October 31, 2023

Ventura County Transportation Commission, (VCTC)

SESPE CREEK BRIDGE OVERFLOW SANTA PAULA BRANCH LINE, FILLMORE, CA

90% Submittal - Design Submittal Report

Prepared by:
Julina Corona
RailPros Inc.
811 Wilshire Blvd
Suite 1820
Los Angeles, CA 90017

Table of Contents

1. List of Items Transmitted

1.1 Design Drawing Sheets

Surveying Control Points: 90% Survey Control sheets are included.
Track Design Sheets: Track general notes, track plan and profile sheets have been updated to 90 percent design which address comments provided by Sierra Northern and VCTC during the 30 percent design submittal review process.
Structural Design Sheets: The Structural Design details have been prepared in conformance with the detail required at the 90 percent level. The proposed solution consists of two new bents, two new cast-in-place (CIP) pier caps, and two new spans that are each 49-feet in length. Each bent is comprised of two cast-in-drilled-hole (CIDH) piles 30% design diameter of the pile is taken as 6 feet pending final Geotech memo. The abutment and wingwall will be constructed at the end of the bridge. This design is shown on S-001 General Plan No. 1 Plan and Elevation.

1.2 Structural Design Calculations

Bridge Design Calculation Package: Structural calculations are based on the 90 percent design drawings.

1.3 Cultural Memorandum

Memo: Cultural memo was prepared to demonstrate a thorough evaluation was conducted for the eligibility of the Sespe Creek Overflow Bridge for listing in the National Register of Historic Places (NRHP).

1.4 Geotechnical Investigation Report

Report: Includes geotechnical recommendations to the engineer based on soil testing results of geotechnical boring samples taken on-site. The report has been updated to be consistent with 90% design.

1.5 Hydraulics Report

Report: Includes results from HEC-RAS development of the memo is dependent on confirmation of boring sample lab results and is consistent with the 90 percent design level.

2. Statement of Design Accomplishments

The design submittal is at 90 percent level. So far, the following have been completed:

- Geotechnical Report at 90 percent design level
- Hydraulics Report at 90 percent design level
- Cultural Resources Memorandum
- Bridge Design Calculations at 90 percent level
- Utility investigation as part of 30 percent submittal
- Rough Order Magnitude Cost as part of 30 percent submittal

3. Design Changes

Structural Design for Repair

Structural components are entirely re-designed and meet or exceed Metrolink standard specifications. The superstructure required re-design so the proposed repair could geometrically to tie into the existing portion of the bridge while adopting current industry engineering standards and E-80 demand load criteria.

Handrail Repair

Handrail repair was designed to match existing handrail. Not a Metrolink standard handrail.

Track Repair

Track repair to match existing rail size or nearest rail size and to match jointed rail connection. Not a Metrolink standard CWR rail.

Hydraulic capacity

Hydraulics report findings show existing bridge does not meet SCRRA design criteria. Consequentially the proposed bridge repair will also not meet these criteria as addressing this issue would be beyond the scope this rehabilitation project.

Cofferdam design

During 30 percent design development it was identified by biologist and SWPPP lead that a cofferdam design for water diversion would be needed to address the condition of a continuously wet creek bed to meet requirements of the Regional Water Quality Control Board permits.

4. Environmental and Permitting Progress Update

Items in progress: SWPPP/Construction General Permit, 404 Permit, 401 Certification, CDFW LSAA (1600) Permit

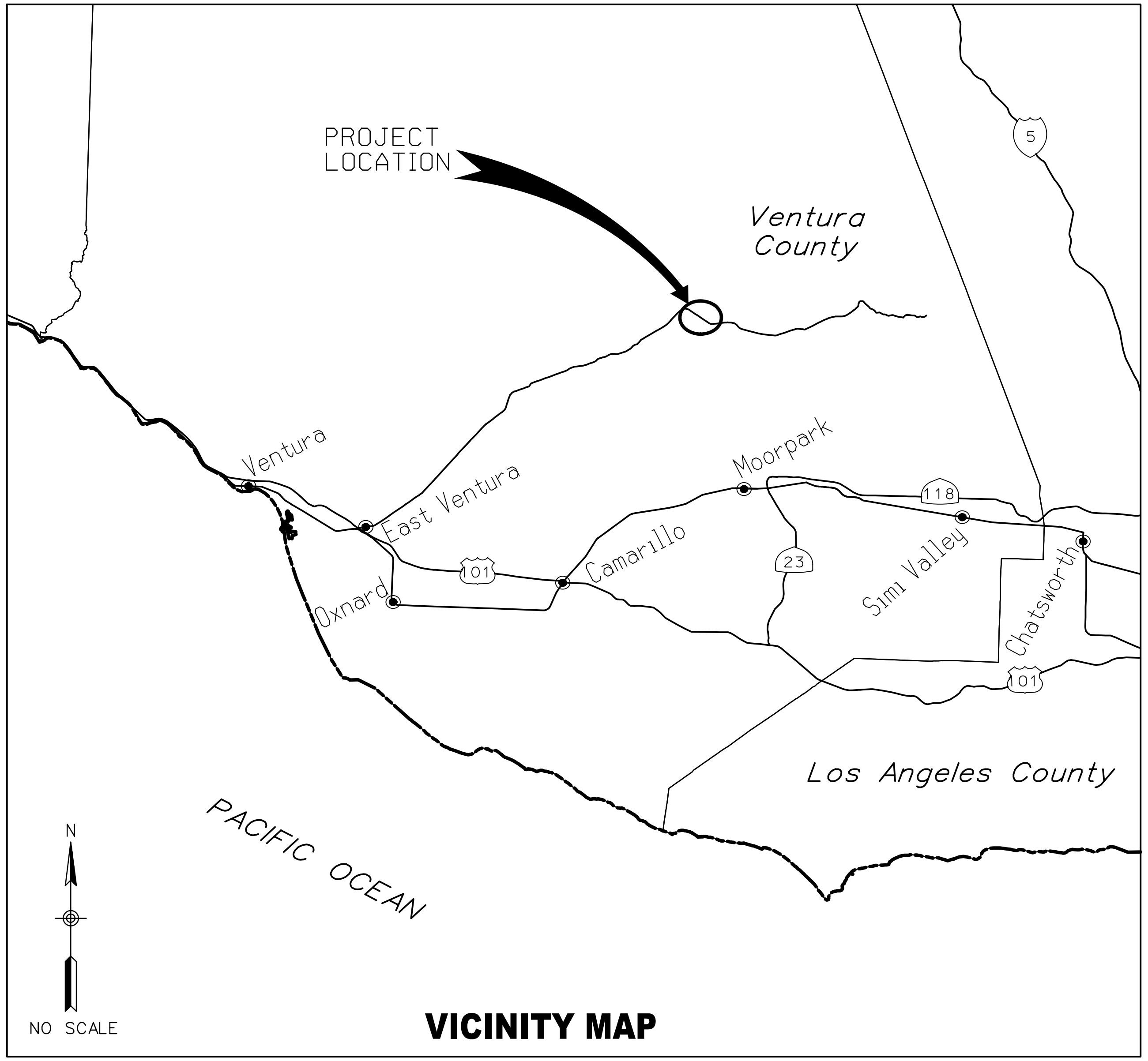
5. Quality Assurance / Quality Control (QA/QC)

RailPros follows a detailed internal QA/QC program. Quality Assurance for the 90 percent design submittal was performed by Adam Hall. Julina Corona performed the QC review for the track design drawings. Sarwar Naveed performed the QC review for the structural design drawings. Calculations were reviewed by Sarwar Naveed, Kurt Thomsen and Demi Yang.

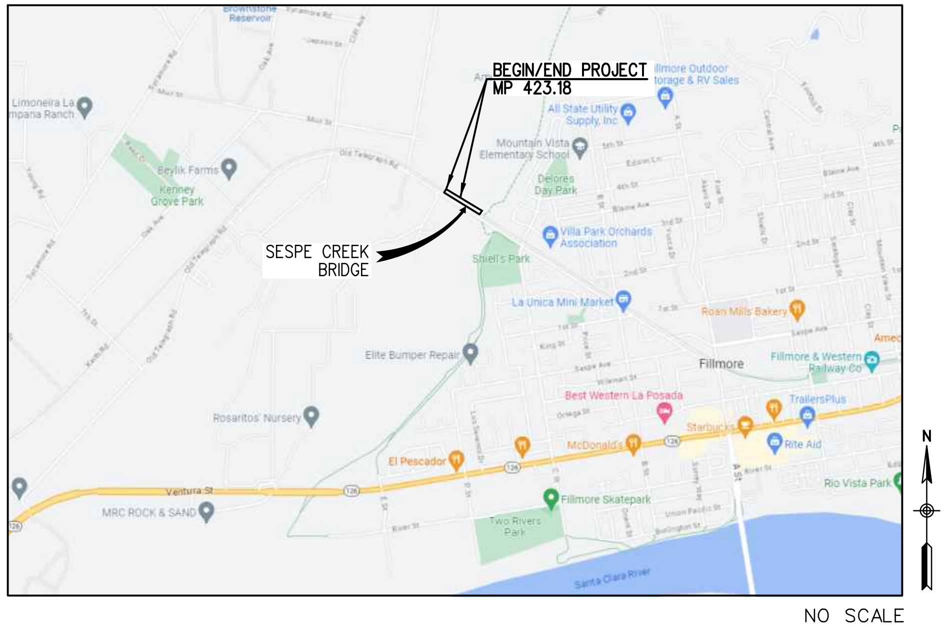
6. Cost Estimate

Quantities will be submitted week of November 6th 2023 with the cost and source information to be checked by Julina Corona.

VENTURA COUNTY TRANSPORTATION COMMISSION
SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA



VICINITY MAP



LOCATION MAP

OCTOBER 31, 2023

90% SUBMITTAL

NOT FOR CONSTRUCTION

RAILPROS

APPROVED BY: ______________________ DATE: __________

SUBMITTED BY: ______________________ DATE: __________
JULINA CORONA, P.E.
PROJECT MANAGER, RAILPROS

VCTC

10/31/2023 USER = jackson.ziegler
Z:\Engineering\VCTC\Sespe Creek Bridge Overflow\900 CADD\950 Drawings\Track\VCTC_SCB_G-001.dgn
Z:\Engineering\SCRRA\140 SCORE\03 Phase 1 Final Design - Burbank\900 CADD\950 Drawings\Plot Drivers\PlotStamp.tbl
Z:\Engineering\VCTC\Sespe Creek Bridge Overflow\900 CADD\950 Drawings\Plot Drivers\SCRRA-11X17-CLR-PDF-HW-CLR.pltcfg

SHT NO.	DWG. NO.	REV. NO.	TITLE
GENERAL			
1	G-001	0	TITLE SHEET
2	G-002	0	INDEX OF DRAWINGS
3	G-003	0	STANDARD ABBREVIATIONS
4	G-004	0	STANDARD SYMBOLS
5	G-005	0	GENERAL NOTES
6	G-006	0	SURVEY CONTROL EXHIBIT
TRACK			
7	TD-001	0	TYPICAL SECTION
8	RP-001	0	TRACK PLAN AND PROFILE - STA 98+50 TO STA 110+50
9	DIV-001	0	TEMPORARY CREEK DIVERSION PLAN
STRUCTURES			
10	S-001	0	GENERAL PLAN NO. 1
11	S-002	0	GENERAL PLAN NO. 2
12	S-003	0	GENERAL NOTES AND INDEX OF DRAWINGS
13	S-004	0	STAGE CONSTRUCTION PLAN
14	S-005	0	FOUNDATION PLAN
15	S-006	0	ABUTMENT DETAILS NO. 1
16	S-007	0	ABUTMENT DETAILS NO. 2
17	S-008	0	ROCK SLOPE PROTECTION
18	S-009	0	BENT DETAILS NO. 1
19	S-010	0	BENT DETAILS NO. 2
20	S-011	0	BENT DETAILS NO. 3
21	S-012	0	GIRDER DETAILS NO. 1
22	S-013	0	GIRDER DETAILS NO. 2
23	S-014	0	HANDRAIL REPLACEMENT PLAN
24	S-015	0	HANDRAIL DETAILS
25	S-016	0	MISCELLANEOUS DETAILS NO. 1
26	S-017	0	MISCELLANEOUS DETAILS NO. 2
GEOTECHNICAL			
27	GE-001	0	LOG OF TEST BORINGS
28	GE-002	0	SOIL LEGEND 1 OF 2 - LOG OF TEST BORINGS
29	GE-003	0	SOIL LEGEND 2 OF 2 - LOG OF TEST BORINGS

PRELIMINARY 90%
NOT FOR CONSTRUCTION

REV.	DATE		BY / SUB.	APP.
			CG	NO

DESIGNED BY M. WHITE
DRAWN BY J. ZIEGLER
CHECKED BY M. WHITE
APPROVED BY N. ORTEGA
DATE 10-31-2023

VENTURA COUNTY
TRANSPORTATION
COMMISSION

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SUBMITTED: ______________
JULINA CORONA, P.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

INDEX OF DRAWINGS

CONTRACT NO.
DRAWING NO. G-002
REVISION
SHEET NO. 2 OF 29
SCALE NTS

UTILITY AND AGENCY CONTACTS

SCRRA (METROLINK)	(909) 392-8463 CHRISTOS SOURMELIS
CITY OF SANTA CLARITA	(661) 286-4172 LESLIE FRAZIER
AT&T TRANSMISSION	(714) 963-7964 JOSEPH FORKERT
PLAINS ALL AMERICAN PIPELINE	(562) 728-2371 PAULA BAWDEN
MCI(VERIZON BUSINESS)	(469) 886-4238 DEAN BOYERS
QWEST/CENTURYLINK/LEVEL 3 COMMUNICATIONS	(303) 992-9931 GEORGE MCELVAIN
SPRINT	(800) 659-9698 TIBOR LAKY
WILCON (WILSHIRE CONNECTION LLC)	(213) 542-0100 NOC
CHARTER COMMUNICATIONS/TWC	(818) 295-3030 JERRY BAYLES
SOUTHERN CALIFORNIA GAS	(818) 701-3245 SAM SIFUENTES

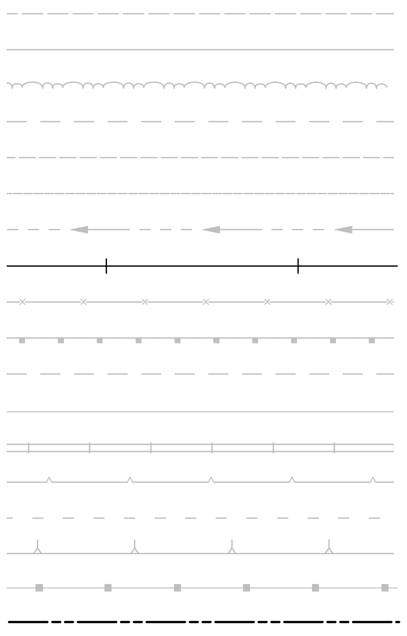
ABBREVIATIONS

ADS	ADVANCED DRAINAGE SYSTEMS
AVE	AVENUE
AT&T	AMERICAN TELEPHONE AND TELEGRAPH COMPANY
AWW	ABSOLUTE WORK WINDOW
BLVD	BOULEVARD
CI	CAST IRON
₵	CENTERLINE
CMPA	CORRUGATED METAL PIPE ARCH
CONT	CONTINUED
CP	CONTROL POINT
CPUC	CALIFORNIA PUBLIC UTILITIES COMMISSION
CWR	CONTINUOUS WELDED RAIL
Dc	DEGREE OF CURVE
θ_s	DEFLECTION ANGLE - SPIRAL
DI	DRAINAGE INLET
DOT	DEPARTMENT OF TRANSPORTATION (U.S.)
DWG	DRAWING
EA	EACH
Ea	ACTUAL SUPERELEVATION
Eu	UNBALANCED SUPERELEVATION
ELEV	ELEVATION
ES	ENGINEERING STANDARDS (SCRRA STANDARD DRAWINGS)
EG	EXISTING GROUND
EWD	EASTWARD DIRECTION
EXIST, EX, (E)	EXISTING
FL	FLOW LINE
FT	FEET, FOOT
FWY	FREEWAY
GPS	GLOBAL POSITIONING SYSTEM
HMA	HOT MIX ASPHALT
HR	HOUR
HTTO	HAND THROW TURNOUT
HDPE	HIGH DENSITY POLY ETHYLENE
HST	HOLLOW STEEL TIE
IJ	INSULATED JOINT
JCT	JUNCTION
L	LENGTH
LA	LOS ANGELES
LACMTA	LOS ANGELES COUNTY METROPOLITAN TRANSPORTATION AUTHORITY
LACTC	LOS ANGELES COUNTY TRANSPORTATION COMMISSION
Lc	LENGTH OF CIRCULAR CURVE
Ls	LENGTH OF SPIRAL
LF	LINEAL FOOT
LH	LEFT HAND
LLT	LAST LONG TIE
LT	LEFT
LG	LIP OF GUTTER
LWW	LIMITED WORK WINDOW
MCI	MICROWAVE COMMUNICATIONS INC.
MFS	MERCANTILE FREIGHT SERVICE
MH	MANHOLE
MIN	MINUTE
MIN	MINIMUM
MP	MILEPOST
MPH	MILES PER HOUR
MT	MAIN TRACK
NAD 83	NORTH AMERICAN DATUM OF 1983
NAD 88	NORTH AMERICAN DATUM OF 1988
NO	NUMBER
NTS	NOT TO SCALE
OH	OVERHEAD
OTM	OTHER TRACK MATERIAL
OFF	OFFSET
O.C.	ON CENTER

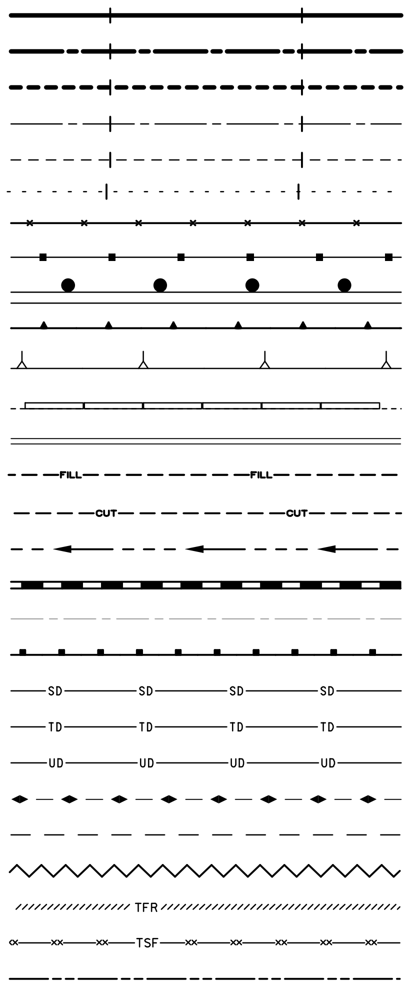
ABBREVIATIONS (CONT.)

PCC	PORTLAND CEMENT CONCRETE
PED	PEDESTRIAN
PH	POT HOLE
PITO	POINT OF INTERSECTION OF TURNOUT
POB	POINT OF BEGINNING
POE	POINT OF ENDING
POTO	POWER OPERATED TURNOUT
PROP	PROPOSED
PS	POINT OF SWITCH
PI	POINT OF INTERSECTION
SPI	POINT OF INTERSECTION - SPIRAL
SC	POINT OF SPIRAL TO CIRCULAR CURVE
CS	POINT OF CIRCULAR CURVE TO SPIRAL
ST	POINT OF SPIRAL TO TANGENT
TS	POINT OF TANGENT TO SPIRAL
PT	POINT OF TANGENCY
PTC	POSITIVE TRAIN CONTROL
PVI	POINT OF VERTICAL INTERSECTION
PVT	POINT OF VERTICAL TANGENT
PVC	POINT OF VERTICAL CURVE
QWEST	QWEST ENGINEERING
R	RADIUS
RBM	RAIL BOUND MANGANESE
RR	RAILROAD
RH	RIGHT HAND
RCB	REINFORCED CONCRETE BOX
ROW, R/W	RIGHT-OF-WAY
RT	RIGHT
RWIC	RAILROAD WORKER IN CHARGE
SCRRA	SOUTHERN CALIFORNIA REGIONAL RAIL AUTHORITY
STA	STATION
ST	STREET
SD	STORM DRAIN
SUB	SUBDIVISION
SWT	SWITCH
TCE	TEMPORARY CONSTRUCTION EASEMENT
TF	TURNOUT
TO	TRACK FOOT
TOR, T/R	TOP OF RAIL
TWC	TIME WARNER CABLE
TYP	TYPICAL
UPRR	UNION PACIFIC RAILROAD
V	VELOCITY
VERT	VERTICAL
WSM	WELDED SPRING MANGANESE
WWD	WESTWARD DIRECTION
WWM	WELDED WIRE MESH
XING	CROSSING

EXISTING LINESTYLES


ASPHALT SURFACE
BUILDING
BRUSH LINE/TREE LINE
CONCRETE SURFACE
CURB
DIRT SURFACE
FLOW LINE
EXISTING TRACK
FENCE AND HANDRAILS
GUARD RAIL
GUTTER
PROPERTY LINE
RAILROAD TRACK
RETAINING WALL
ROAD STRIPING
TOP OF SLOPE
SCRRA INTERTRACK FENCE/WWM
SCRRA RIGHT-OF-WAY

PROPOSED LINESTYLES



PROPOSED TRACK
PROPOSED RESURFACE TRACK
PROPOSED SHIFT TRACK
EXISTING RESURFACE TRACK
EXISTING SHIFT TRACK
TRACK TO BE REMOVED
FENCE
SCRRA INTERTRACK FENCE/WWM
ROADWAY GUARDRAIL
RETAINING WALL / GRAVITY WALL
TOP OF SLOPE
K-RAIL
PLATFORM HANDRAIL
FILL
CUT
FLOW LINE
BLOCK WALL
CENTERLINE OF ROAD
GUARDRAIL
STORM DRAIN
TRENCH DRAIN
UNDER DRAIN
PLATFORM EDGE FENCE
LIMITS OF CONSTRUCTION BOUNDARY
CONST JOINT
FIBER ROLLS
SILT CONTROL FENCE
PROPOSED TEMPORARY CONSTRUCTION EASEMENT

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PRELIMINARY 90%
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INFORMATION CONFIDENTIAL: All plans, drawings, specifications, and or information furnished herewith shall remain the property of the the Southern California Regional Rail Authority and shall be held confidential; and shall not be used for any purpose not provided for in agreements with the Southern California Regional Rail Authority.

DESIGNED BY
M. WHITE
DRAWN BY
J. ZIEGLER
CHECKED BY
M. WHITE
APPROVED BY
N. ORTEGA
DATE
10-31-2023

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SUBMITTED: ______________________
JULINA CORONA, P.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

STANDARD ABBREVIATIONS

CONTRACT NO.
DRAWING NO.
G-003
REVISION
SHEET NO.
3 OF 29
SCALE
NTS

GENERAL SYMBOLS

DESCRIPTION	EXISTING	PROPOSED
ATCS/PTC ANTENNA		
BILLBOARD		
BUILDING		
BUMPER		
COORDINATE	N2,800,500	
CROSSING GATE & FLASHERS		
CURVE NUMBER	C12	C12
ELECTROLIER WITH POLE		
ELECTROLIERS, DOUBLE WITH POLE		
ELECTROLIER WITHOUT POLE		
FIRE HYDRANT		
FLASHERS		
FLAG POLE		
FLARED END SECTION		
FLOW		
GRID TICK	+	+
GROUND CONTROL POINT (AERIAL)	△	△
GUY WIRE		
HEADWALL		
MANHOLE	MH	MH
NORTH ARROW		
PHOTOELECTRIC CELL	PC	PC
POLE-MOUNTED LUMINAIRE		
POT HOLE LOCATION		
POWER POLE/TELEPHONE POLE		
RAILROAD MILEPOST	XX MP 2.27	XX (XX) SG MP 59.00 OR MP 59.00 (MP 2.27)
SANITARY SEWER MANHOLE	S	S
SIGN		
RAILROAD SIGNAL		
RAILROAD CANTILEVER SIGNAL		
STATION EQUALITY	EQ	EQ
STORM DRAIN CATCH BASIN	CB	CB
STORM DRAIN DROP INLET	DI	
STORM DRAIN MANHOLE	SD	
TELEPHONE MANHOLE	T	T
THIRD PARTY PROJECTS		
TRAFFIC SIGNAL		
TRAFFIC SIGNAL WITH ARM ONLY		
TRAFFIC SIGNAL WITH ARM AND POLE		
TREE		
TREE PALM		
TREE LINE, SHRUBBERY		
TIME CLOCK	TC	TC

SWITCHES AND DERAILS

DESCRIPTION	EXISTING	PROPOSED
POINT OF SWITCH (HAND-THROW TURNOUT)		
POINT OF SWITCH (POWER-OPERATED TURNOUT)		
DERAIL SWITCH POINT	SP	SP
DERAIL POWERED SWITCH POINT		
DERAIL BI-DIRECTIONAL WITH CROWDER	▽	▼
RAIL LUBRICATOR	RL	RL

SURVEY CONTROL SYMBOLS

HORIZONTAL CONTROL POINT	△
HORIZONTAL AND VERTICAL CONTROL POINT	△
VERTICAL CONTROL POINT	○
BENCHMARK	

SIGNAL HOUSES, CASES, SECURITY AND UTILITY BOXES & MANHOLES

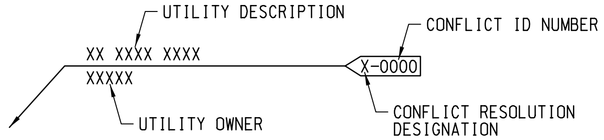
DESCRIPTION	EXISTING	PROPOSED
SIGNAL HOUSE	□	□
10x10 SIGNAL CASE	10x10	10x10
BATTERY BOX	□	□
CCTV, SECURITY MANHOLE	⊟	⊟
TELEVISION MANHOLE	⊞	⊞
ELECTRIC MANHOLE	⧄	⧄
WATER VALVE BOX	□	□
TRAFFIC CONTROL BOX	◪	◪

HATCHES AND PATTERNS

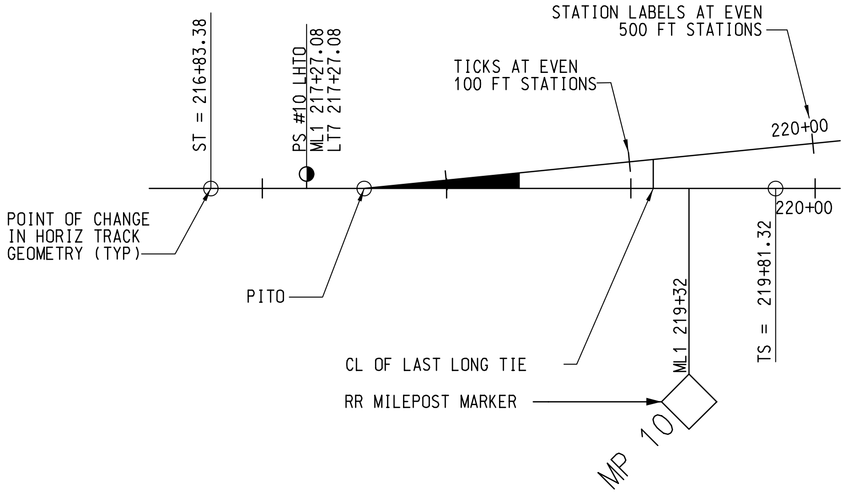
	PATTERNS
STONE/BRICK PAVING	▨
BALLAST	▨
TIMBER	▨
SUBGRADE, EARTH	▨
SUBBALLAST	▨
AGGREGATE BASE	▨
CONCRETE	▨
PEDESTRIAN CROSSING PANEL	▨
TACTILE WARNING TILES	▨
GRADED/LANDSCAPED AREA	▨
GRADE CROSSING PANELS	▨
HOT MIX ASPHALT CONCRETE	▨
SAWCUT EXISTING ASPHALT	▨

EXISTING TO BE REHABBED

UTILITIES LEGEND



TYPICAL ILLUSTRATION OF TRACK AND TURNOUT



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SUBMITTED: ______
JULINA CORONA, P.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

STANDARD SYMBOLS

CONTRACT NO.
DRAWING NO. G-004
REVISION
SHEET NO. 4 OF 29
SCALE NTS

GENERAL NOTES

1. THE CONTRACTOR SHALL COMPLY WITH ALL LOCAL, STATE, AND FEDERAL SAFETY CODES REGULATIONS, AND SPECIFICATIONS FOR THIS CONTRACT.

2. ALL CONSTRUCTION ACTIVITIES SHALL BE SCHEDULED AND COORDINATED WITH THE ENGINEER AND THE VARIOUS COMPANIES, AGENCIES, AND OTHER CONTRACTORS WHO MAY BE AFFECTED BY THIS WORK.

3. HORIZONTAL AND VERTICAL CONTROL POINTS FOR THE SITE LAYOUT ARE IDENTIFIED IN THE CONTRACT DOCUMENTS. IT SHALL BE THE CONTRACTOR'S RESPONSIBILITY TO UTILIZE THESE CONTROL POINTS TO ASSURE THAT ALL FACILITIES INCLUDED IN PROJECT ARE CONSTRUCTED AT THE CORRECT HORIZONTAL AND VERTICAL LOCATIONS.

4. SECTION 4216/4217 OF THE GOVERNMENT CODE REQUIRES A DIG ALERT IDENTIFICATION NUMBER BE ISSUED BEFORE A "PERMIT TO EXCAVATE" IS VALID. THE CONTRACTOR SHALL CALL THE UNDERGROUND SERVICE ALERT (1-800-422-4133) TWO (2) WORKING DAYS PRIOR TO CONSTRUCTION TO OBTAIN A DIG ALERT ID NUMBER.

5. CALIFORNIA SENATE BILL 1359 (APPROVED 2006) OUTLINES PROCEDURES FOR LOCATING UTILITIES BY HAND EXCAVATION. THE CONTRACTOR SHALL BECOME FAMILIAR WITH THIS LEGISLATION AND COMPLY WITH ITS DIRECTIVE. PRIOR TO EACH CONSTRUCTION ACTIVITY WITHIN RAILROAD RIGHT-OF-WAY, THE CONTRACTOR SHALL NOTIFY RAILROAD'S SIGNAL REPRESENTATIVE.

6. SIERRA NORTHERN & VCTC ARE NOT MEMBERS OF DIG ALERT. THE CONTRACTOR SHALL CALL SIERRA NORTHERN'S 24-HOUR EMERGENCY NUMBER A MINIMUM OF FIVE DAYS PRIOR TO BEGINNING CONSTRUCTION TO MARK SIGNAL AND COMMUNICATION CABLES AND CONDUITS. TO ASSURE CABLES AND CONDUITS HAVE BEEN MARKED, NO WORK WAY PROCEED UNTIL THE CONTRACTOR HAS BEEN PROVIDED WITH WRITTEN AUTHORIZATION TO PROCEED FROM SIERRA NORTHERN. IN CASE OF SIGNAL EMERGENCIES OR GRADE CROSSING PROBLEMS, THE CONTRACTOR SHALL CALL THE 24-HOUR EMERGENCY NUMBER PROVIDED.

7. THE CONTRACTOR SHALL FIELD VERIFY ALL DIMENSIONS FOR CONFLICTS WITH EXISTING UTILITIES, SIGNAL CABLES/EQUIPMENT, FIBER OPTIC LINES, AND/OR OTHER ITEMS THAT MIGHT IMPAIR CONSTRUCTION ACTIVITIES. INCONSISTENCIES FOUND SHALL BE REPORTED TO THE ENGINEER.

8. REPAIRS TO THE DAMAGED MATERIALS OR FACILITIES INTENDED TO REMAIN IN PLACE SHALL BE MADE BY THE CONTRACTOR AT THE CONTRACTOR'S EXPENSE UNLESS OTHERWISE STATED BY THE ENGINEER.

9. ALL EXCAVATED WASTE MATERIAL SHALL BE IMMEDIATELY REMOVED FROM THE SITE. ON SITE STORAGE OF EXCAVATED WASTE MATERIAL SHALL NOT BE PERMITTED AT ANY TIME.

10. DEFINITIONS:

    A. TRACK OUTAGE: TRACK WHICH IS OUT OF SERVICE FOR A GIVEN PERIOD OF TIME.

    B. ACTIVE TRACK: TRACK ON WHICH TRAINS ARE OPERATING AND INTERRUPTION OF SERVICE MAY OCCUR ONLY WITHIN AN APPROVED "WINDOW" AS DEFINED BELOW.

    C. FOULED TRACK: TRACK IS FOULED WHEN AN OBSTRUCTION IS PLACED WITHIN FOUR (4) FEET FROM THE NEAREST RAIL OF THE TRACK OR WHEN AN OVERHEAD OBSTRUCTION IS PLACED WITHIN TWENTY-TWO AND A HALF FEET (22'-6") ABOVE THE TOP OF RAIL.

    D. WINDOW: A GIVEN PERIOD OF TIME BETWEEN OPERATING TRAINS WHERE A TRACK MAY BE FOULED WITH THE STIPULATION THAT THE TRACK SHALL BE BACK IN SERVICE AT THE END OF THE GIVEN PERIOD OF TIME. A FORM OF POSITIVE PROTECTION SHALL ALSO BE REQUIRED.

    E. EXCLUSIVE TRACK WINDOW / ABSOLUTE WORK WINDOW (AWW): AN APPROVED WORK WINDOW IN WHICH NO TRAIN MOVEMENTS WILL OPERATE ON ANY TRACK WITHIN THE WINDOW LIMITS. THE CONTRACTOR MAY DISMANTLE, REMOVE, RECONSTRUCT, OR OTHERWISE OBSTRUCT TRACKS WITHIN THE LIMITS OF SUCH A WINDOW THIS WORK WAY BE PROTECTED BY TRACK OUT OF SERVICE, TRACK AND TIME LIMITS, OR BY FORM B TRACK BULLETIN.

    F. LIMITED TRACK WINDOW / LIMITED WORK WINDOW (LWW): AN APPROVED WORK WINDOW FOR SOME, BUT NOT ALL TRACKS WITHIN A GENERAL WORK AREA (E.G. ONE TRACK REMAINS FOR OPERATION OF TRAINS, OTHER TRACKS ARE AVAILABLE FOR THE CONTRACTOR'S WORK), MOVEMENT OF TRAINS OVER THE TRACK(S) OF A LIMITED TRACK WINDOW IS UNDER THE CONTROL OF THE SIERRA NORTHERN EMPLOYEE-IN CHARGE (EIC) WHO WILL NOT AUTHORIZE TRAIN MOVEMENT UNLESS AND UNTIL THE CONTRACTOR PERSONNEL AND EQUIPMENT ARE CLEAR OF THE OPERATING TRACK. THE CONTRACTOR MAY REMOVE, CONSTRUCT, OR OBSTRUCT ONLY THE TRACK DESIGNATED BY THE SSWP AND MUST ARRANGE THE WORK SO THAT TRAINS CAN OPERATE WITHOUT DELAY ON THE REMAINING TRACK(S) IN THE WORK AREA. THIS WORK MAY BE PROTECTED BY TRACK OUT OF SERVICE, TRACK AND TIME, OR BY FORM B TRACK BULLETIN.

    G. WORK WINDOW: AN APPROVED WORK WINDOW IN WHICH PASSENGER, FREIGHT AND ALL OTHER TRAINS AND ON-TRACK EQUIPMENT MOVEMENTS CAN BE PROHIBITED FROM ENTERING THE DEFINED LIMITS OF A SEGMENT OF TRACK. THE "FORM B" WORK WINDOW DOES NOT ALLOW THE CONTRACTOR TO REMOVE FROM SERVICE OR MODIFY THE TRACKS, SIGNALS. BRIDGES, STATIONS OR OTHER ELEMENTS OF THE OPERATING SYSTEM IN A MANNER, WHICH WILL DELAY OR IN ANY WAY AFFECT THE SAFE OPERATION OF THE TRAINS. THE "FORM B" WORK WINDOW ALLOWS THE CONTRACTOR THE ABILITY TO ENTER THE OPERATING ENVELOPE AND PERFORM CONSTRUCTION ACTIVITIES SUBJECT TO THE CONDITIONS ABOVE. AN EIC/FLAGMAN FROM SIERRA NORTHERN WILL EXERCISE STRICT CONTROL OVER THE CONTRACTOR'S CONSTRUCTION ACTIVITIES IN CONJUNCTION WITH ROADWAY WORKER PROTECTION REQUIREMENTS, TO ASSURE THAT THE CONTRACTOR'S ACTIVITIES DO NOT DELAY OR IMPACT TRAIN SERVICE.

    H. TRACK AND TIME: AN APPROVED WORK WINDOW IN WHICH THE DISPATCHER WILL AUTHORIZE MEN AND EQUIPMENT TO OCCUPY A TRACK OR TRACKS WITHIN LIMITS FOR A CERTAIN TIME PERIOD. THE DISPATCHER AUTHORITY SHALL INCLUDE AUTHORITY NUMBER, TRACK DESIGNATION, LIMITS AND TIME. MOVEMENTS MAY BE MADE IN EITHER DIRECTION WITHIN THE SPECIFIED LIMITS UNTIL THE LIMITED ARE RELEASED.

11. PRIOR TO COMMENCING WORK, ALL EXISTING SITE CONDITIONS SHALL BE FIELD VERIFIED WITH THE ENGINEER TO ASCERTAIN THE LIMITS OF WORK ACTIVITIES. THE CONTRACTOR SHALL SUBMIT AND RECEIVE THE ENGINEER'S APPROVAL OF THE PROJECT SCHEDULE AND OPERATIONS PLAN. EACH ITEM OF WORK SHALL BE DESCRIBED AND ACCOUNTED FOR IN THE CONTRACT DOCUMENTS. THE CONTRACTOR SHALL REFER TO THE SPECIFICATIONS FOR FURTHER INFORMATION REGARDING SUBMITTAL REQUIREMENTS.

GENERAL NOTES (CONTNUED)

12. RAIL TRAFFIC DISRUPTIONS SHALL BE KEPT TO A MINIMUM. DISRUPTIONS IN RAIL TRAFFIC THAT MAY BE REQUIRED SHALL BE COORDINATED WITH THE ENGINEER BEFOREHAND. NO SUCH WORK SHALL COMMENCE WITHOUT THE ENGINEER'S APPROVAL. WORK AFFECTING THE MOVEMENT OF TRAINS WILL BE UNDER THE AUTHORITY AND OVERALL CONTROL OF THE ENGINEER OR HIS REPRESENTATIVE.

13. THE CONTRACTOR SHALL NOT PLACE MATERIAL AND/OR EQUIPMENT WITHIn TWENTY (20) FEET OF AN ACTIVE TRACK AT ANY TIME WITHOUT PRIOR APPROVAL FROM THE ENGINEER.

14. WALKWAYS SHALL BE PLACED AS REQUIRED BY CALIFORNIA PUBLIC UTILITIES COMMISSION GENERAL ORDER NO. 118 AND 26D AND SCRRA ENGINEERING STANDARD ES2109 FOR ALL NEW CONSTRUCTION, UNLESS OTHERWISE NOTED.

15. THE CONTRACTOR AGREES THAT IN ACCORDANCE WITH GENERALLY ACCEPTED CONSTRUCTION PRACTICES , THE CONTRACTOR WILL BE REQUIRED TO ASSUME SOLE AND COMPLETE RESPONSIBILITY FOR JOB SITE CONDITIONS DURING THE COURSE OF CONSTRUCTION OF THE PROJECT, INCLUDING SAFETY OF ALL PERSONS AND PROPERTY, THAT THIS REQUIREMENT SHALL BE MADE TO APPLY CONTINUOUSLY AND NOT LIMITED TO NORMAL WORKING HOURS, AND THE CONTRACTOR FURTHER AGREES TO DEFEND, INDEMNIFY HOLD SIERRA NORTHERN, VCTC, VENTURA COUNTY AND THE DESIGN PROFESSIONAL HARMLESS FROM ANY AND ALL LIABILITY, REAL OR ALLEGED, IN CONNECTION WITH THE PERFORMANCE OF WORK ON THIS PROJECT.

16. THE LOCATIONS AND DIMENSIONS SHOWN ON THE PLANS FOR EXISTING FACILITIES ARE IN ACCORDANCE WITH AVAILABLE INFORMATION WITHOUT UNCOVERING AND MEASURING. THE ENGINEER DOES NOT GUARANTEE THE ACCURACY OF THIS INFORMATION OR THAT ALL EXISTING UNDERGROUND FACILITIES ARE SHOWN.

17. ALL WORK SHALL BE DONE IN ACCORDANCE WITH THE APPLICABLE CODES, ORDINANCES, AND STANDARD SPECIFICATIONS OF ALL AGENCIES THAT HAVE THE RESPONSIBILITY OF REVIEWING PLANS AND SPECIFICATIONS FOR CONSTRUCTION OF ALL ITEMS PER THESE PLANS AND SPECIFICATIONS IN THIS LOCALITY.

18. THE CONTRACTOR SHALL OBTAIN ALL THE NECESSARY PERMITS AND PAY PERMIT FEES AS REQUIRED FOR CONSTRUCTION OF THIS PROJECT.

