$ = **0.003**

Nominal concrete shear strength - eqn.22.5.5.1 — $V_c = \min(8 \times \lambda_s \times \lambda \times \rho^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1 \text{ psi})} \times d$ = **11546** lb/ft

Strength reduction factor — ϕ_s = **0.75**

Design concrete shear strength - cl.7.6.3.1 — $\phi V_c = \phi_s \times V_c$ = **8660** lb/ft

$V / \phi V_c$ = **0.356**

PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.7.6.1.1 — $A_{sx.req} = 0.0018 \times t_{stem}$ = **0.389** in^2/ft

Transverse reinforcement provided — No.6 bars @ 12" c/c each face

Area of transverse reinforcement provided — $A_{sx.prov} = 2 \times \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **0.884** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

As_min = 0.003*h*(12 in/ft) = 0.648 in^2/ft (both faces)
Note that t_stem = h = 18 in.
As_min < As_prov = 0.884 in^2/ft (both faces)

Check base design at heel

Depth of section — h = **16.5** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 4 — M = **11211** lb_ft/ft

Depth of tension reinforcement — $d = h - c_{bt} - \phi_{bt} / 2$ = **13.125** in

Compression reinforcement provided — No.6 bars @ 12" c/c

Area of compression reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **0.442** in^2/ft

Tension reinforcement provided — No.6 bars @ 12" c/c

Area of tension reinforcement provided — $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **0.442** in^2/ft

Maximum reinforcement spacing - cl.7.7.2.3 — $s_{max} = \min(18 \text{ in}, 3 \times h)$ = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block — $a = A_{bt.prov} \times f_y / (0.85 \times f'_c)$ = **0.577** in

Neutral axis factor - cl.22.2.2.4.3 — $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85)$ = **0.825**

Depth to neutral axis — $c = a / \beta_1$ = **0.7** in

Strain in reinforcement — $\varepsilon_t = 0.003 \times (d - c) / c$ = **0.05325**

Section is in the tension controlled zone

Strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / 0.003, 0.65), 0.9)$ = **0.9**

Nominal flexural strength — $M_n = A_{bt.prov} \times f_y \times (d - a / 2)$ = **28354** lb_ft/ft

Design flexural strength — $\phi M_n = \phi_f \times M_n$ = **25519** lb_ft/ft

$M / \phi M_n$ = **0.439**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis — $A_{bt.des}$ = **0.192** in^2/ft

Minimum area of reinforcement - cl.7.6.1.1 — $A_{bt.min} = 0.0018 \times h$ = **0.356** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Ab_min = 0.003*h*(12 in/ft) = 0.594 in^2/ft (both faces)
Ab_min < Ab_prov = (0.442+0.442) in^2/ft (both faces)

Rectangular section in shear - Section 22.5

Design shear force — V = **3116** lb/ft

Concrete modification factor - cl.19.2.4 — λ = **1**

Depth of tension reinforcement — d = **13.13** in

Size effect modification factor - cl. 22.5.5.1.3 — $\lambda_s = \min(\sqrt{2 / (1 + (d / 1 \text{ in}) / 10)}, 1.0)$ = **0.93**

Reinforcement ratio $\rho = A_{bt.prov} / d = \mathbf{0.003}$

Nominal concrete shear strength - eqn.22.5.5.1 $V_c = \min(8 \times \lambda \times \rho^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\ psi)} \times d = \mathbf{11920}$ lb/ft

Strength reduction factor $\phi_s = \mathbf{0.75}$

Design concrete shear strength - cl.7.6.3.1 $\phi V_c = \phi_s \times V_c = \mathbf{8940}$ lb/ft

$V / \phi V_c = \mathbf{0.349}$

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

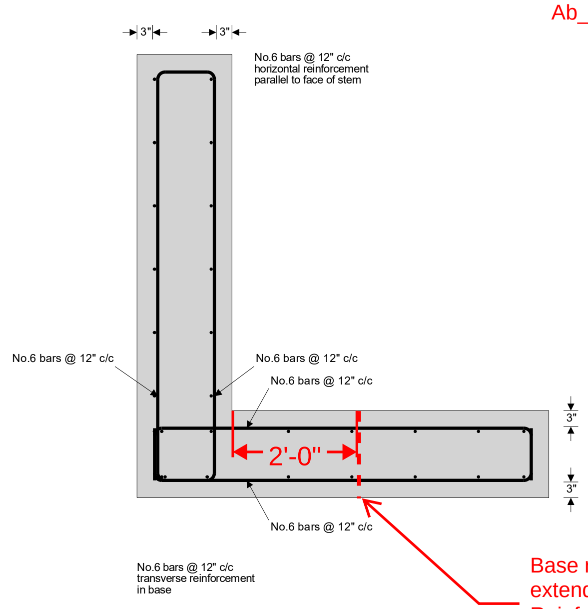
Minimum area of reinforcement - cl.7.6.1.1 $A_{bx.req} = 0.0018 \times t_{base} = \mathbf{0.356}\ in^2/ft$

Transverse reinforcement provided No.6 bars @ 12" c/c each face

Area of transverse reinforcement provided $A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{0.884}\ in^2/ft$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Ab_min = 0.003*h*(12 in/ft) = 0.594 in^2/ft (both faces)
Note that t_base = h = 16.5 in.
Ab_min < Ab_prov = 0.884 in^2/ft (both faces)



Base reinforcement only extends to CJ. Reinforcement for culvert roof per Caltrans standard.

Reinforcement details

CROSS CHANNEL BOXES (8 FT)

Calculation is based on user defined combination values

Overall design summary provided by Tedds software does not account for overall structural system. Design explanations provided in results.

Tedds calculation version 2.9.11

Analysis summary

~~**Design summary**~~

~~Overall design utilisation 3.67~~

~~Overall design status Fail~~

Description	Unit	Capacity	Applied	F o S	Result
~~Sliding stability~~	~~plf~~	~~1462~~	~~5367~~	~~0.272~~	~~FAIL~~
~~Overturning stability~~	~~lb_ft/ft~~	~~30947~~	~~23126~~	~~1.338~~	~~PASS~~
~~Bearing pressure~~	~~psf~~	~~4000~~	~~928~~	~~4.310~~	~~PASS~~

By inspection, cross channel wall is externally stable due to box design/layout and connections with overall system and existing cross channel.

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Stem p0 rear face - Flexural reinforcement	in²/ft	0.802	0.620	0.773	PASS
Stem p0 - Shear resistance	lb/ft	8203	6185	0.754	PASS
Base bottom face - Flexural reinforcement	in²/ft	0.802	0.785	0.979	PASS
Base - Shear resistance	lb/ft	8203	2962	0.361	PASS
Transverse stem reinforcement	in²/ft	0.614	0.259	0.422	PASS
Transverse base reinforcement	in²/ft	0.614	0.259	0.422	PASS

Retaining wall details

Stem type — Cantilever

Stem height — h_{stem} = **8** ft

Stem thickness — t_{stem} = **12** in

Angle to rear face of stem — α = **90** deg

Stem density — γ_{stem} = **150** pcf

Toe length — l_{toe} = **13** ft

Used for design purposes only with Tedds software to avoid software errors.

Base thickness — t_{base} = **12** in

Base density — γ_{base} = **150** pcf

Height of retained soil — h_{ret} = **8** ft

Angle of soil surface — β = **0** deg

Conservative retained soil parameters for design purposes.

Depth of cover — d_{cover} = **0** ft

Retained soil properties

Soil type — Dense fine or silty sand

Moist density — γ_{mr} = **120** pcf

Saturated density — γ_{sr} = **120** pcf

Based on Sept. 2020 Geotechnical Memorandum by Gannett Fleming (Geotech Memo).

Base soil properties

Soil type — Dense fine or silty sand

Soil density — γ_b = **120** pcf

Gross allowable bearing pressure — q_{allow_gross} = **4000** psf

Based on Geotech Memo for seismic loading plus all other loads.

Seismic details

Horizontal seismic acceleration factor — K_h = **0.472**

Vertical seismic acceleration factor — K_v = **0**

Kh = 0.5*(PGAm) = 0.5*(0.943), from Geotech Memo; Kv = 0 (conservative assumption)

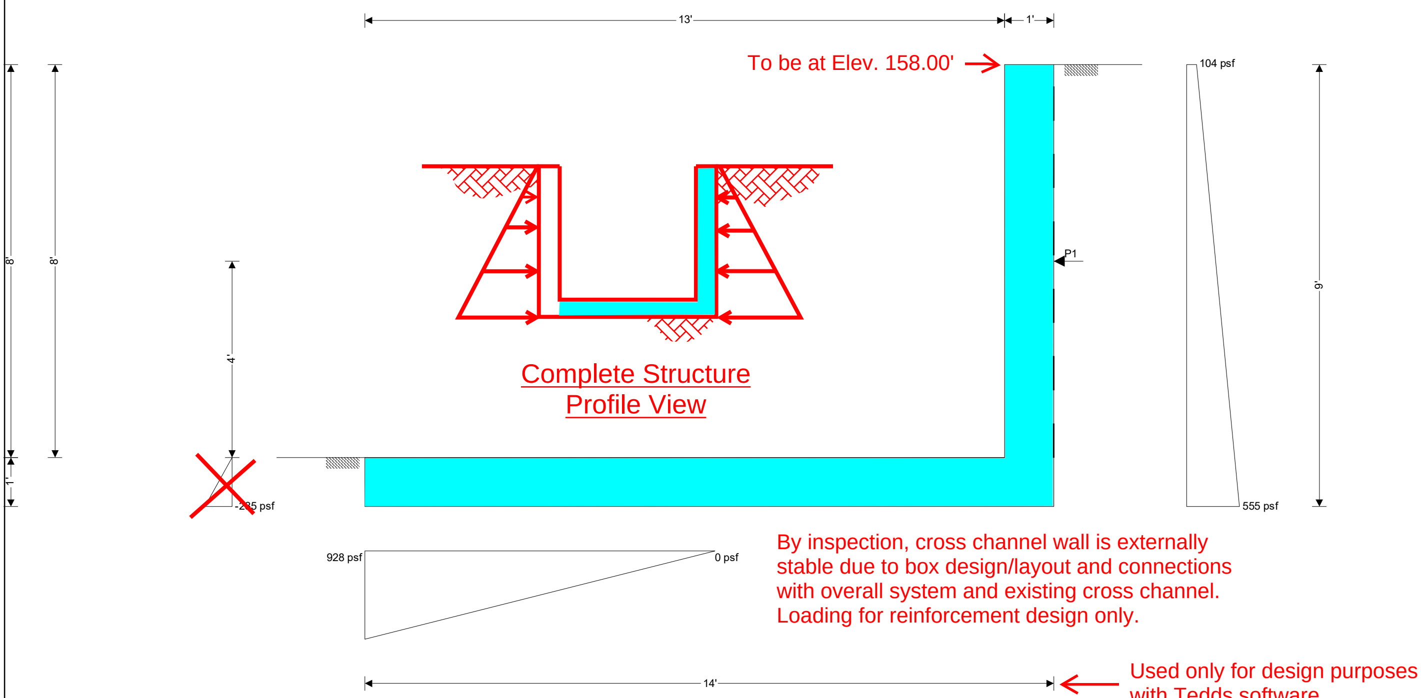
Seismic acceleration angle — $\theta = \text{atan}(K_h / (1 - K_v))$ = **25.267** deg