19. THE CONTRACTOR SHALL CLEAN UP ALL DEBRIS AND MATERIALS RESULTING FROM HIS OPERATION AND RESTORE ALL SURFACES, STRUCTURES, DITCHES, AND PROPERTY TO ITS ORIGINAL CONDITION TO THE SATISFACTION OF THE ENGINEER.

20. CONTRACTOR SHALL PROVIDE FOR THE CONTINUOUS OPERATION OF THE EXISTING FACILITY WITHOUT INTERRUPTION DURING CONSTRUCTION EXCEPT DURING EXCLUSIVE TRACK WINDOWS OUTLINED IN THE SPECIFICATIONS AND UNLESS SPECIFICALLY AUTHORIZED OTHERWISE BY SIERRA NORTHERN.

21. CONTRACTOR TO IDENTIFY DEPTH AND LOCATION OF ALL EXISTING UNDERGROUND UTILITIES. FOR LOCATION OF SIGNALS AND COMMUNICATION CONDUITS CONTACT RAILROAD SIGNAL DEPARTMENT.

22. TIMBER TIES SHALL BE SPACED AT 19 1/2 INCHES ON CENTER. CONCRETE TIES SHALL BE SPACED AT 24 INCHES ON CENTER.

23. TEMPORARY FACILITIES CONSTRUCTED AND REMOVED BY THE CONTRACTOR TO PROVIDE FOR MAINTENANCE RAIL OPERATIONS DURING THE PHASING OF CONSTRUCTION (SUCH AS PLACEMENT OF A TEMPORARY TRACK PANEL AT THE LOCATION OF A TURNOUT TO BE CONSTRUCTED AT A FUTURE PHASE) WILL BE CONSIDERED INCIDENTAL TO OTHER ITEMS BEING CONSTRUCTED. NO SEPARATE MEASUREMENT OR PAYMENT WILL BE MADE FOR PROVIDING FOR THE CONTINUOUS OPERATION OF RAIL TRAFFIC.

24. EXISTING RAILROAD SIGNAGE (INCLUDING SPEED SIGNS) SHALL BE MAINTAINED DURING CONSTRUCTION PERIOD. ALL RAILROAD SIGNAGE SHALL BE FULLY RESTORED UPON COMPLETION OF EACH WORK PERIOD IN ACCORDANCE WITH SCRRA ENGINEERING STANDARDS. PRIOR TO CONSTRUCTION, SCRRA STANDARD PROJECT NOTICE SIGNS SHALL BE PLACED AT LOCATIONS AS DIRECTED BY THE ENGINEER. NO TRESPASSING SIGNS SHALL BE PLACED IN ACCORDANCE WITH ES5214 AND AS SHOWN ON THE DRAWINGS.

25. CONTACT VCTC'S CONSULTANT /CONTRACTOR TO ARRANGE FOR FLAGGING SERVICES. FLAGGING SERVICE IS DEPENDENT ON THE EIC AVAILABILITY AND MAY REQUIRE A MINIMUM OF FIFTEEN WORKING DAYS PRIOR TO BEGINNING WORK. PRIOR NOTIFICATION OF FLAGGING SERVICES DOES NOT GUARANTEE THE AVAILABILITY OF THE EIC FOR THE PROPOSED DATE OF WORK.

26. CONTACT VCTC'S CONSULTANT /CONTRACTOR TO ARRANGE FOR THIRD PARTY SAFETY TRAINING, ALLOW 5 WORKING DAYS FROM THE REQUEST FOR SAFETY TRAINING TO ARRANGE THE TRAINING.

27. NO MECHANIZED EXCAVATION WITHIN 2 FEET OF FIBER LINE IS ALLOWED. QWEST, VCTC AND MFS TO BE PRESENT FOR ANY ACTIVITY WITHIN 5 FEET HORIZONTALLY OR VERTICALLY OF FIBER LINES. NO FACILITIES MAY BE ADDED CLOSER THAN 2 FEET VERTICALLY OR HORIZONTALLY TO QWEST, LACTC AND MFS'S STRUCTURES, INCLUDING THE ENCASEMENT. CONTRACTOR SHALL POTHOLE ALL FIBER LINES WITHIN THE WORK LIMITS BEFORE BEGINNING WORK IN THAT VICINITY. IF CONSTRUCTION EQUIPMENT INTENDS TO DRIVE OVER THE FIBER LINE, CONTRACTOR SHALL PLACE STEEL PLATES OVER THE FIBER LINE BEFORE CONSTRUCTION CREWS DRIVE OVER FIBER.

DESIGN CRITERIA

SCRRA DESIGN CRITERIA MANUAL, FEBRUARY 2022

PRELIMINARY 90%
NOT FOR CONSTRUCTION

REV. | DATE | BY/SUB. | APP.

INFORMATION CONFIDENTIAL:
All plans, drawings, specifications, and or information furnished herewith shall remain the property of the the Southern California Regional Rail Authority and shall be held confidential; and shall not be used for any purpose not provided for in agreements with the Southern California Regional Rail Authority.

DESIGNED BY M. WHITE
DRAWN BY J. ZIEGLER
CHECKED BY M. WHITE
APPROVED BY N. ORTEGA
DATE 10-31-2023

VENTURA COUNTY
TRANSPORTATION
COMMISSION

RAILPROS
811 WILSHIRE, SUITE 1820
LOS ANGELES, CA 90017
WWW.RAILPROS.COM
EMAIL: INFO@RAILPROS.COM
PHONE: (213)627-0044

SUBMITTED: ______________________
JULINA CORONA, P.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

GENERAL NOTES

CONTRACT NO.
DRAWING NO. G-005
REVISION
SHEET NO. 5 OF 29
SCALE NTS

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TO EAST VENTURA ←

RR WEST

TO FILLMORE →

RR EAST

PROJECT CONTROL				
POINT NUMBER	NORTHING	EASTING	ELEVATION	DESCRIPTION
500	1971511.827	6280526.913	457.84'	CUT X IN CONC ON WB SIDE OF BRIDGE 27' EAST OF WEST EXPANSION JOINT
501	1971316.983	6280828.833	458.67'	CUT X IN CONC ON WB SIDE OF BRIDGE 94' WEST OF EAST EXPANSION JOINT
502	1971336.612	6280917.852	446.28'	3.5" USC&GS BRASS BM DISK STAMPED "S121B8, 1971" ON SE ABUTMENT, CONC WALKWAY
503	1971201.537	6281085.270	458.32'	MAGNAIL & SPIKE IN GROUND 5.15' FROM CONC CURBING AT GATE TO RR ABUTMENT ON SE SIDE OF RR TRX

LEGEND:

△ PROJECT CONTROL POINT

BASIS OF COORDINATES:

THE BASIS OF HORIZONTAL CONTROL IS THE NORTH AMERICAN DATUM OF 1983, 2011 ADJUSTMENT (NAD83-2011), MUTI-YEAR CORS SOLUTION 2 (MYSC2) ESTABLISHED BY USING THE SMARTNET SYSTEM OF CONTINUOUSLY OPERATING REFERENCE STATIONS (CORS).

COORDINATES ARE IN CALIFORNIA STATE PLANE COORDINATE SYSTEM, ZONE 5, EPOCH 2023.25, US SURVEY FT.

VERTICAL SURVEY CONTROL VALUES HEREON ARE BASED UPON THE NORTH AMERICAN VERTICAL DATUM OF 1988, GNSS-DERIVED BY FAST STATIC SURVEY METHODS USING GEIOD18 PER CALIFORNIA PUBLIC RESOURCES CODE 8890, DEFINED AS CALIFORNIA ORTHOMETRIC HEIGHTS OF 1988 (CH88).

ALL POSITIONS ARE CALCULATED PER A FULLY CONSTRAINED LEAST SQUARES ADJUSTMENT USING STARNET V11 LEAST SQUARES ADJUSTMENT SOFTWARE.

N

30' 0' 30' 60'

GRAPHIC SCALE

$USER$

$TIME$

$DATE$
$FILE$
$PENTBLL$
$PLTDRVL$

90% SUBMITTAL
NOT FOR CONSTRUCTION

REV. DATE BY SUB. APP.

INFORMATION CONFIDENTIAL: All plans, drawings, specifications, and or information furnished herewith shall remain the property of the the Southern California Regional Rail Authority and shall be held confidential; and shall not be used for any purpose not provided for in agreements with the Southern California Regional Rail Authority.

DESIGNED BY M. CUSICK
DRAWN BY M. CUSICK
CHECKED BY C. FESTA
APPROVED BY C. FESTA
DATE 10-27-2023

CODY J FESTA
LS9340

VENTURA COUNTY
TRANSPORTATION COMMISSION

RSE, INC.
1075 OLD COUNTY ROAD, STE. D
BELMONT, CA 94002
WWW.RSECORP.COM

SUBMITTED: ______
CODY FESTA, P.L.S.
SURVEY MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA
SURVEY CONTROL EXHIBIT

CONTRACT NO.
DRAWING NO. G-006
REVISION
SHEET NO. 6 OF 29
SCALE AS SHOWN

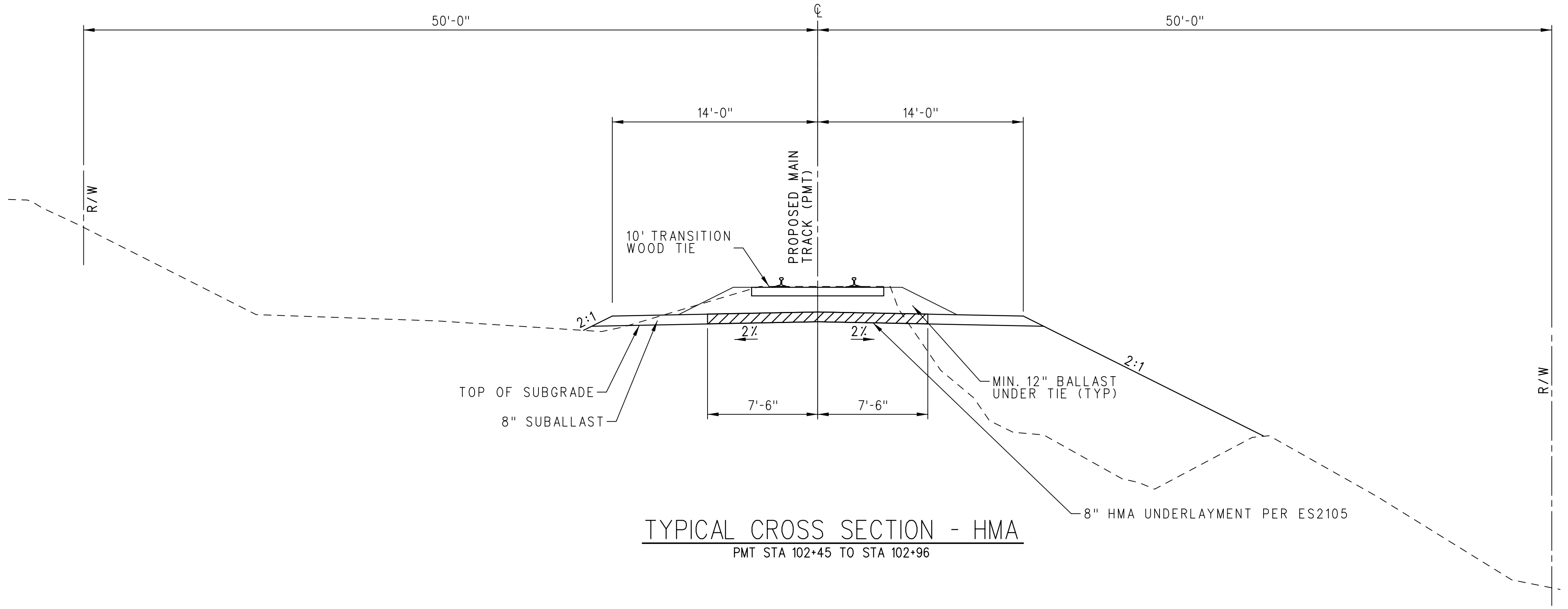
NOTES:

1) CONTRACTOR TO REMOVE AND REINSTALL TRACK FOR BRIDGE CONSTRUCTION AND HMA UNDERLAYMENT.

2) CONTRACTOR SHALL MAINTAIN A MINIMUM WALKWAY PER ES 2109 FOR ALL REINSTALLED AND RESURFACED TRACK.

3) SEE STRUCTURAL PLANS FOR PROPOSED BRIDGE.

4) CONTRACTOR TO FIELD VERIFY EXISTING HMA. IF HMA IS ENCOUNTERED, CONTRACTOR TO REMOVE EXISTING HMA PRIOR TO PROPOSED HMA INSTALLATION FOR BRIDGE APPROACH.



TYPICAL CROSS SECTION - HMA

PMT STA 102+45 TO STA 102+96

PRELIMINARY 90%
NOT FOR CONSTRUCTION

REV.	DATE	BY/SUB.	APP.

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DESIGNED BY M. WHITE
DRAWN BY J. ZIEGLER
CHECKED BY M. WHITE
APPROVED BY N. ORTEGA
DATE 10-31-2023

VENTURA COUNTY
TRANSPORTATION
COMMISSION

RAILPROS
811 WILSHIRE, SUITE 1820
LOS ANGELES, CA 90017
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EMAIL: INFO@RAILPROS.COM
PHONE: (213)627-0044

SUBMITTED: ______________________
JULINA CORONA, P.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

TYPICAL SECTION

CONTRACT NO.
DRAWING NO. TD-001
REVISION
SHEET NO. 7 OF 29
SCALE NTS

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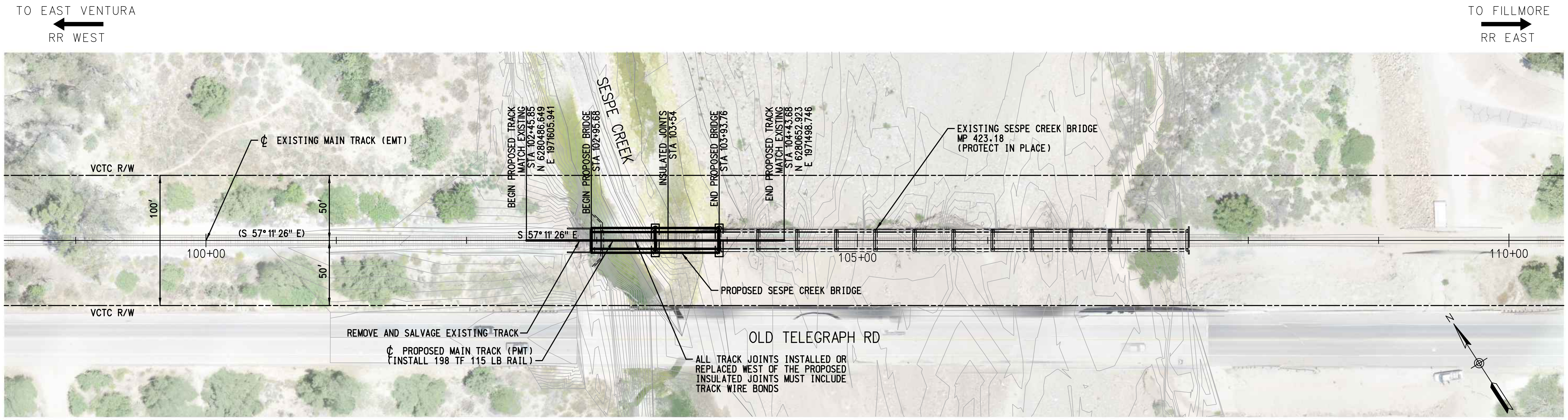


NOTES:

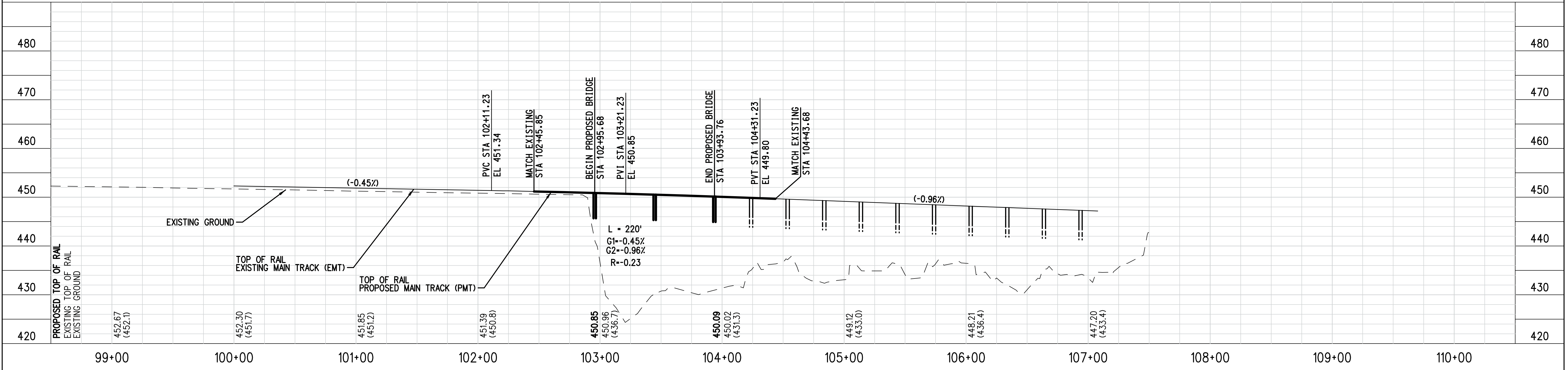
1. EXISTING RIGHT-OF-WAY IS BASED ON VCTC TRACK CHARTS

LEGEND:

▬	PROPOSED TRACK
─	EXISTING TRACK
— - —	EXISTING VCTC R/W

40' 0' 40' 80'
HORIZONTAL GRAPHIC SCALE

10' 0' 10' 20'
VERTICAL GRAPHIC SCALE



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PRELIMINARY 90%
NOT FOR CONSTRUCTION

REV. DATE BY SUB. APP.

INFORMATION CONFIDENTIAL: All plans, drawings, specifications, and or information furnished herewith shall remain the property of the the Southern California Regional Rail Authority and shall be held confidential; and shall not be used for any purpose not provided for in agreements with the Southern California Regional Rail Authority.

DESIGNED BY M. WHITE
DRAWN BY J. ZIEGLER
CHECKED BY M. WHITE
APPROVED BY N. ORTEGA
DATE 10-31-2023

VENTURA COUNTY
TRANSPORTATION
COMMISSION

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PHONE: (213)627-0044

SUBMITTED: ____________
JULINA CORONA, P.E.
PROJECT MANAGER

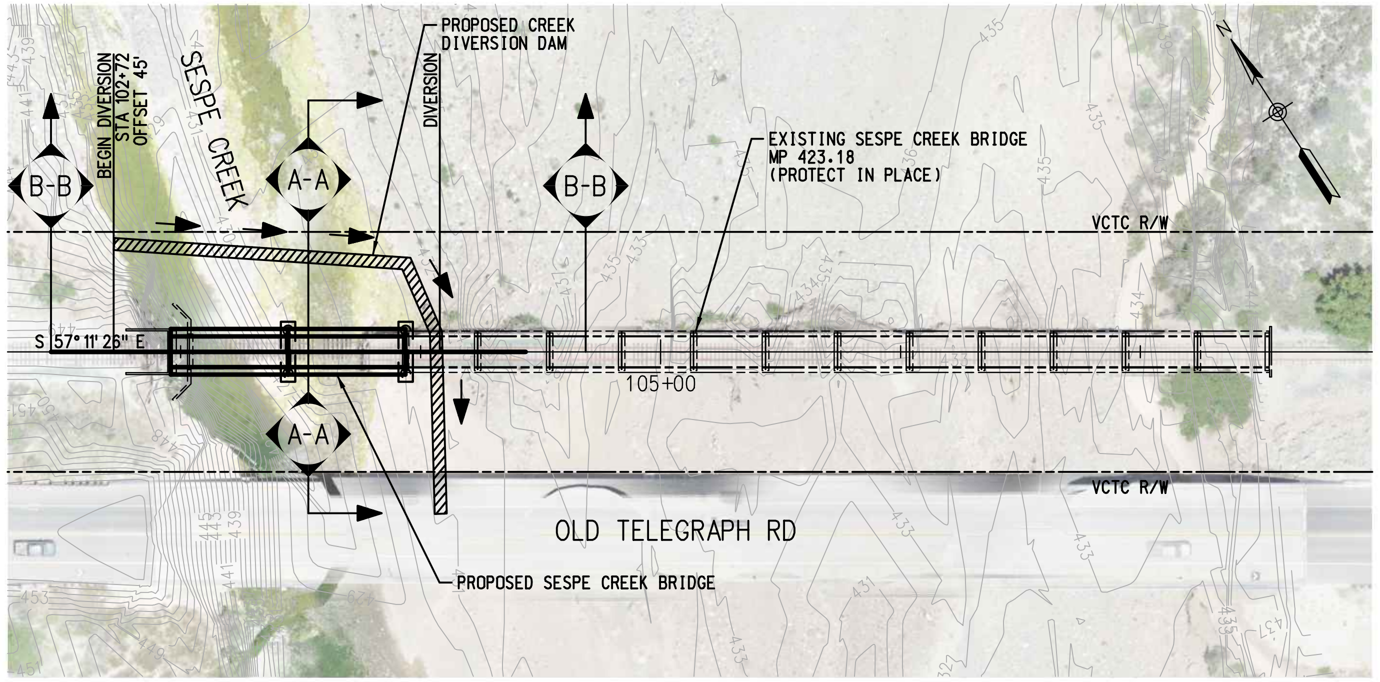
SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

TRACK PLAN AND PROFILE
STA 98+50 TO STA 110+50

CONTRACT NO.
DRAWING NO. RP-001
REVISION
SHEET NO. 8 OF 29
SCALE AS SHOWN

TO EAST VENTURA ← RR WEST

TO FILLMORE → RR EAST



CONSTRUCTION NOTES:

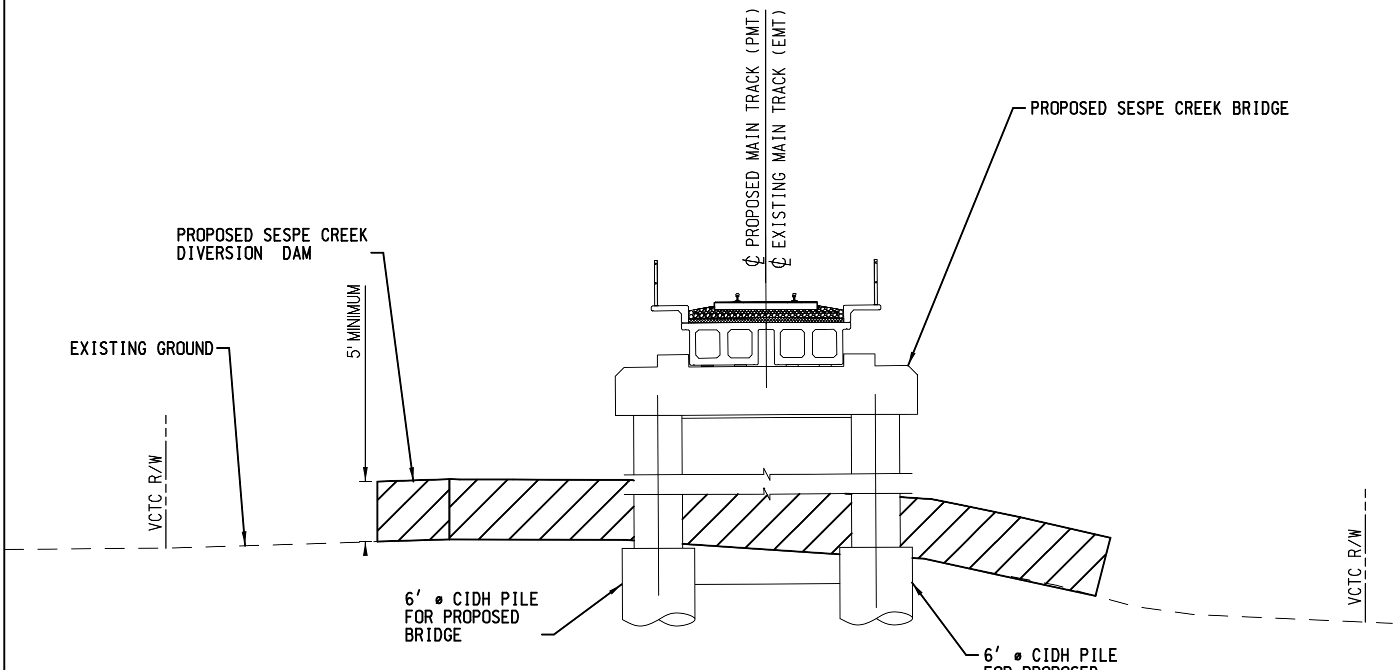
1. CONSTRUCTION OF COFFERDAM SYSTEM SHALL NOT BEGIN UNTIL ALL REQUIRED PERMITS HAVE BEEN OBTAINED - SEE SPECIFICATIONS.
2. STATIONS AND OFFSETS ARE APPROXIMATE AND SHALL BE VERIFIED IN FIELD BY ENGINEER.
3. INSTALL COFFERDAM SYSTEM AS NEEDED TO MAINTAIN POSITIVE FLOW IN THE CREEK AND DIVERT CREEK FLOW FROM THE WORK SITE ENCLOSED.
4. COFFERDAM SYSTEM SHALL BE SUBMITED TO ENGINEER FOR APPROVAL PRIOR TO CONSTRUCTION AND MUST MEET SUPPLIERS MINIMUM DIMENSIONS AND CRITERIA. PRODUCT DATA AND INSTALLATION METHODS MUST BE SUBMITTED FOR REVIEW.
5. UPON COMPLETION OF CREEK BED CONSTRUCTION OF ABUTMENT AND PIERS, COFFERDAM SYSTEM SHALL BE COMPLETELY REMOVED.
6. DIVERSION COFFERDAM SYSTEM SHALL NOT BE PLACED WHEN WATER SURFACE ELEVATION EXCEEDS EL. 431.
7. WORK SITE MUST BE CLEARED UNTIL A REMIDIAL ACTION PLAN IS DEVELOPED IF THERE IS ANTICIPATED POTENTIAL FOR THE WSE TO BE GREATER THAN EL. 431 OR IF WATER SEEPAGE OCCURS
8. COFFERDAM ENDPOINTS DOWNSTREAM AND UPSTREAM SHALL BE AS NEEDED TO PREVENT ANY FLOW TO THE WORKSITE BEING ENCLOSED INCLUDING BACKFLOW.
9. COFFERDAM DOWNSTREAM ENDPOINT SHALL TERMINATE BEYOND THE OLD TELEGRAPH RD BRIDGE AND EXTEND NO LESS THAN 130 FEET FROM THE RAILROAD BRIDGE STRUCTURE.
10. THIS PLAN IS FOR MINIMUM CRITERIA AND CONTRACTOR IS REQUIRED TO REVIEW THE REQUIREMENTS OF THE CONSTRUCTION GENERAL PERMIT FOR THIS PROJECT IF A WATER DIVERSION AND/OR COFFERDAM SYSTEM IS USED FOR CONSTRUCTION.

40' 0' 40' 80'

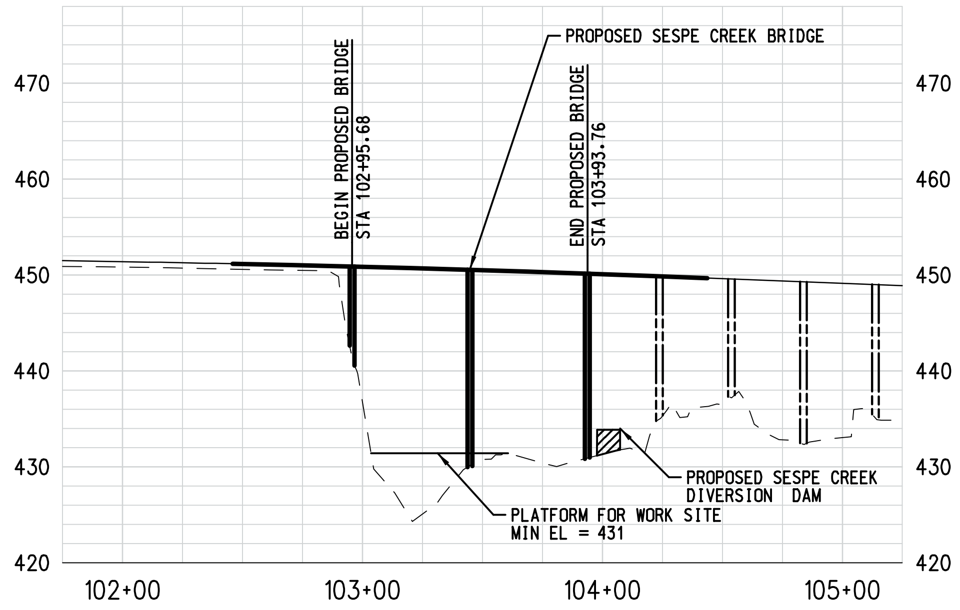
HORIZONTAL GRAPHIC SCALE

LEGEND:

━━━	PROPOSED TRACK
───	EXISTING TRACK
─ ─ ─	EXISTING VCTC R/W
▨	PROPOSED BARRIER
→	PROPOSED WATER DIVERSION FLOW



SECTION A-A
PMT 102+72 TO PMT 104+28
NOT TO SCALE



SECTION B-B
PROFILE PMT 101+75 TO PMT 105+25
NOT TO SCALE

PRELIMINARY 90%
NOT FOR CONSTRUCTION

REV. DATE BY SUB. APP.

DESIGNED BY A. SILVER
DRAWN BY J. ZIEGLER
CHECKED BY M. WHITE
APPROVED BY N. ORTEGA
DATE 10-31-2023

VENTURA COUNTY
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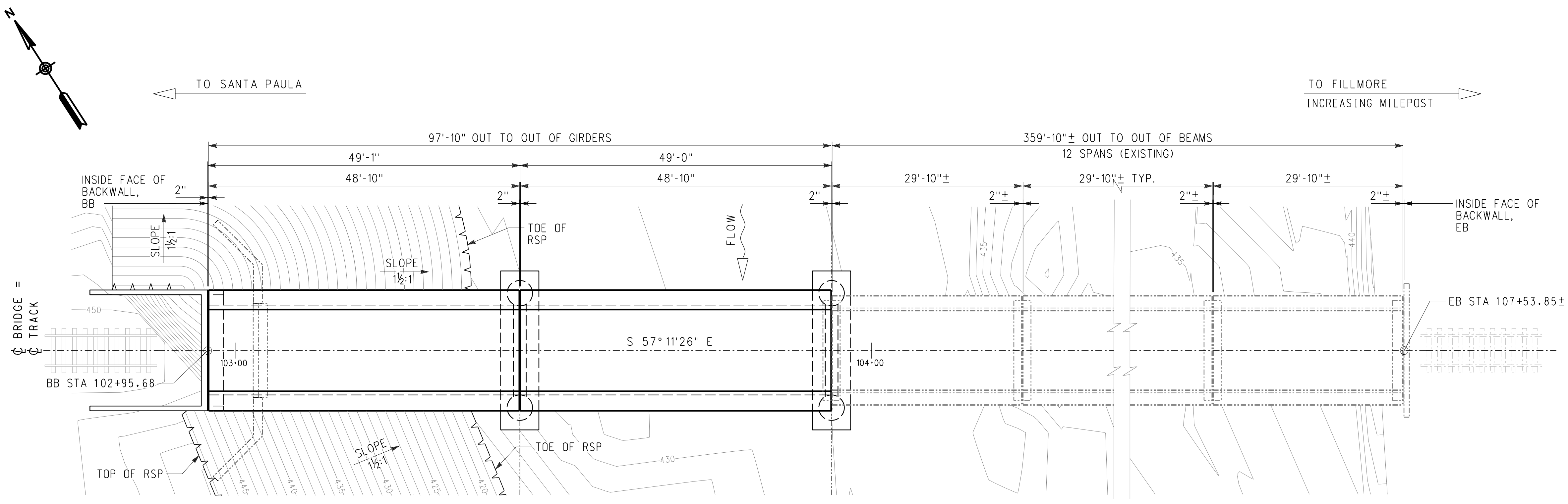
PHONE: (213)627-0044

SUBMITTED: ______________
JULINA CORONA, P.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

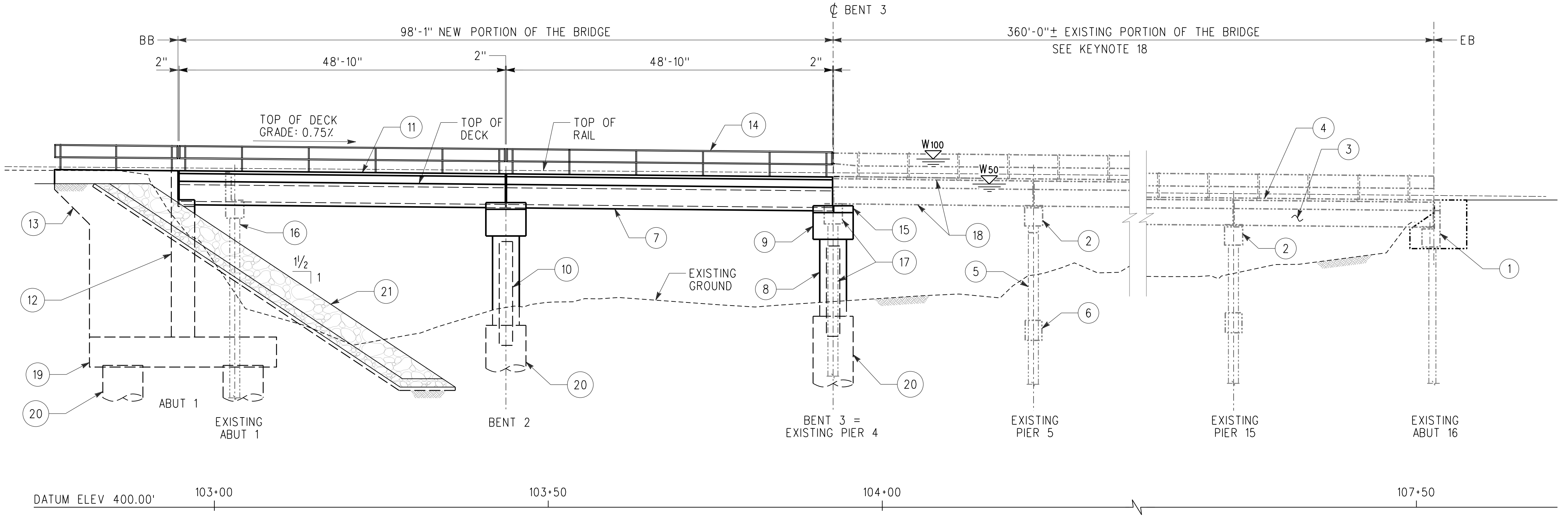
TEMPORARY CREEK DIVERSION PLAN

CONTRACT NO.
DRAWING NO. DIV-001
REVISION
SHEET NO. 9 OF 29
SCALE AS SHOWN



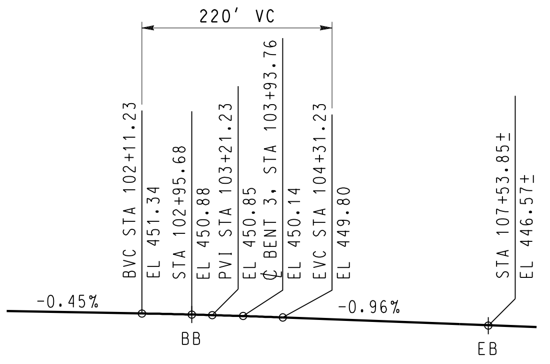
PLAN

SCALE: 1"=10'



ELEVATION

SCALE: 1"=10'



PROFILE GRADE - ℄ TRACK

NO SCALE

RAILROAD DATA

MILEPOST:	423.18
SUBDIVISION:	FILLMORE & WESTERN RAILWAY CO
DOT #:	NONE
CITY:	FILLMORE
COUNTY:	VENTURA
STATE:	CALIFORNIA
LATITUDE:	34° 24' 22.78" N
LONGITUDE:	118° 55' 55.13" W

KEYNOTES

①	EXISTING CONCRETE ABUTMENT
②	EXISTING CONCRETE BENT CAP
③	EXISTING CONCRETE GIRDER
④	EXISTING CONCRETE WALKWAY
⑤	EXISTING STEEL PILES WITH IN-FILL WALL
⑥	EXISTING CONCRETE COLLAR
⑦	PRECAST PRESTRESSED CONCRETE DOUBLE-BOX BRIDGE
⑧	CONCRETE COLUMN, 4'-0"Ø
⑨	CONCRETE BENT CAP
⑩	CONCRETE IN-FILL WALL
⑪	CONCRETE WALKWAY
⑫	CONCRETE ABUTMENT
⑬	CONCRETE WINGWALL
⑭	HANDRAIL
⑮	CATCHER BLOCK
⑯	EXISTING ABUTMENT, IN-FILL WALL & STEEL PILES TO BE REMOVED
⑰	EXISTING BENT, IN-FILL WALL & STEEL PILES TO BE REMOVED
⑱	EXIST CONC GIRDERS, RAILING AND WALKWAYS TO BE REMOVED AND RE-INSTALLED BETWEEN NEW BENT 3 AND EXIST PIERS 5
⑲	CONCRETE PILE CAP
⑳	CIDH CONCRETE PILES, 6'-0"Ø
㉑	ROCK SLOPE PROTECTION (RSP)

LEGEND

-------	INDICATES EXISTING STRUCTURE
―――	INDICATES NEW STRUCTURE
W100 ▽	INDICATES 100-YEAR FLOOD LEVEL = ELEV 452.18
W50 ▽	INDICATES 50-YEAR FLOOD LEVEL = ELEV 448.45

NOTES

1. FOR TYPICAL SECTIONS, SEE "GENERAL PLAN NO. 2" SHEET.
2. THE SCOPE OF WORK IS LIMITED TO REHABILITATION NEW SPANS 1 AND 2, INCLUDING THE NEW BENTS. EXISTING REMAINING STRUCTURE EVALUATION NOT CONSIDERED.

REV.	DATE		BY	SUB.	APP.