Loading details

Live surcharge load — $Surcharge_L$ = **250** psf

Horizontal line load at 4 ft — P_{D1} = **376** plf

Type 60K Concrete Barrier Load = 900 plf x Ko
Load included conservatively in case barriers are placed at cross channel inlet/outlet structures.



General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Used only for design purposes with Tedds software. Usage of the toe length and overall base length was to avoid errors within Tedds.

Base length	$l_{base} = l_{toe} + t_{stem}$ = **14** ft
Moist soil height	$h_{moist} = h_{soil}$ = **8** ft
- Distance to horizontal seismic component	$x_{seismic\_h} = 0.6 \times (h_{soil} + h_{base})$ = **5.4** ft
Length of surcharge load	$l_{sur} = l_{heel}$ = **0** ft
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2$ = **14** ft
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret}$ = **9** ft
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2$ = **4.5** ft
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem}$ = **8** ft^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2$ = **13.5** ft
Area of wall base	$A_{base} = l_{base} \times t_{base}$ = **14** ft^2
- Distance to vertical component	$x_{base} = l_{base} / 2$ = **7** ft

Soil coefficients

Coefficient of friction to back of wall	K_{fr} = **0.400**
Coefficient of friction to front of wall	K_{fb} = **0.400**
Coefficient of friction beneath base	K_{fbb} = **0.400**
At rest pressure coefficient	K_0 = **0.417**

Based on Geotech Memo.

Ko = 50 pcf / 120 pcf, from Geotech Memo.

Tekla Tedds Gannett Fleming 2251 Douglas Boulevard Suite 200 Roseville, CA 95661	Project NHC/United Water - Freeman Diversion - 3BC				Job Ref. 067376	
	Section Cross Channel Boxes (8 FT)				Sheet No./Rev. 3 / Rev.0	
	Calc. by RC/SMU	Date 7/11/2023	Chk'd by J. Allen	Date 7/18/2023	App'd by T. Sell	Date 8/11/2023

~~Passive pressure coefficient~~ ~~K_P = **2.375**~~

Using Mononobe-Okabe theory

Active dynamic pressure coefficient K_{AE} = **0.754**

~~Passive dynamic pressure coefficient~~ ~~K_{PE} = **2.374**~~

Not applicable.

User defined combination

Load combination 1 — 1 × Dead + 1 × Live + 1 × Lateral earth

Sliding check

Vertical forces on wall

Wall stem — $F_{stem} = A_{stem} \times \gamma_{stem}$ = **1200** plf

Wall base — $F_{base} = A_{base} \times \gamma_{base}$ = **2100** plf

Total — $F_{total\_v} = F_{stem} + F_{base}$ = **3300** plf

Horizontal forces on wall

Surcharge load — $F_{sur\_h} = K_0 \times Surcharge_L \times h_{eff}$ = **938** plf

Line loads — $F_{P\_h} = P_{D1}$ = **376** plf

Moist retained soil — $F_{moist\_h} = K_0 \times \gamma_{mr} \times h_{eff}^2 / 2$ = **2027** plf

Total — $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h}$ = **3341** plf

By inspection, cross channel wall is externally stable due to box design/layout and connections with overall system and existing cross channel.

Check stability against sliding

Base soil resistance — $F_{exc\_h} = K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2$ = **143** plf

Base friction — $F_{friction} = F_{total\_v} \times K_{fbb}$ = **1320** plf

Resistance to sliding — $F_{rest} = F_{exc\_h} + F_{friction}$ = **1462** plf

Factor of safety — $FoS_{sl} = F_{rest} / F_{total\_h}$ = **0.438** < 1.5

FAIL - Factor of safety against sliding is inadequate

Overturning check

Vertical forces on wall

Wall stem — $F_{stem} = A_{stem} \times \gamma_{stem}$ = **1200** plf

Wall base — $F_{base} = A_{base} \times \gamma_{base}$ = **2100** plf

Total — $F_{total\_v} = F_{stem} + F_{base}$ = **3300** plf

Horizontal forces on wall

Surcharge load — $F_{sur\_h} = K_0 \times Surcharge_L \times h_{eff}$ = **938** plf

Line loads — $F_{P\_h} = P_{D1}$ = **376** plf

Moist retained soil — $F_{moist\_h} = K_0 \times \gamma_{mr} \times h_{eff}^2 / 2$ = **2027** plf

Base soil — $F_{exc\_h} = -K_P \times \gamma_b \times (h_{pass} + h_{base})^2 / 2$ = **-143** plf

Total — $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{exc\_h}$ = **3198** plf

Overturning moments on wall

Surcharge load — $M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h}$ = **4222** lb_ft/ft

Line loads — $M_{P\_OT} = abs(P_{D1}) \times (p_1 + t_{base})$ = **1880** lb_ft/ft

Moist retained soil — $M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h}$ = **6080** lb_ft/ft

Total — $M_{total\_OT} = M_{sur\_OT} + M_{P\_OT} + M_{moist\_OT}$ = **12182** lb_ft/ft

Restoring moments on wall

Wall stem — $M_{stem\_R} = F_{stem} \times x_{stem}$ = **16200** lb_ft/ft

Wall base — $M_{base\_R} = F_{base} \times x_{base}$ = **14700** lb_ft/ft

Base soil $M_{exc\_R} = -F_{exc\_h} \times x_{exc\_h} =$ **47** lb_ft/ft

Total $M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{exc\_R} =$ **30947** lb_ft/ft

Check stability against overturning

Factor of safety $FoS_{ot} = M_{total\_R} / M_{total\_OT} =$ **2.54** > 1.5

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} =$ **1200** plf

Wall base $F_{base} = A_{base} \times \gamma_{base} =$ **2100** plf

Total $F_{total\_v} = F_{stem} + F_{base} =$ **3300** plf

By inspection, cross channel wall is externally stable due to box design/layout and connections with overall system and existing cross channel.

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_0 \times Surcharge_L \times h_{eff} =$ **938** plf

Line loads $F_{P\_h} = P_{D1} =$ **376** plf

Moist retained soil $F_{moist\_h} = K_0 \times \gamma_{mr} \times h_{eff}^2 / 2 =$ **2027** plf

Base soil $F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 =$ **-143** plf

Total $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{pass\_h} - F_{total\_v} \times K_{fbb} =$ **1878** plf

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} =$ **16200** lb_ft/ft

Wall base $M_{base} = F_{base} \times x_{base} =$ **14700** lb_ft/ft

Surcharge load $M_{sur} = -F_{sur\_h} \times x_{sur\_h} =$ **-4222** lb_ft/ft

Line loads $M_P = -(P_{D1} \times (p_1 + t_{base})) =$ **-1880** lb_ft/ft

Moist retained soil $M_{moist} = -F_{moist\_h} \times x_{moist\_h} =$ **-6080** lb_ft/ft

Base soil $M_{pass} = -F_{pass\_h} \times x_{pass\_h} =$ **47** lb_ft/ft

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{moist} + M_{pass} =$ **18766** lb_ft/ft

Check bearing pressure

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} =$ **5.687** ft

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 =$ **-1.313** ft

Loaded length of base $l_{load} = l_{base} =$ **14** ft

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) =$ **368** psf

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) =$ **103** psf

Allowable bearing capacity $q_{allow} = q_{allow\_gross} =$ **4000** psf

Factor of safety $FoS_{bp} = q_{allow} / \max(q_{toe}, q_{heel}) =$ **10.858**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

User defined combination

Load combination 2 1 × Dead + 1 × Live + 1 × Earthquake + 1 × Lateral earth

Sliding check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} =$ **1200** plf

Wall base $F_{base} = A_{base} \times \gamma_{base} =$ **2100** plf

Total $F_{total\_v} = F_{stem} + F_{base} =$ **3300** plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_0 \times Surcharge_L \times h_{eff} = \mathbf{938}$ plf

Line loads $F_{P\_h} = P_{D1} = \mathbf{376}$ plf

Moist retained soil $F_{moist\_h} = K_0 \times \gamma_{mr} \times h_{eff}^2 / 2 = \mathbf{2027}$ plf

Seismic $F_{seismic\_h} = K_0 \times \gamma_{mr} \times (h_{soil} + h_{base})^2 / 2 = \mathbf{2027}$ plf

Total $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{seismic\_h} = \mathbf{5367}$ plf

Check stability against sliding

Base soil resistance $F_{exc\_h} = K_{PE} \times \gamma_b \times (h_{pass} + h_{base})^2 / 2 = \mathbf{142}$ plf

Base friction $F_{friction} = F_{total\_v} \times K_{fbb} = \mathbf{1320}$ plf

Resistance to sliding $F_{rest} = F_{exc\_h} + F_{friction} = \mathbf{1462}$ plf

Factor of safety $FoS_{sl} = F_{rest} / F_{total\_h} = \mathbf{0.272} < 1.1$

FAIL - Factor of safety against sliding is inadequate

Overturning check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{1200}$ plf

Wall base $F_{base} = A_{base} \times \gamma_{base} = \mathbf{2100}$ plf

Total $F_{total\_v} = F_{stem} + F_{base} = \mathbf{3300}$ plf

By inspection, cross channel wall is externally stable due to box design/layout and connections with overall system and existing cross channel.