DESIGNED BY H. KAZEM
DRAWN BY G. ESTEPA
CHECKED BY H. YANG
APPROVED BY S. NAVEED
DATE 10-31-2023

PROGRESS SUBMITTAL 90% NOT FOR CONSTRUCTION

VENTURA COUNTY TRANSPORTATION COMMISSION

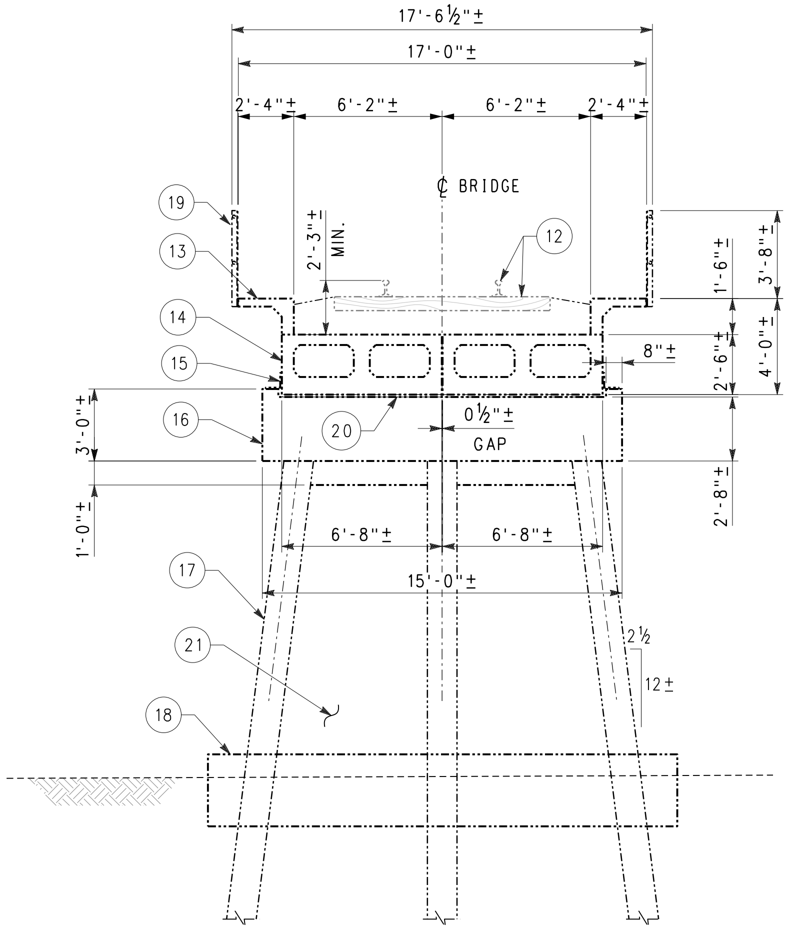
RAILPROS

SUBMITTED: ____________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

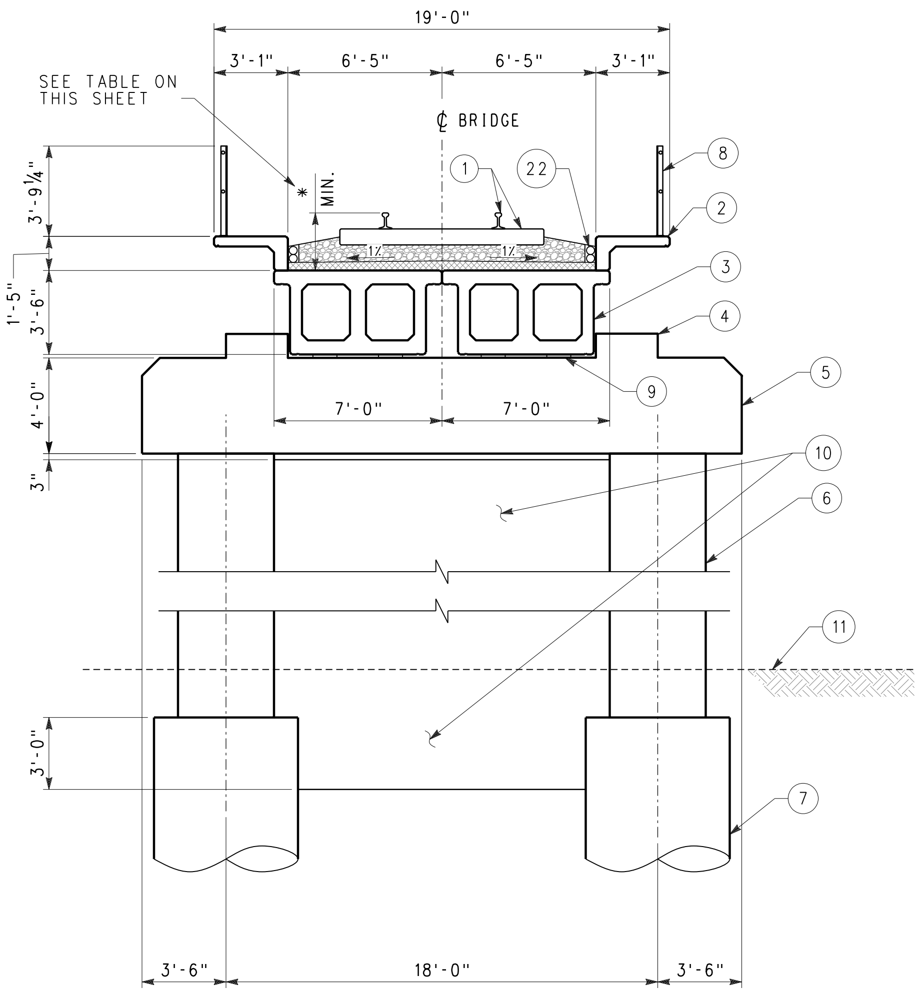
GENERAL PLAN NO. 1

CONTRACT NO.
DRAWING NO. S-001
REVISION
SHEET NO. 10 OF 29
SCALE AS SHOWN



TYPICAL SECTION - EXISTING BENTS

SCALE: 1/4" = 1'-0"



TYPICAL SECTION - NEW BENTS 2 & 3

SCALE: 1/4" = 1'-0"

* DEPTH TOP/RAIL TO TOP/DECK	
8"	RAIL & TIE PLATE
8"	CONCRETE TIE
8"	MINIMUM BALLAST
4"	MAXIMUM HMA AT CENTERLINE AND VARIES WITH 1% CROSS SLOPE
2'-4"	TOTAL (SEE NOTE 2)

KEYNOTES

① RAIL AND CONCRETE TIES
② PRECAST CONCRETE BALLAST CURB & SIDEWALK
③ PRECAST PRESTRESSED CONCRETE DOUBLE BOX GIRDER
④ CONCRETE SHEAR KEY
⑤ CAST-IN-PLACE CONCRETE BENT CAP
⑥ CONCRETE COLUMN, 4'-0"Ø
⑦ CIDH CONCRETE PILE, 6'-0"Ø
⑧ HANDRAIL
⑨ BEARING PAD
⑩ CONCRETE IN-FILL WALL
⑪ EXISTING GRADE
⑫ EXISTING RAIL AND TIES
⑬ EXISTING BALLAST CURB & SIDEWALK
⑭ EXISTING PRECAST PRESTRESSED CONCRETE DOUBLE BOX GIRDER
⑮ EXISTING STEEL ANGLE
⑯ EXISTING CONCRETE BENT CAP
⑰ EXISTING STEEL PILE
⑱ EXISTING CONCRETE BRACE
⑲ EXISTING HANDRAIL
⑳ EXISTING BEARING PAD, ¾"± THK
㉑ EXISTING CONCRETE IN-FILL WALL
㉒ 2~4" ID GALVANIZED METAL CONDUIT WITH CONDUIT BRACKET EACH SIDE OF BRIDGE STRUCTURE (TOTAL 4) PER SCRRA STANDRAD PLAN ES6001-05 & ES6002-14

NOTES

1. ALL EXISTING DIMENSIONS ARE APPROXIMATE AND SHALL BE FIELD MEASURED AND CONFIRMED BEFORE START OF WORK OR ORDERING MATERIALS.
2. DIMENSIONS LISTED ARE MINIMUM AND SHALL BE ADJUSTED AS NEEDED TO MAINTAIN THE EXISTING TRACK PROFILE.

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REV.	DATE		BY/SUB.	APP.

INFORMATION CONFIDENTIAL:

DESIGNED BY
H. KAZEM
DRAWN BY
G. ESTEPA
CHECKED BY
H. YANG
APPROVED BY
S. NAVEED
DATE
10-31-2023

PROGRESS SUBMITTAL
90%
NOT FOR CONSTRUCTION

VENTURA COUNTY
TRANSPORTATION
COMMISSION

RAILPROS

SUBMITTED: ____________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

GENERAL PLAN NO. 2

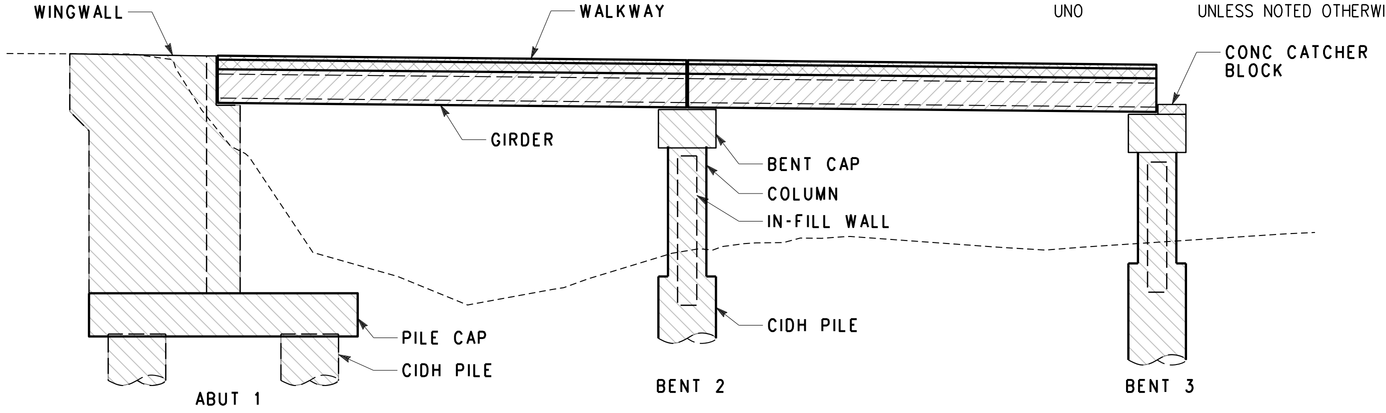
CONTRACT NO.
DRAWING NO. S-002
REVISION
SHEET NO. 11 OF 29
SCALE AS SHOWN

GENERAL NOTES:

DESIGN CRITERIA:	AMERICAN RAILWAY AND MAINTENANCE-OF-WAY ASSOCIATION (AREMA), 2023 EDITION SOUTHERN CALIFORNIA REGIONAL RAILROAD AUTHORITY (SCRRA) DESIGN CRITERIA FEB, 2022
LIVE LOAD:	COOPER E-80
PROJECT SPECIFICATIONS:	SCRRA STANDARD SPECIFICATIONS MAY 2022, AND CALTRANS STANDARD SPECIFICATIONS 2023 WHERE REFERRED TO BY SCRRA SPECIFICATIONS OR AS NOTED ON DRAWING
GEOTECHNICAL DATA:	GEOTECHNICAL REPORT RECONSTRUCT A PORTION OF THE SESPE CREEK OVERFLOW RAILROAD BRIDGE CITY OF FILLMORE, CALIFORNIA, PROJECT NO. 2023-010 DATED: OCTOBER 13, 2023, PREPARED BY: DIAZ & YOURMAN & ASSOCIATES (1616 EAST 17TH STREET, SANTA ANA, CA 92705-8509, (714) 245-2920)
LATERAL EARTH PRESSURE:	UNIT WEIGHT OF EARTH FILLING MATERIALS, γs = 120 PCF EQUIVALENT AT-REST PRESSURE COEFFICIENT, k0 = 0.47 EQUIVALENT ACTIVE PRESSURE COEFFICIENT, ka = 0.31 EQUIVALENT PASSIVE PRESSURE COEFFICIENT, kp = 3.25
SEISMIC LATERAL DATA:	AREMA LEVEL 1 ∆kae, 95YR (SERVICEABILITY) = 0.07 AREMA LEVEL 2 ∆kae, 475YR (ULTIMATE) = 0.15 AREMA LEVEL 3 ∆kae, 2475YR (SURVIVABILITY) = 0.35 CALTRANS ∆kae, 975YR = 0.28
PGA:	AREMA LEVEL 1, 95YR (SERVICEABILITY) = 0.19G AREMA LEVEL 2, 475YR (ULTIMATE) = 0.44G AREMA LEVEL 3, 2475YR (SURVIVABILITY) = 0.82G CALTRANS, 975YR = 0.72G

CONCRETE STRENGTH AND TYPE LIMITS

REINFORCED CONCRETE:	f'c = 4.0 KSI @ 28 DAYS UNLESS NOTED OTHERWISE
REINFORCING BARS:	fy = 60 KSI, ASTM A706 GRADE 60
REINFORCING BAR COUPLERS:	REINFORCING BAR MECHANICAL COUPLERS SHALL BE "SERVICE SPLICE" SELECTED FROM CALTRANS AUTHORIZED MATERIAL LIST AT "HTTPS://DOT.CA.GOV/PROGRAMS/ENGINEERING-SERVICES/AUTHORIZED-MATERIALS-LISTS"



LEGEND:

▨	STRUCTURAL PRECAST CONCRETE, (f'c = 4 KSI AT 28 DAYS)
▨	PRESTRESSED CONCRETE, SEE "GIRDER DETAILS NO. 2" SHEET
▨	STRUCTURAL CONCRETE BRIDGE, (f'c = 4 KSI AT 28 DAYS)

ABBREVIATIONS:

AREMA	AMERICAN RAILWAY ENGNIEERING AND MAINTENANCE OF WAY ASSOCIATION
ASTM	AMERICAN SOCIETY FOR TESTING AND MATERIALS
BB	BEGINNING OF BRIDGE
BC	BEGINNING OF CURVE
BOT	BOTTOM
BRG	BEARING
BVC	BEGINNING OF VERTICAL CURVE
CALTRANS	CALIFORNIA DEPARTMENT OF TRANSPORTATION
CIDH	CAST-IN-DRILLED HOLE
CIP	CAST-IN-PLACE
CLR	CLEAR, CLEARANCE
CONC	CONCRETE
EA	EACH
EB	END OF BRIDGE
EC	END OF CURVE
ELEV, EL	ELEVATION
EMBED	EMBEDMENT
EVC	END OF VERTICAL CURVE
EXIST	EXISTING
EXP JT	EXPANSION JOINT
FG	FINISHED GRADE
FT	FOOT, FEET
HMA	HOT MIXED ASPHALT
KIPS	1000 POUNDS-FORCE
KSI	1000 POUNDS-FORCE PER SQUARE INCH
LOL	LAYOUT LINE
MAX	MAXIMUM
MIN	MINIMUM
MP	MILEPOST
NA, N/A	NOT APPLICABLE
NO.	NUMBER
PC	PRECAST
PCF	POUND-FORCE PER CUBIC FOOT
PCI	POUND-FORCE PER CUBIC INCH
PS	PRESTRESSED
PVI	POINT OF VERTICAL INTERSECTION
REINF	REINFORCING
RSP	ROCK SLOPE PROTECTION
R/W, ROW	RIGHT OF WAY
RW	RETAINING WALL
RWLOL	RETAINING WALL LAYOUT LINE
SCRRA	SOUTHERN CALIFORNIA REGIONAL RAILROAD AUTHORITY
SSPWC	STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION
SYM	SYMMETRICAL
T/R, TOR	TOP OF RAIL
TOC	TOP OF CONCRETE
TOT	TOTAL
TYP	TYPICAL
UNO	UNLESS NOTED OTHERWISE

INDEX OF DRAWINGS:

CONSTRUCTION NOTE:

1. CONTRACTOR SHALL FIELD VERIFY AND CALCULATE THE SEAT ELEVATIONS FOR THE NEW ABUTMENT AND BENTS TO MAINTAIN THE TRACK PROFILE BEFORE FABRICATION OR ORDERING ANY MATERIALS.

REV.	DATE		BY/SUB.	APP.

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DESIGNED BY H. KAZEM
DRAWN BY G. ESTEPA
CHECKED BY H. YANG
APPROVED BY S. NAVEED
DATE 10-31-2023

PROGRESS SUBMITTAL ★ 90% ★ NOT FOR CONSTRUCTION

VENTURA COUNTY
TRANSPORTATION
COMMISSION

RAILPROS

SUBMITTED: ____________________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

GENERAL NOTES AND INDEX OF DRAWINGS

CONTRACT NO.
DRAWING NO. S-003
REVISION
SHEET NO. 12 OF 29
SCALE NO SCALE

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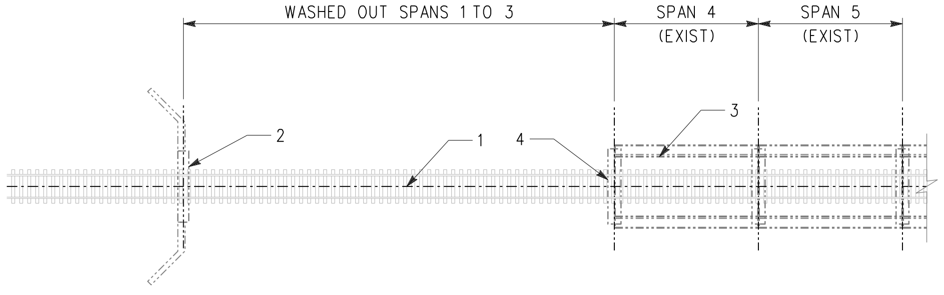

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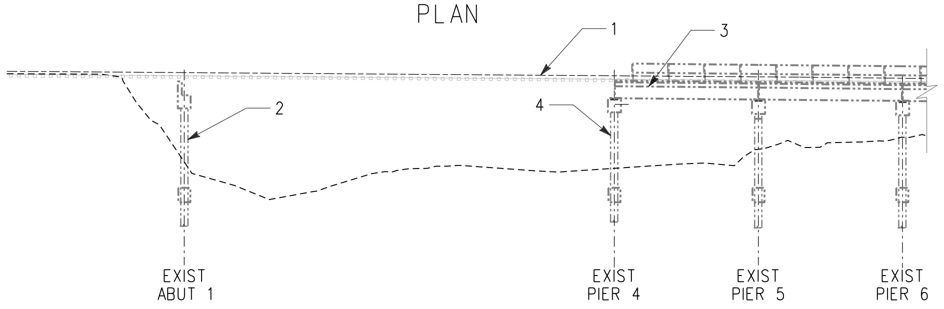
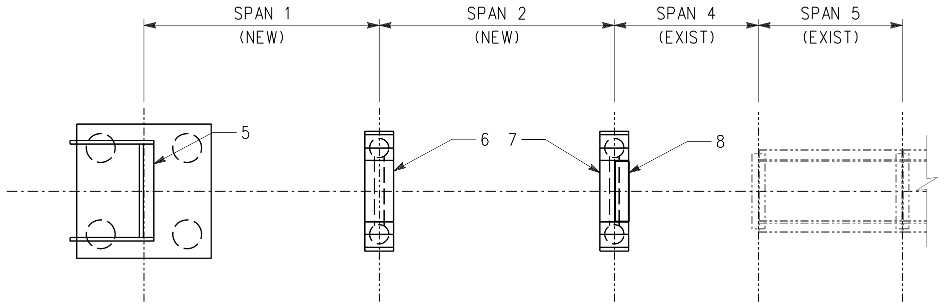


ELEVATION

CONSTRUCTION STAGE 1

NOTES - STAGE 1:

1. REMOVE EXISTING TRACKS & TIES. CUT RAILS FROM 50 FT BEFORE ABUTMENT 1 TO MIDPOINT OF EXISTING SPAN BETWEEN EXISTING PIERS 5 AND 6
2. DEMOLISH EXISTING LEFT OVER ABUTMENT 1 AND REMOVE EXISTING STEEL PILES
3. REMOVE EXISTING CONCRETE GIRDERS & HANDRAILS IN SPAN BETWEEN EXISTING PIERS 4 AND 5. EXISTING GIRDERS TO BE REPAIRED (AS NEEDED)/REINSTALLED AT A LATER CONSTRUCTION STAGE
4. DEMOLISH EXISTING PIER 4 AND REMOVE EXISTING STEEL PILES, IN-FILL WALL & CONCRETE BRACE



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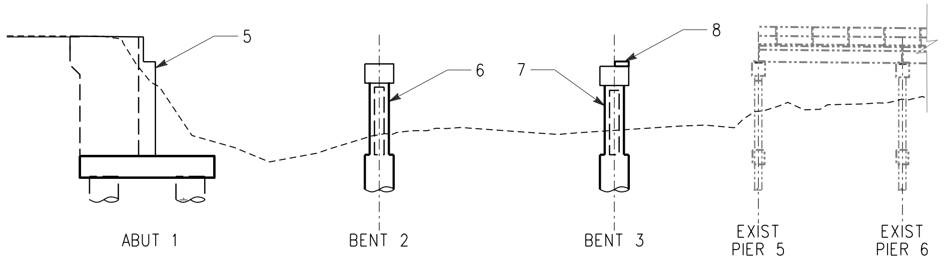
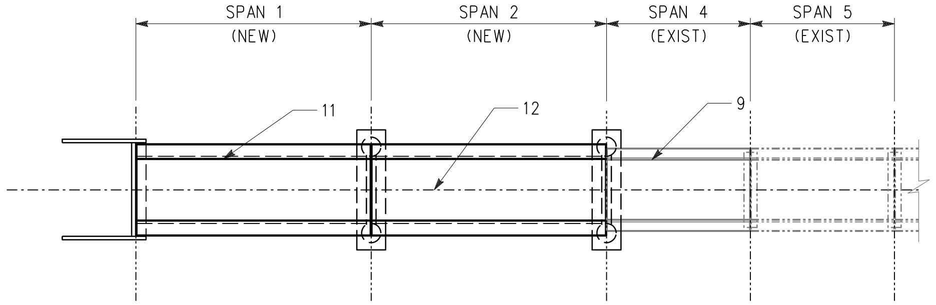


ELEVATION

CONSTRUCTION STAGE 2

NOTES - STAGE 2:

5. BUILD ABUTMENT 1 AND WINGWALLS
6. BUILD BENT 2 AND INFILL WALL
7. BUILD BENT 3 AND INFILL WALL
8. INSTALL PRECAST CONCRETE CATCHER BLOCK ON BENT 3



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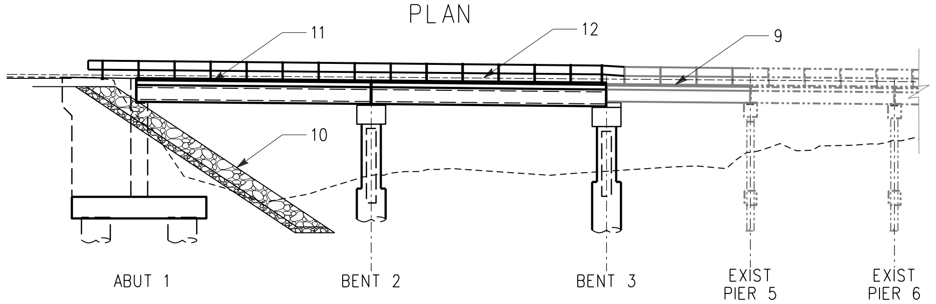

ELEVATION

CONSTRUCTION STAGE 3 - FINAL

NOTES - STAGE 3, FINAL:

9. RE-INSTALL SPAN 4 SUPERSTRUCTURE INCLUDING GIRDERS, WALKWAYS & HANDRAILS
10. BUILD ROCK SLOPE PROTECTION FOR ABUTMENT 1
11. INSTALL NEW SUPERSTRUCTURE ON SPANS 1 AND 2 INCLUDING WALKWAYS AND HANDRAILS
12. INSTALL STEEL PLATES, GIRDER RESTRAINERS, HMA, BALLAST, TRACKS & TIES

REV.	DATE		BY	SUB.	APP.

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VENTURA COUNTY
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RAILPROS

SUBMITTED: ____________________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

STAGE CONSTRUCTION PLAN

CONTRACT NO.
DRAWING NO. S-004
REVISION
SHEET NO. 13 OF 29
SCALE NO SCALE

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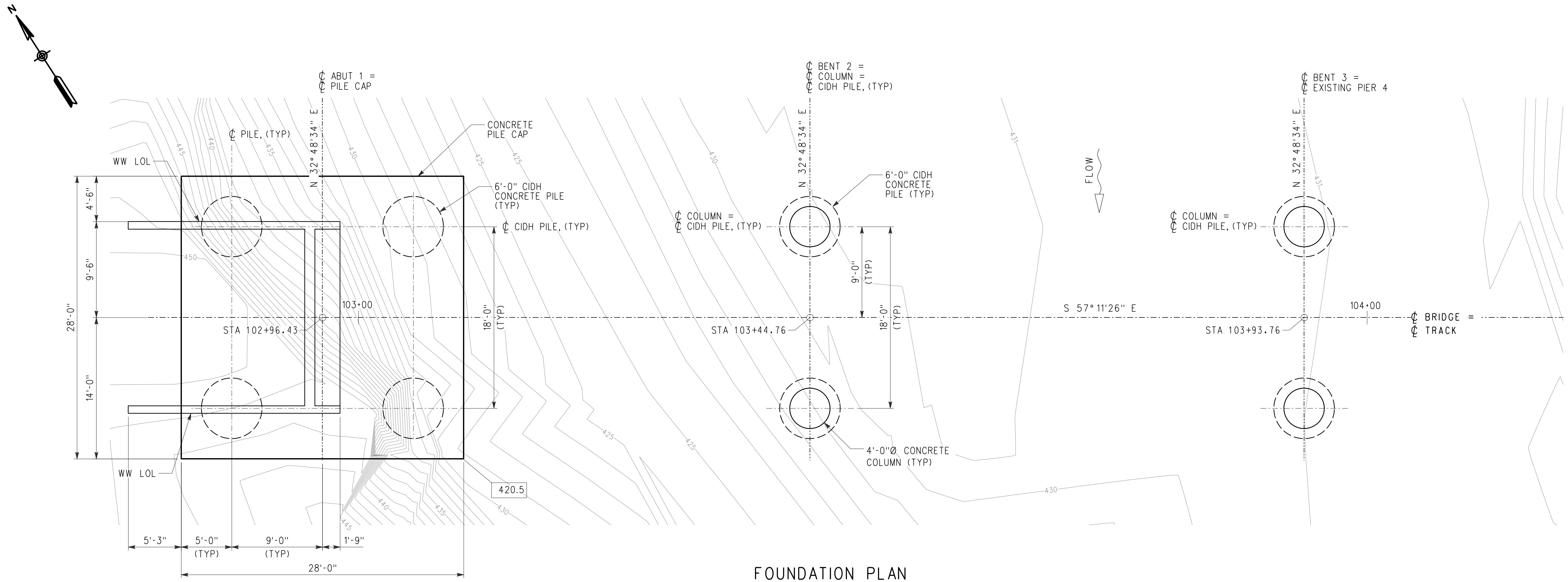

FOUNDATION PLAN

SCALE: 3/16" = 1'-0"

HYDRAULOGICAL DATA

50 YEAR FLOOD LEVEL = 448.45
100 YEAR FLOOD LEVEL = 452.18

PILE DATA TABLE

LOCATION	PILE TYPE	NOMINAL RESISTANCE (kips)		PILE CUT-OFF ELEVATION (ft)	DESIGN TIP ELEVATION (ft)	SPECIFIED TIP ELEVATION (ft)	NOMINAL DRIVING RESISTANCE (kips)
		COMPRESSION	TENSION				
ABUT 1	72"Ø CIDH	716	0	420.75	(a) 322.25 (c) 378.25 (d) 355.75	322.25	N/A
BENT 2	72"Ø CIDH	778	304	425.00	(a) 350.0 (b) 392.0 (c) 364.0 (d) 355.0	350.00	N/A
BENT 3	72"Ø CIDH	778	304	429.00	(a) 354.0 (b) 396.0 (c) 368.0 (d) 359.0	354.00	N/A

NOTES:
1. DESIGN TIP ELEVATIONS ARE CONTROLLED BY: (a) COMPRESSION, (b) TENSION, (c) SETTLEMENT, AND (d) LATERAL LOAD.
2. THE SPECIFIED TIP ELEVATION FOR DRIVEN PILES MUST NOT BE RAISED ABOVE THE DESIGN TIP ELEVATIONS FOR SETTLEMENT AND LATERAL LOAD. THE SPECIFIED TIP ELEVATION FOR CIDH PILES MUST NOT BE RAISED.

BENCH MARK

POINT NUMBER	NORTHING	EASTING	ELEV (FT)	DESCRIPTION
500	1971511.827	6280526.913	457.84′	CUT X CONC ON WB SIDE OF BRIDGE 27′ EAST OF WEST EXP JT
501	1971316.983	62808728.833	458.67′	CUT X CONC ON WB SIDE OF BRIDGE 94′ EAST OF WEST EXP JT
502	1971336.612	6280917.852	446.28′	3.5″ USC&GS BRASS BM DISK STAMPED ″S12188, 1971″ ON SE ABUTMENT, CONC WALKWAY
503	1971201.537	6281085.270	458.32′	MAGNAIL & SPIKE IN GROUND 5.15′ FROM CONC CURBING AT GATE TO RR ABUTMENT ON SESIDE OF RR TRACK

SURVEY CONTROL: THE BASIC HORIZONTAL CONTROL IS THE NORTH AMERICAN DATUM OF 1983, 2011 ADJUSTMENT (NAD83-2011), MUTI-YEAR CORS SOLUTION 2 (MYSC2) ESTABLISHED BY USING THE SMARTNET SYSTEM OF CONTINUOUSLY OPERATING REFERENCE STATIONS (CORS).

COORDINATE ARE IN CALIFORNIA STATE PLAN COORDINATE SYSTEM, ZONE 5, EPOCH 2023.25, US SURVEY FT.

VERTICAL SURVEY CONTROL VALUES HEREON ARE BASED UPON THE NORTH AMERICAN VERICAL DATUM OF 1988, GNSS-DERIVED BY FAST STATIC SURVEY METHODS USING GEIOD18 PER CALIFORNIA PUBLIC RESOURCES CODE 8890, DEFINED AS CALIFORNIA ORTHOMETRIC HEIGHTS OF 1988 (CH88).

ALL POSITION ARE CALCULATED PER A FULLY CONSTRAINED LEAST SQUARES ADJUSTMENT USING STARNET V11 LEAST SQUARES ADJUSTMENT SOFTWARE.

LEGEND

—	NEW STRUCTURE
◌	72" Ø CIDH PILE
XXX.X	BOTTOM OF PILE CAP ELEVATION
◁	DIRECTION OF FLOW

NOTES

1. ONLY NEW STRUCTURE SHOWN FOR CLARITY. EXISTING STRUCTURE PORTION THAT REMAINS IN PLACE IS NOT SHOWN. SEE GENERAL PLAN AND STAGE CONSTRUCTION PLAN FOR DETAILS.

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CHECKED BY H. YANG
APPROVED BY S. NAVEED
DATE 10-31-2023

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VENTURA COUNTY
TRANSPORTATION
COMMISSION

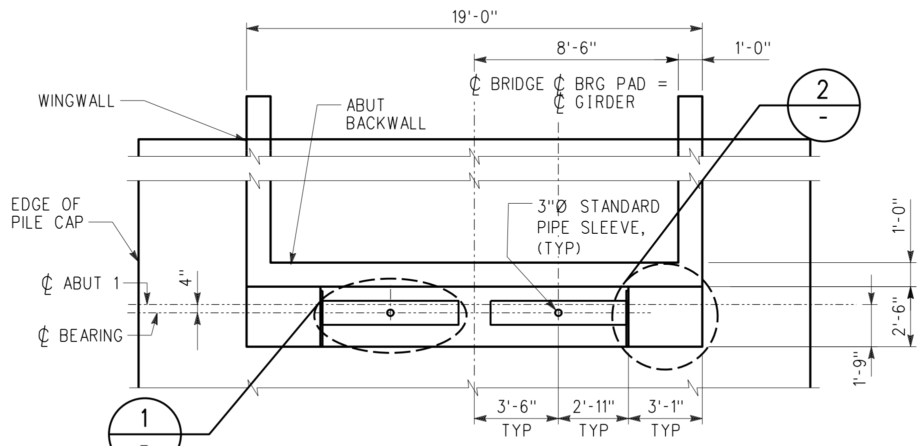
RAILPROS

SUBMITTED: ____________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

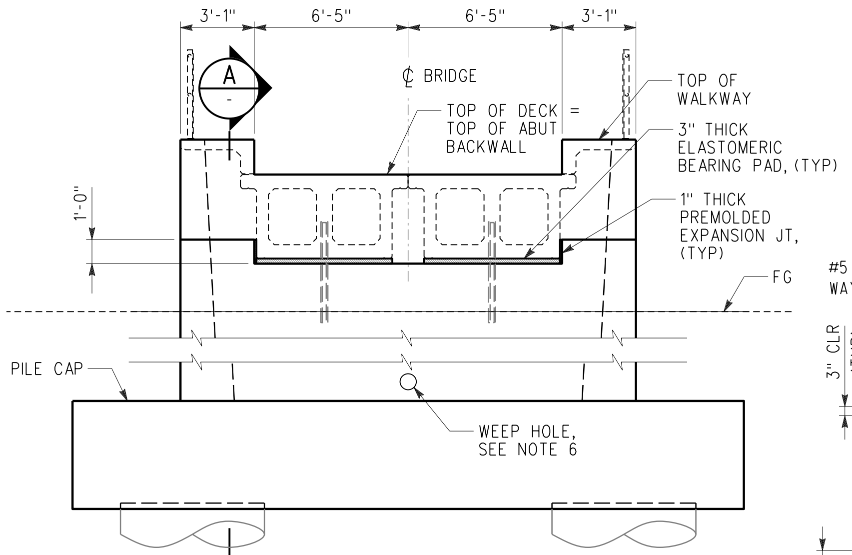
SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

FOUNDATION PLAN

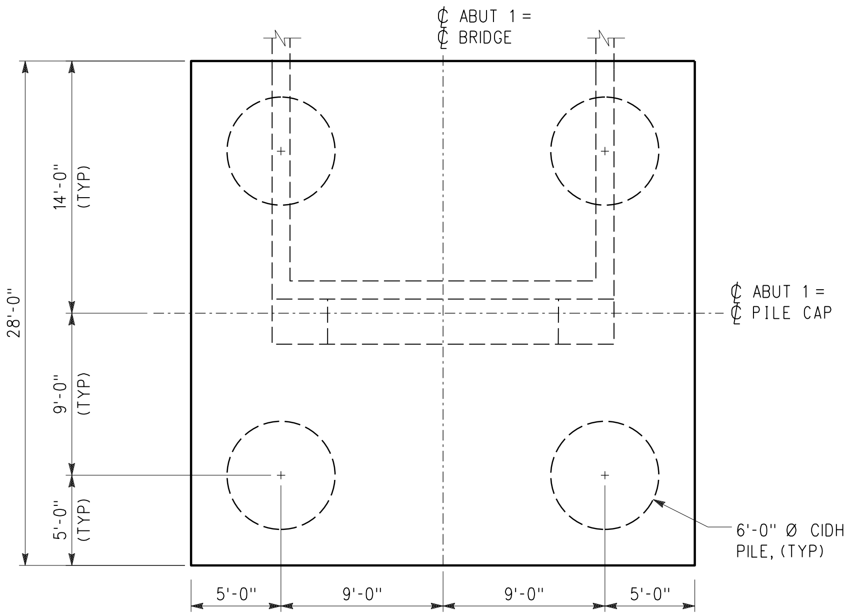
CONTRACT NO.
DRAWING NO. S-005
REVISION
SHEET NO. 14 OF 29
SCALE AS SHOWN



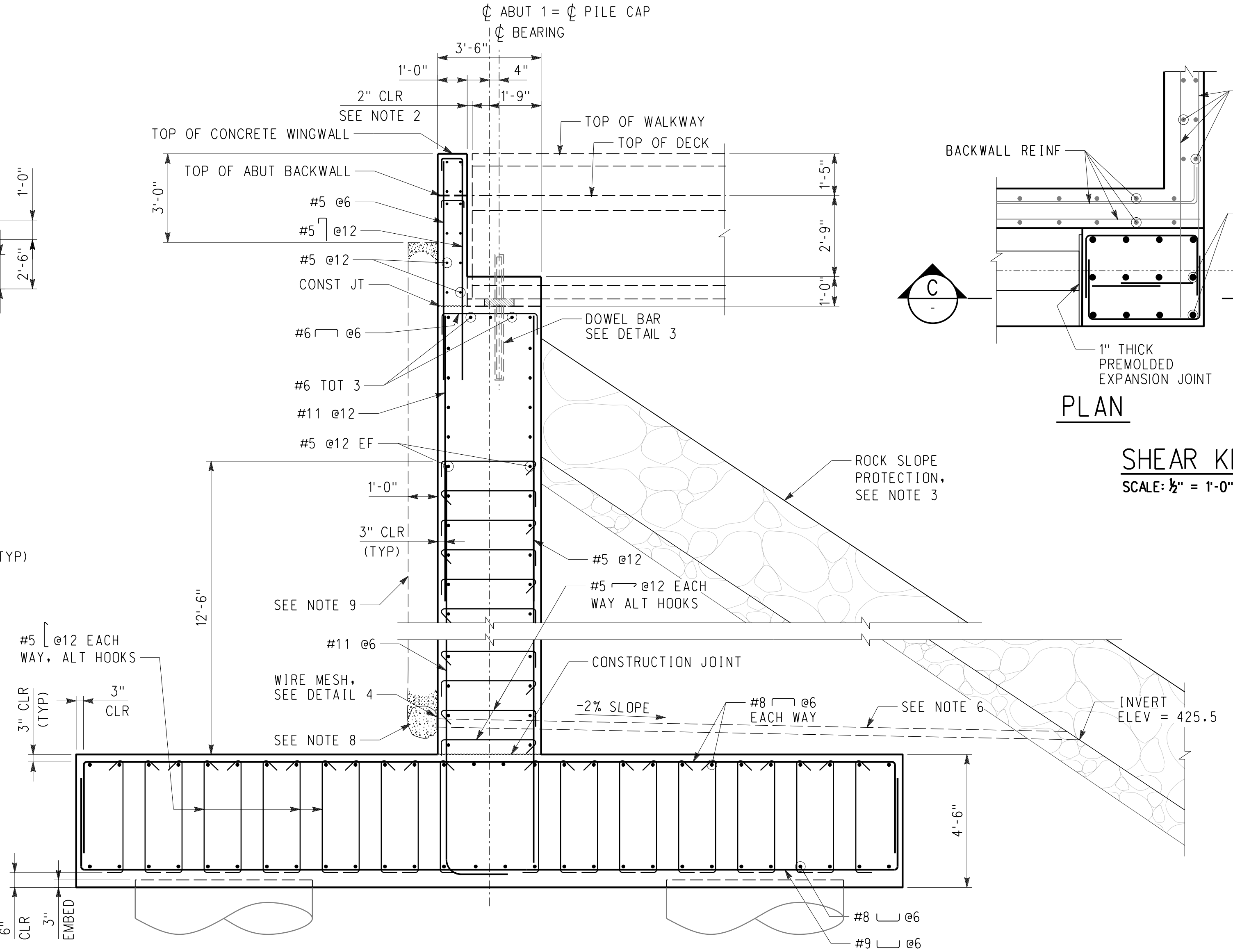
PLAN
SCALE: ¼" = 1'-0"



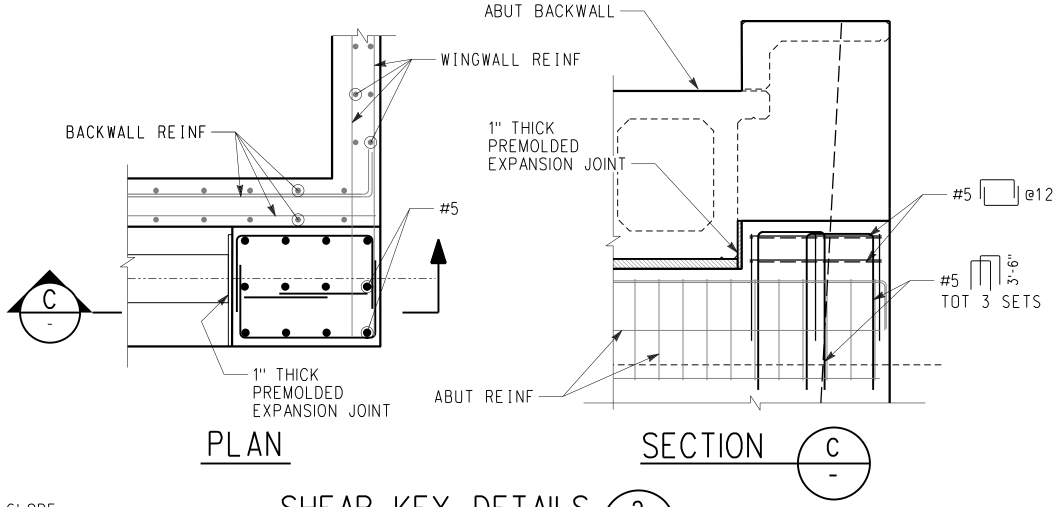
ELEVATION
SCALE: ¼" = 1'-0"



PILE LAYOUT
SCALE: 3⁄16" = 1'-0"



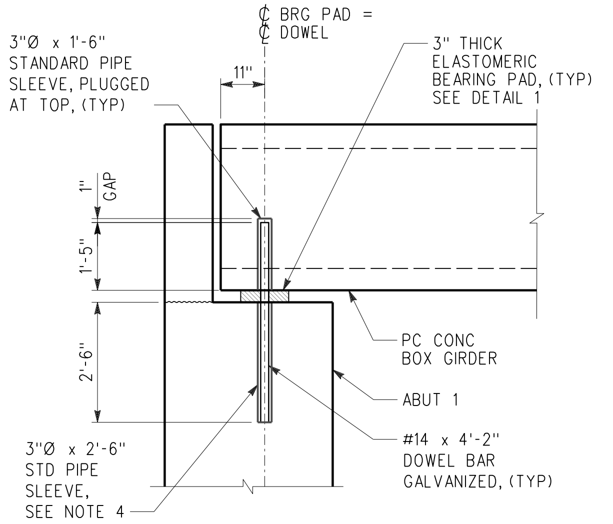
SECTION A
SCALE: ⅜" = 1'-0"



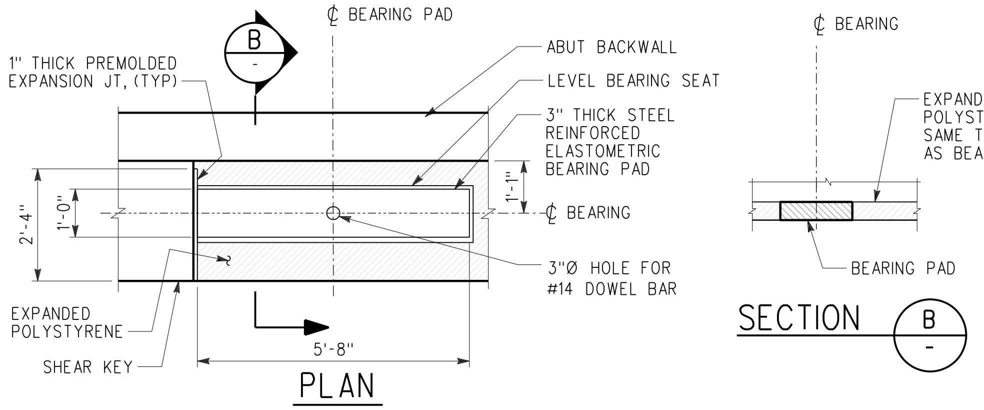
PLAN

SECTION C

SHEAR KEY DETAILS 2
SCALE: ½" = 1'-0"



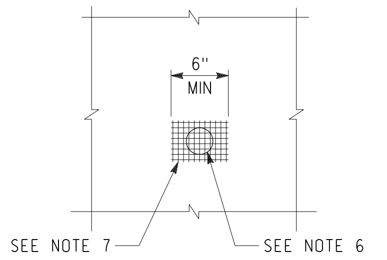
DOWEL BAR DETAIL 3
SCALE: ½" = 1'-0"



PLAN

SECTION B

BEARING PAD DETAIL 1
SCALE: ½" = 1'-0"



WIRE MESH DETAIL 4
NO SCALE

NOTES:

1. SLOPE ABUT SEAT TO DRAIN EXCEPT AS SHOWN IN BEARING PAD DETAILS
2. FOR HMA OVER THE JOINT DETAILS, SEE SCRRA STANDARD PLAN ES6001-03
3. FOR RSP DETAILS, SEE "ROCK SLOPE PROTECTION DETAILS" SHEET
4. LOWER PIPE SLEEVE TO BE FILLED WITH NON-SHRINK GROUT AFTER INSTALLATION OF #14 DOWEL BAR
5. FOR ABUTMENT PILE DETAILS, SEE "ABUTMENT DETAILS NO.2" SHEET
6. 4"Ø DRAINS AT CENTER OF ABUTMENT.
7. 6" SQUARE ALUMINUM OR GALVANIZED STEEL WIRE 1/4" MESH HARDWARE CLOTH, MINIMUM WIRE DIAMETER 0.025". ANCHOR FIRMLY TO BACKFACE.
8. ONE CUBIC FOOT PERVIOUS BACKFILL MATERIAL IN A NONWOVEN FILTER FABRIC, SECURELY TIED.
9. PERVIOUS BACKFILL MATERIAL CONTINUOUS BEHIND ABUTMENT.