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_0 \times Surcharge_L \times h_{eff} = \mathbf{938}$ plf

Line loads $F_{P\_h} = P_{D1} = \mathbf{376}$ plf

Moist retained soil $F_{moist\_h} = K_0 \times \gamma_{mr} \times h_{eff}^2 / 2 = \mathbf{2027}$ plf

Base soil $F_{exc\_h} = -K_{PE} \times \gamma_b \times (h_{pass} + h_{base})^2 / 2 = \mathbf{-142}$ plf

Seismic $F_{seismic\_h} = K_0 \times \gamma_{mr} \times (h_{soil} + h_{base})^2 / 2 = \mathbf{2027}$ plf

Total $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{exc\_h} + F_{seismic\_h} = \mathbf{5225}$ plf

Overturning moments on wall

Surcharge load $M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h} = \mathbf{4222}$ lb_ft/ft

Line loads $M_{P\_OT} = abs(P_{D1}) \times (p_1 + t_{base}) = \mathbf{1880}$ lb_ft/ft

Moist retained soil $M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h} = \mathbf{6080}$ lb_ft/ft

Seismic $M_{seismic\_OT} = F_{seismic\_h} \times x_{seismic\_h} = \mathbf{10944}$ lb_ft/ft

Total $M_{total\_OT} = M_{sur\_OT} + M_{P\_OT} + M_{moist\_OT} + M_{seismic\_OT} = \mathbf{23126}$ lb_ft/ft

Restoring moments on wall

Wall stem $M_{stem\_R} = F_{stem} \times x_{stem} = \mathbf{16200}$ lb_ft/ft

Wall base $M_{base\_R} = F_{base} \times x_{base} = \mathbf{14700}$ lb_ft/ft

Base soil $M_{exc\_R} = -F_{exc\_h} \times x_{exc\_h} = \mathbf{47}$ lb_ft/ft

Total $M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{exc\_R} = \mathbf{30947}$ lb_ft/ft

Check stability against overturning

Factor of safety $FoS_{ot} = M_{total\_R} / M_{total\_OT} = \mathbf{1.338} > 1.1$

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{1200}$ plf

Wall base $F_{base} = A_{base} \times \gamma_{base} =$ **2100** plf

Total $F_{total\_v} = F_{stem} + F_{base} =$ **3300** plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_0 \times Surcharge_L \times h_{eff} =$ **938** plf

Line loads $F_{P\_h} = P_{D1} =$ **376** plf

Moist retained soil $F_{moist\_h} = K_0 \times \gamma_{mr} \times h_{eff}^2 / 2 =$ **2027** plf

Base soil $F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 =$ **-143** plf

Seismic $F_{seismic\_h} = K_0 \times \gamma_{mr} \times (h_{soil} + h_{base})^2 / 2 =$ **2027** plf

Total $F_{total\_h} = F_{sur\_h} + F_{P\_h} + F_{moist\_h} + F_{pass\_h} + F_{seismic\_h} - F_{total\_v} \times K_{fbb} =$ **3905** plf

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} =$ **16200** lb_ft/ft

Wall base $M_{base} = F_{base} \times x_{base} =$ **14700** lb_ft/ft

Surcharge load $M_{sur} = -F_{sur\_h} \times x_{sur\_h} =$ **-4222** lb_ft/ft

Line loads $M_P = -(P_{D1} \times (p_1 + t_{base})) =$ **-1880** lb_ft/ft

Moist retained soil $M_{moist} = -F_{moist\_h} \times x_{moist\_h} =$ **-6080** lb_ft/ft

Base soil $M_{pass} = -F_{pass\_h} \times x_{pass\_h} =$ **47** lb_ft/ft

Seismic $M_{seismic} = -F_{seismic\_h} \times x_{seismic\_h} =$ **-10944** lb_ft/ft

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{moist} + M_{pass} + M_{seismic} =$ **7822** lb_ft/ft

By inspection, cross channel wall is externally stable due to box design/layout and connections with overall system and existing cross channel.

Check bearing pressure

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} =$ **2.37** ft

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 =$ **-4.63** ft

Loaded length of base $l_{load} = 3 \times \bar{x} =$ **7.111** ft

Bearing pressure at toe $q_{toe} = 2 \times F_{total\_v} / l_{load} =$ **928** psf

Bearing pressure at heel $q_{heel} =$ **0** psf

Allowable bearing capacity $q_{allow} = q_{allow\_gross} =$ **4000** psf

Factor of safety $FoS_{bp} = q_{allow} / \max(q_{toe}, q_{heel}) =$ **4.31**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

CROSS CHANNEL BOXES (8 FT)

Retaining wall design in accordance with ACI 318-19

In conformance with USACE EM 1110-2-2104 (Nov. 2016), Sections 2 and 3. Additional information provided, where applicable.

Tedds calculation version 2.9.11

Concrete details

Compressive strength of concrete $f'_c =$ **4500** psi

Concrete type Normal weight

Reinforcement details

Yield strength of reinforcement $f_y =$ **60000** psi

Modulus of elasticity or reinforcement $E_s =$ **29000000** psi

Compression-controlled strain limit $\varepsilon_{ty} =$ **0.002**

Cover to reinforcement

In conformance with USACE EM 1110-2-2104 (Nov. 2016), Table 2-1, for "formed and screeded surfaces such as stilling basin walls, chute spillway slabs, and channel lining slabs on grade: greater than 12 in. and less than 24 in. thick".

Front face of stem $c_{sf} =$ **3** in

Rear face of stem $c_{sr} =$ **3** in

Top face of base $c_{bt} =$ **3** in

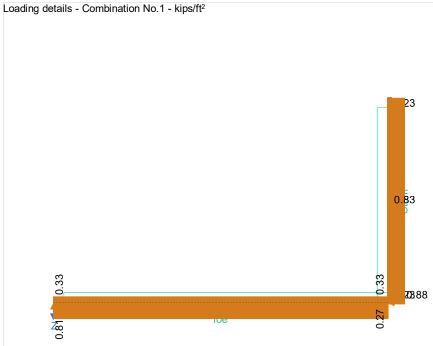
Bottom face of base $c_{bb} =$ **3** in

User defined load combinations

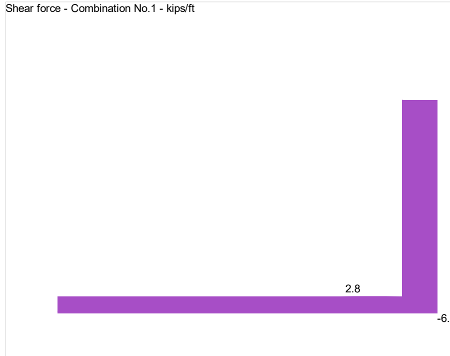
In conformance with USACE EM 1110-2-2104 (Nov. 2016), Table 3-1. Loads with "f" are considered "favorable" conditions by Tedds, but calculations below only use "unfavorable" (non-"f") loads.

Load combination no.1	2.2D + 1Df + 2.2L + 1Lf + 2.2H + 1Hf	Static Case
Load combination no.2	2.2D + 1Df + 2.2L + 1Lf + 2.2H + 1Hf	
Load combination no.3	1.2D + 1Df + 1L + 1Lf + 1.25E + 1Ef + 1.5H + 1Hf	Seismic Case
Load combination no.4	1.2D + 1Df + 1L + 1Lf + 1.25E + 1Ef + 1.5H + 1Hf	

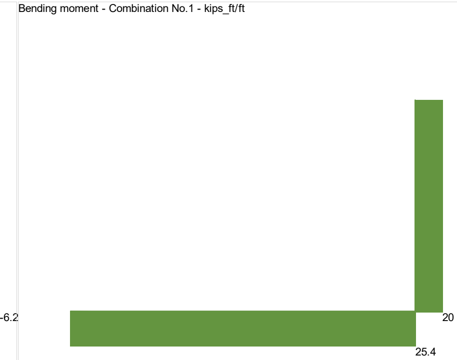
Loading details - Combination No.1 - kips/ft²



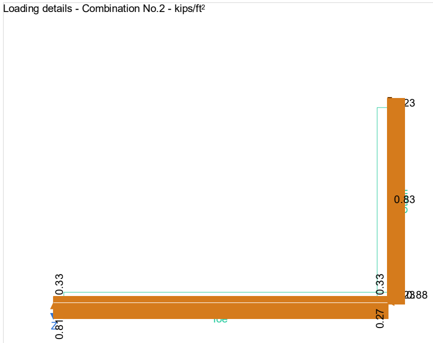
Shear force - Combination No.1 - kips/ft



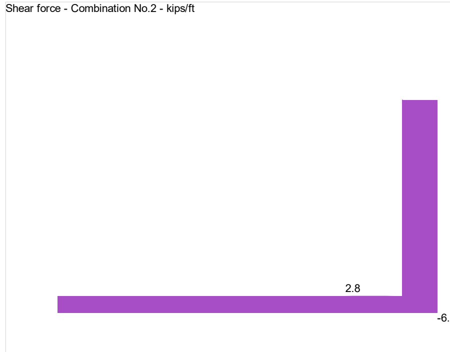
Bending moment - Combination No.1 - kips_ft/ft



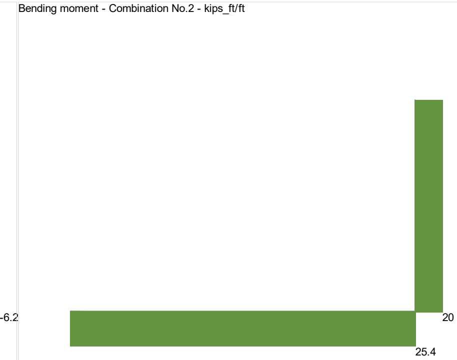
Loading details - Combination No.2 - kips/ft²



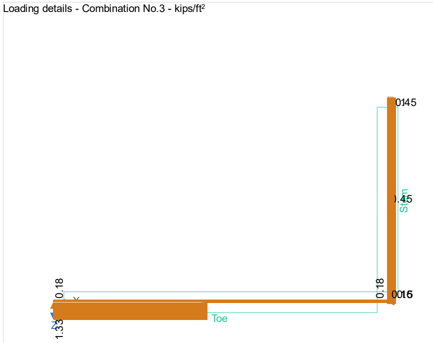
Shear force - Combination No.2 - kips/ft



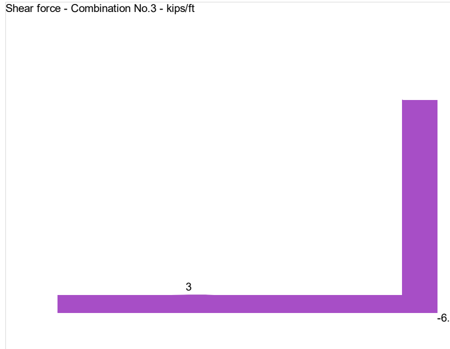
Bending moment - Combination No.2 - kips_ft/ft