REV.	DATE	BY / SUB.	APP.

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DRAWN BY G. ESTEPA
CHECKED BY H. YANG
APPROVED BY S. NAVEED
DATE 10-31-2023

PROGRESS SUBMITTAL 90% NOT FOR CONSTRUCTION

VENTURA COUNTY
TRANSPORTATION
COMMISSION

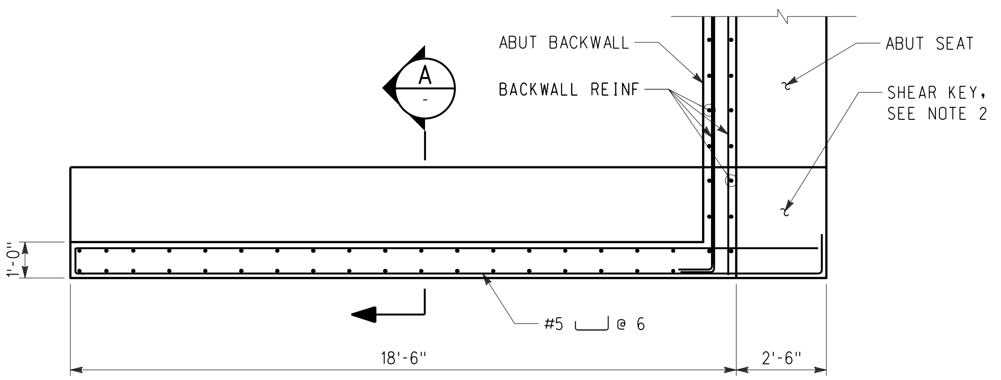
RAILPROS

SUBMITTED: DANIELLE LIBRING, P.E., T.E. PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

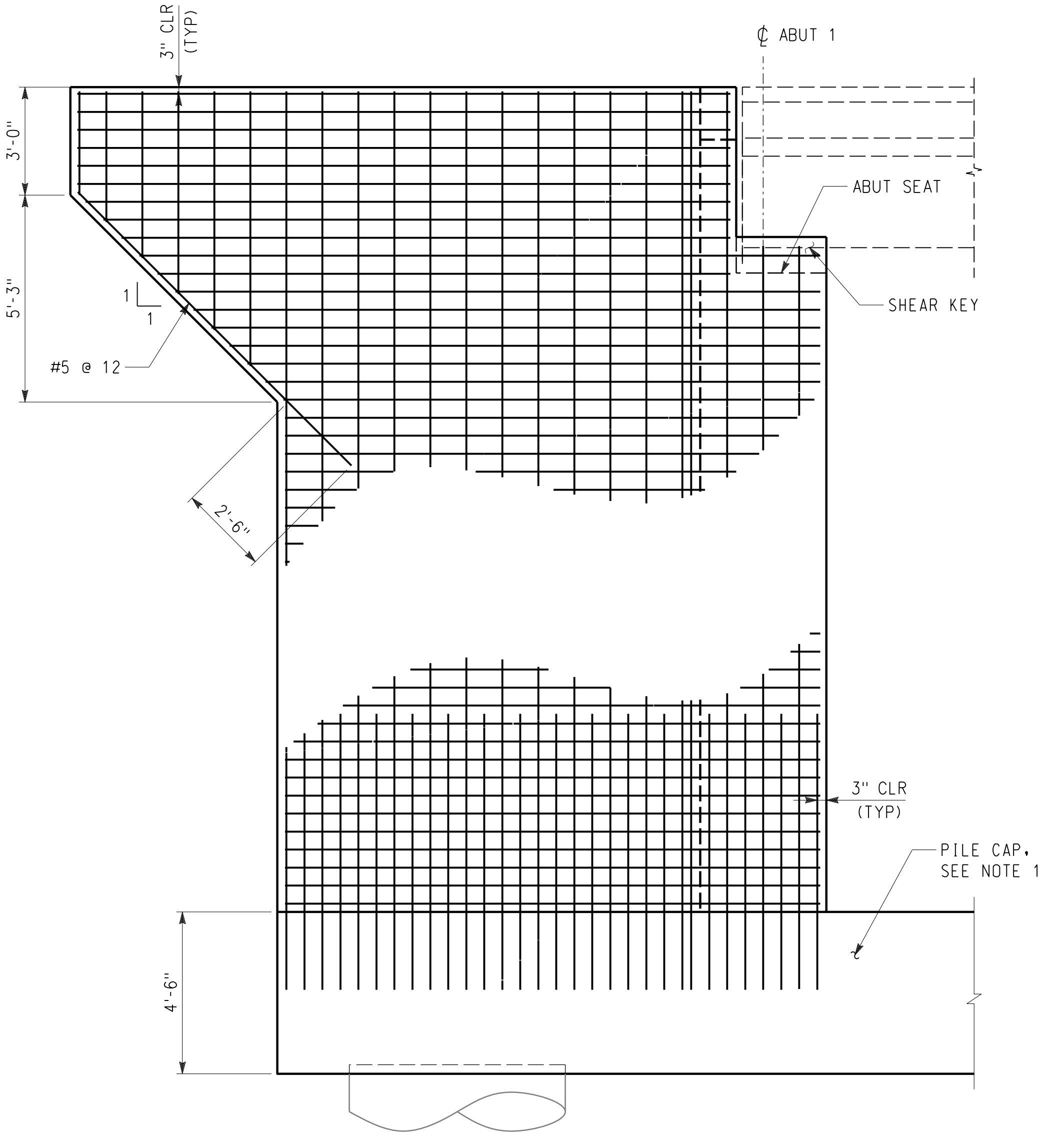
ABUTMENT DETAILS NO.1

CONTRACT NO.
DRAWING NO. S-006
REVISION
SHEET NO. 15 OF 29
SCALE AS SHOWN



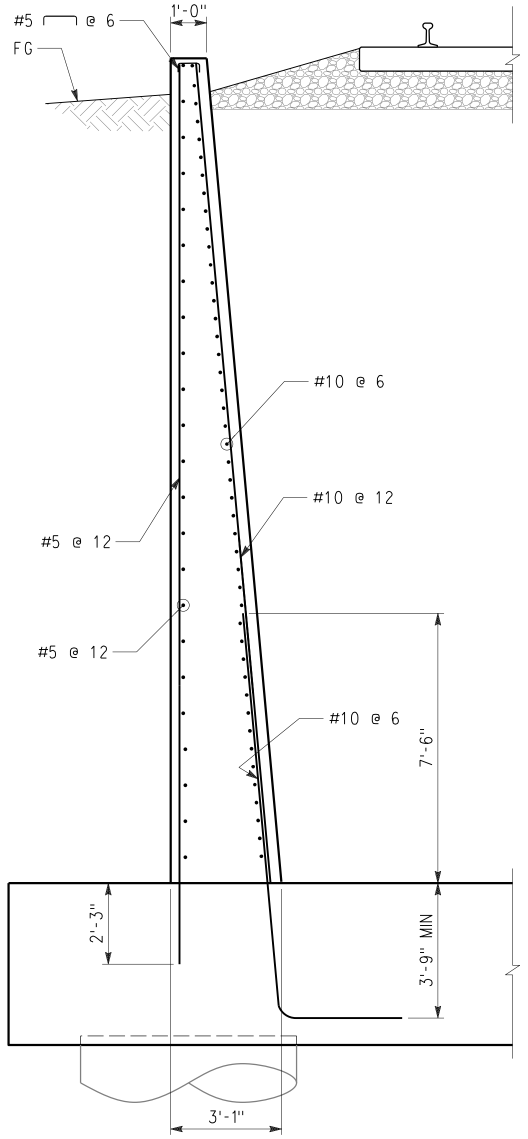
PLAN – ABUTMENT WINGWALL

SCALE: ⅜" = 1'-0"



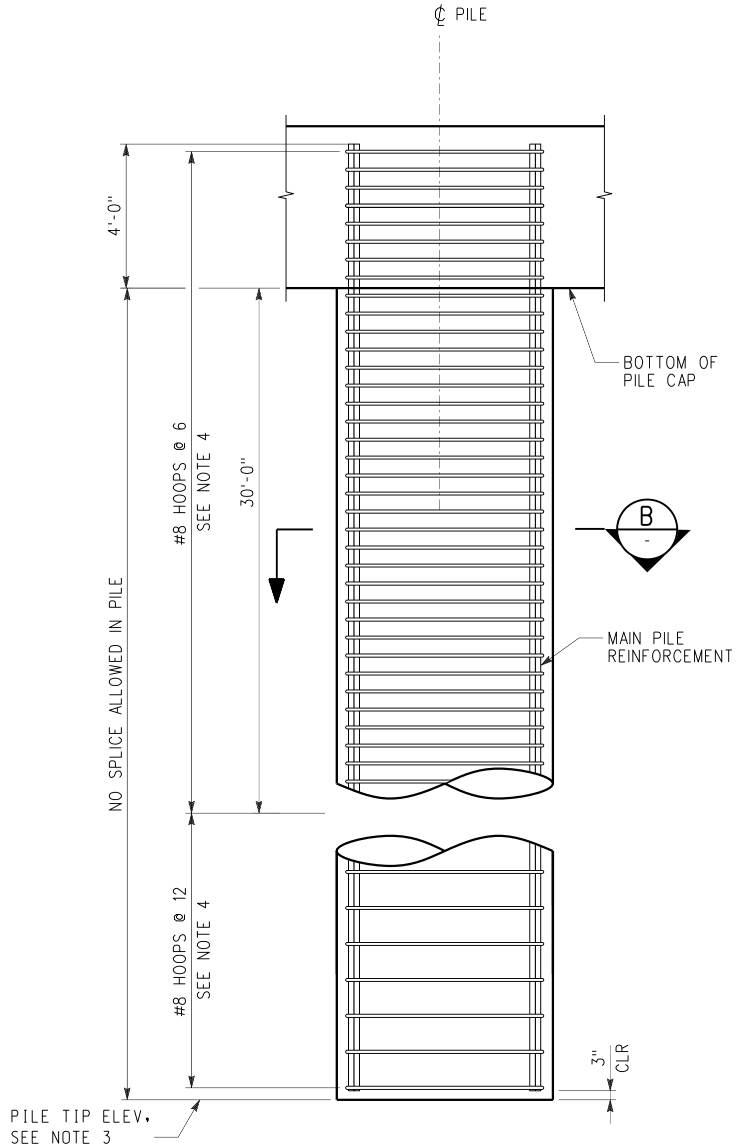
ELEVATION – ABUTMENT WINGWALL

SCALE: ⅜" = 1'-0"



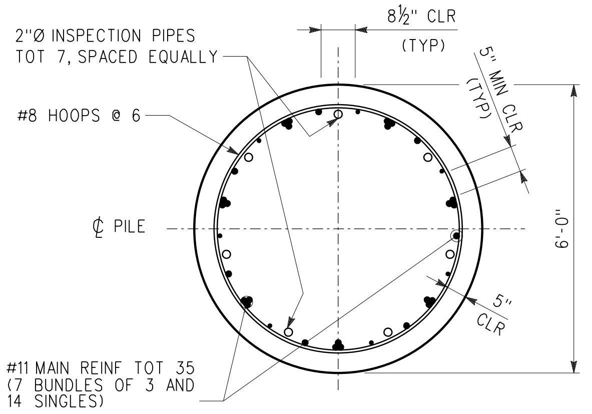
SECTION A/-

SCALE: ⅜" = 1'-0"



ABUTMENT PILE ELEVATION

SCALE: ⅜" = 1'-0"



SECTION B/-

SCALE: ½" = 1'-0"

NOTES:

1. FOR PILE CAP DIMENSIONS AND REINFORCEMENT, SEE "ABUTMENT DETAILS NO. 1"
2. FOR SHEAR KEY REINFORCEMENT, SEE "ABUTMENT DETAILS NO. 1"
3. FOR PILE TIP ELEVATION SEE "FOUNDATION PLAN" SHEET
4. ALL HOOPS ARE ULTIMATE BUTT SPLICES

10/26/2023 9:38:47 AM USER = gerry.estepa
Z:\Engineering\VCTC\Sespe Creek Bridge Overflow\900 CADD\950 Drawings\S-007_Abut Details 2.sht
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REV.	DATE		BY/SUB.	APP.

DESIGNED BY H. KAZEM
DRAWN BY G. ESTEPA
CHECKED BY H. YANG
APPROVED BY S. NAVEED
DATE 10-31-2023

PROGRESS SUBMITTAL 90% NOT FOR CONSTRUCTION

VENTURA COUNTY TRANSPORTATION COMMISSION

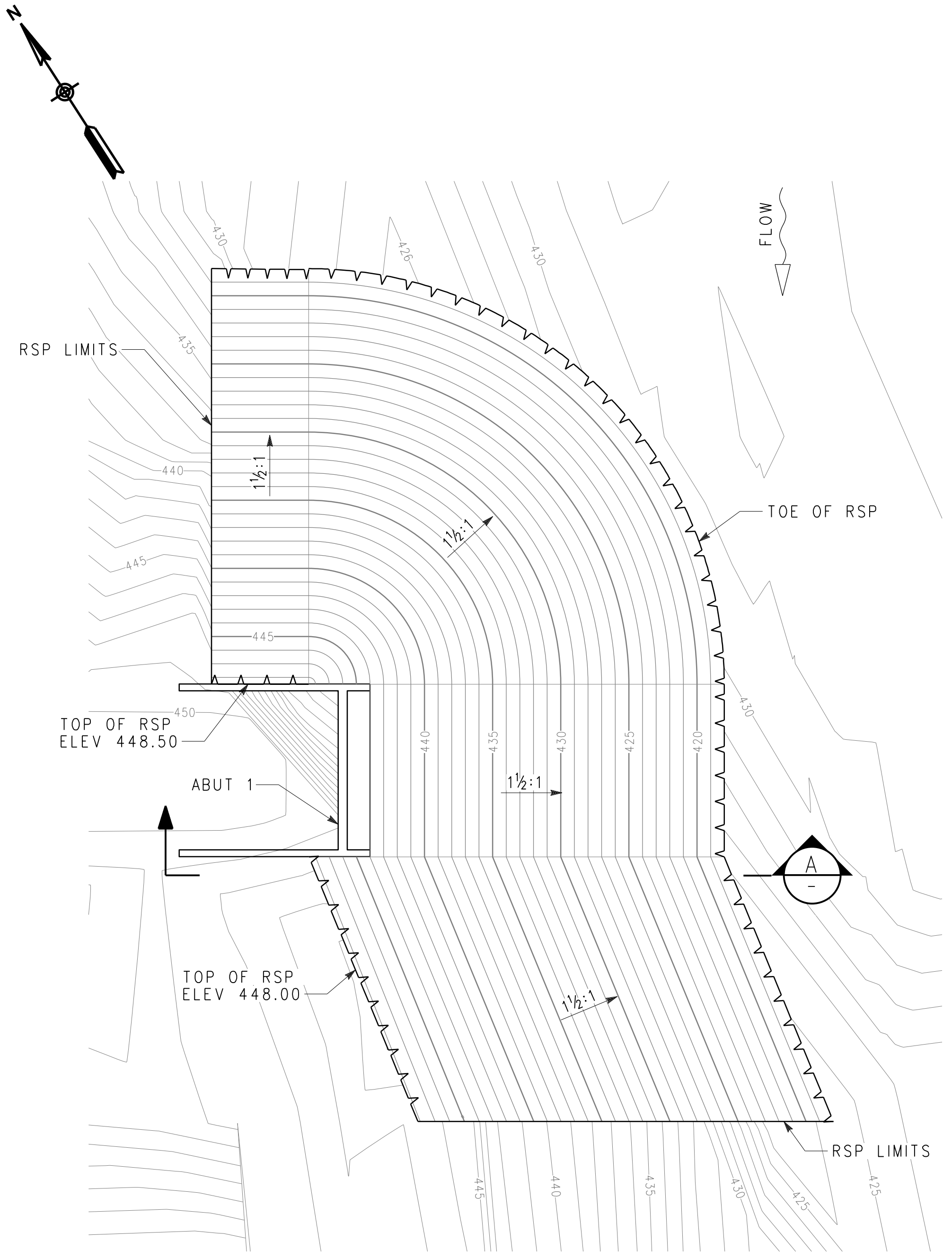
RAILPROS

SUBMITTED: ______
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

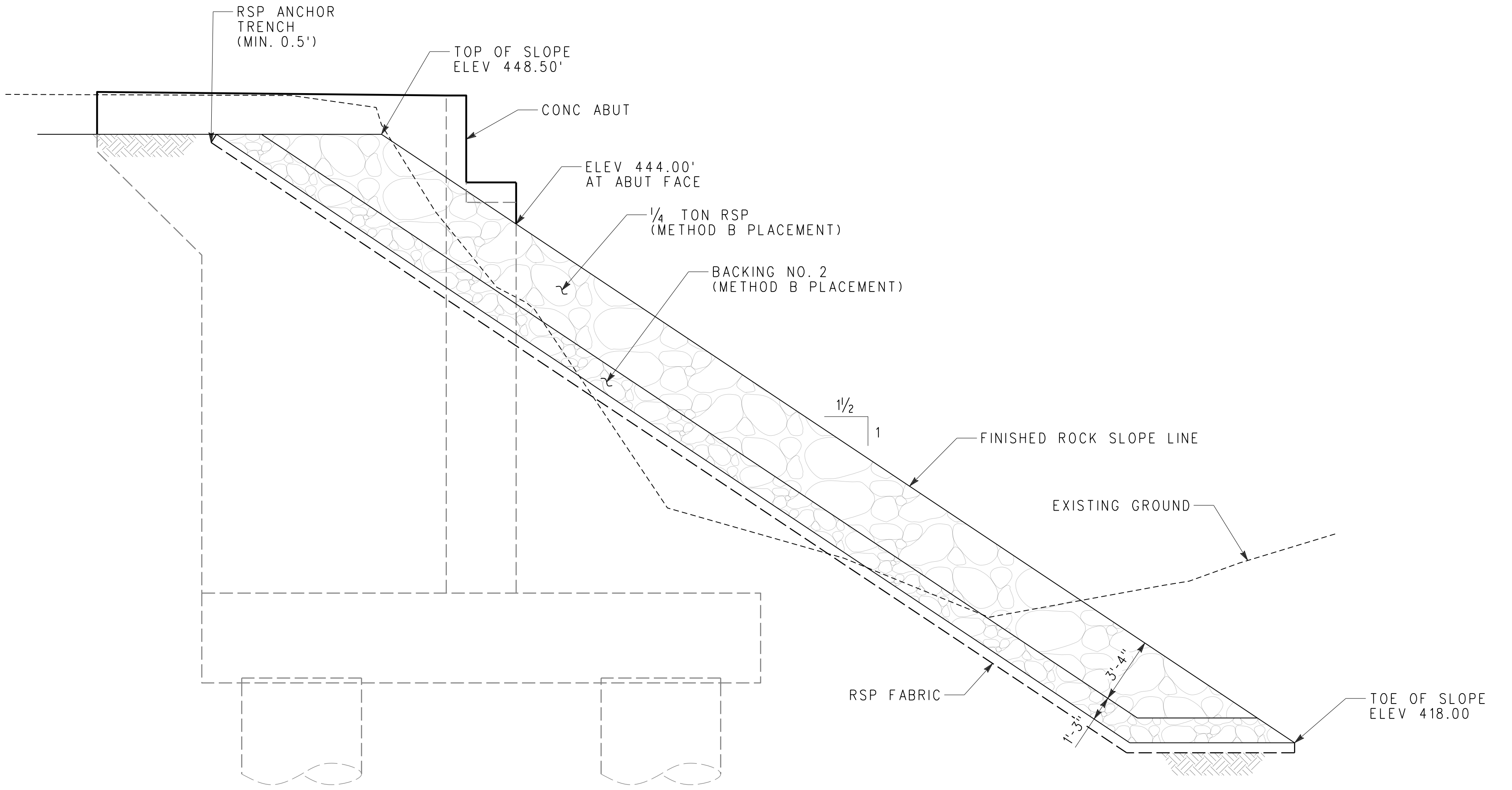
ABUTMENT DETAILS NO. 2

CONTRACT NO.
DRAWING NO. S-007
REVISION
SHEET NO. 16 OF 29
SCALE AS SHOWN



PLAN

SCALE: 1"=10'



SECTION A/-

SCALE: 1/4" = 1'-0"

NOTE:

1. ROCK SLOPE PROTECTION SHALL BE PER CALTRANS STANDARD SPECIFICATIONS SECTION 72.

10/26/2023 9:38:56 AM USER = gerry.estepa
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DESIGNED BY H. KAZEM
DRAWN BY G. ESTEPA
CHECKED BY H. YANG
APPROVED BY S. NAVEED
DATE 10-31-2023

PROGRESS SUBMITTAL
90%
NOT FOR CONSTRUCTION

VENTURA COUNTY
TRANSPORTATION
COMMISSION

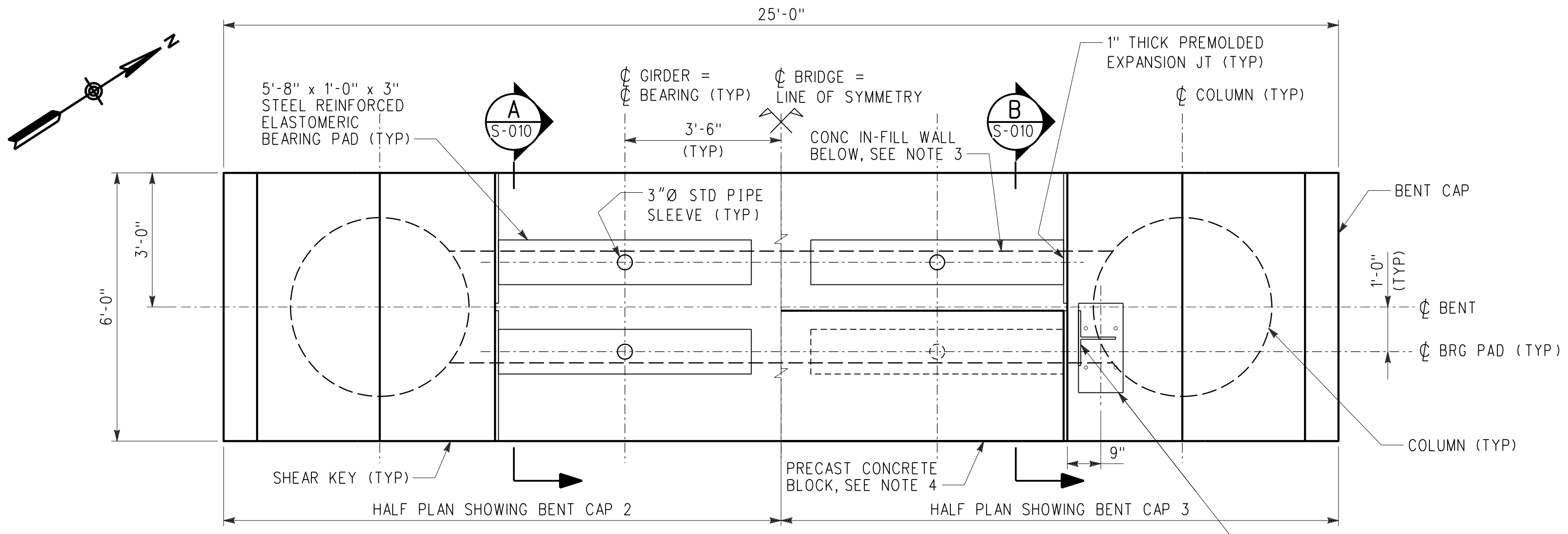
RAILPROS

SUBMITTED: ____________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

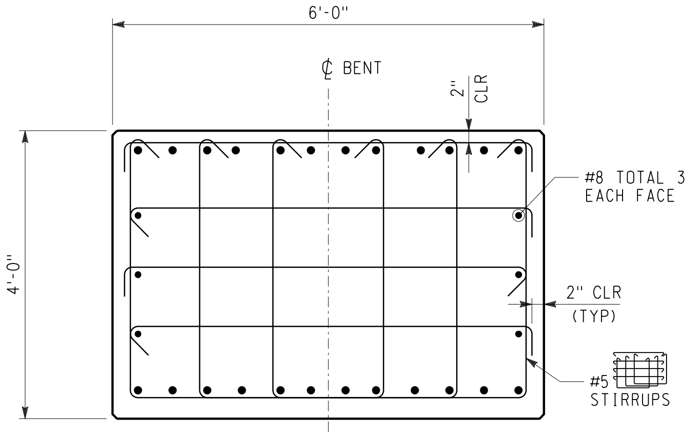
SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

ROCK SLOPE PROTECTION

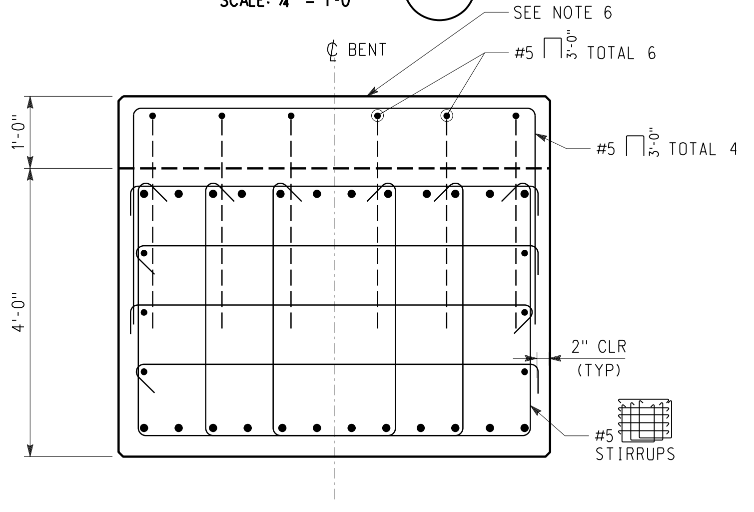
CONTRACT NO.
DRAWING NO. S-008
REVISION
SHEET NO. 17 OF 29
SCALE AS SHOWN



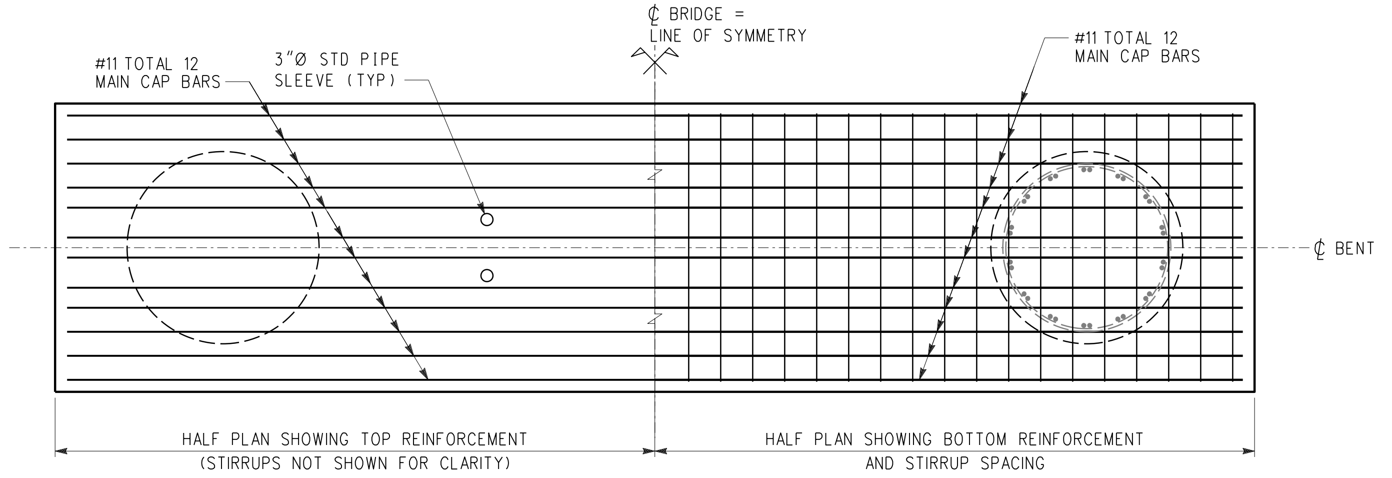
PLAN - BENT CAP
1
-
SCALE: ½" = 1'-0"



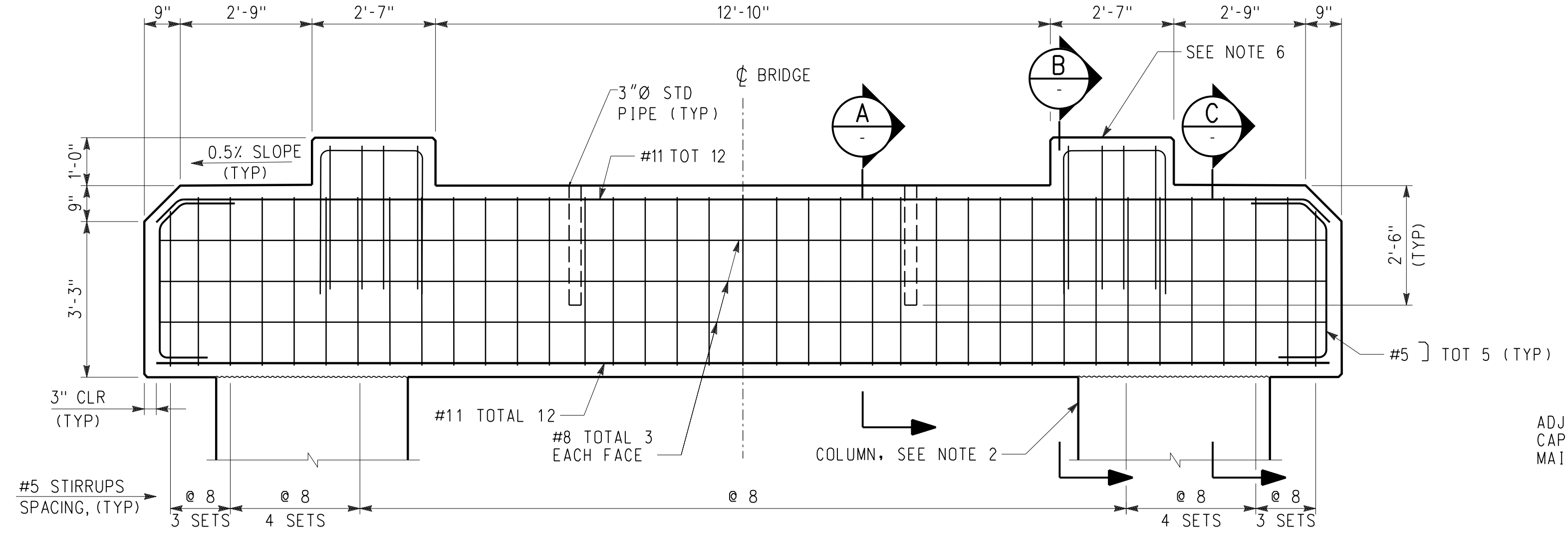
SECTION A
-
SCALE: ¾" = 1'-0"



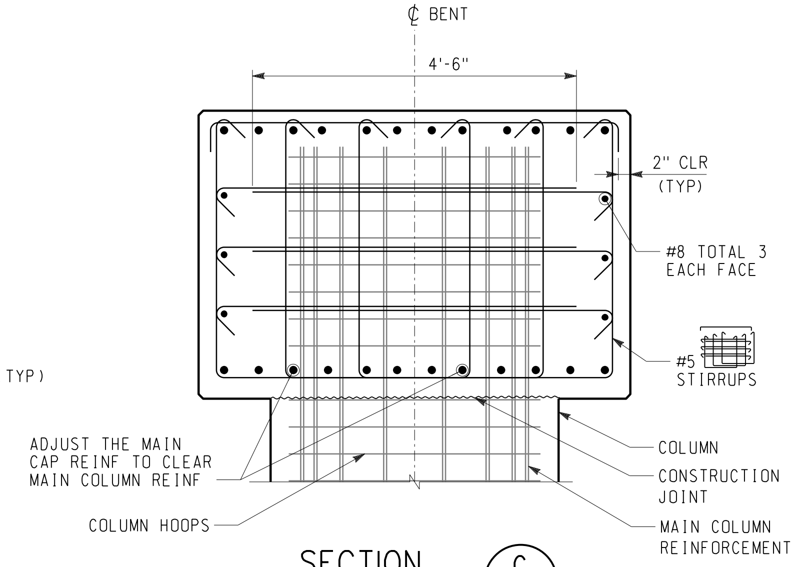
SECTION B
-
SCALE: ¾" = 1'-0"



PLAN – BENT CAP REINFORCEMENT
SCALE: ½" = 1'-0"



ELEVATION – BENT CAP REINFORCEMENT
SCALE: ½" = 1'-0"



SECTION C
-
SCALE: ¾" = 1'-0"

NOTES:

1. NO SPLICES ALLOWED IN MAIN BENT CAP REINFORCEMENT.
2. COLUMN REINFORCEMENT NOT SHOWN FOR CLARITY. FOR REINFORCEMENT, SEE "BENT DETAILS NO. 2" SHEET.
3. FOR CONCRETE IN-FILL WALL DIMENSIONS AND REINFORCEMENT, SEE "BENT DETAILS NO. 2" SHEET.
4. FOR SIZE AND REINFORCEMENT OF PRECAST CONCRETE CATCHER BLOCK, SEE "BENT DETAILS NO. 2" SHEET. AT BENT 3 UP-STATION ONLY.
5. FOR GIRDER STOP PLACEMENT DETAIL, SEE "MISCELLANEOUS DETAILS NO. 1" SHEET. FOR GIRDER STOP AND EMBED PLATE DETAILS, SEE "MISCELLANEOUS DETAILS NO. 2" SHEET.
6. EMBEDDED PLATE AND GIRDER STOP NOT SHOWN FOR CLARITY.

10/26/2023 9:39:03 AM USER : gerry.estepa
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REV.	DATE		BY	SUB.	APP.