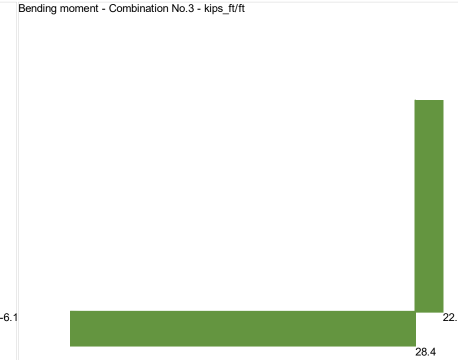
Loading details - Combination No.3 - kips/ft²

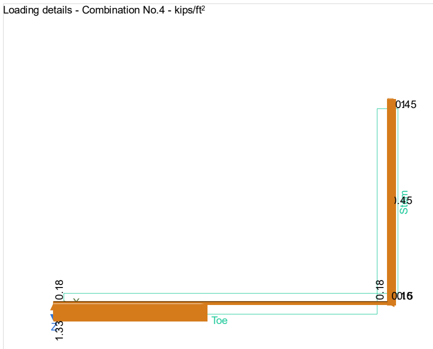


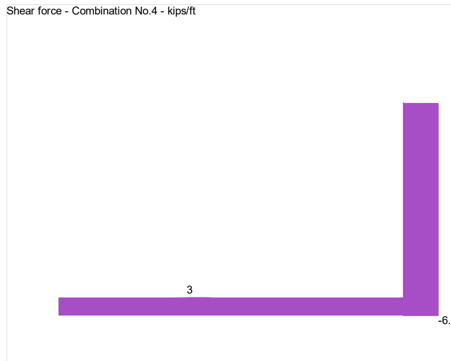
Shear force - Combination No.3 - kips/ft

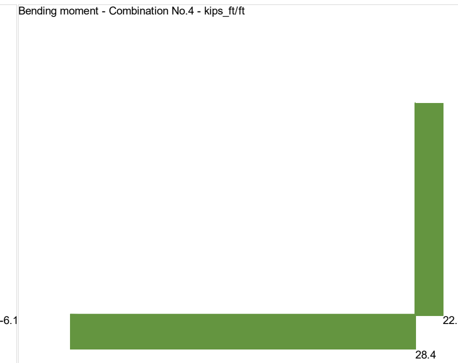


Bending moment - Combination No.3 - kips_ft/ft









Check stem design at base of stem

Depth of section $h = \mathbf{12}$ in

Rectangular section in flexure - Section 22.3

Design bending moment combination 4 $M = \mathbf{22755}$ lb_ft/ft

Depth of tension reinforcement $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{8.563}$ in

Compression reinforcement provided No.7 bars @ 9" c/c

Area of compression reinforcement provided $A_{sf.prov} = \pi \times \phi_{sf}^2 / (4 \times s_{sf}) = \mathbf{0.802}$ in²/ft

Tension reinforcement provided No.7 bars @ 9" c/c

Area of tension reinforcement provided $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{0.802}$ in²/ft

Maximum reinforcement spacing - cl.7.7.2.3 $s_{max} = \min(18\text{ in}, 3 \times h) = \mathbf{18}$ in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{sr.prov} \times f_y / (0.85 \times f'_c) = \mathbf{1.048}$ in

Neutral axis factor - cl.22.2.2.4.3 $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\text{ ksi}) / 1\text{ ksi}, 0.65), 0.85) = \mathbf{0.825}$

Depth to neutral axis $c = a / \beta_1 = \mathbf{1.27}$ in

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c = \mathbf{0.017221}$

Section is in the tension controlled zone

Strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / 0.003, 0.65), 0.9) = \mathbf{0.9}$

Nominal flexural strength $M_n = A_{sr.prov} \times f_y \times (d - a / 2) = \mathbf{32225}$ lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n = \mathbf{29002}$ lb_ft/ft

$M / \phi M_n = \mathbf{0.785}$

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{sr.des} = \mathbf{0.62}$ in²/ft

Minimum area of reinforcement - cl.7.6.1.1 $A_{sr.min} = 0.0018 \times h = \mathbf{0.259}$ in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

USACE EM 1110-2-2104 (Nov. 2016), Section 2.9 requires 0.003 instead of 0.0018.
As_min = 0.003*h*(12 in/ft) = 0.432 in^2/ft (both faces)
As_min < As_prov = (0.802+0.802) in^2/ft (both faces)

Rectangular section in shear - Section 22.5

Design shear force $V = \mathbf{6185}$ lb/ft

Concrete modification factor - cl.19.2.4 $\lambda = \mathbf{1}$

Depth of tension reinforcement $d = \mathbf{8.56}$ in

Size effect modification factor - cl. 22.5.5.1.3 $\lambda_s = \min(\sqrt{2 / (1 + (d / 1\text{ in}) / 10)}, 1.0) = \mathbf{1}$

Reinforcement ratio $\rho = A_{sr.prov} / d = \mathbf{0.008}$

Nominal concrete shear strength - eqn.22.5.5.1 $V_c = \min(8 \times \lambda_s \times \lambda \times \rho^{1/3}, 5 \times \lambda) \times \sqrt{f'_c \times 1\text{ psi}} \times d = \mathbf{10937}$ lb/ft

Strength reduction factor $\phi_s = \mathbf{0.75}$

Design concrete shear strength - cl.7.6.3.1 — $\phi V_c = \phi_s \times V_c =$ **8203** lb/ft

$V / \phi V_c =$ **0.754**

PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.7.6.1.1 — $A_{sx.req} = 0.0018 \times t_{stem} =$ **0.259** in^2/ft