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DESIGNED BY H. KAZEM
DRAWN BY G. ESTEPA
CHECKED BY H. YANG
APPROVED BY S. NAVEED
DATE 10-31-2023

PROGRESS SUBMITTAL 90% NOT FOR CONSTRUCTION

VCTC
VENTURA COUNTY
TRANSPORTATION
COMMISSION

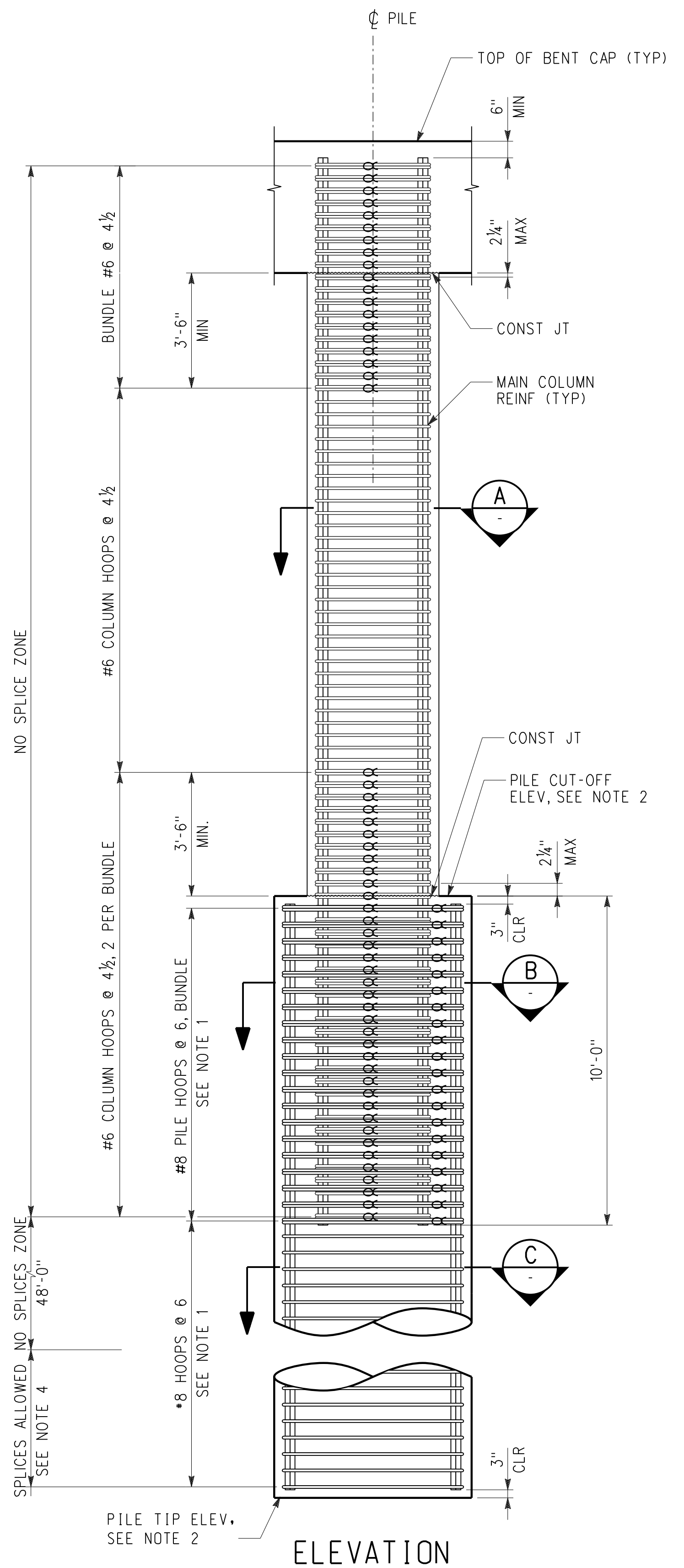
RAILPROS

SUBMITTED:
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

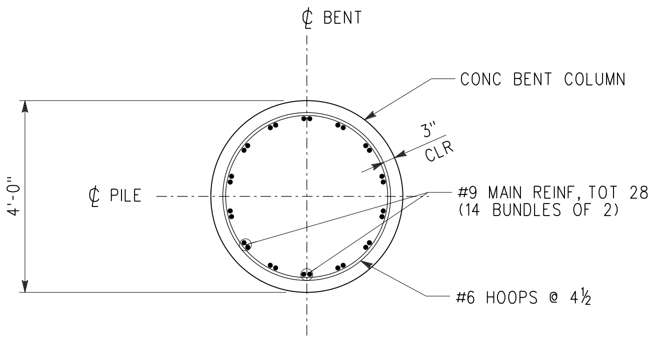
BENT DETAILS NO. 1

CONTRACT NO.
DRAWING NO. S-009
REVISION
SHEET NO. 18 OF 29
SCALE AS SHOWN



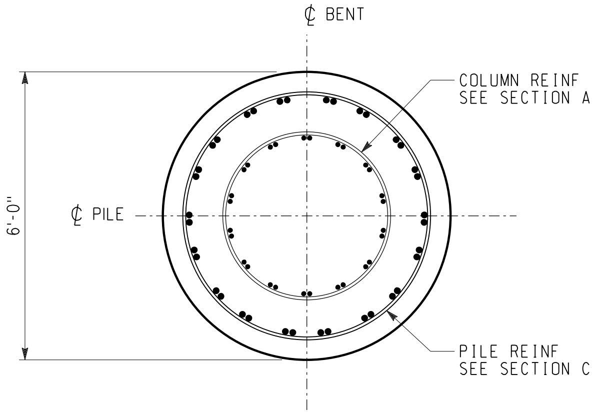
ELEVATION

SCALE: ⅜" = 1'-0"



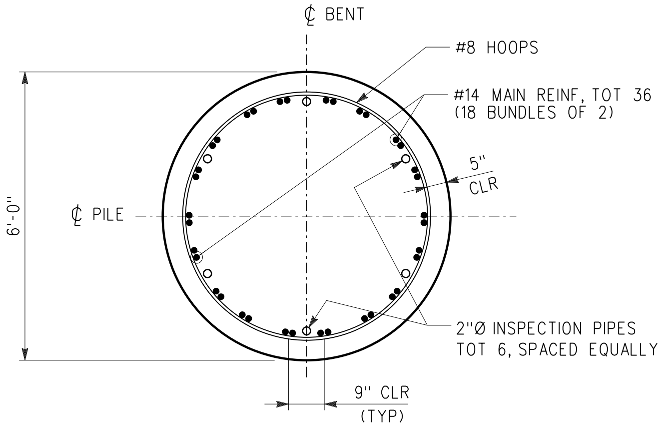
SECTION A -

SCALE: ½" = 1'-0"



SECTION B -

SCALE: ½" = 1'-0"



SECTION C -

SCALE: ½" = 1'-0"

NOTES:

1. ALL HOOPS ARE ULTIMATE BUTT SPLICES
2. FOR PILE TIP AND CUT-OFF ELEVATION, SEE PILE DATA TABLE ON "FOUNDATION PLAN" SHEET
3. NO SPLICES ALLOWED IN THE COLUMN MAIN REINFORCEMENT
4. SPLICES SHALL BE SERVICE SPLICES "MECHANICAL COUPLERS"

LEGEND

∝ INDICATES BUNDLED BARS

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DESIGNED BY H. KAZEM
DRAWN BY G. ESTEPA
CHECKED BY H. YANG
APPROVED BY S. NAVEED
DATE 10-31-2023

PROGRESS SUBMITTAL ★ 90% ★ NOT FOR CONSTRUCTION

VENTURA COUNTY
TRANSPORTATION
COMMISSION

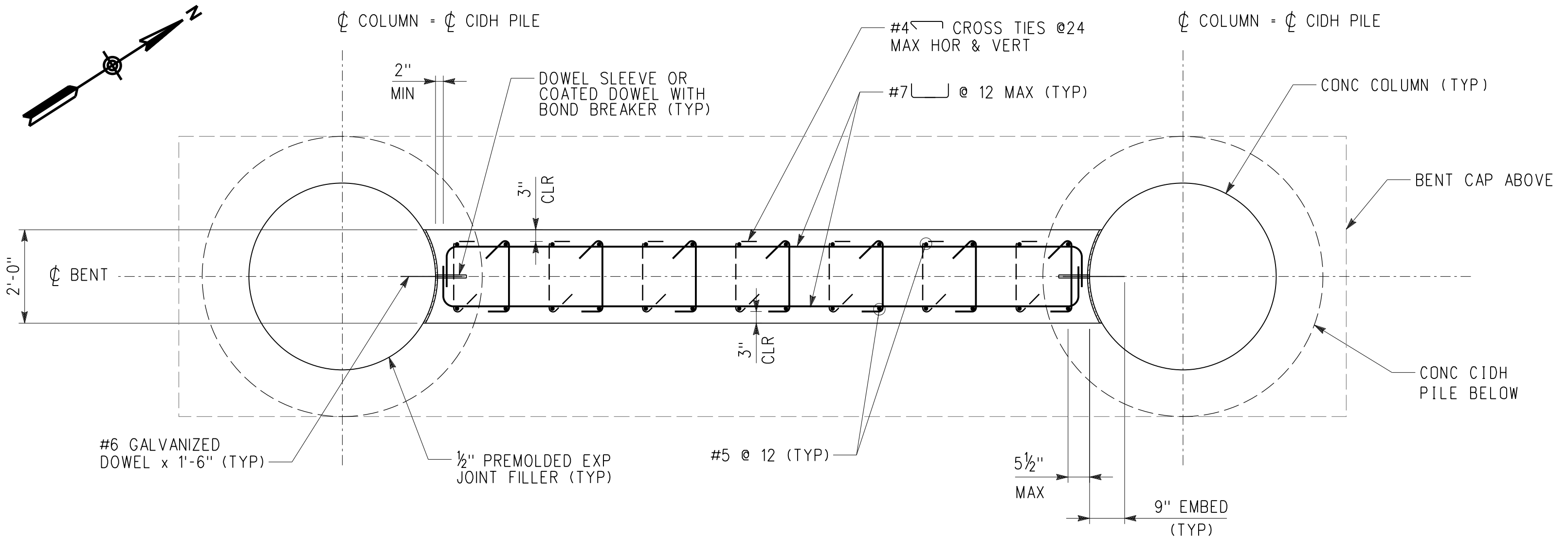
RAILPROS

SUBMITTED: ____________________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

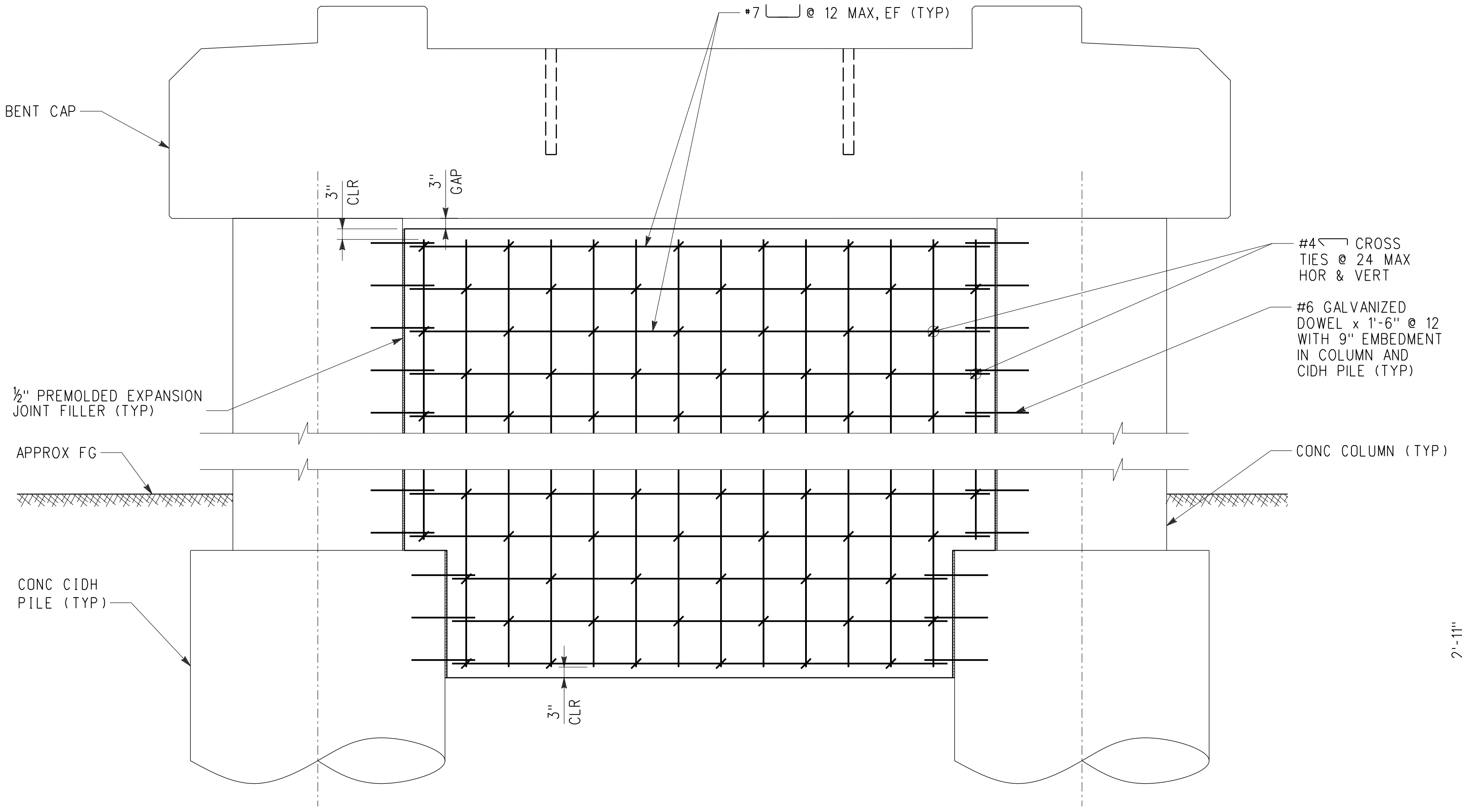
BENT DETAILS NO. 2

CONTRACT NO.
DRAWING NO. S-010
REVISION
SHEET NO. 19 OF 29
SCALE AS SHOWN



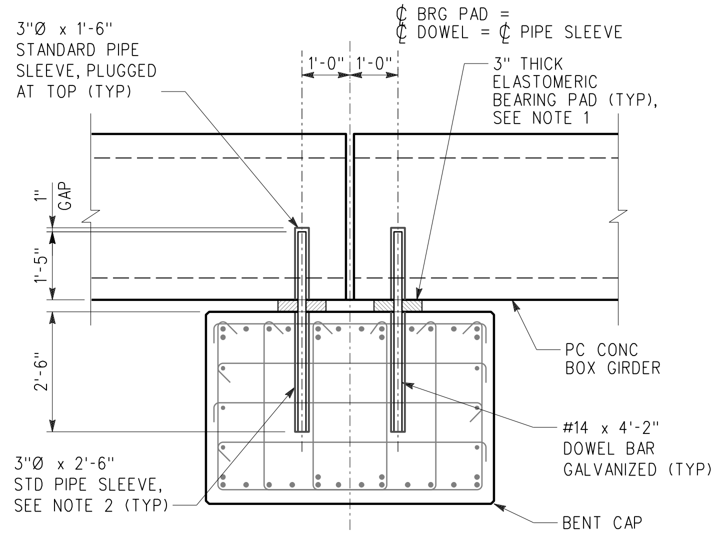
PLAN – IN-FILL WALL

SCALE: ½" = 1'-0"



ELEVATION – IN-FILL WALL

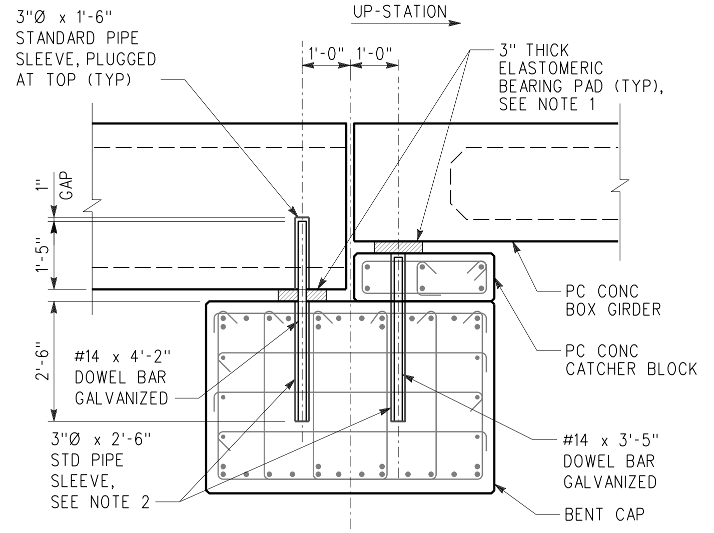
SCALE: ½" = 1'-0"



SECTION - BENT 2 CAP

A / S-008

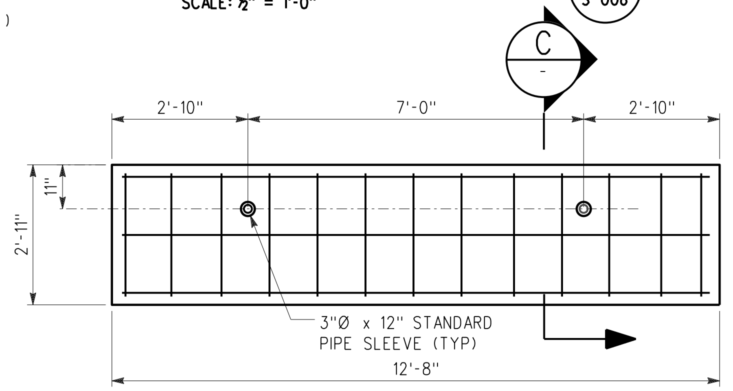
SCALE: ½" = 1'-0"



SECTION - BENT 3 CAP

B / S-008

SCALE: ½" = 1'-0"

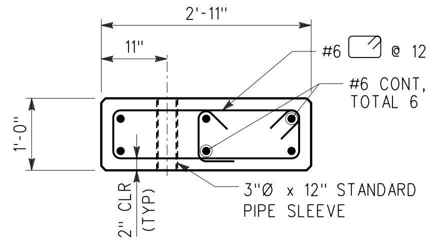


PLAN

PC CONCRETE CATCHER BLOCK DETAIL

1 / -

SCALE: ½" = 1'-0"



SECTION

C / -

SCALE: ¾" = 1'-0"

NOTES:

1. FOR BEARING PAD DETAILS, SEE DETAIL 1 ON "ABUTMENT DETAILS NO. 1" SHEET.
2. PIPE SLEEVE TO BE FILLED WITH NON-SHRINK GROUT AFTER INSTALLATION OF #14 DOWEL BAR.
3. BENT CAP REINFORCEMENT TO BE ADJUSTED AS NEEDED TO PROVIDE 1" CLEARANCE TO THE PIPE SLEEVE.

REV.	DATE		BY	SUB.	APP.

DESIGNED BY H. KAZEM
DRAWN BY G. ESTEPA
CHECKED BY H. YANG
APPROVED BY S. NAVEED
DATE 10-31-2023

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VENTURA COUNTY
TRANSPORTATION
COMMISSION

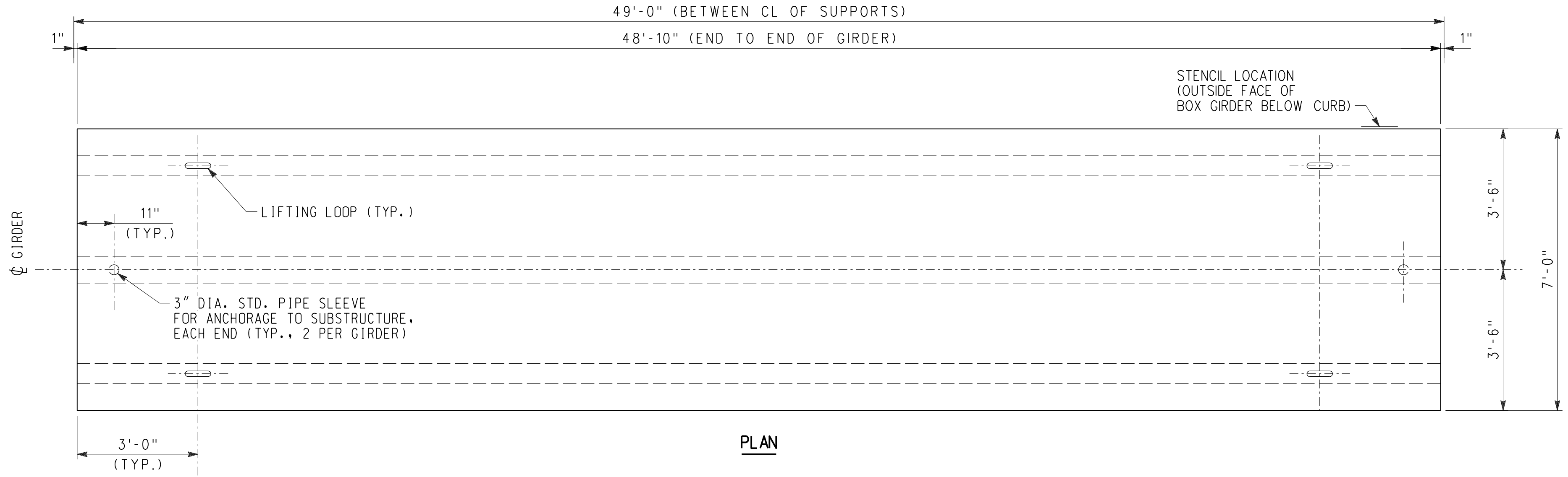
RAILPROS

SUBMITTED: ____________________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

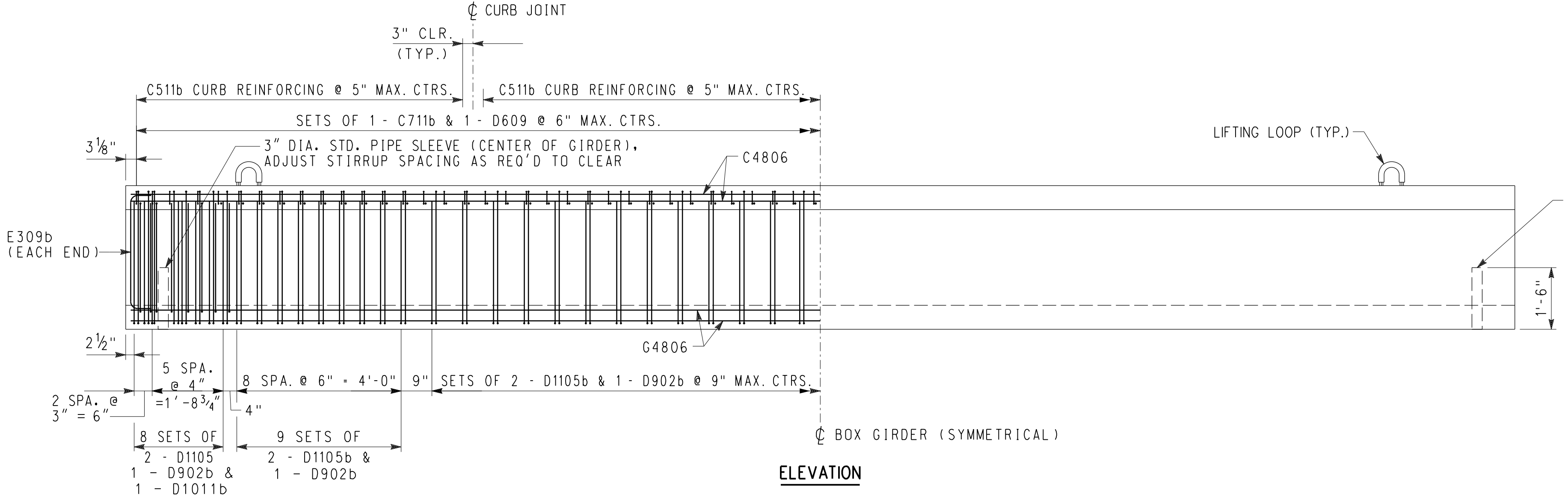
SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

BENT DETAILS NO. 3

CONTRACT NO.
DRAWING NO. S-011
REVISION
SHEET NO. 20 OF 29
SCALE AS SHOWN



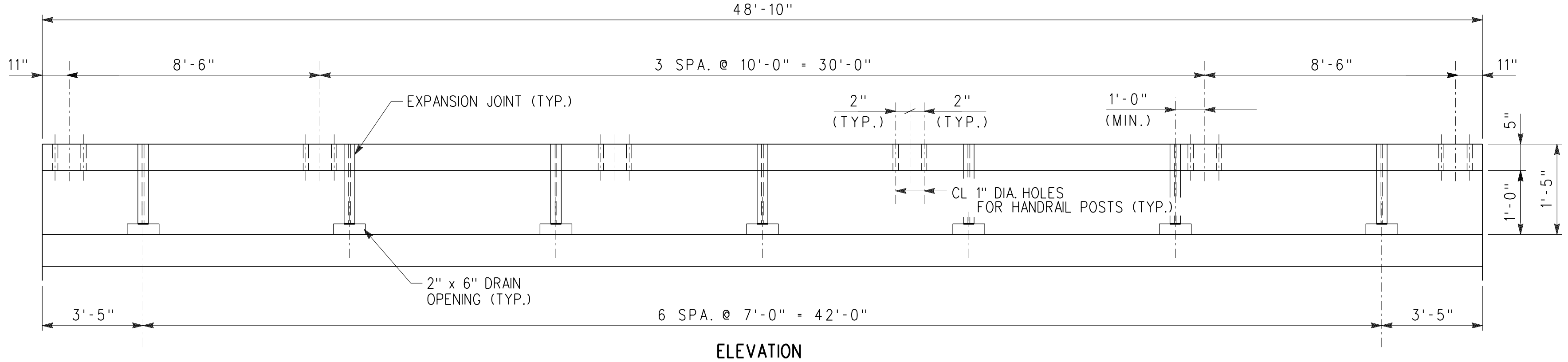
PLAN



ELEVATION

ELEVATION w/ CONVENTIONAL REINFORCING

SCALE: NONE



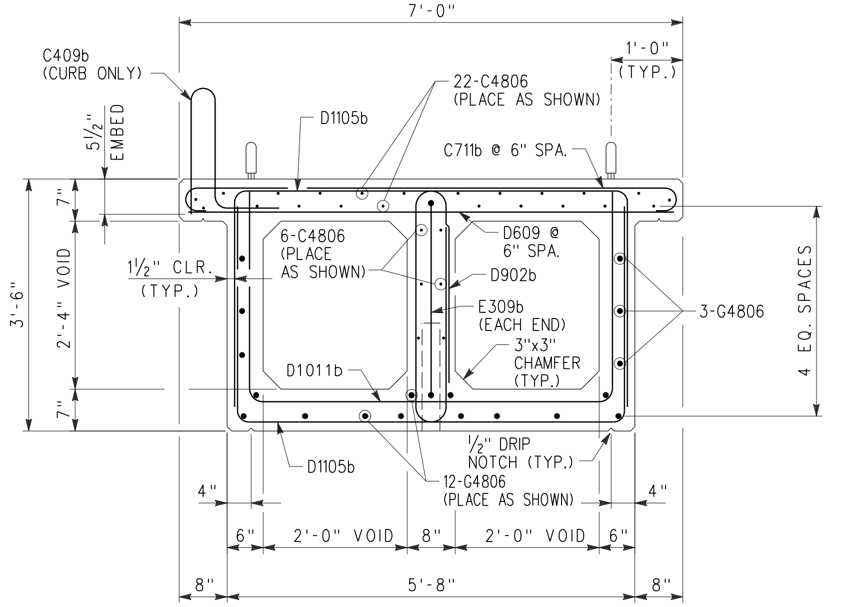
ELEVATION

BALLAST CURB AND SIDEWALK

SCALE: NONE

NOTE:

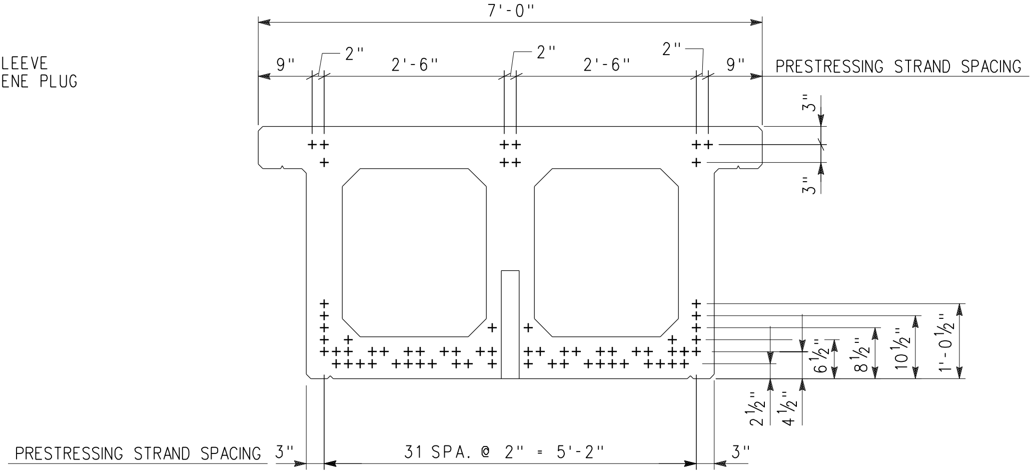
FOR GENERAL GIRDER NOTES AND REINFORCING SCHEDULE, SEE "GIRDER DETAILS NO. 2" SHEET.



TYPICAL MILD REINFORCING SECTION

SCALE: ¾" = 1'-0"

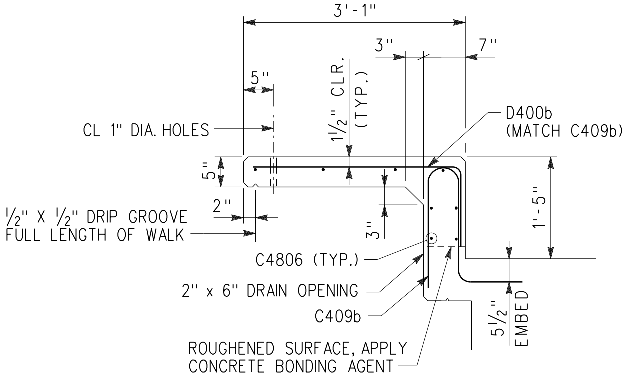
(PRESTRESSING STRANDS NOT SHOWN FOR CLARITY)



PRESTRESSING STRAND PATTERN

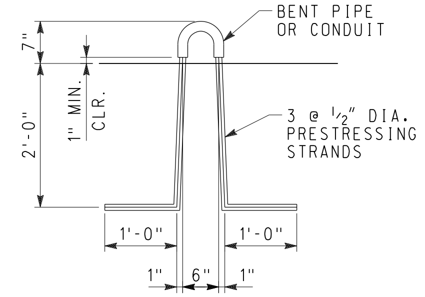
SCALE: ¾" = 1'-0"

+ (66 - 0.6" 270 KSI PRESTRESSING STRANDS)



CURB AND WALK DETAIL

SCALE: ¾" = 1'-0"



LIFTING LOOP DETAIL

SCALE: ¾" = 1'-0"

(FABRICATOR IS RESPONSIBLE FOR ADEQUACY OF LIFTING LOOPS, SEE NOTES ON "GIRDER DETAILS NO.2" SHEET)

REV.	DATE		BY	SUB.	APP.

DESIGNED BY K. THOMSEN
DRAWN BY G. SMITH
CHECKED BY H. YANG
APPROVED BY S. NAVEED
DATE 10-31-2023

PROGRESS SUBMITTAL 90% NOT FOR CONSTRUCTION

VENTURA COUNTY TRANSPORTATION COMMISSION

RAILPROS

SUBMITTED: DANIELLE LIBRING, P.E., T.E. PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

GIRDER DETAILS NO. 1

CONTRACT NO.
DRAWING NO. S-012
REVISION
SHEET NO. 21 OF 29
SCALE AS SHOWN

GENERAL NOTES

CONCRETE:

CONCRETE MATERIAL, PLACING AND CURING SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS SPECIFIED IN SCRRA STANDARD SPECIFICATIONS AND THE CURRENT EDITION OF CHAPTER 8 OF THE AREMA MANUAL FOR RAILWAY ENGINEERING.

THE COMPRESSIVE STRENGTH OF THE CONCRETE SHALL BE 6,500 PSI AT THE TRANSFER OF THE PRESTRESSING FORCE AND 8,000 PSI AT 28 DAYS.

MINIMUM COMPRESSIVE STRENGTH OF CURB CONCRETE SHALL BE 4,000 PSI AT 28 DAYS.

AIR ENTRAINING AGENTS SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS SPECIFIED IN THE CURRENT EDITION OF ASTM C260. THE TOTAL ENTRAINED AIR CONTENT SHALL BE 6% +/- 1% BY VOLUME OF THE PLASTIC CONCRETE.

CONCRETE AGGREGATE SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS SPECIFIED IN THE CURRENT EDITION OF ASTM C33. COARSE AGGREGATE SHALL BE SIZE NO. 67.

PRESTRESSING STRAND:

PRESTRESSING STRAND SHALL BE 0.6 INCH DIAMETER, SEVEN WIRE, UNCOATED, LOW RELAXATION PRESTRESSING STRAND WHICH IS IN ACCORDANCE WITH THE REQUIREMENTS SPECIFIED IN ASTM A416. THE PRESTRESSING STRAND SHALL HAVE AN ULTIMATE TENSILE STRENGTH OF 270 KSI. THE INITIAL PRESTRESS SHALL BE 43,400 LBS. PER PRESTRESSING STRAND UNLESS NOTED OTHERWISE.

PRESTRESSING STRAND SHALL BE TESTED IN ACCORDANCE WITH PCI RECOMMENDATIONS (MOUSTAFA METHOD) AND CERTIFIED BY THE FABRICATOR AS HAVING ADEQUATE BOND CHARACTERISTICS TO SATISFY THE PREDICTION EQUATIONS FOR TRANSFER AND DEVELOPMENT LENGTH GIVEN IN THE AREMA MANUAL FOR RAILWAY ENGINEERING.

AN ALTERNATE PRESTRESSING STRAND PATTERN WHICH HAS THE SAME ECCENTRICITY AS THE PATTERN SHOWN ON THIS PLAN AND IS BETTER SUITED TO THE MANUFACTURER'S FACILITIES WILL BE CONSIDERED. MANUFACTURER MUST SUBMIT PLANS AND COMPUTATIONS FOR ENGINEER'S APPROVAL PRIOR TO CASTING.

REINFORCING STEEL:

REINFORCING STEEL SHALL BE DEFORMED, PER CURRENT ASTM A615 SPECIFICATION AND MEET GRADE 60 REQUIREMENTS, EXCEPT BARS CROSSING CURB JOINT TO BE PER CURRENT ASTM A1035 SPECIFICATION. BARS REQUIRED TO MEET ASTM A1035 ARE NOTED IN THE BENDING DIAGRAMS.

FABRICATION OF REINFORCING STEEL SHALL BE PER CHAPTER 7 OF THE CRSI MANUAL OF STANDARD PRACTICE. DIMENSIONS OF BENDING DETAILS ARE OUT TO OUT OF BAR.

REINFORCING STEEL IS TO BE BLOCKED TO PROPER LOCATION AND SECURELY WIRED AGAINST DISPLACEMENT. USE PLASTIC PROTECTED REINFORCING SUPPORTS, MEETING CRSI SPECIFICATIONS CHAPTER 3, CLASS 1. TACK WELDING OF REINFORCING IS PROHIBITED. MINIMUM CONCRETE COVER ON REINFORCEMENT SHALL MEET CURRENT AREMA REQUIREMENTS.

DESIGN LOADS:

DEAD LOAD (ASSUMED - LB. PER LIN. FT. OF TRACK):

TRACK, FASTENERS, ETC.	200
BALLAST	4,065
CURB, WALK, & HANDRAIL	580
GIRDERS	3,600
TOTAL	8,445

THE FABRICATOR SHALL CAMBER THE GIRDERS AS REQUIRED TO RESULT IN A NET VERTICAL DEFLECTION OF 0" DUE TO MAXIMUM DEAD LOADS SHOWN BELOW.

DEAD LOAD (ASSUMED - LB. PER LIN. FT. OF ONE GIRDER):

TRACK, FASTENERS, ETC.	100
BALLAST	2,035
CURB, WALK, & HANDRAIL	290
GIRDERS	1,800
TOTAL	4.225

LIVE LOAD: COOPER E80; CENTRIFUGAL FORCE AND EFFECTS OF ECCENTRICITY AND SUPERELEVATION APPLIED PER PROVISIONS OF AREMA CHAPTER 8. DESIGN SUPERELEVATION IS 5" WITH 2" UNBALANCE.

IMPACT: $\frac{225}{\sqrt{\ell}}$ % (WHERE ℓ = L - 14")

GIRDERS HAVE BEEN DESIGNED TO ACCOMMODATE A MAXIMUM OFFSET BETWEEN THE CENTERLINE OF TRACK AND THE CENTER OF THE LONGITUDINAL JOINT BETWEEN GIRDERS OF 6 INCHES. GIRDERS SHALL BE SUPPLIED WITH CURB.

MANUFACTURE:

PRODUCTION PROCEDURES AND DIMENSIONAL TOLERANCES FOR THE MANUFACTURE OF PRECAST, PRESTRESSED GIRDERS SHALL BE IN ACCORDANCE WITH THE AREMA MANUAL FOR RAILWAY ENGINEERING AND THE PRECAST CONCRETE INSTITUTE'S CURRENT MANUAL MNL 116 FOR QUALITY CONTROL.

TOLERANCE FOR LOCATION OF LIFTING LOOPS SHALL BE +/-½".

THE ENDS OF THE PRESTRESSING STRANDS SHALL BE CUT FLUSH WITH THE END OF THE PRODUCT AND PAINTED WITH AN APPROVED COATING.

CURB SHALL BE CAST AFTER GIRDER IS REMOVED FROM FORM.

CONCRETE BONDING AGENT: REFER TO SPECIFICATIONS.

SURFACES SHALL BE FORMED IN A MANNER WHICH WILL PRODUCE A SMOOTH AND UNIFORM APPEARANCE WITHOUT RUBBING OR PLASTERING. UNLESS OTHERWISE NOTED, EXPOSED EDGES OF 90-DEGREES OR LESS ARE TO BE CHAMFERED ¾"x¾". UNFORMED SURFACES SHALL HAVE A SMOOTH FINISH FREE OF ALL FLOAT AND TROWEL MARKS.

THE FABRICATOR SHALL STENCIL THE FABRICATOR'S NAME, DATE OF FABRICATION, PIECE MARK, AND ACTUAL LIFTING WEIGHT AT LOCATION SHOWN.

VOID DIMENSIONS SHOWN ARE MAXIMUM AND MUST NOT BE EXCEEDED AT ANY POINT INCLUDING SPLICES OF VOID FORM.

GIRDERS SHALL BE SUPPORTED BY BLOCKING WITHIN 1'-6" OF ENDS DURING STORAGE AND TRANSPORT. STORE AND TRANSPORT GIRDERS IN LEVEL POSITIONS.

INSPECTION, LOADING, AND SECURING FOR SHIPMENT: REFER TO SPECIFICATIONS.

LIFTING LOOPS:

THE AREA AROUND LIFTING LOOPS SHALL NOT BE RECESSED. LIFTING LOOPS TO BE REMOVED IN FIELD FLUSH WITH CONCRETE SURFACE.

IF LIFTED WITH SLINGS INSTEAD OF LIFTING LOOPS, SLINGS MUST NOT BE PLACED MORE THAN 3'-0" FROM ENDS OF GIRDERS.

FABRICATOR IS RESPONSIBLE FOR DEVELOPING LIFTING LOOP ANCHORAGE DETAIL TO PROVIDE SAFETY FACTOR OF 4 ON WORKING LOAD. DETAIL SHALL BE PROOF-TESTED WITH TEST RESULTS KEPT ON FILE BY FABRICATOR AND AVAILABLE FOR INSPECTION BY THE ENGINEER.

WEIGHTS (ONE GIRDER)

NOMINAL GIRDER LENGTH (L)	NOMINAL WEIGHT *		MAX LIFTING WEIGHT **	
	WEIGHT (WITH CURB & WALK)		WEIGHT (WITH CURB & WALK)	
	LB.	TON	LB.	TON
49'	98,230	49.1	103,455	51.8

* Computed weights using nominal dimensions. For planning purposes only. Fabricator to determine actual lifting weight.
If scale weight not available, use maximum weights.

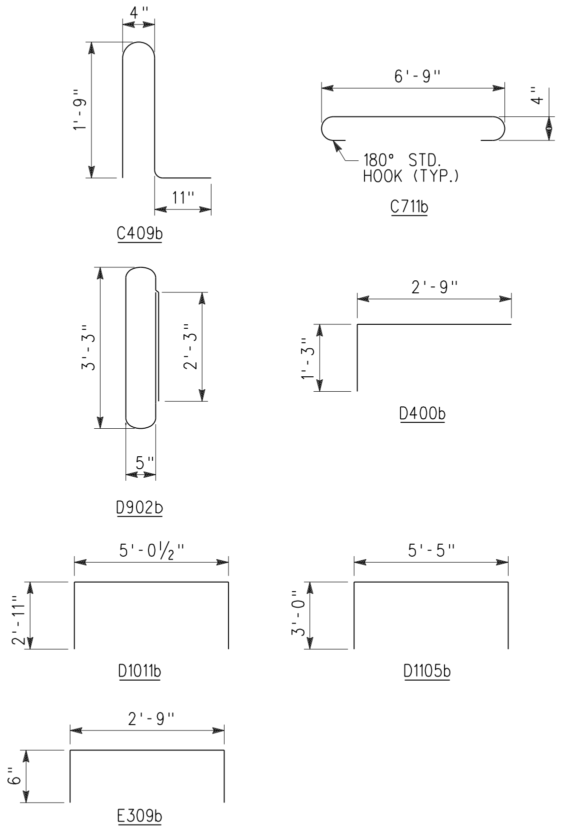
** Computed weights using maximum dimensions per allowable tolerances. Use for lifting weight if scale weight is not available.

REINFORCING SCHEDULE
(QUANTITY PER ONE 42" DOUBLE CELL BOX GIRDER)

REQ'D	MARK	SIZE	LENGTH	SHAPE
116	C409b	#4	4'-9"	⎍
98	C711b	#4	7'-11"	⊂⊃
36	C4806	#4	48'-6"	—
116	D400b	#5	4'-0"	¬
98	D609	#5	6'-9"	—
80	D902b	#5	9'-2"	⬭
16	D1011b	#5	10'-11"	⊓
160	D1105b	#5	11'-5"	⊓
2	E309b	#6	3'-9"	⊓
18	G4806	#8	48'-6"	—

EST. WT. OF REINFORCING STEEL = 8,425 LB.

BENDING DIAGRAM
(DIMENSIONS ARE OUT TO OUT)



NOTE:

BAR DESIGNATIONS CONSIST OF BAR SIZE & LENGTH FOLLOWED BY THE LETTER "b" IF BENT. BAR SIZES ARE REPRESENTED BY THE LETTERS A THROUGH L CORRESPONDING TO BAR SIZE #2 THROUGH #18. BAR LENGTHS ARE GIVEN IN FEET AND INCHES; THE LAST TWO DIGITS ARE INCHES.

10/26/2023 9:39:22 AM USER = gerry.estepa
Z:\Engineering\VCTC\Sespe Creek Bridge Overflow\900 CADD\950 Drawings\S-013_Girder Details 2.sht
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REV. | DATE | BY | SUB. | APP.

INFORMATION CONFIDENTIAL:
All plans, drawings, specifications, and or information furnished herewith shall remain the property of the Ventura County Transportation Commission and shall be held confidential; and shall not be used for any purpose not provided for in agreements with the Ventura Country Transportation Commission.

DESIGNED BY K. THOMSEN
DRAWN BY G. SMITH
CHECKED BY
APPROVED BY
DATE 10-31-2023

PROGRESS SUBMITTAL 90% NOT FOR CONSTRUCTION

VENTURA COUNTY
TRANSPORTATION
COMMISSION

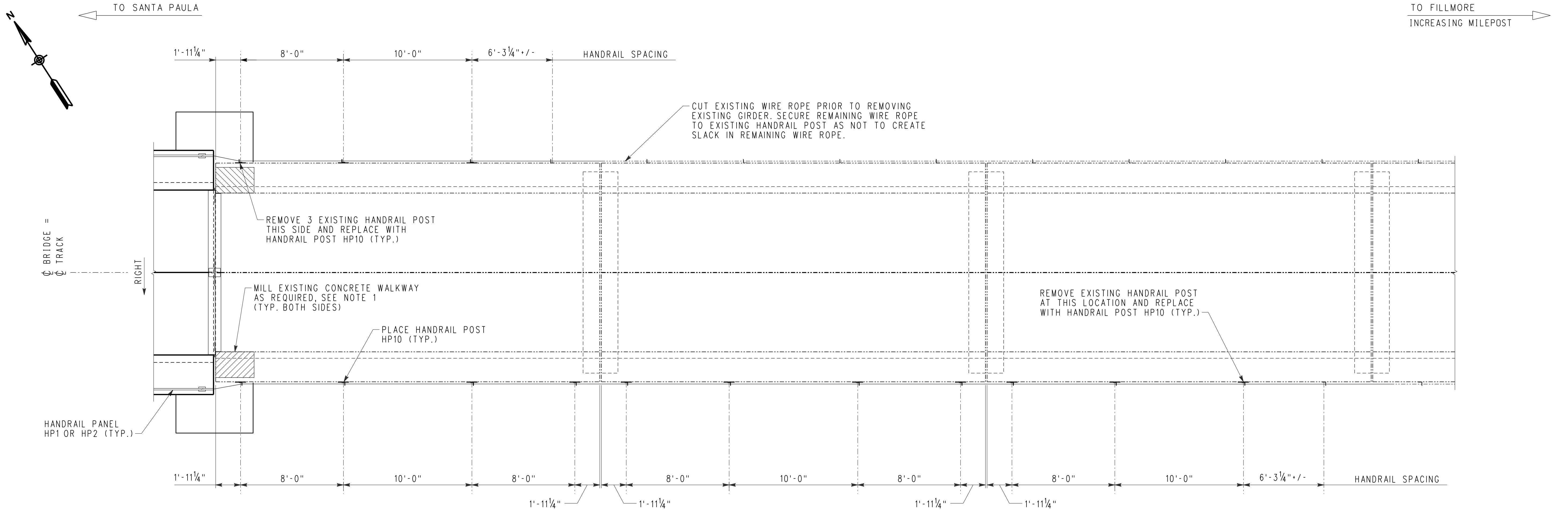
RAILPROS

SUBMITTED: ______
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

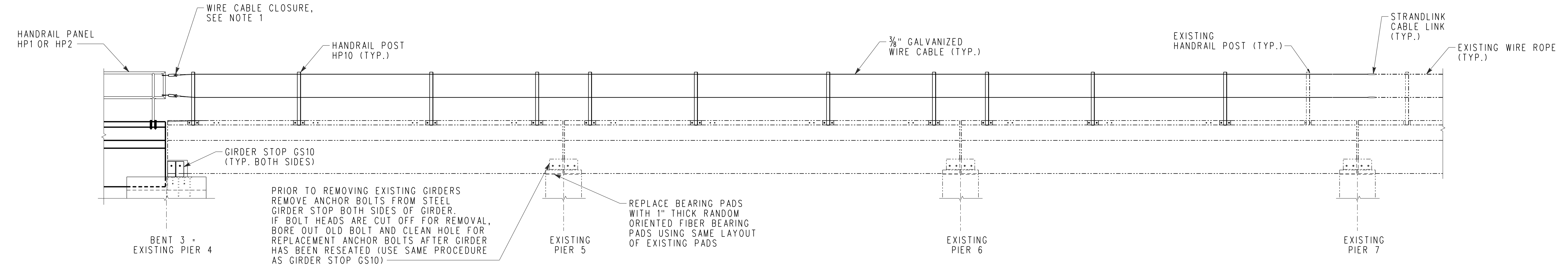
GIRDER DETAILS NO. 2

CONTRACT NO.
DRAWING NO. S-013
REVISION
SHEET NO. 22 OF 29
SCALE AS SHOWN



HANDRAIL REPLACEMENT PLAN

SCALE: ¼" = 1'-0"



HANDRAIL REPLACEMENT ELEVATION

SCALE: ¼" = 1'-0"

NOTE:

1. FOR INSTALLATION DETAILS, SEE "MISCELLANEOUS DETAILS NO.1" SHEET.

10/26/2023 9:39:27 AM USER = gerry.estepa
Z:\Engineering\VCTC\Sespe Creek Bridge Overflow\900 CADD\950 Drawings\S-014_HandrailReplacement Plan.sht
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REV.	DATE		BY / SUB.	APP.

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DESIGNED BY K. THOMSEN
DRAWN BY G. SMITH
CHECKED BY
APPROVED BY
DATE 10-31-2023

PROGRESS SUBMITTAL 90% NOT FOR CONSTRUCTION

VENTURA COUNTY TRANSPORTATION COMMISSION

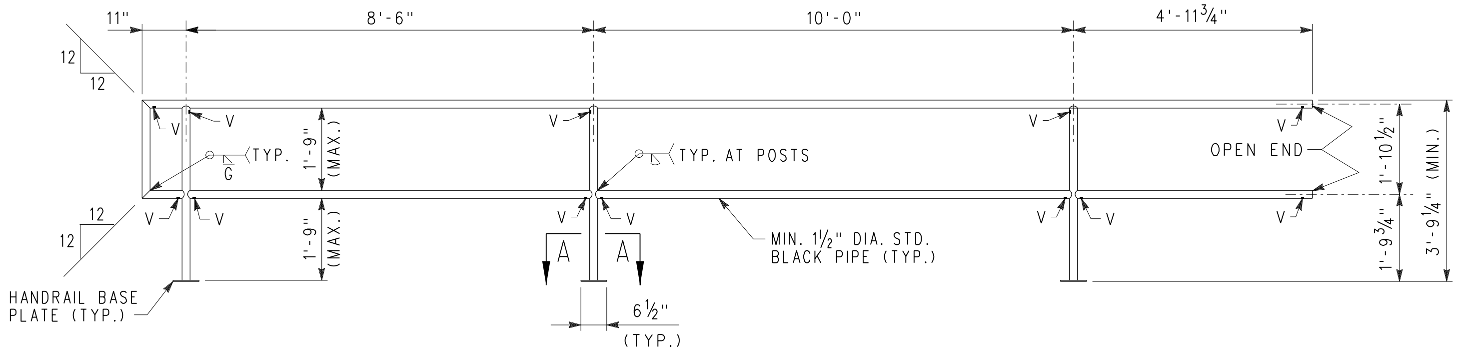
RAILPROS

SUBMITTED: DANIELLE LIBRING, P.E., T.E. PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

HANDRAIL REPLACEMENT PLAN

CONTRACT NO.
DRAWING NO. S-014
REVISION
SHEET NO. 23 OF 29
SCALE AS SHOWN



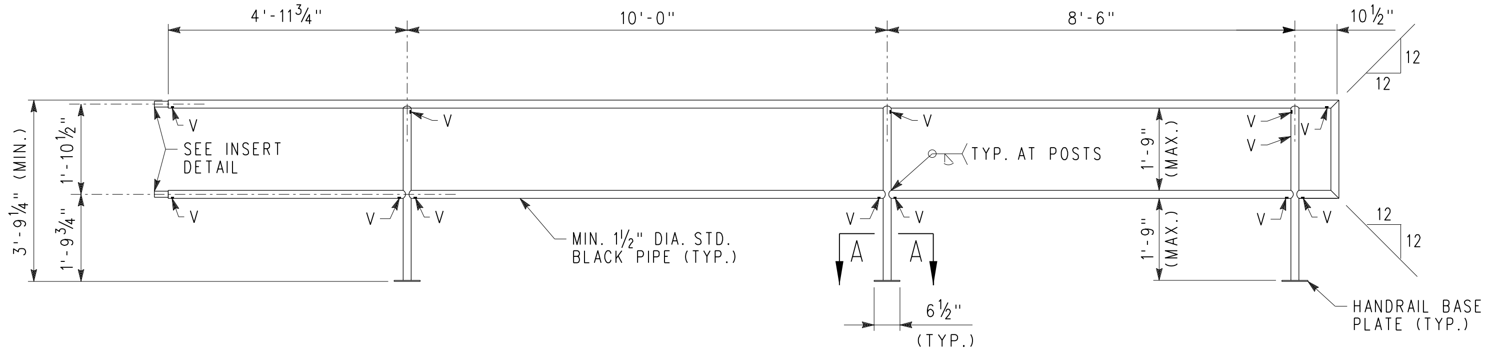
HANDRAIL PANEL HP1

SCALE: ½" = 1'-0"

GALVANIZE AFTER FABRICATION

NOTE:

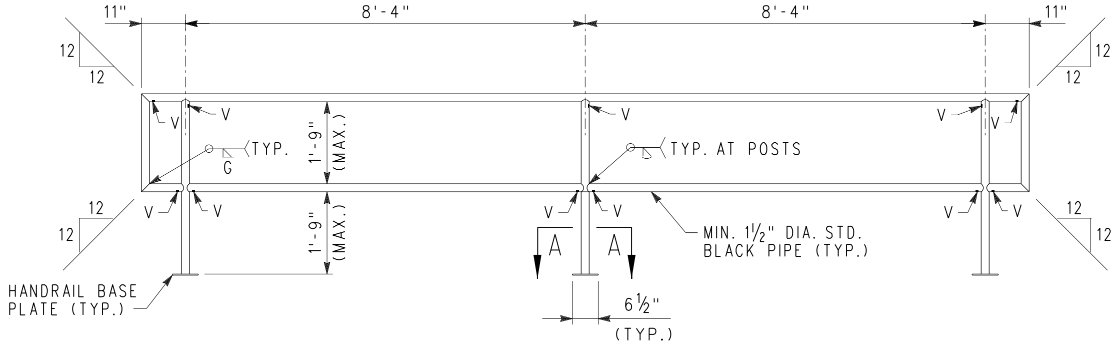
V = ⅜"ø DRILLED VENT HOLE 1" FROM JOINT.



HANDRAIL PANEL HP2

SCALE: ½" = 1'-0"

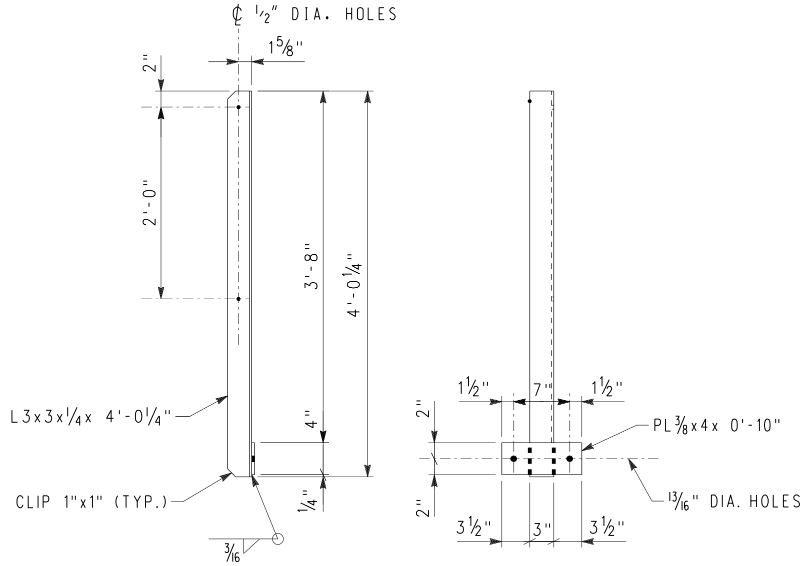
GALVANIZE AFTER FABRICATION.



WINGWALL HANDRAIL PANEL

SCALE: ½" = 1'-0"

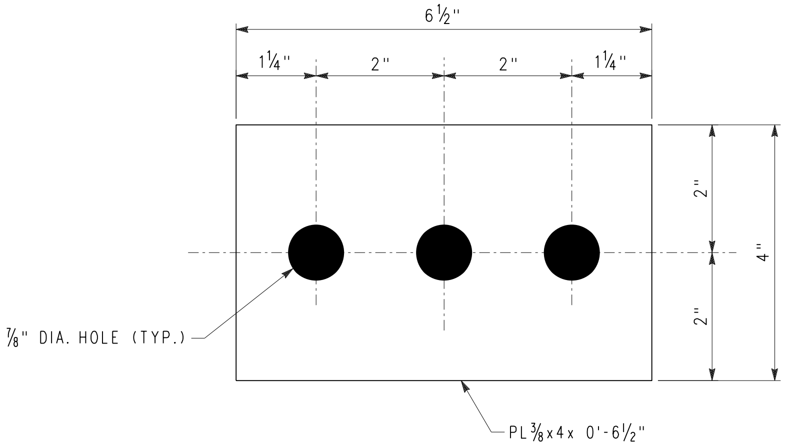
GALVANIZE AFTER FABRICATION.



HANDRAIL POST HP10

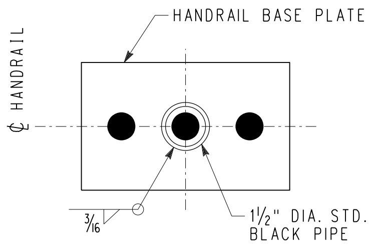
SCALE: 1" = 1'-0"

GALVANIZE AFTER FABRICATION.

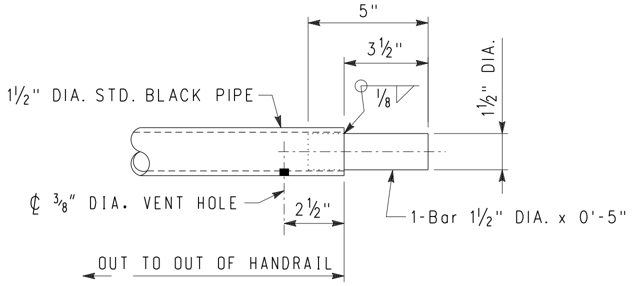


HANDRAIL BASE PLATE DETAIL

WEIGHT = 2.5 LB.



SECTION A-A



INSERT DETAIL

NOTES:

MATERIAL:

STRUCTURAL STEEL BARS SHALL MEET THE REQUIREMENTS OF THE CURRENT A.S.T.M. DESIGNATION: A36.

STANDARD BLACK PIPE SHALL MEET THE REQUIREMENTS OF THE CURRENT A.S.T.M. DESIGNATION: A53. UNCOATED PIPE SHALL BE USED.

SHOP NOTES:

FABRICATION AND ARC WELDING OF STRUCTURAL STEEL AND HANDRAIL PANELS SHALL BE IN ACCORDANCE WITH CHAPTER 15, PART 3 OF THE CURRENT A.R.E.M.A. MANUAL FOR RAILWAY ENGINEERING. MIG WELDING SHALL BE USED ON HANDRAIL PANELS. OPEN HOLES: AS NOTED. SHOP PAINT: NONE.

GALVANIZING:

HANDRAIL PANELS SHALL BE GALVANIZED AFTER FABRICATION IN ACCORDANCE WITH THE CURRENT A.S.T.M. DESIGNATION: A123.

AFTER GALVANIZING ALL ELEMENTS SHALL BE FREE OF FINS, ABRASIONS, ROUGH OR SHARP EDGES AND OTHER SURFACE DEFECTS.

10/26/2023 9:39:32 AM USER = gerry.estepa
Z:\Engineering\VCTC\Sespe Creek Bridge Overflow\900 CADD\950 Drawings\S-015_HandrailDetails.sht
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REV.	DATE		BY/SUB.	APP.

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DESIGNED BY K. THOMSEN
DRAWN BY G. SMITH
CHECKED BY
APPROVED BY
DATE 10-31-2023

PROGRESS SUBMITTAL 90% NOT FOR CONSTRUCTION

VENTURA COUNTY
TRANSPORTATION
COMMISSION

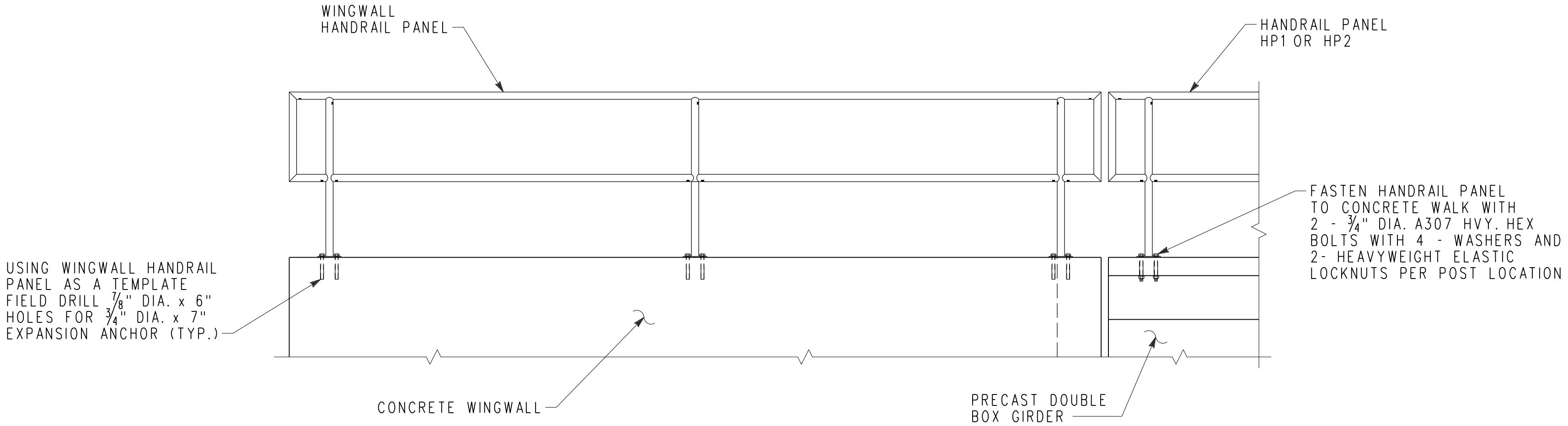
RAILPROS

SUBMITTED:
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

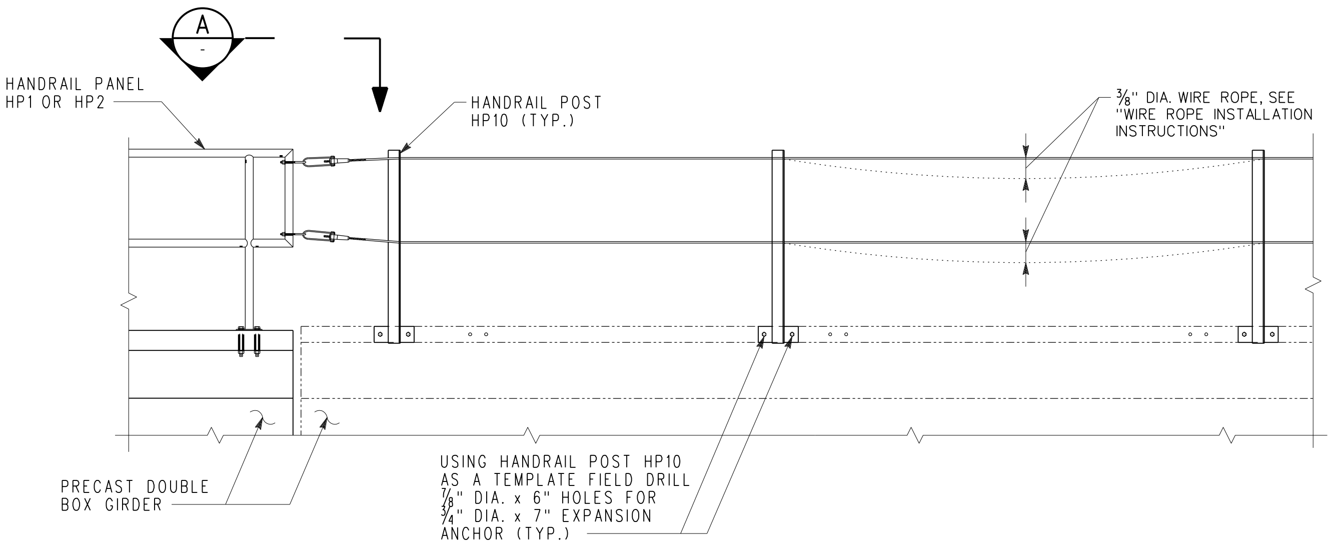
HANDRAIL DETAILS

CONTRACT NO.
DRAWING NO. S-015
REVISION
SHEET NO. 24 OF 29
SCALE AS SHOWN



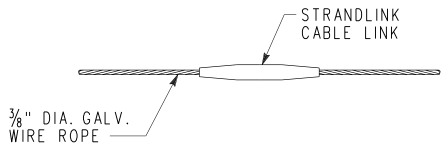
HANDRAIL PANEL INSTALLATION DETAIL

SCALE: 1/2" = 1'-0"



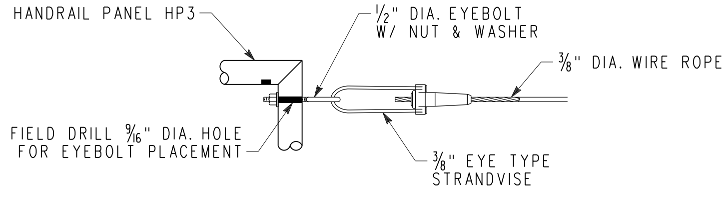
HANDRAIL POST INSTALLATION DETAIL

SCALE: 1/2" = 1'-0"



WIRE ROPE SPLICE DETAIL

SCALE: NONE

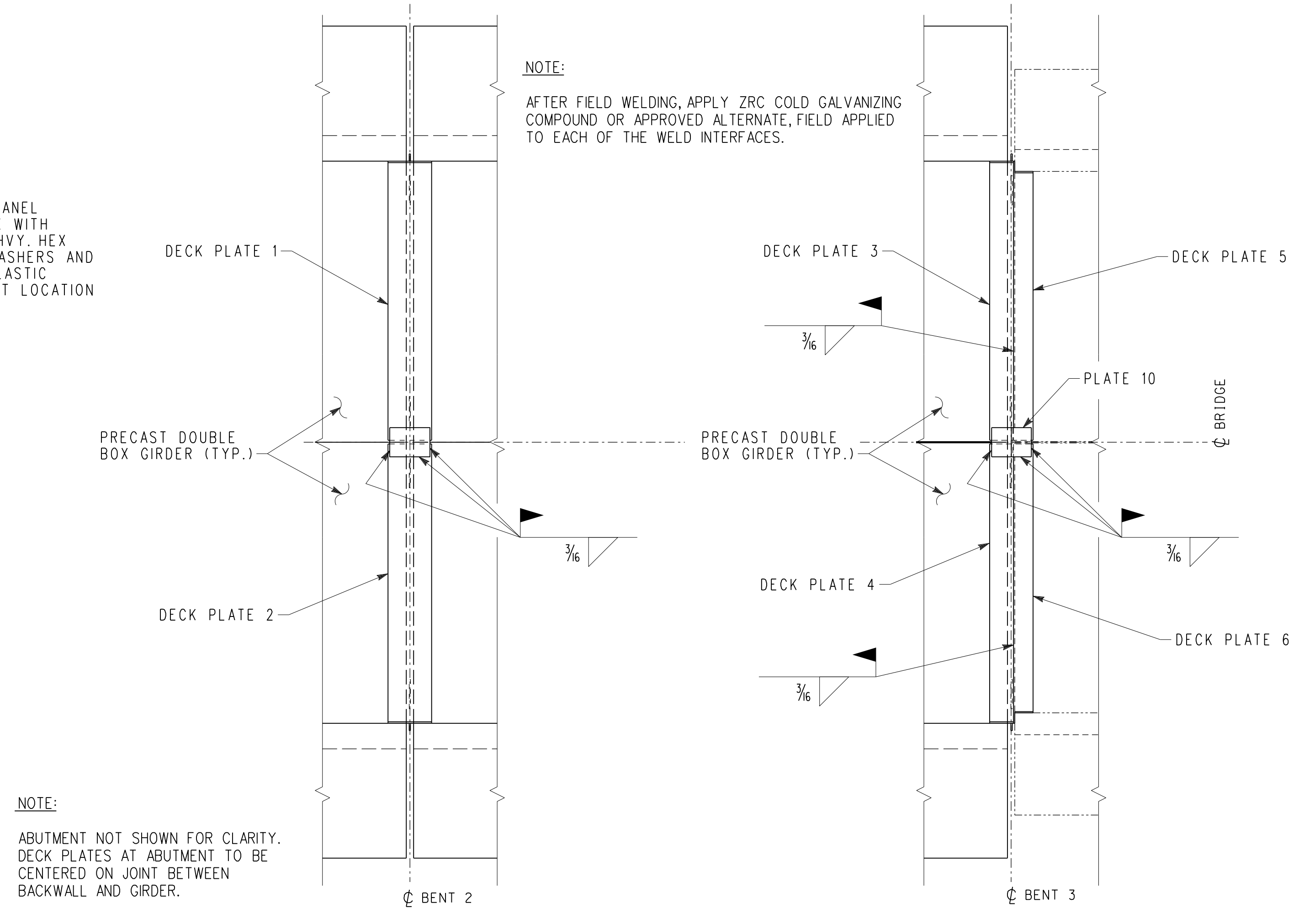


VIEW A -

SCALE: 1 1/2" = 1'-0"

WIRE ROPE INSTALLATION INSTRUCTIONS:

1. THREAD WIRE ROPE THROUGH ALL CLIPS AND BARREL ANCHORS AND SEAT RETAINING WEDGES ON ONE END HANDRAIL POST.
2. STRETCH WIRE ROPE, HANG A MINIMUM OF 10 LB. ON CABLE BETWEEN TWO POSTS AND REMOVE ALL SAG TO A MAXIMUM OF 2 INCHES.
3. SEAT RETAINING WEDGES AT REMAINING END HANDRAIL POST.
4. REMOVE WEIGHTS.
5. TIGHTEN CLIPS AT INTERMEDIATE POSTS.
6. CUT & REMOVE EXCESS WIRE ROPE, COAT CUT PORTIONS OF WIRE ROPE WITH COLD GALVANIZING COMPOUND.



NOTE:

AFTER FIELD WELDING, APPLY ZRC COLD GALVANIZING COMPOUND OR APPROVED ALTERNATE, FIELD APPLIED TO EACH OF THE WELD INTERFACES.

NOTE:

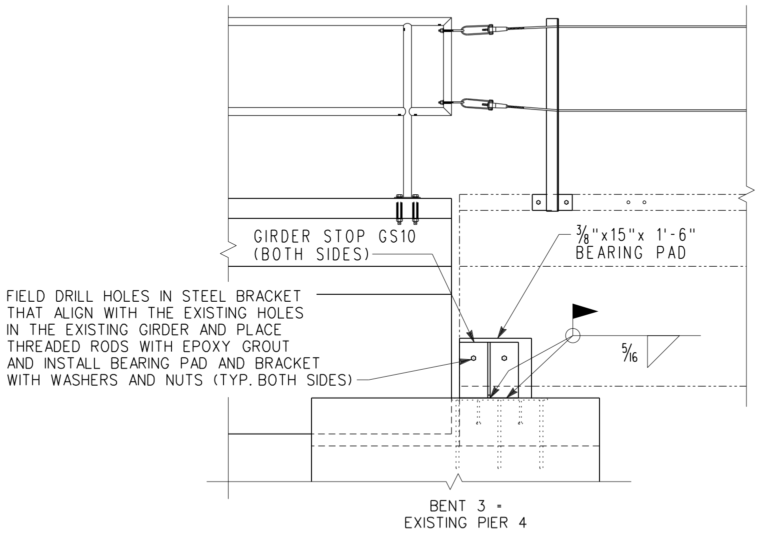
ABUTMENT NOT SHOWN FOR CLARITY. DECK PLATES AT ABUTMENT TO BE CENTERED ON JOINT BETWEEN BACKWALL AND GIRDER.

ABUT. #1 & BENT #2

BENT #3

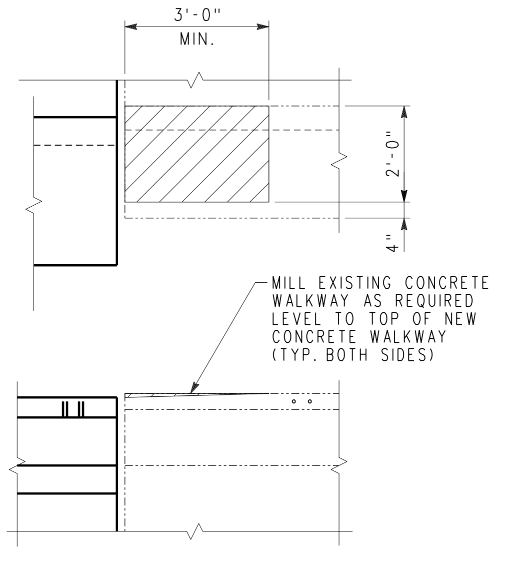
DECK PLATE WELD DETAIL

SCALE: 1/2" = 1'-0"



GIRDER STOP PLACEMENT DETAIL

SCALE: 1/2" = 1'-0"



CONCRETE WALKWAY MILLING

SCALE: 1/2" = 1'-0"

REV.	DATE		BY	SUB.	APP.

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DESIGNED BY K. THOMSEN
DRAWN BY G. SMITH
CHECKED BY
APPROVED BY
DATE 10-31-2023

PROGRESS SUBMITTAL 90% NOT FOR CONSTRUCTION

VENTURA COUNTY
TRANSPORTATION
COMMISSION

RAILPROS

SUBMITTED: ____________________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

MISCELLANEOUS DETAILS NO. 1

CONTRACT NO.
DRAWING NO. S-016
REVISION
SHEET NO. 25 OF 29
SCALE AS SHOWN

10/26/2023 9:39:37 AM USER = gerry.estepa
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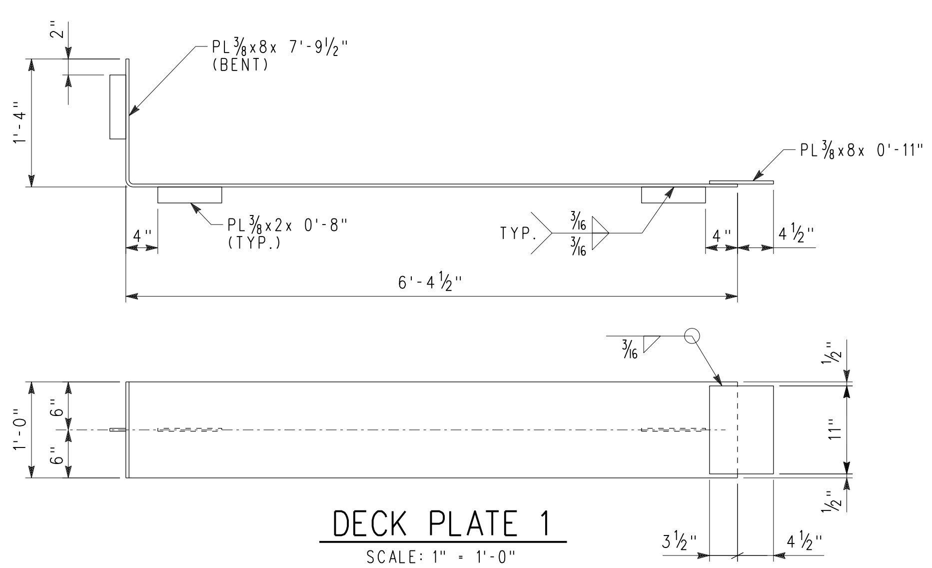
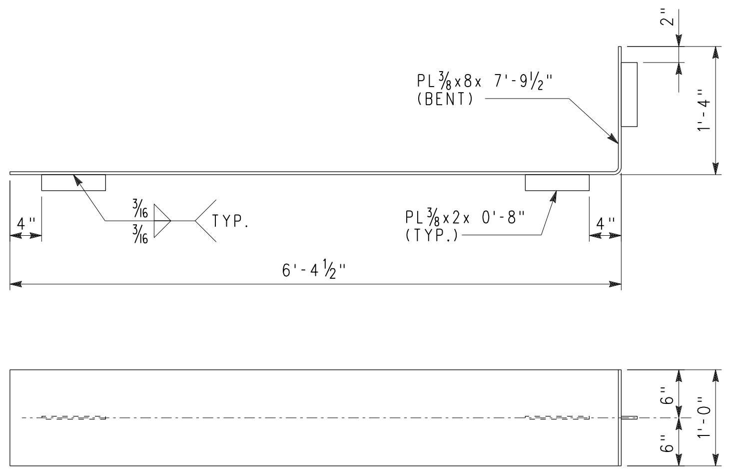


DECK PLATE 1

SCALE: 1" = 1'-0"

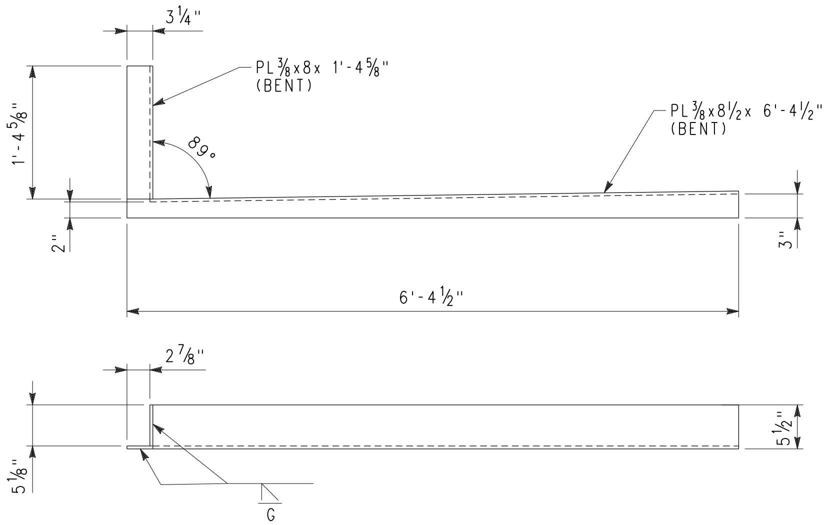
GALVANIZE AFTER FABRICATION
WEIGHT = 134 LB.



DECK PLATE 2

SCALE: 1" = 1'-0"

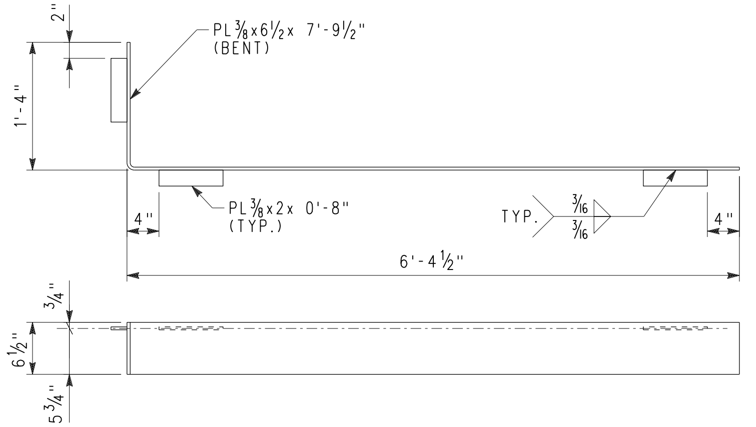
GALVANIZE AFTER FABRICATION
WEIGHT = 134 LB.



DECK PLATE 5

SCALE: 1" = 1'-0"

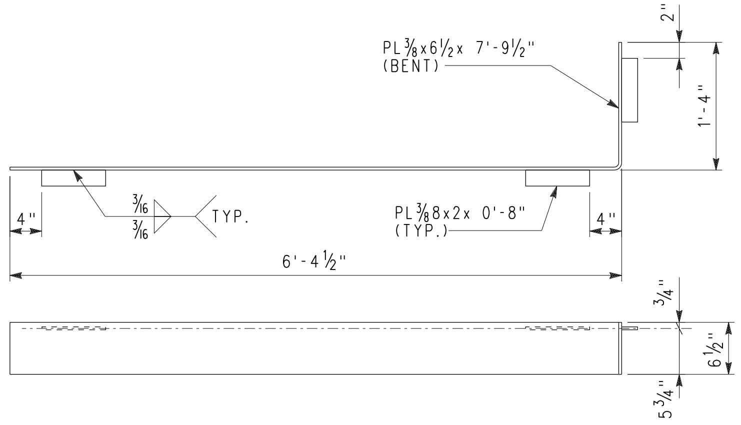
GALVANIZE AFTER FABRICATION
WEIGHT = 84 LB.



DECK PLATE 3

SCALE: 1" = 1'-0"

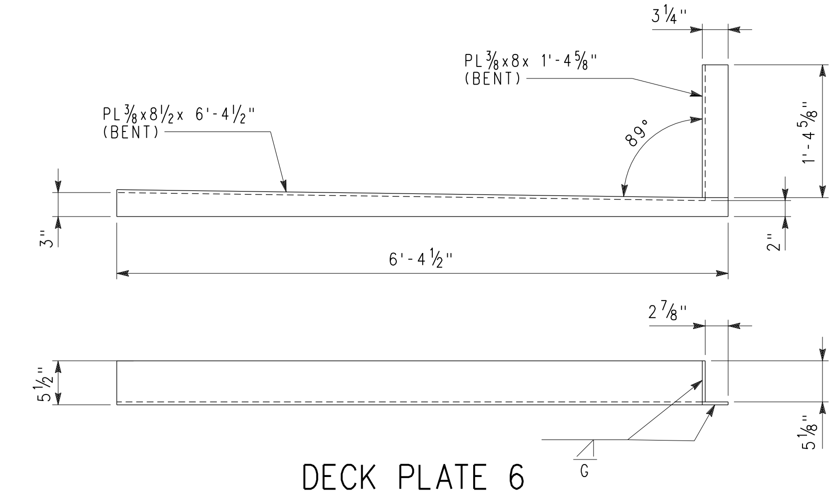
GALVANIZE AFTER FABRICATION
WEIGHT = 70 LB.



DECK PLATE 4

SCALE: 1" = 1'-0"

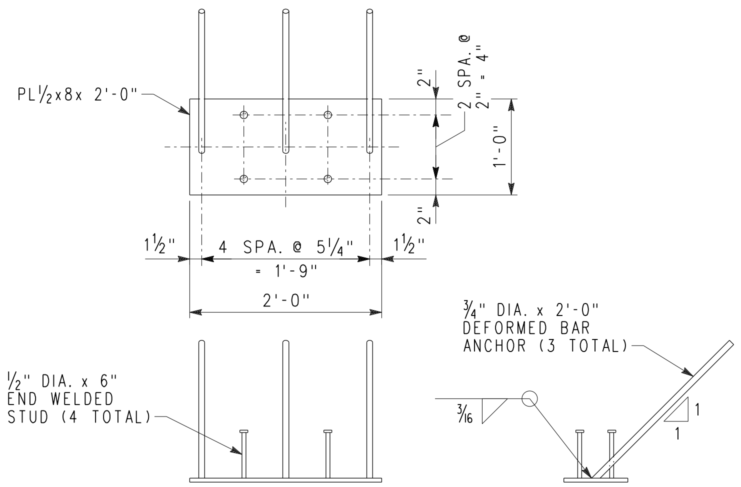
GALVANIZE AFTER FABRICATION
WEIGHT = 70 LB.



DECK PLATE 6

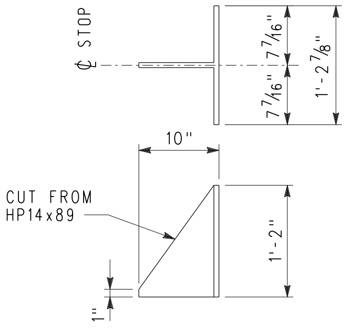
SCALE: 1" = 1'-0"

GALVANIZE AFTER FABRICATION
WEIGHT = 84 LB.



EMBED PLATE ½x8x 2'-0"

SCALE: 1" = 1'-0"



GIRDER STOP GS10

SCALE: 1" = 1'-0"

GALVANIZE AFTER FABRICATION
WEIGHT = 49 LB.

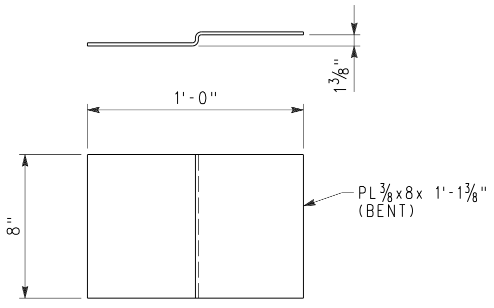


PLATE 10

SCALE: 3" = 1'-0"

GALVANIZE AFTER FABRICATION
WEIGHT = 12 LB.

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Z:\Engineering\VCTC\Sespe Creek Bridge Overflow\900 CADD\950 Drawings\S-017_Miscellaneous Details 2.sht
Y:\Microstation\CADD Standard (All Agency)\MetroLink-SCRRA\WorkSpace\Standards\Tables\Pen\PlotStamp-HalfSize.tbl
Y:\Microstation Connect\Configuration\WorkSpaces\SCRRA-Structures\Standards\Pltcfg\pdf_11x17.pltcfg

REV.	DATE		BY / SUB.	APP.

DESIGNED BY K. THOMSEN
DRAWN BY G. SMITH
CHECKED BY
APPROVED BY
DATE 10-31-2023

PROGRESS SUBMITTAL 90% NOT FOR CONSTRUCTION

VENTURA COUNTY
TRANSPORTATION
COMMISSION

RAILPROS

SUBMITTED: ______________________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

MISCELLANEOUS DETAILS NO. 2

CONTRACT NO.
DRAWING NO. S-017
REVISION
SHEET NO. 26 OF 29
SCALE AS SHOWN

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
07	VENTURA	--	423.18	3	3

REGISTERED GEOTECHNICAL ENGINEER   DATE

PLANS APPROVAL DATE

SOMADEVAN NIRANJANAN
No. GE2819
EXP 09/30/15

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

RAILPROS
250 COMMERCE STE 200
IRVINE, CALIFORNIA 92602

DIAZ YOURMAN & ASSOC.
1616 E 17TH STREET
SANTA ANA, CALIFORNIA 92705

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010).

DYB23-02

ASPHALT black; - 2 inches.

Poorly graded SAND with SILT (SP-SM); brown; moist; coarse to fine GRAVEL; coarse to fine SAND; cobbles; micaceous.

Very dense.

Difficult drilling.

SILTY, CLAYEY SAND with GRAVEL (SC-SM); dense; reddish brown; moist; coarse to fine SAND; trace coarse to fine GRAVEL; micaceous; difficult drilling.

Easy drilling.

SILTY SAND with GRAVEL (SM); medium dense; reddish brown; moist; coarse to fine GRAVEL; coarse to fine SAND; micaceous.

Very dense; difficult drilling.

No recovery.

CLAYEY SAND with GRAVEL (SC); very dense; brown; moist; coarse to fine GRAVEL; coarse to fine SAND; micaceous.

Wet.

SANDY lean CLAY (CL); stiff; light brown; wet; trace coarse to fine GRAVEL; coarse to fine SAND; micaceous.

SILTY SAND (SM); hard; reddish brown; wet; coarse to fine SAND; trace coarse to fine GRAVEL; micaceous.

Poorly graded GRAVEL with CLAY and SAND (GP-GC); very dense; brown; wet; coarse to fine GRAVEL; coarse to fine SAND; micaceous.

CLAYEY GRAVEL with SAND (GC); very dense; brown; wet; coarse to fine GRAVEL; coarse to fine SAND; micaceous.

Grades same as above.

Olive brown.

Rig chattering at 80 to 82 feet.

SILTY CLAYEY SAND WITH GRAVEL (SC-SM) very dense; very dense; brown; wet; coarse GRAVEL; coarse to fine SAND.

Poorly graded GRAVEL with SILT and SAND (GP-GM); very dense; very dense; olive brown; wet; coarse to fine GRAVEL; coarse to fine SAND.

Loss of drilling fluid; possible cobbles.

DYB23-01

Poorly graded GRAVEL with SILT and SAND (GP-GM); loose; light brown; moist; coarse to fine GRAVEL; coarse to fine SAND; loose when hand augering.

Poorly graded SAND with SILT (SP-SM); dense; olive brown; wet; coarse to fine SAND; trace coarse to fine GRAVEL; micaceous.