Transverse reinforcement provided — No.5 bars @ 12" c/c each face

Area of transverse reinforcement provided — $A_{sx.prov} = 2 \times \pi \times \phi_{sx}^2 / (4 \times s_{sx}) =$ **0.614** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

As_min = 0.003*h*(12 in/ft) = 0.432 in^2/ft (both faces)
Note that t_stem = h = 12 in.
As_min < As_prov = 0.614 in^2/ft (both faces)

Check base design at toe

Depth of section — h = **12** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 4 — M = **28420** lb_ft/ft

Depth of tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2 =$ **8.563** in

Compression reinforcement provided — No.7 bars @ 9" c/c

Area of compression reinforcement provided — $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) =$ **0.802** in^2/ft

Tension reinforcement provided — No.7 bars @ 9" c/c

Area of tension reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) =$ **0.802** in^2/ft

Maximum reinforcement spacing - cl.7.7.2.3 — $s_{max} = \min(18 \text{ in}, 3 \times h) =$ **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block — $a = A_{bb.prov} \times f_y / (0.85 \times f'_c) =$ **1.048** in

Neutral axis factor - cl.22.2.2.4.3 — $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) =$ **0.825**

Depth to neutral axis — $c = a / \beta_1 =$ **1.27** in

Strain in reinforcement — $\varepsilon_t = 0.003 \times (d - c) / c =$ **0.017221**

Section is in the tension controlled zone

Strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / 0.003, 0.65), 0.9) =$ **0.9**

Nominal flexural strength — $M_n = A_{bb.prov} \times f_y \times (d - a / 2) =$ **32225** lb_ft/ft

Design flexural strength — $\phi M_n = \phi_f \times M_n =$ **29002** lb_ft/ft

$M / \phi M_n =$ **0.980**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis — $A_{bb.des} =$ **0.785** in^2/ft

Minimum area of reinforcement - cl.7.6.1.1 — $A_{bb.min} = 0.0018 \times h =$ **0.259** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Ab_min = 0.003*h*(12 in/ft) = 0.432 in^2/ft (both faces)
Ab_min < Ab_prov = (0.802+0.802) in^2/ft (both faces)

Rectangular section in shear - Section 22.5

Design shear force — V = **2962** lb/ft

Concrete modification factor - cl.19.2.4 — $\lambda =$ **1**

Depth of tension reinforcement — d = **8.56** in

Size effect modification factor - cl. 22.5.5.1.3 — $\lambda_s = \min(\sqrt{2 / (1 + (d / 1 \text{ in}) / 10)}, 1.0) =$ **1**

Reinforcement ratio — $\rho = A_{bb.prov} / d =$ **0.008**

Nominal concrete shear strength - eqn.22.5.5.1 — $V_c = \min(8 \times \lambda \times \rho^{1/3}, 5 \times \lambda) \times \sqrt{f'_c \times 1 \text{ psi}} \times d =$ **10937** lb/ft

Strength reduction factor — $\phi_s =$ **0.75**

Design concrete shear strength - cl.7.6.3.1 — $\phi V_c = \phi_s \times V_c =$ **8203** lb/ft

$V / \phi V_c =$ **0.361**

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

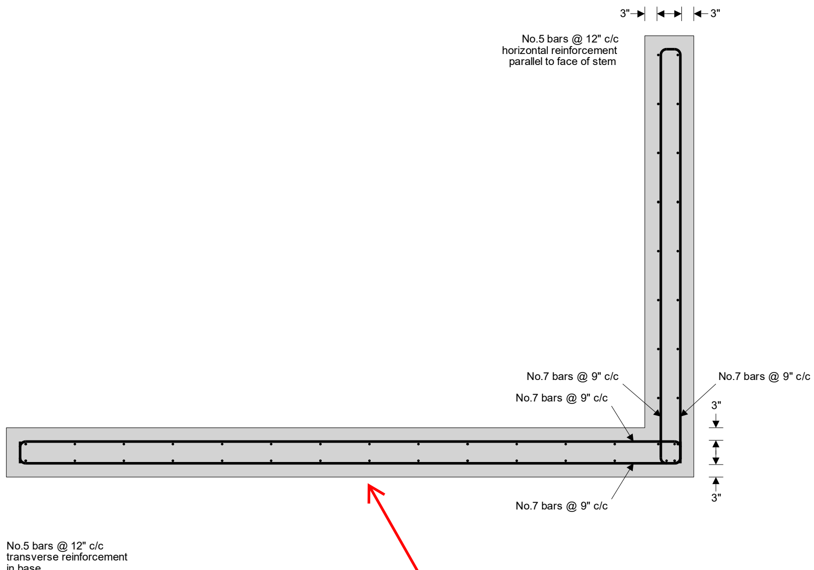
Minimum area of reinforcement - cl.7.6.1.1 $A_{bx.req} = 0.0018 \times t_{base} = \mathbf{0.259}$ in^2/ft

Transverse reinforcement provided No.5 bars @ 12" c/c each face

Area of transverse reinforcement provided $A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{0.614}$ in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

Ab_min = 0.003*h*(12 in/ft) = 0.432 in^2/ft (both faces)
Note that t_base = h = 12 in.
Ab_min < Ab_prov = 0.614 in^2/ft (both faces)



Reinforcement details

Base reinforcement design was used as a basis for detailing cross channel inlet/outlet structures' footing.