Poorly graded GRAVEL with SILT and SAND (GP-GM); very dense; olive brown; wet; coarse to fine GRAVEL; coarse to fine SAND; trace lean CLAY nodules; trace cobbles; micaceous.

CLAYEY SAND with GRAVEL (SC); very dense; brown; wet; coarse to fine GRAVEL; coarse to fine SAND; trace CLAY nodules; micaceous.

Well-graded GRAVEL with SILT and SAND (GW-GM); very dense; olive brown; wet; coarse to fine GRAVEL; coarse to fine SAND; micaceous; loss of drilling fluid.

Loss of drilling fluid.

Poorly graded SAND with SILT (SP-SM); very dense; brown; wet; coarse to fine SAND; trace coarse to fine GRAVEL; micaceous.

CLAYEY GRAVEL with SAND (GC); very dense; olive brown; wet; coarse to fine GRAVEL; coarse to fine SAND.

CLAYEY SAND (SC); very dense; brown; wet; coarse to fine SAND; few coarse to fine GRAVEL; micaceous; iron oxide stains.

Mottled with pale brown; no iron oxide stains.

SILTY SAND (SM); very dense; black; wet; coarse to fine SAND; trace coarse to fine GRAVEL; micaceous.

CLAYEY SAND (SC); very dense; brown; wet; coarse to fine GRAVEL; coarse to fine SAND; micacious.

Well-graded GRAVEL with SILT and SAND (GW-GM); very dense; brown; wet; coarse to fine GRAVEL; coarse to fine SAND; micaceous; iron oxide stains.

CLAYEY SAND with GRAVEL (SC); very dense; olive gray; wet; coarse to fine GRAVEL; coarse to fine SAND.

Difficult drilling and fluid loss.

Poorly graded SAND with SILT (SP-SM); very dense; olive brown; wet; fine GRAVEL; trace coarse GRAVEL; coarse to fine SAND; micaceous.

Loss of drilling fluid.

SILTY SAND (SM); very dense; olive brown; wet; coarse to fine SAND; trace coarse to fine GRAVEL.

Loss of drilling fluid.

SILTY GRAVEL with SAND (GM); very dense; pale brown; wet; coarse to fine GRAVEL; coarse to fine SAND; micaceous.

90% SUBMITTAL
NOT FOR CONSTRUCTION

REV. | DATE | BY/SUB. | APP.

DESIGNED BY A. SCHOLDER
DRAWN BY A. SCHOLDER
CHECKED BY T. REINERT
APPROVED BY T. REINERT
DATE 10–18–2023

PROGRESS SUBMITTAL 90% NOT FOR CONSTRUCTION

VENTURA COUNTY TRANSPORTATION COMMISSION

SUBMITTED: ____________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

LOG OF TEST BORINGS SHEET 1 OF 1

CONTRACT NO.
DRAWING NO. GE-001
REVISION
SHEET NO. 27 OF 29
SCALE AS SHOWN

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
07	VENTURA	--	423.18	3	3

REGISTERED GEOTECHNICAL ENGINEER DATE

PLANS APPROVAL DATE

SOMADEVAN NIRANJANAN
No. GE2819
EXP 09/30/15

RAILPROS
250 COMMERCE STE 200
IRVINE, CALIFORNIA 92602

DIAZ YOURMAN & ASSOC.
1616 E 17TH STREET
SANTA ANA, CALIFORNIA 92705

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010).

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

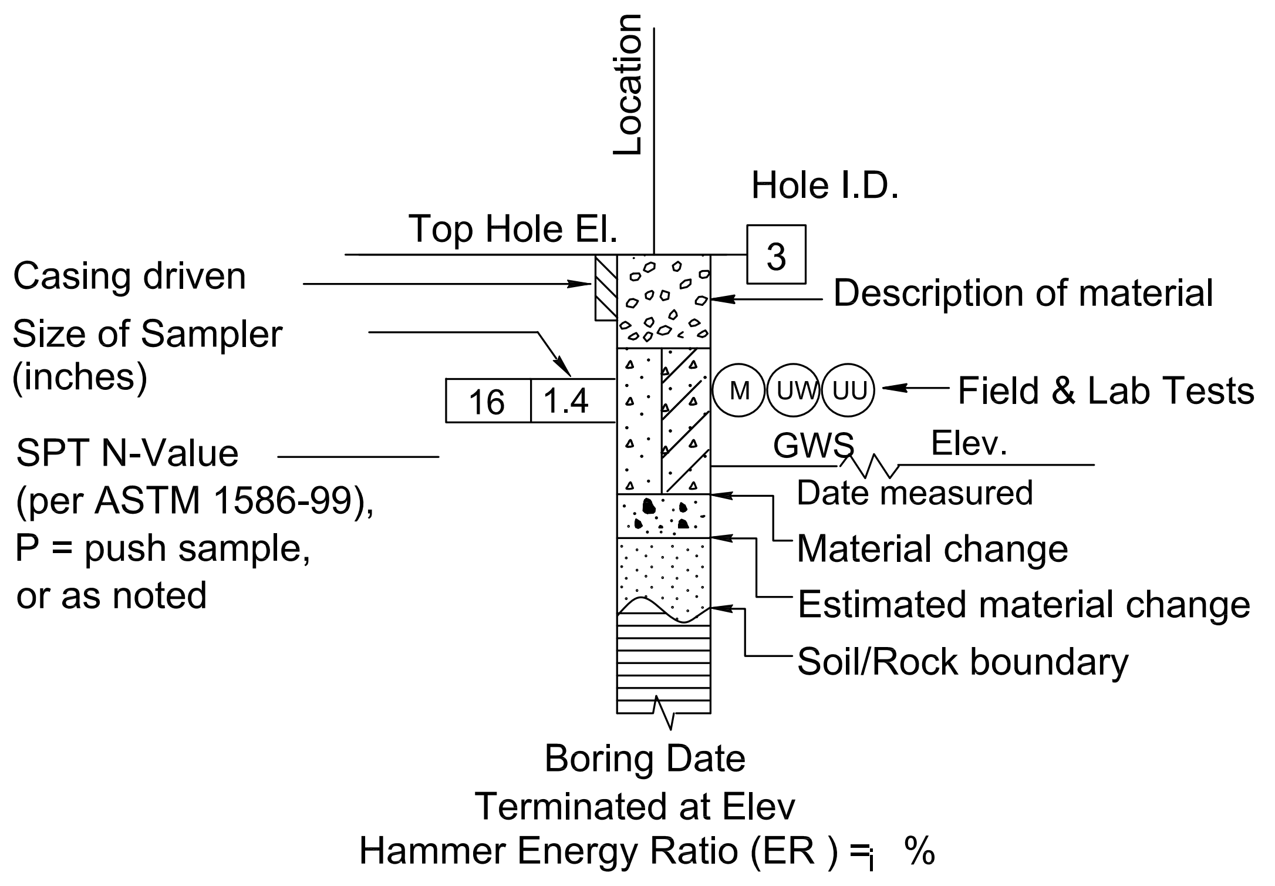
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring (hollow or solid stem bucket)
Size	R RW RC P	Rotary drilled boring (conventional) Rotary drilled with self-casing wire-line Rotary core with continuously-sampled, self-casing wire-line Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
◎	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778)
☐	O	Other (note on LOTB)

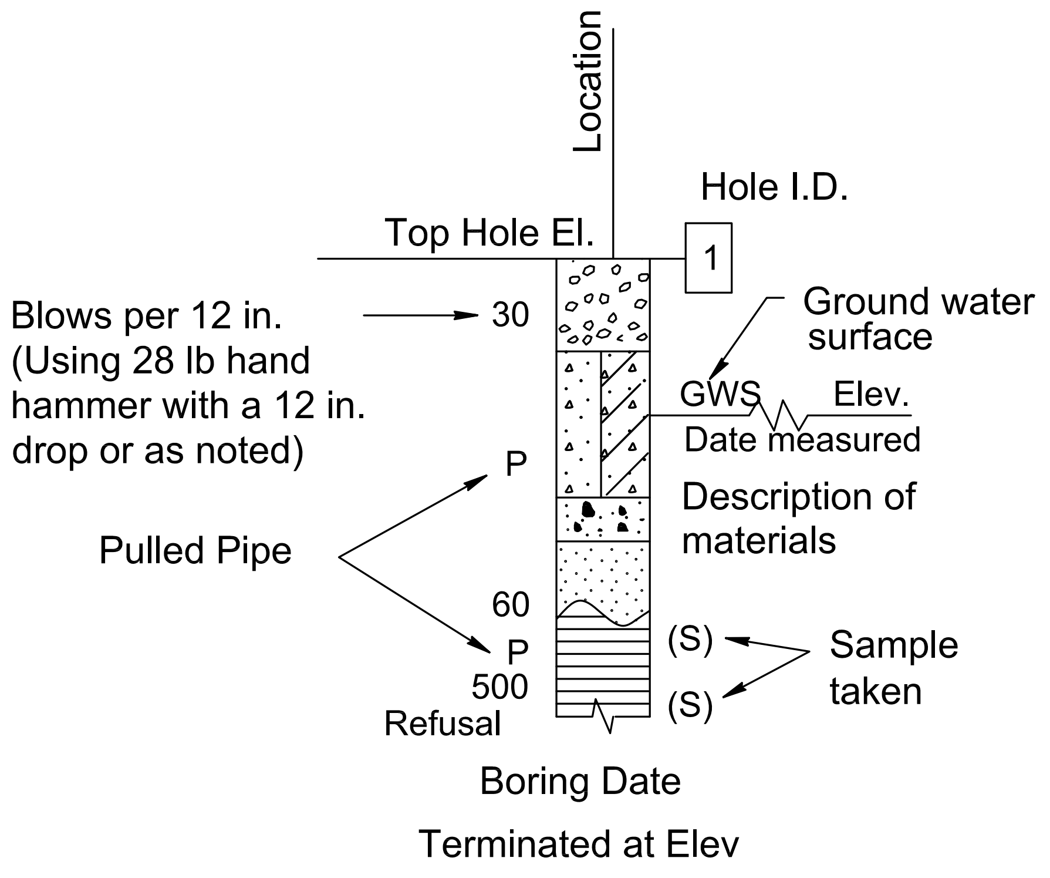
Note: Size in inches.

CONSISTENCY OF COHESIVE SOILS

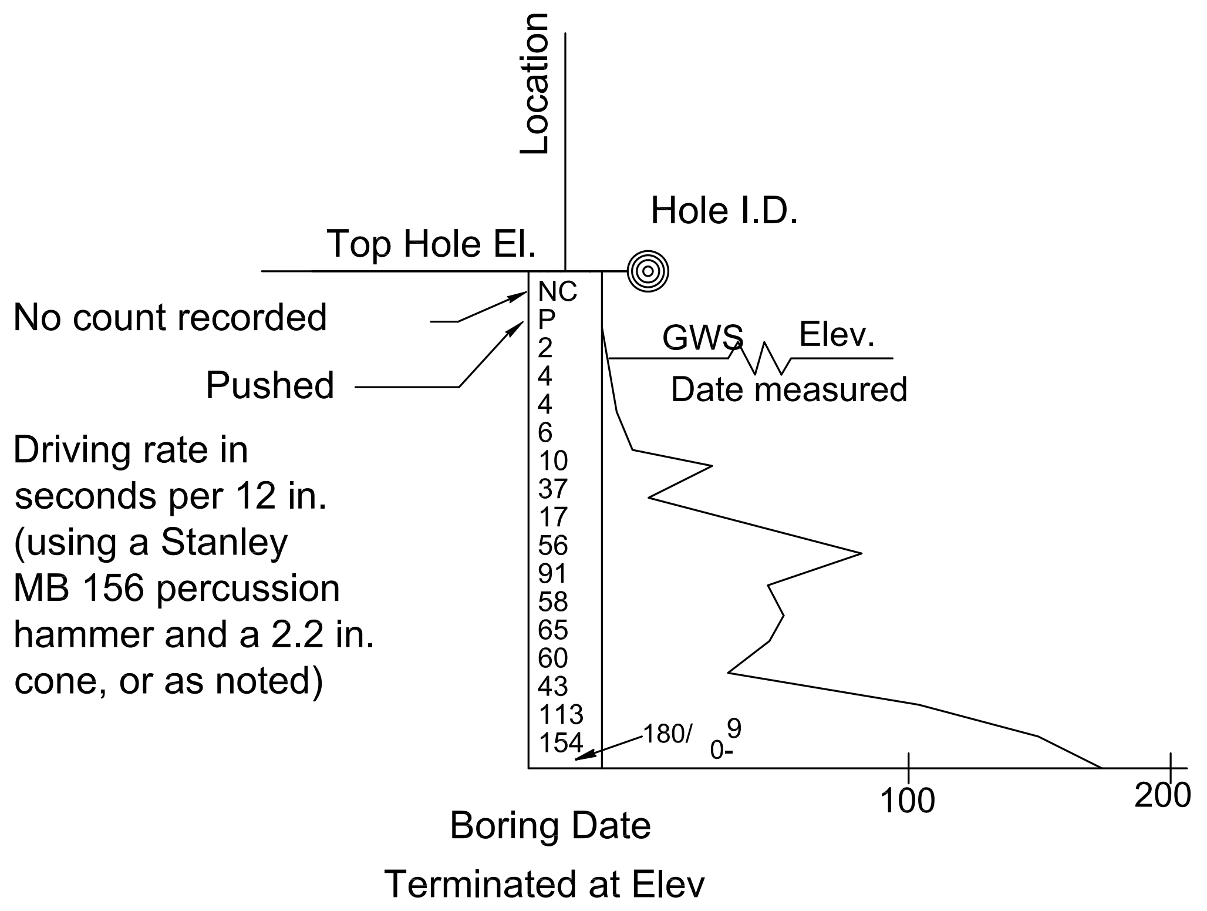
Description	Shear Strength (tsf)	Pocket Penetrometer Measurement, PP, (tsf)	Torvane Measurement, TV, (tsf)	Vane Shear Measurement, VS, (tsf)
Very Soft	Less than 0.12	Less than 0.25	Less than 0.12	Less than 0.12
Soft	0.12 - 0.25	0.25 - 0.5	0.12 - 0.25	0.12 - 0.25
Medium Stiff	0.25 - 0.5	0.5 - 1	0.25 - 0.5	0.25 - 0.5
Stiff	0.5 - 1	1 - 2	0.5 - 1	0.5 - 1
Very Stiff	1 - 2	2 - 4	1 - 2	1 - 2
Hard	Greater than 2	Greater than 4	Greater than 2	Greater than 2



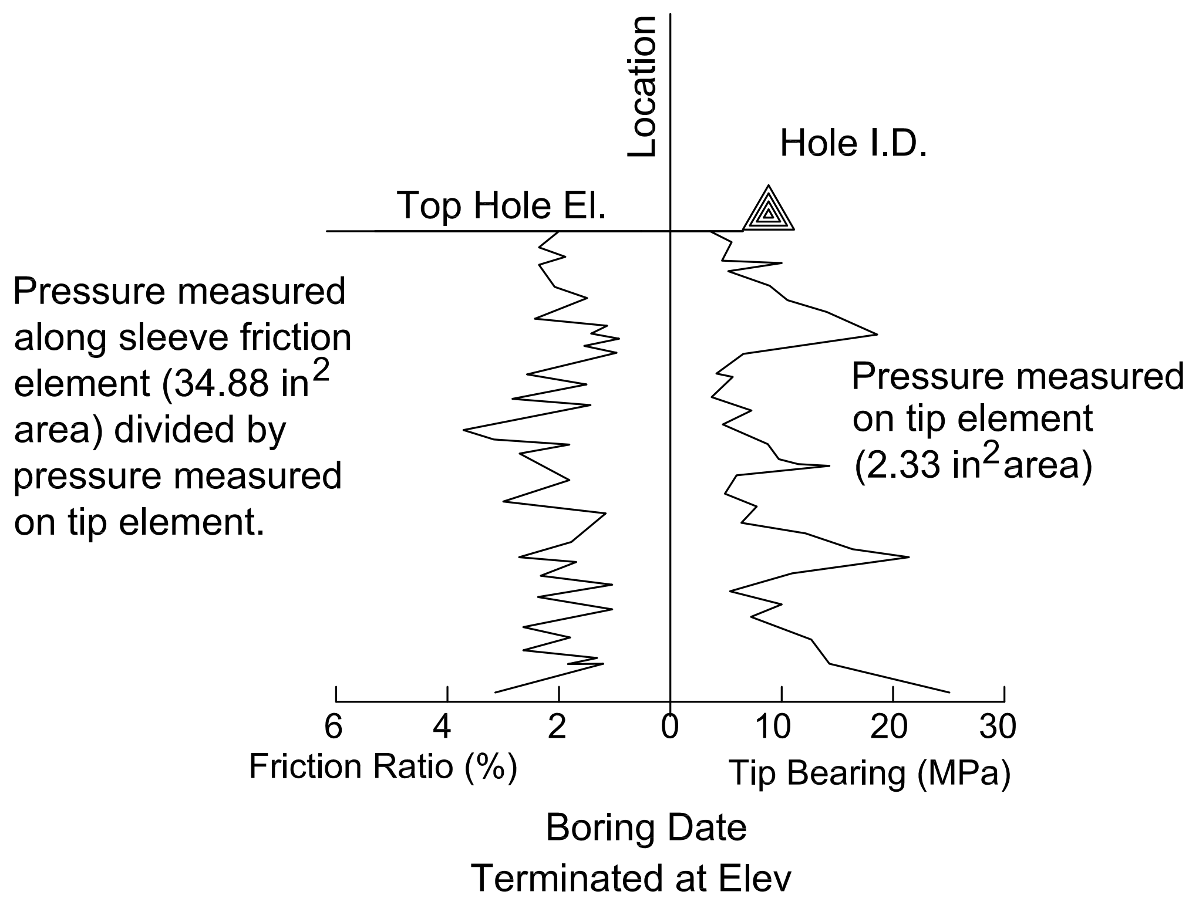
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) BORING

$USER$
$TIME$
$DATE$
$FILEL$
$PENTBLL$
$PLTDRVL$

90% SUBMITTAL
NOT FOR CONSTRUCTION

REV. DATE BY/SUB. APP.

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DESIGNED BY A. SCHOLDER
DRAWN BY A. SCHOLDER
CHECKED BY T. REINERT
APPROVED BY T. REINERT
DATE 10-18-2023

PROGRESS SUBMITTAL 90% NOT FOR CONSTRUCTION

VENTURA COUNTY
TRANSPORTATION
COMMISSION

SUBMITTED: ______________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

SOIL LEGEND 1 OF 2
LOG OF TEST BORINGS

CONTRACT NO.
DRAWING NO. GE-002
REVISION
SHEET NO. 28 OF 29
SCALE AS SHOWN

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

GROUP SYMBOLS AND NAMES

Graphic/Symbol		Group Names
▨	GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
▨	GP	Poorly-graded GRAVEL Poorly-graded GRAVEL with SAND
▨	GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
▨	GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
▨	GP-GM	Poorly-graded GRAVEL with SILT Poorly-graded GRAVEL with SILT and SAND
▨	GP-GC	Poorly-graded GRAVEL with CLAY (or SILTY CLAY) Poorly-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
▨	GM	SILTY GRAVEL SILTY GRAVEL with SAND
▨	GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
▨	GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
▨	SW	Well-graded SAND Well-graded SAND with GRAVEL
▨	SP	Poorly-graded SAND Poorly-graded SAND with GRAVEL
▨	SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
▨	SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
▨	SP-SM	Poorly-graded SAND with SILT Poorly-graded SAND with SILT and GRAVEL
▨	SP-SC	Poorly-graded SAND with CLAY (or SILTY CLAY) Poorly-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
▨	SM	SILTY SAND SILTY SAND with GRAVEL
▨	SC	CLAYEY SAND CLAYEY SAND with GRAVEL
▨	SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
▨	PT	PEAT
▨		COBBLES COBBLES and BOULDERS BOULDERS

Graphic/Symbol		Group Names
▨	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
▨	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
▨	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
▨	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
▨	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
▨	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
▨	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
▨	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
▨	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
▨	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTING

Ⓒ	Consolidation (ASTM D 2435)
(CL)	Collapse Potential (ASTM D 5333)
(CP)	Compaction Curve (CTM 216)
(CR)	Corrosivity Testing (CTM 643, CTM 422, CTM 417)
(CU)	Consolidated Undrained Triaxial (ASTM D 4767)
(DS)	Direct Shear (ASTM D 3080)
(EI)	Expansion Index (ASTM D 4829)
Ⓜ	Moisture Content (ASTM D 2216)
(OC)	Organic Content-% (ASTM D 2974)
Ⓟ	Permeability (CTM 220)
(PA)	Particle Size Analysis (ASTM D 422)
(PI)	Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
(PL)	Point Load Index (ASTM D 5731)
(PM)	Pressure Meter
Ⓡ	R-Value (CTM 301)
(SE)	Sand Equivalent (CTM 217)
(SG)	Specific Gravity (AASHTO T 100)
(SL)	Shrinkage Limit (ASTM D 427)
(SW)	Swell Potential (ASTM D 4546)
(UU)	Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
(UU)	Unconsolidated Undrained Triaxial (ASTM D 2850)
(UW)	Unit Weight (ASTM D 4767)

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
07	VENTURA	--	423.18	3	3

REGISTERED GEOTECHNICAL ENGINEER

DATE

PLANS APPROVAL DATE

SOMADEVAN NIRANJANAN
No. GE2819
EXP 09/30/15

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

RAILPROS
250 COMMERCE STE 200
IRVINE, CALIFORNIA 92602

DIAZ YOURMAN & ASSOC.
1616 E 17TH STREET
SANTA ANA, CALIFORNIA 92705

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010).

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 in.)
Very Loose	0 - 5
Loose	5 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Greater than 50

MOISTURE

Description	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visible free water

PERCENT OR PROPORTION OF SOILS

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5% - 10%
Little	15% - 25%
Some	30% - 45%
Mostly	50% - 100%

PARTICLE SIZE

Description		Size (in.)
Boulder		Greater than 12
Cobble		3 - 12
Gravel	Coarse	3/4 - 3
	Fine	1/5 - 3/4
Sand	Coarse	1/16 - 1/5
	Medium	1/64 - 1/16
	Fine	1/300 - 1/64
Silt and Clay		Less than 1/300

$USER$

$TIME$

$DATE$
$FILEL$
$PENTBLL$
$PLTDRVL$

90% SUBMITTAL
NOT FOR CONSTRUCTION

REV.	DATE		BY SUB.	APP.

INFORMATION CONFIDENTIAL: All plans, drawings, specifications, and or information furnished herewith shall remain the property of the the Southern California Regional Rail Authority and shall be held confidential; and shall not be used for any purpose not provided for in agreements with the Southern California Regional Rail Authority.

DESIGNED BY A. SCHOLDER
DRAWN BY A. SCHOLDER
CHECKED BY T. REINERT
APPROVED BY T. REINERT
DATE 10-18-2023

PROGRESS SUBMITTAL
90%
NOT FOR CONSTRUCTION

VENTURA COUNTY
TRANSPORTATION
COMMISSION

SUBMITTED: ______________
DANIELLE LIBRING, P.E., T.E.
PROJECT MANAGER

SESPE CREEK BRIDGE OVERFLOW
SANTA PAULA BRANCH LINE, FILLMORE, CA

SOIL LEGEND 2 OF 2
LOG OF TEST BORINGS

CONTRACT NO.
DRAWING NO. GE-003
REVISION
SHEET NO. 29 OF 29
SCALE AS SHOWN

Memorandum

Subject	Preliminary Historical Evaluation of the Sespe Creek Overflow Bridge, Fillmore (vicinity), Ventura County, California
Attention	Lisa Patterson, Jacobs Engineering Group, Inc.
From	Patricia Ambacher, MA, Matthew Sterner, MA, RPA, Jacobs Engineering Group, Inc.
Date	August 2, 2023

---

1. Introduction

At the request of VCTC, Jacobs Engineering Group, Inc. (Jacobs) prepared this preliminary cultural resources evaluation of the Sespe Creek Overflow Bridge for environmental compliance under Section 106 of the National Historic Preservation Act (Section 106). The project site surrounds the Sespe Creek bridge, a railroad bridge that crosses the Sespe Creek in the vicinity of Fillmore, Ventura County, California.

1.1 Project Location and Description

The Project is located where the rail line crosses Sespe Creek at approximately Mile Post 423.44, immediately east of Old Telegraph Road. The legal description of the Project location falls in Section 25, Township 4 North, Range 20 West (Figures 1 and 2).

The proposed project requires the bridge to be repaired after approximately 90-feet was washed away during winter storms in January 2023. Several piers were destroyed, leaving unsupported track spanning the northern portion of the Sespe Creek channel (Figure 3). Repairs will likely be undertaken from a combination of track and channel construction efforts.

2. Regulatory Context

The proposed project needs a U.S. Army Corps of Engineers Section 404 permit, which requires the project to comply with Section 106.

2.1 Section 106 of the National Historic Preservation Act

The project is subject to the requirements of Section 106, as amended and its implementing regulations (Title 36 Code of Federal regulations [CFR], Part 800 [36 CFR 800] (Section 106). Section 106 calls for considerable consultation with the State Historic Preservation Officer (SHPO), Native American tribes, and interested members of the public throughout the process. The four key steps of the Section 106 process are as follows:

1. Initiate Section 106 (36 CFR 800.3).
2. Identify historic properties, resources eligible for inclusion in the National Register of Historic Places (NRHP) (36 CFR 800.4).
3. Assess the effects of the undertaking to historic properties in the APE (36 CFR 800.5).
4. Resolve adverse effects (36 CFR Part 800.6).

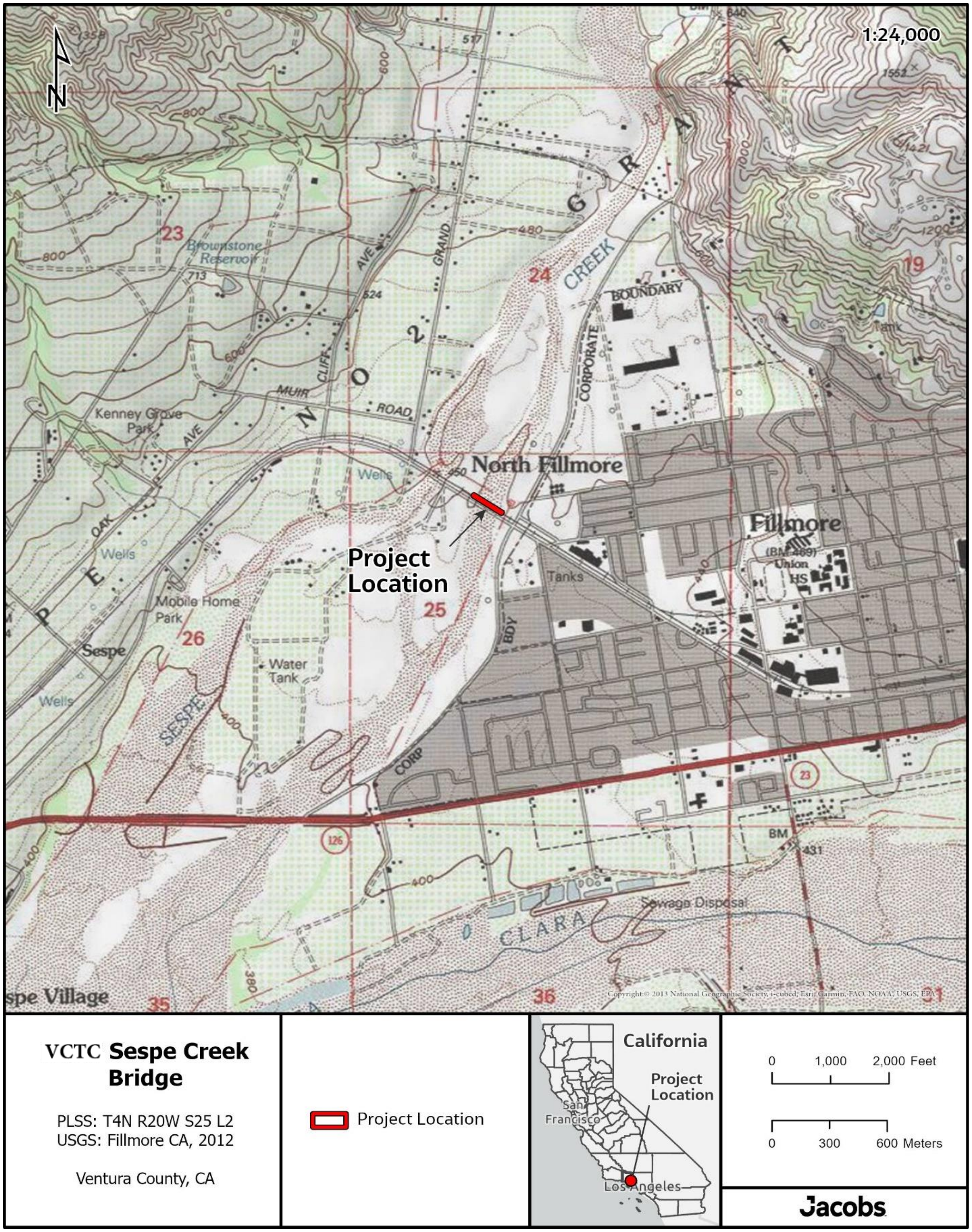

Figure 1. VCTC **Sespe Bridge Location.**

Figure 2. Aerial View of VCTC Sespe Creek Bridge.

Figure 3. Photographs of VCTC Sespe Creek Bridge, January 2023.

2.1.1 National Register of Historic Places

The NRHP is the nation's official list of buildings, structures, objects, sites, and districts in the U.S. that are significant in American history, architecture, engineering, archaeology, and culture. A property must be at least 50 years old to be evaluated for eligibility, or it must possess exceptional significance. The criteria used to evaluate historic properties for inclusion in the NRHP are summarized below:

A. Event – Properties associated with events that have made a significant contribution to the broad patterns of our history.

B. Person – Properties associated with the lives of persons significant in our past.

C. Architecture/Engineering – Properties that embody the distinctive characteristics of a type, period, or method of construction, or that represent the work of a master, or that possess high artistic values, or that represent a significant and distinguishable entity whose components may lack individual distinction.

D. Archaeology – Properties that have yielded, or may be likely to yield, information important in prehistory or history.

In addition to meeting one of these evaluation criteria a historic property must retain integrity in order to convey its significance. Integrity is measured by seven aspects:

- Location – The place where the historic property was constructed or where the historic event occurred.
- Design – The combination of elements that create the form, plan, space, structure, and style of the property.
- Setting – The physical environment of the historic property.
- Materials – The physical elements that were combined or deposited during a particular period of time and in a particular pattern or configuration to form the historic property.
- Workmanship – The physical evidence of the crafts of a particular culture or people during any given period in history or prehistory.
- Feeling – The property's expression of the aesthetic or historic sense of a particular period of time.
- Association – The direct link between an important historic event or person and a historic property.

2.1.2 Effects Assessment

If historic properties are identified, it must be determined if they will be adversely affected by the undertaking. The federal agency shall assess adverse effects, if any, in accordance with the Criteria of Adverse Effect (36 CFR 800.5(a)1). Adverse effects on historic properties include, but are not limited to:

1. Physical destruction of or damage to all or part of the property.
2. Alteration of the property, including restoration, rehabilitation, repair, maintenance, stabilization, hazardous material remediation, and provision of handicapped access, that is not consistent the SOI's Standards for the Treatment of historic Properties (36 CFR 68) and applicable guidelines.
3. Removal of the property from its historic location.

4. Changing the character of the property's use of physical features within the property's setting that contribute to its historic significance.
5. Introduction of visual, atmospheric, or audible elements that diminish the integrity of the property's significant historic features.
6. Neglect of the property that causes its deterioration, except where such neglect and deterioration are recognized qualities of the property of religious or cultural significance to Native American tribe or Native Hawaiian organization.
7. Transfer, lease, or sale of the property out of federal ownership or control without adequate and legally enforceable restrictions or conditions to ensure long-term preservation of the property's historic significance.

Adverse effects on historic properties often are resolved through preparation of a memorandum of agreement or a programmatic agreement developed in consultation with the lead federal agency, SHPO, Native American tribes, and interested members of the public. The Advisory Council on Historic Preservation is also invited to participate.

3. Historic Context

To accurately assess the potential historical significance of the Sespe Creek Overflow Bridge, the following section provides an evaluative historic context covering the themes: Ventura County and Fillmore's development, railroad development, and railroad bridge construction and design.

3.1 Ventura County and Fillmore Development

Ventura County was established in 1873 and Ventura was the county seat (Hoover and Kyle 1990). At that time, the county's population was more than 3,000 and agriculture was the principal source of revenue. By the early 1900s, the oil industry joined agriculture as an economic driving industry for Ventura County (County of Ventura 2023). During World War II, the U.S. military took temporary control of Port Hueneme for the transportation of cargo to support the war effort. Like most of California, Ventura experienced a population boom after World War II and the county's boundaries expanded. By 2022, agriculture remains an important industry for Ventura County in addition to healthcare and the technology industries.

Fillmore was established by the Southern Pacific Railroad (SPRR) and was named for the company's general superintendent, Jerome Fillmore. The SPRR and the Sespe Land and Water Company promoted Fillmore's location and in 1888, a street map of the town was recorded with Ventura County. By 1900, Fillmore had a population of 150 (Fillmore Historical Museum 2023a). The train depot became the center of the Fillmore community bringing new settlers who began planting orchards and crops. Agriculture became a leading industry for the Fillmore (Fillmore Historical Museum 2023b).

3.2 Railroad Development

Railroads were developed in the U.S. during the early nineteenth century and the earliest U.S. railroads were constructed without strict government oversight, which resulted with the highest density of tracks appearing in the northeast (Library of Congress 2018). Over time, the need for increased railroad connectivity from the east to the west coast was required if increased trade, settlement, prosperity, and population growth were to occur in the new western frontier (American-Rails 2021).

The California Gold Rush attracted a mass of people westward subjecting them to arduous travel by ship or wagon (Bennett 1915; History.com 2021). An interconnected railroad network throughout the country was necessary to revolutionize transportation and westward expansion (Bennett 1915). The completion of the transcontinental railroad became a reality when engineer Theodore Judah petitioned an agreement from investors Collis P. Huntington, Mark Hopkins, Charles Crocker, and Leland Stanford, known as the Big Four, to establish the Central Pacific Railroad (CPRR) in Sacramento (Galloway 1989). The passage of the Pacific Railroad Act of 1862 and the western and eastern routes secured, Judah could move his plans with CPRR forward. The Union Pacific Railroad (UPRR) was formed to build the middle route through the Rocky Mountains (Borneman 2010). The transcontinental railroad was completed in 1869 when the two railroads met at Promontory Point in Utah.

The period between the 1880s and 1920s is often referred to as the “Golden Age” of railroads, as the industry experienced an era of profitability and expansion. By 1916, there were more than 254,000 miles of track nationwide (American-Rails 2021). During the 1930s, also referred to as the “Silver Age,” streamliner design attempted to breathe new life into the railroad. Designers used new technologies and methods to promote new engines, including diesel, as a last-ditch attempt to retain passenger traffic (Solomon 2008). The aerodynamic machines fell out of favor by the 1950s, as passengers opted for the personal automobile.

3.2.1 Southern Pacific Railroad

The Southern Pacific Railroad (SPRR) was started as a branch line from San Francisco to San Diego by the Big Four and was incorporated in 1865 (UPRR 2023). That same year, the SPRR took operational control of the CPRR (Yenne 1996:51). By 1877, these two railroads controlled more than 85 percent of all the railroads in California that totaled more than 2,300 miles of track (Daggett 1922:140). The CPRR eventually merged with the SPRR in 1959 and the SPRR merged with the Union Pacific Railroad (UPRR) in 1996 (UPRR 2023).

The SPRR’s coastal route, the Coast Line, between Los Angeles and San Francisco began in 1887 (County of Ventura 2023). The preferred route was through the Santa Clara River Valley to Ventura with train stops situated approximately 10 miles apart. The railroad also wanted to be far enough from the Santa Clara River and its tributaries to prevent flooding (Fillmore Historical Museum 2023). Sugar beets, citrus fruits, and beans were popular railroad freight and the railroad connected to the wharf along the Santa Barbara Channel, which would become Port of Hueneme (Ventura County 2023).

An approximately 30-miles long segment of the former Southern Pacific branch extends from the eastern edge of the city of Ventura to the town of Piru to the east. It eventually became known as the Santa Paula Branch after mainline Coast Line traffic was rerouted following the opening of the Santa Susana Tunnel in 1904. The right-of-way originally extended farther into the Santa Clarita Valley to Saugus, where it connected with SP’s line to the San Joaquin Valley via Tehachapi Pass (Lustig 2021a).

The VCTC bought the Santa Paul Branch Line railroad in 1995 (VCTC 2023). For nearly 30 years 13 miles was used by a contractor, Fillmore & Western, which operated a tourist train, and used it for movie and television filming of the branch line for (Lustig 2021b). In 2021, VCTC entered into a 35-year Railroad Lease and Operations Agreement with Sierra Northern Railway, a county-owned line between Ventura and Fillmore (VCTC 2023).

4. Railroad Bridge Construction and Design

In 1840 and 1846, the emergence of the Howe Truss patents propelled railroad bridge design in a new direction. Other patents soon followed, such as that of father and son team Caleb and Thomas Pratt who received a patent in 1844, and James Warren who patented a design in 1848 in England. All patents were variations on how the trusses transferred loads or weight. These new designs would favor cast and wrought iron members over timber, and Pratt and Warren truss designs, or a variation of them, became standard by the late nineteenth century (Solomon 2008).

The U.S. railroad and steel industries emerged during a crucial time in western expansion. Steel quickly became the favored material. The development of the steel industry produced new designs to support the demand for longer, taller, and stronger bridges. Companies started to mass-produce steel bridges using standardized plans and patented truss designs, which resulted in common steel truss bridge types throughout the country (Kramer 2004).

Bridges with movable spans were a practical solution to the problem of how to cross a waterway while maintaining access for ship traffic. As early as the mid-nineteenth century, movable span bridges were constructed using timber truss systems that rotated on a central pier. These early swing bridges were manually operated (Jensen, n.d.). The introduction of steel spans and electric motors allowed for the development of larger, more permanent swing span bridges. By 1870, the center pivot swing span became one of the more common types of movable span steel bridges constructed in the U. S. (Jensen, n.d.; Mead & Hunt 2009).

By the early- to mid-twentieth century, the dominance of the swing span design had diminished in favor of other movable bridge types, such as bascule and vertical lift designs (Parsons Brinkerhoff and Engineering and Industrial Heritage 2005). By the 1920s, new swing span bridges were “virtually obsolete” around the country, although “because of their basic economy of materials and simplified construction, the swing bridge was utilized during the Depression for large work-relief bridge projects” (Jensen, n.d.).

In 1950, the Portland Cement Association began load testing a prestressed concrete railway trestle slab in their laboratories in Illinois. Full scale load tests began in October 1953 by the Association of American Railroads on a 19-foot prestressed concrete trestle slab in Colorado. In 1954, the first prestressed concrete railroad bridge was in use in the U.S. Between 1954 and 1957, engineers designed and constructed longer spans using slabs and boxes to replace timber trestles. These early spans were 20 to 30 feet and were typically supported on prestressed concrete piles. Prestressed concrete box girder bridges became commonplace in railroad bridge design for the next 30 years (Goldberg 1983).

5. Eligibility Determination

5.1 Description

This description is based on photographs and bridge inspections reports from 2021 (Wilson & Company 2021).

The bridge is a concrete double box girder constructed in 1969 (Wilson & Company 2021). Its total length is 450 feet with a vertical clearance of 14 feet. Its 15 spans are 29 feet-10 inches each (VCTC 2023). It has a ballast bridge deck with timber ties and concrete abutments (Wilson & Company 2021). The bridge was partially washed away (see Figure 3) during heavy rain storms in the early part of January 2023 (*Ventura County Star* 2023). Three spans, approximately 90 feet, on the western end of the bridge were washed out.

5.2 Evaluation

The Sespe Creek Overflow Bridge does not meet the criteria for the NRHP because of a lack of significance and integrity. Under Criterion A, this bridge did not play a significant role in the railroad development or in the development of Fillmore or Ventura County. The bridge was constructed in 1969, which is well past the peak of the railroad's influence on the development of the region. Therefore, it does not meet Criterion A. Research did not reveal that the bridge is associated with individuals who made significant contributions to history as required under Criterion B. Under Criterion C, the bridge is a common example of a double box girder prestressed concrete bridge. A bridge type introduced in railroad engineering and design in the early 1950s. It lacks high artistic values and research did not reveal it was designed by a master engineer. In consideration of all the elements of Criterion C, this bridge is not significant. As a built environment resource, it is not the sole source of important information to history and does not meet Criterion D.

In addition to not meeting the NRHP evaluation criteria, the loss of 90 feet of the bridge has impacted its integrity of design, materials, and workmanship. Integrity of materials was previously affected by the repairing and replacing of ties and some ballast tamping (Lustig 2021b). It retains integrity of location, setting, feeling, and association.

An archaeological evaluation surrounding the bridge location has not been performed to date. That said, Sespe Creek is a highly active watercourse with demonstrated high-energy scouring events, indicating that the likelihood of *in situ* archaeological materials remaining in the immediate vicinity of the bridge is extremely low to non-existent. Construction efforts associated with repair or replacement of the VCTC Sespe Creek Bridge presents little to no risk to archaeological resources.

6. Conclusion

The Sespe Creek Overflow Bridge, constructed in 1969, lacks historical and engineering significance and integrity. Based on the present evaluation, the bridge is recommended not eligible for listing in the NRHP. While this report presents preliminary findings only, adequate information has been presented to recommend a finding of **no historic properties affected** for the project under Section 106. This memorandum further concludes there are no further management recommendations needed.

7. References

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A Report Prepared for:

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GEOTECHNICAL REPORT
RECONSTRUCT A PORTION OF THE SESPE CREEK OVERFLOW RAILROAD BRIDGE
CITY OF FILLMORE, CALIFORNIA

Project No. 2023-010

by

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October 26, 2023

REVISION NO.	DATE	REVISION DESCRIPTION
Draft V1	10/13/2023	Preliminary Draft for Internal Review – 90% Design
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QUALITY CONTROL REVIEWER

Saroj P Weeraratne, PhD, PE, GE
Principal Engineer

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1 INTRODUCTION

This report presents the results of the geotechnical design services performed by Diaz•Yourman & Associates (DYA) in connection with planning, design, and environmental compliance to reconstruct the Sespe Creek Overflow Railroad Bridge on the Santa Paula Branch Line services ("Project"). The geotechnical services were performed to provide professional services to Ventura County Transportation Commission (VCTC) ("Owner" and “Client”) with DYA as a subconsultant to RailPros. RailPros authorized this work on June 19, 2023, with a written contract.

The Sespe Creek Overflow Railroad Bridge (Bridge) is located at approximately Mile Post 423.44, west of Fillmore, California, as shown on the Vicinity Map.

Figure 1. In early January, heavy rain, stream flow, and debris accumulated during a series of storms and washed out three spans, or approximately 90 feet, of the Bridge. Three spans on the western end of the Bridge were destroyed and require reconstruction to restore pre-disaster design, capacity, and function to resume rail services on the Bridge. Additionally, an earthwork abutment was partially washed out and will be replaced with a concrete abutment and wingwalls.

The approximate layout of the Project improvements is shown on the Site Plan, Figure 2. Project drawings (Railpros, 2023a) are presented in Appendix A.

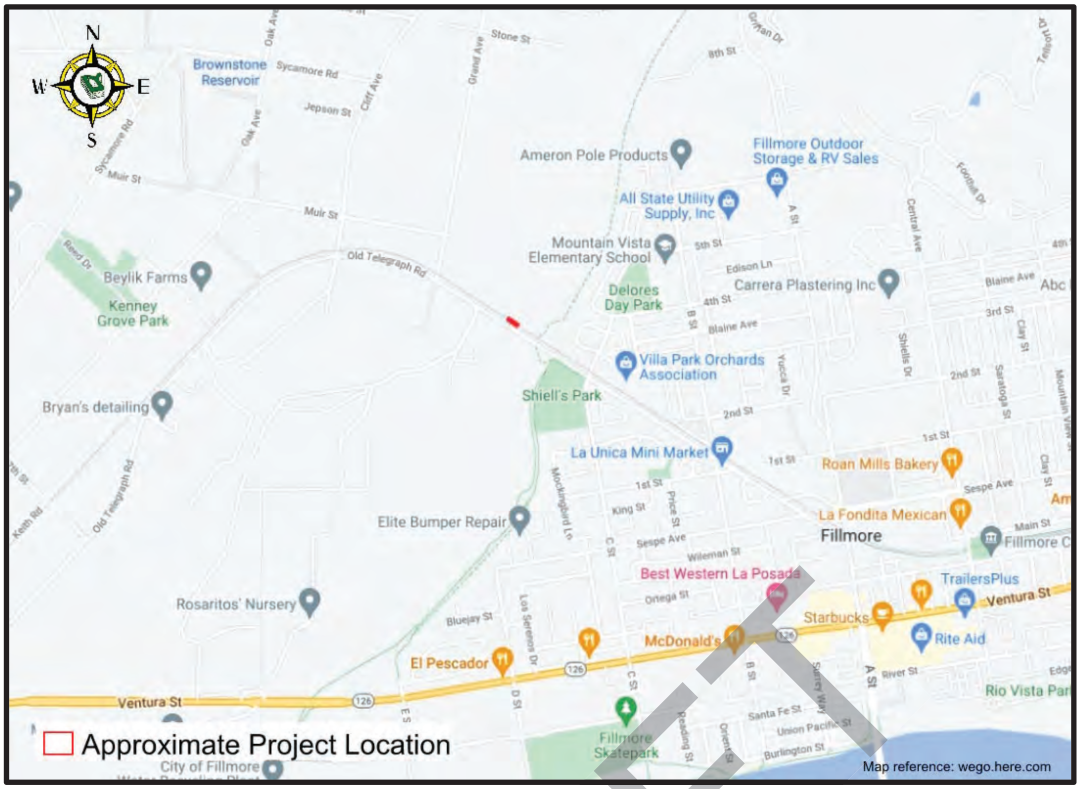

Figure 1 - VICINITY MAP

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Figure 2 - SITE PLAN

The purpose of DYA's services was to provide geotechnical input for the design of the Project. The scope of our services consisted of the following tasks:

- Reviewing existing geotechnical and geological data.
- Conducting a limited field exploration.
- Performing limited laboratory tests on selected soil samples.
- Performing engineering analyses to develop conclusions and recommendations regarding the following:
  - Subsurface conditions
  - Geologic and seismic hazards
  - Site preparation and grading
  - Foundation types and deep foundations
  - Estimated total and differential foundation settlement
  - Resistance to lateral loads
  - Lateral earth pressures
  - Soil corrosion potential
- Preparing this report.

Engineering analysis is restricted to the bents and abutment that have currently been observed to have failed. Further analyses for the existing other bridge bents and abutment were not within DYA's scope. Our scope of services also specifically excluded any investigation needed to evaluate the presence or absence of hazardous or toxic materials at the site in the soil, surface water, or groundwater.

2 DATA REVIEW, FIELD EXPLORATION, AND LABORATORY TESTING

The information provided in this report is based on DYA's review of the available regional geologic maps, existing subsurface and groundwater data gathered in the Project vicinity, a limited field exploration, limited laboratory testing, and discussions with Project designer members. Available Caltrans logs of test borings (LOTBs) for the Old Telegraph Road Bridge (Moore and Taber, 1982), which is located adjacent to the failed Bridge, are presented in Appendix B. A list of the documents reviewed is presented in the bibliography (Section 7).

The field exploration, conducted from July 17 through July 26, 2023, consisted of drilling two borings using rotary-wash techniques, each to a depth of approximately 100 feet. The boring locations are shown on Figure 2. One boring (DYB23-02) was drilled on the shoulder of Old Telegraph Road near the location of the washed-out abutment, and the second boring (DYB23-01) was drilled within the Sespe Creek bed near the location of the washed-out bents. As the stream is active in the location of the two washed-out bents, our field exploration was limited to the vicinity of the existing abutment and remaining interior bent. Prior to drilling, the borings were marked and underground service alert (USA) was contacted in order to mark out utility locations. A geophysical survey was also performed prior to drilling to locate any further utilities. Due to the shallow groundwater conditions anticipated at the site, mud-rotary wash drilling techniques were implemented for the field exploration. Because of the difficult access conditions to the channel bottom, a track-mounted, mud rotary wash drill rig was used for the field exploration. In order for the track-mounted, mud rotary drill rig to access the boring location within the creek bed, a pathway was created using a skip loader to move aside cobbles and boulders within the creek bed. Traffic control was provided during drilling and geophysics activities on the roadway. The field exploration implemented standard penetration testing (SPT) to obtain and collect subsurface data and samples for geotechnical engineering properties. Details of the field exploration, including sampling procedures and borings, are presented in Appendix C.

Because of the restrictions to access across the channel bed, drilling deep borings using a drill rig was not possible at the failed bent locations within the three spans on the western end of the Bridge. Therefore, a seismic refraction survey was also performed across the channel bed along the western edge of the Bridge. The location(s) of these seismic refraction survey lines are shown on Figure 2. The purpose of the survey was to develop subsurface velocity profiles of the site and to characterize the subsurface soils at deeper depths (depths deeper than 20 feet) and possibly to estimate the depth to bedrock at the failed bent locations. The refraction survey seismic profiles (Atlas, 2023) are shown in Appendix D.

Soil samples collected from the borings were re-examined in the laboratory to substantiate field classifications. Selected soil samples were tested for moisture content, dry density, grain-size distribution, Atterberg limits, shear strength, and corrosion potential (pH, electrical resistivity, soluble chlorides, and soluble sulfates). The soil samples tested are identified on the boring logs. Laboratory test data are summarized on the boring logs in Appendix C and presented on individual test reports in Appendix E.

3 SITE CONDITIONS

3.1 REGIONAL GEOLOGY

The Project site lies within the east Ventura basin portion of the western Transverse Ranges named for their east-west orientation, roughly perpendicular to most of California’s mountain ranges. The east Ventura Basin is generally east-west trending and contains the Santa Clara River into which Sespe Creek drains near Fillmore (

Figure 1 - Regional Geology; Bedrossian and Roffers, 2012). Sespe Creek (including the Project site) contains young wash (river) deposits (map symbol Qw) and is bordered on the west by younger (Qya) and older alluvium (Qoa), and younger alluvial fan (Qyf) deposits. East of Sespe Creek is predominantly Qyf and shale (Tsh) bedrock.

Southern California is a seismically active region with many faults, some of which are capable of producing large-scale earthquakes of approximately 7.0 to 8.0 magnitude (M) on the Richter scale. One such Holocene active fault (Figure 2 - Regional Fault Map; California Geological Survey [CGS], Fault Activity Map website, 2023a) is the San Cayetano Fault that borders the bedrock approximately 5,000 to 10,000 feet east of the Project site and approximately 12,500 feet west of the site. Such earthquakes can trigger severe ground shaking, possible surface fault rupture near the fault, and liquefaction in loose, unconsolidated soils in areas of shallow groundwater.

3.2 LOCAL GEOLOGY

The Project alignment lies within the east Ventura physiographic basin, which is part of the Transverse Ranges geomorphic province. The Santa Clara River-Sespe Creek area of the east Ventura Basin is alluviated lowland that is bound to the north by the Topatopa Mountains and on the south by the Santa Susana Mountains and by South Mountain. The Project site railroad bridge alignment area is mainly mapped as Holocene alluvial wash deposits (Qw) and young alluvium deposits (Qya; Figure 3- Project Site Geology Map).

Qw deposits, beneath the eastern three-quarters of the alignment, are composed of unconsolidated gravel and sand deposits in the active channel deposited from upstream sources in the valley which may contain loose to moderately loose sand and silty sand. Qya deposits, beneath the western one-quarter, are unconsolidated to moderately consolidated boulder, cobble, gravel, sand, and silt deposits. Logs of two test borings (LOTBs B-1 and B-2; Moore & Taber, 1982) just south of the Project alignment indicate that the Qw deposits are 5- to 10-feet thick and consist of coarse gravel, cobbles, and boulders with a medium to coarse sand matrix. LOTB B-1 encountered groundwater at a depth of approximately 40 feet indicating Qya deposits may be susceptible to liquefaction because this is a seismically active region (California Geological Survey, Earthquake Zones of Required Investigation website, 2023b). The nearby San Cayetano reverse fault is believed to be capable of at least a 7.2 magnitude earthquake (Dolan, 2009; Olsen, 2021).

The surface geology units mapped at this site are shown on Figure 3.

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3.3 SURFACE CONDITIONS

At the time of our exploration, two piers of the Bridge had been washed out with a third being pushed out of plumb. The west Bridge abutment was also in the process of failure from erosion. The other intact bridge piers also had a significant buildup of tree debris which may cause significant lateral pressures in the event of another flood. The riverbed was mostly uneven, with numerous small to large boulders. The riverbed had an active stream flowing on the west edge between the west-most pier and the adjoining abutment. The roadway on Old Telegraph Road was in relatively good condition with no noticeable potholes or significant cracks.

3.4 SUBSURFACE CONDITIONS

Based on our limited field exploration, the subsurface soils were significantly difficult to drill through due to the various large-sized boulders encountered and the significant fluid loss experienced. Subsurface soils were primarily sandy gravels, clayey gravel, and silty clayey sands with gravel.

Approximately 20 feet of dense sand and silty sand were present at the abutment location. A five-foot-thick lean clay layer was present at elevation 412 to 407 at the abutment location only. The bottom of the creek bed was estimated to be at elevation 430 feet based on the North American Vertical Datum (NAVD88).

The thicknesses of the different subsurface materials at the abutment location and the channel bottom were idealized along the bridge improvement alignment are presented in Table 1 - IDEALIZED SOIL PROFILE – SESPE CREEK

Note that due to the geological depositional nature of the soils in the creek bed over time, the layers reported in Table 2 may not be present at the same thicknesses at all locations. The site is highly variable with layers of boulders, cobbles, and gravel, and those materials can be encountered at any depth.

Table 1 - IDEALIZED SOIL PROFILE – SESPE CREEK

SOIL LAYER^{1,2}	ELEVATION^{3} (feet)	DEPTH (feet)	TOTAL UNIT WEIGHT (pcf)	SHEAR STRENGTH		
				Total	Effective	
				S_u (psf^3)	ϕ' (degrees)	c' (psf)
Poorly-Graded Sand with Silt (SP-SM); Silty Sand (SM); ABUTMENT FILL	450 to 430	0 to 20	120	--	34	50
Poorly-Graded Sand with Silt and Gravel (SP-SM); Silty Sand with Gravel (SM); Clayey Sand with Gravel (SC); Poorly-Graded Gravel (GP); CREEK BED	430 to 412^4	20 to 38	125	--	38	50
Silty Sand with Gravel (SM); Clayey Sand with Gravel (SC); Lean Clay with Sand and Gravel (CL)^5	412 to 407	38 to 43	125	2,000^5	38	50
Poorly-Graded Gravel with Silt and Sand (GP-GM); Clayey Sand with Gravel (SC); Silty Sand with Gravel (SM)	407 to 378	43 to 72	125	--	38	50
Clayey Gravel with Sand (GC); Silty, Clayey Gravel with Sand (GC-GM); Silty Sand with Gravel (SM)	378 to 330	72 to 120	125	--	38	50

Note(s):

1. Unified Soil Classification System.
2. Soils are not homogeneous and not in layers. Simplified geotechnical design profile was developed considering the proposed lightly loaded structures and subsurface conditions encountered at the site.
3. Elevation based on NAVD88.
4. Groundwater encountered at an elevation of 423 feet.
5. The 5-foot sandy lean clay layer at elevation 412 to 407 applies to the Abutment 1 location only.

- pcf = pounds per cubic foot.
- The site is highly variable with layers boulders, cobbles, and gravel, and those materials can be encountered at any depth.
- This profile can be used for both the abutments and the bents. See Note 5 for the layer that corresponds to the abutment location only.

3.5 GROUNDWATER LEVEL

Groundwater was encountered during the field exploration in Boring DYB23-01 at 7 feet bgs (elevation 423 feet) and in Boring DYB23-02 at 35 feet bgs (elevation 415 feet). The depth to the historically highest groundwater level near the Project site has been reported to range from 10 to 20 feet (CGS, 2002a). Based on information obtained from the Caltrans LOTBs (Appendix B), the groundwater level was reported at an elevation of 387 feet dating back to 1982 (see Appendix B for details of groundwater elevations encountered). Therefore, the design depth to groundwater was assumed to be at an elevation of 423 feet. Accordingly, design groundwater depth was assumed to be at 7 feet bgs within the creek bed. Note that seasonal variations in water level may occur and that the groundwater can be even closer to ground surface.

4 CONCLUSIONS AND RECOMMENDATIONS

Based on geotechnical considerations, the site is suitable for the proposed Project. The primary geotechnical considerations at the site include the large seismic ground motions, potential liquefaction of loose soils present below the historically highest groundwater levels, scour potential at the abutment locations, and heavy loading of the bridge structure.

The proposed bridge spans at the western end of the Bridge and the abutment can be supported on deep pile foundations. Design recommendations to address the primary geotechnical considerations are presented herein and were developed in accordance with the AASHTO LRFD Bridge Design Specifications (AASHTO, 2017) and the Caltrans Amendments to the AASHTO LRFD Bridge Design Specifications (Caltrans, 2019a).

4.1 SEISMIC/GEOLOGIC HAZARDS

4.1.1 Ground Motion

The site, like most of Southern California, will be subject to strong ground shaking during major earthquakes. The site is outside the Alquist-Priolo Special Study Zone (CGS, 2021) and Landslide Zone (CGS, 2002b). The nearest known active or potentially active faults are summarized in Table 2.

Table 2 - MAJOR FAULT CHARACTERIZATION IN THE PROJECT VICINITY

FAULT[1]	Distance[2] (miles)	SLIP SENSE	DIP (degrees)	DIP (direction)	M_{MAX}
San Cayetano	1.27	Thrust	42	N	7.2
Oak Ridge Connected	2.44	Reverse	53	Unspecified	7.4
Oak Ridge (Onshore)	2.44	Reverse	65	S	7.2
Santa Susana, alt 1	9.91	Reverse	55	N	6.9
Hoser, alt 1	10.39	Reverse	58	S	6.8

Note(s):

1. Based on United States Geological Survey (USGS) online Seismic Hazard Maps (USGS, 2023a).
2. Distance to nearest portion of the project.

- M_{MAX} = maximum earthquake magnitude.
- N = North, S = South

Design earthquake magnitudes ranged from 6.8 to 7.4 for the return periods (USGS, 2023a).

Seismic hazard analyses for the bridge structure consisted of development of acceleration response spectra (ARS). The American Railway Engineering and Maintenance-of-Way Association (AREMA) guidelines (AREMA, 2021) were used for the evaluation of the rail bridge structure in accordance with the SCRRA Design Criteria Manual (2021a).

Seismic hazard analyses were performed using a probabilistic approach in accordance with Chapter 9 of the AREMA manual (2021). The AREMA manual specified three ground-motion levels, which correspond to three performance criteria: serviceability, ultimate, and survivability for seismic design. Probabilistic seismic hazards were evaluated for the Project using the USGS Unified Hazards tool (USGS, 2023b). The return periods and the corresponding peak ground acceleration (PGA) values corresponding to each of the three design ground motion levels are summarized in Table 2. The horizontal acceleration coefficients and return period relationship for the proposed site are summarized in Table 3.

Table 3 - SUMMARY OF AREMA PEAK GROUND ACCELERATIONS

AREMA SEISMIC GROUND MOTION LEVEL	PERFORMANCE CRITERIA	RETURN PERIODS (years)	PEAK GROUND ACCELERATION (PGA, g)
1	Serviceability	95	0.19
2	Ultimate	475	0.44
3	Survivability	2,475	0.82

Note(s)

- Values presented in table are based on return periods stated in the SCRRA Design Criteria Manual (SCRRA, 2021a and AREMA, 2021).

Table 4 - AREMA SEISMIC RESPONSE COEFFICIENTS

PERIOD (seconds)	AREMA SEISMIC RESPONSE COEFFICIENT (C_m)[1,2,3]		
	95-Year Return Period[4]	475-Year Return Period[5]	2,475-Year Return Period[6]
	C_m (g)	C_m (g)	C_m (g)
0.01	0.1932	0.4390	0.8190
0.05	0.2938	0.6106	1.2178
0.10	0.4313	0.9521	1.9670
0.20	0.4313	0.9521	1.9670
0.30	0.4313	0.9521	1.9670
0.40	0.4169	0.9521	1.9670
0.50	0.3335	0.8549	1.7225
0.60	0.2780	0.7124	1.4354
0.70	0.2382	0.6107	1.2304
0.80	0.2085	0.5343	1.0766
0.90	0.1853	0.4750	0.9569
1.00	0.1668	0.4275	0.8613
1.10	0.1516	0.3886	0.7830
1.20	0.1390	0.3562	0.7177
1.30	0.1283	0.3288	0.6625
1.40	0.1191	0.3053	0.6152
1.50	0.1112	0.2850	0.5742
2.00	0.0834	0.2137	0.4306
2.50	0.0667	0.1710	0.3445
3.00	0.0556	0.1425	0.2871
3.50	0.0476	0.1221	0.2461
4.00	0.0417	0.1069	0.2153

Note(s):

1. Seismic response spectra determined in accordance with AREMA, 2021.
2. Seismic response coefficient for the m^th mode, C_m, per AREMA (2021), Chapter 9, Paragraph 1.4.4.3.
3. Low period reduced response may be calculated in accordance with AREMA (2021), Chapter 9, Paragraph 1.4.4.4; seismic response coefficient above does not include this adjustment.
4. Level 1 Seismic Ground Motion (AREMA, 2021) corresponding to Earthquake return period of 95 years; Site Class D.
5. Level 2 Seismic Ground Motion (AREMA, 2021) corresponding to Earthquake return period of 475 years; Site Class D.
6. Level 3 Seismic Ground Motion (AREMA, 2021) corresponding to Earthquake return period of 2,475 years; Site Class D.

Ground motion and acceleration response spectra (ARS) were also evaluated using the USGS Unified Hazard Tool (2023b) and Caltrans Seismic Design Criteria (2019b), respectively. The Caltrans procedure considers probabilistic response spectra based on a 5% probability of exceedance in 50 years (975-year return period). Based on the results obtained from the Caltrans ARS online tool (2023) and the USGS Unified Hazard Tool (2023b), the peak ground acceleration (PGA) and earthquake modal magnitude, respectively, for the Project location are presented in Table 3. Caltrans design ARS for the Project are presented in Table 6

Table 5 - SUMMARY OF CALTRANS SEISMIC DESIGN PARAMETERS

Location	**Magnitude[1]**	**PGA[2]**
34.406311º, -118.931937º	7.15	0.72

Note(s):

1. Based on United States Geological Survey (USGS) Unified Hazard Tool (USGS, 2023b). Magnitude is based on the maximum value of the mean and modal magnitude values.
2. Based on Caltrans ARS Online Tool V3 (Caltrans, 2023).

Table 6 - CALTRANS ACCELERATION RESPONSE SPECTRUM

PERIOD (seconds)	**SPECTRAL ACCELERATION (g)**
0	0.72
0.1	1.33
0.2	1.73
0.3	1.79
0.5	1.53
0.75	1.33
1	1.14
2	0.56
3	0.36
4	0.26
5	0.20

Note(s):

- Based on United States Geological Survey (USGS) Unified Hazard Tool (USGS, 2023b). Magnitude is based on the maximum value of the mean and modal magnitude values.
- Based on Caltrans ARS Online Tool V3 (Caltrans, 2023).

4.1.2 Liquefaction Potential

Depth to groundwater was assumed to be at elevation 423 feet at the site. Due to the presence of dense to very dense cohesionless soils at the two bridge sites below the design groundwater level, the potential for liquefaction is considered to be low. Therefore, seismic-induced settlements at the site are anticipated to be minimal. Since the site is not located near a free-face, we judge that potential for lateral spreading is low.

4.2 EARTHWORK

Earthwork is anticipated to be required for the bridge bents and abutment. Deep excavations may be required with shoring adjacent to the roadway or other structures for the construction of the concrete abutment and wingwalls.

4.2.1 Site Preparation and Grading

Prior to the start of construction, the following should be performed:

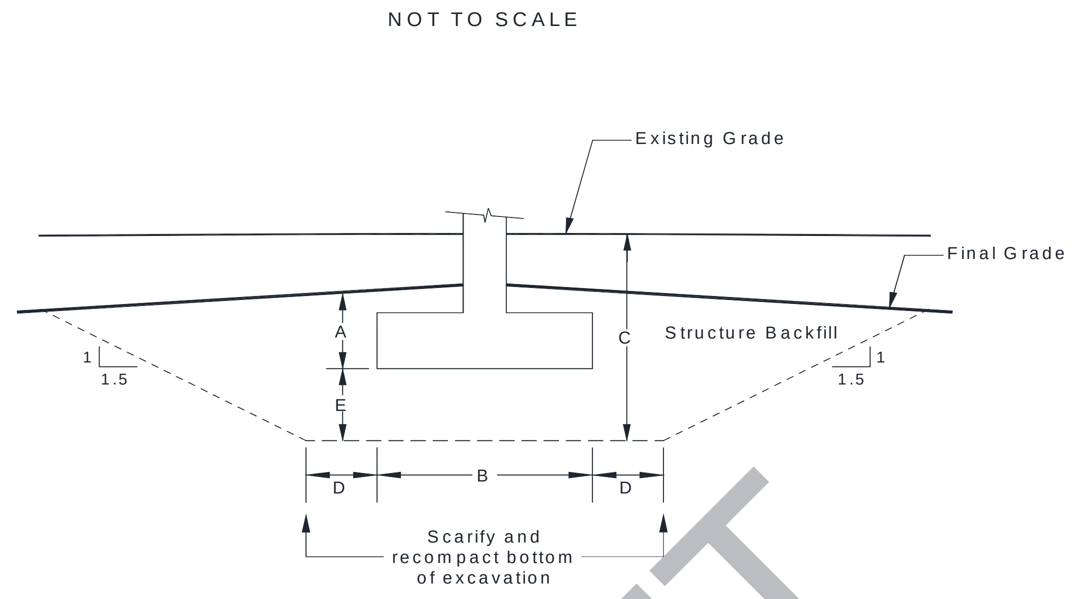
- All utilities should be located in the field and rerouted, removed, abandoned, or protected where necessary.
- Areas to be graded should be stripped of vegetation and debris, and the material removed from the site.
- Pavement should be separated for recycling.

The upper soil should be excavated and replaced with compacted fill as shown on Figure 4. For the bottom of the excavation, the following should be performed:

- Scarified to a depth of 8 inches.
- Moisture-conditioned to at least 2% above optimum moisture content.
- Compacted to at least 95% relative compaction.[1]

---

[1] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by ASTM International (ASTM) D1557 test method. Optimum moisture content is the moisture content corresponding to the maximum dry density, as determined by the ASTM D1557 test method.



LOCATION	MINIMUM DIMENSIONS (feet)
A. Footing Embedment Below Final Grade	2
B. Footing Width	2
C. Excavation Below Existing Grade	2
D. Excavation Beyond Footing	5
E. Compacted Soil/Fill Below Footing	2
PRESSURE (psf)	
Static (net) Allowable Bearing Capacity (FS≥3)	5,000
Maximum Static Bearing Capacity (FS≥3)[1]	5,000
Maximum Transient Bearing Capacity ((FS≥2)	7,500

Note(s):

1. The static bearing capacity can be increased by 33% to calculate the transient bearing capacity for transient loading conditions.
- FS = factor of safety.

Figure 4 - GRADING/FOUNDATION DETAILS (LIGHTLY LOADED SHALLOW FOOTINGS)

Where the soils at the bottom of the excavation preclude compaction, they should be excavated to a depth sufficient to achieve a firm and unyielding surface at the planned bottom of excavation or the base of fill. Generally, an overexcavation depth of 1 to 2 feet is sufficient. Using geogrids and/or easily compactable material such as crushed rock can reduce the depth of excavation. The geogrids and/or geotextile should satisfy the requirements of Standard Specifications for Public Works Construction ([Greenbook]; Building News, 2018, Table 213-5.2 (D) Biaxial S1.).

Fill and backfill should be compacted by:

- Placing in loose layers less than 8 inches thick.
- Moisture-conditioning to at least 2% above optimum moisture content.
- Compacting to at least 95% relative compaction.

The compacted subgrade soils should be firm, hard, and unyielding.

Concrete flatwork (i.e., hardscape, curbs, and gutters) should be underlain by a minimum of 12 inches of soil compacted to at least 95% relative compaction and at least 2% above optimum moisture content.

Materials for structure backfill should meet the criteria per SCRRA (2021b) Standard Spec 31.20.00. Recommendations provided in Caltrans specifications (Caltrans, 2018)/Greenbook (Building News, 2018)) can be used for import fill material criteria.

Generally, the upper soils encountered in the borings are not expected to meet the criteria for structure backfill per SCRRA Standard Spec 31.20.00 (SCRRA, 2021b).

Site grading may be accomplished with conventional heavy-duty construction equipment. The fill should be compacted using soil compactors as recommended by the Caterpillar Performance Handbook (2018), or equivalent. However, to avoid overstressing retaining walls when placing backfill adjacent to the retaining walls, backfill should be compacted using lightweight compaction equipment or the walls should be braced.

4.3 FOUNDATION DESIGN

4.3.1 Deep Foundations

We judge that the proposed abutments and bents for the structure replacement can be supported on pile foundations. Drilled (cast-in-drilled-hole [CIDH]) piles were considered for the design. Because of potential driving difficulties/refusals in very dense sands, potential pile-driving-induced vibration, and proximity of rail tracks, driven piles may not be feasible at this Project site. Therefore, CIDH piles were selected by the designer for foundation support for the design.

Construction of CIDH concrete piles should address potential caving/sloughing/heaving of granular soils. Based on the subsurface conditions at the site, the CIDH pile tip elevations are anticipated to be below the design groundwater elevation; therefore, wet construction methods are anticipated for CIDH pile construction. It is likely that CIDH pile construction would require a temporary casing or wet drilling method depending on the anticipated groundwater table at the time of construction. Details of CIDH pile construction considerations are discussed in Section 4.6 of this report

For the design, 6-foot-diameter and CIDH piles were selected by the designer for bent and abutment piles, respectively. Pile axial resistances of 6-foot CIDH concrete piles for the abutment and bents were evaluated using SHAFT (Ensoft, 2017) for the Strength Limit and Extreme Limit cases. The CIDH pile axial compression resistance was based on skin friction and neglecting end bearing resistance. An LRFD Strength and Extreme Limit State resistance factor of 0.7 and 1.0, respectively, were considered for skin friction resistance. Based on the AASHTO LRFD Bridge Design Specifications Section 10.8.3.6.3, for a pile group in sand the individual nominal resistance of each pile should be multiplied by an efficiency factor, η, based on pile center-to-center spacing. Based on the bent layouts, the proposed pile center-to-center spacing of the two 6-foot diameter CIDH piles placed in a single row is 18 feet, or 3 diameters (3D). Based on the abutment layout, the proposed pile center-to-center spacing of the four 6-foot diameter piles, placed in a 2 x 2 group, is 18 feet, or 3D. Therefore, pile group reduction factors of 1.0 and 0.8 were applied in the analyses for the bent and abutment pile axial resistances, respectively. Although our borings were performed only to 100 feet deep bgs, our vertical pile capacity analysis on the creek bed (bent) and abutments were performed to a depth of 120 feet by extrapolating the available soil strength parameters from 100 feet to 120 feet.

Based on discussions with the design team, the pile lateral capacity will be performed by the structural engineering team. The structural engineer will provide the recommended pile lengths from their lateral capacity analyses.

Scour is a design concern because the bridge is located within an active streambed. The calculated long-term, local, and total scour depth and the total scour elevation can be found in Table 7 and Table 8, respectively, in the Hydraulics Report for the Sespe Creel Overflow Channel Railroad Bridge prepared by GHD (GHD, 2023). Bottom-of-scour elevations were provided by Railpros (2023b) in accordance with Section 3.7.5 of the Caltrans Amendments to the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (Caltrans, 2019a). The proposed bent and abutment piles should be designed for the local scour, while protecting against potential long-term degradation. Section 7 of the GHD (2023) report provides recommended scour protection countermeasures. Note that the pile cut-off elevations provided in Table 7 and Table

9 were provided by Railpros at a later date, and thus supersede the pile cut-off elevations provided in Appendix A.

The Project structural designer provided the foundation design data, factored design loads and bottom-of-scour elevations for the proposed bent and abutment piles. The foundation design data and bottom-of-scour elevations are presented in Table 7. The factored design loads are presented in Table 8, below. The foundation design recommendations table and pile data table are presented in Table 9 and Table 10, respectively. Settlement of the piles due to Service Limit loading was estimated to be less than 1 inch.

Table 7 - FOUNDATION DESIGN DATA SHEET

SUPPORT NO.	PILE TYPE	CUT-OFF ELEVATION^1 (feet)	BOTTOM-OF-SCOUR ELEVATION (FEET)			PERMISSIBLE SETTLEMENT UNDER SERVICE LOAD (inches)	NUMBER OF PILES PER SUPPORT
			STRENGTH LIMIT STATE	SERVICE LIMIT STATE	EXTREME LIMIT STATE		
Abutment 1	6-foot CIDH	420.75	423.7	411.9	435.4	1”	4
Bent 2	6-foot CIDH	425.00	412.2	406.3	422.1	1”	2
Bent 3	6-foot CIDH	429.00	414.5	406.6	422.4	1”	2

Note:

1. Provided by the structural design team (Railpros, 2023b).

Table 8 - FOUNDATION FACTORED DESIGN LOADS

SUPPORT NO.	SERVICE LIMIT STATE TOTAL LOAD PER PILE (KIPS)	STRENGTH/CONSTRUCTION LIMIT STATE (kips)		EXTREME EVENT LIMIT STATE (kips)	
		COMPRESSION MAX. PER PILE	TENSION MAX. PER PILE	COMPRESSION MAX. PER PILE	TENSION MAX. PER PILE
Abutment 1	887	1,426	0	716	0
Bent 2	550	939	0	778	304
Bent 3	550	939	0	778	304

Note:

- The pile tip elevations should also be checked for lateral loading.

Table 9 - DEEP FOUNDATION DESIGN RECOMMENDATIONS

SUPPORT LOCATION	PILE TYPE	CUT-OFF ELEVATION (feet)	SERVICE-LIMIT STATE LOAD PER PILE (kips)	TOTAL PERMISSIBLE SETTLEMENT SUPPORT SETTLEMENT (inches)	REQUIRED FACTORED NOMINAL RESISTANCE PER PILE (kips)				DESIGN TIP ELEVATIONS (feet)	SPECIFIED TIP ELEVATIONS (feet)
					STRENGTH LIMIT		EXTREME EVENT			
					COMP. (ϕqs = 0.7)	TENSION (ϕqs = 0.7)	COMP. (ϕqs = 1.0)	TENSION (ϕqs = 1.0)		
Abutment 1	72" CIDH	420.75	887	1	1,426	--	716	--	322.75 (a-I) 385.75 (a-II) 372.75 (c) --[1] (d)	322.75
Bent 2	72" CIDH	425.00	550	1	939	--	778	304	353.0 (a-I) 381.0 (a-II) 397.0 (b-II) 371.0 (c) --[1] (d)	353.00
Bent 3	72" CIDH	429.00	550	1	939	--	778	304	355.0 (a-I) 381.0 (a-II) 397.0 (b-II) 371.0 (c) --[1] (d)	353.00

Notes:

1. Design tip elevations for Lateral Load (d) for Bents are not required per discussion with structural engineer. Based on the lateral loads provided, it is assumed that pile tip elevation per lateral load will not control.

- Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (b-I) Tension (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement, (d) Lateral Load.
- The specified tip elevation shall not be raised above the lowest design tip elevation.
- Unsuitable soil layers (i.e., scourable) that do not contribute to the design nominal resistance exist for Abutment 1 and Bents 2 and 3. Bottom-of-scour elevation varies. See Table 7 for bottom-of-scour elevations

Table 10 - PILE DATA TABLE

LOCATION	PILE TYPE	NOMINAL RESISTANCE (kips)		DESIGN TIP ELEVATION (feet)	SPECIFIED TIP ELEVATION (feet)
		COMPRESSION	TENSION		
Abutment 1	6-foot CIDH	2,037	--	322.27 (a) 372.75 (c) -- (d)	322.75
Bent 2	6-foot CIDH	1,174	304	353.0 (a) 397.0 (b) 371.0 (c) -- (d)	353.0
Bent 3	6-foot CIDH	1,174	304	355.0 (a) 397.0 (b) 371.0 (c) -- (d)	355.0

Notes :

- Design tip elevations for abutment and bents are controlled by: (a) Compression, (b) Tension, (c) Settlement, (d) Lateral Load.
- The specified tip elevation should not be raised above the lowest tip elevation.
- Design tip elevation for Lateral Load to be performed by the structural engineer.

4.4 RESISTANCE TO LATERAL LOADS AND LATERAL EARTH PRESSURES

4.4.1 Temporary Shoring

Shoring may be required if excavations for the wingwall are performed adjacent to existing railroad tracks and/or the roadway to avoid undermining the railroad tracks. The contractor should also be prepared to adjust the construction methods based on actual field conditions.

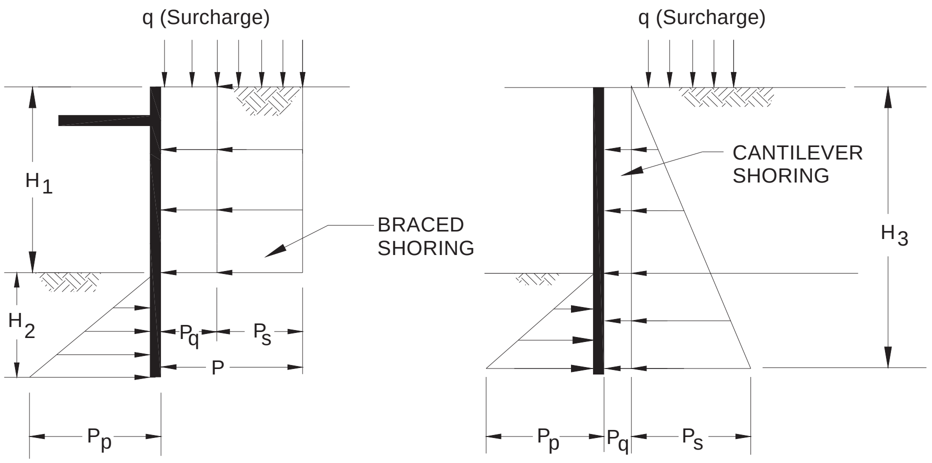
The shoring design is the responsibility of the contractor and should be designed by a registered engineer retained by the contractor. Design of the shoring system will require careful consideration of the existing adjacent improvements, utilities, and foundation systems located close to shored excavations. Shoring design should consider the possible related effects on the surrounding buildings and utilities, deflections of the shoring elements, possible effects of nearby foundation loads on the shoring, and settlements of the retained soil.

The temporary shoring design should incorporate the expected construction procedures, sequence, and loads. In particular, the stockpiling of excavated materials should be considered in design, as well as steel plates for cross traffic and the presence of heavy construction equipment or spoil piles next to the excavations.

Shoring is usually designed as either cantilever (unbraced) or braced. Cantilevered shoring is commonly constructed by either using soldier piles with lagging placed between piles or using sheet piles. If soldier piles and lagging are used, continuous lagging will be required. Difficulty in installing the lagging due to caving cohesionless soils should be anticipated. SCRRA restrictions on the use of shoring based on distance from the operating railroad tracks should be followed (SCRRA, 2021a).

For cantilever shoring, a deflection of 0.5% of the shored height (H1) is necessary to develop active earth pressure (Figure 5 for definition of H1). For braced shoring, the deflection should be less than that for cantilever shoring. We recommend that the design of temporary shoring be performed using shoring pressures equal to or greater than those shown on Figure 5 and Figure 6 and passive resistance equal to or less than that shown on Figure 5. The allowable passive soil pressure outlined on Figure 5 assumes undisturbed existing soils. The upper 1 foot of passive resistance should be neglected.

In using Figure 5, lateral pressures due to rail surcharge on temporary shoring located parallel to the rail tracks can be conservatively estimated assuming lateral coefficients of 0.3 and 0.5 for cantilever and restricted conditions, respectively, and a uniform rail surcharge load (AREMA, 2021).



	BRACED SHORING	**CANTILEVER SHORING**
$P_p = 390 H_2 \leq 4{,}000$ psf	$P = P_s + P_q = 24H_1 + 0.5q$	$P = P_s + P_q = 37H_3 + 0.3q$
	(300 psf minimum)	(300 psf minimum)

Note(s):

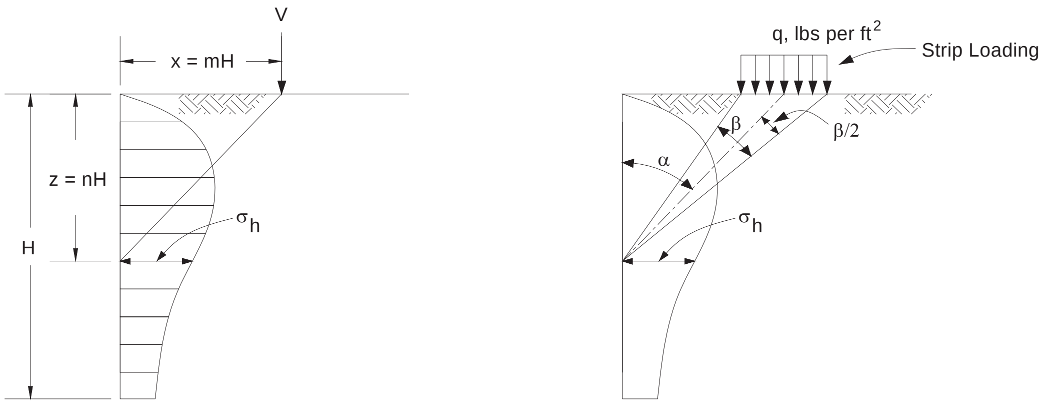
- All values of height (H) in feet and pressure (P) and surcharge (q) in psf.
- Where vehicular traffic from freeway is applicable, assume no less than a 240 psf uniform horizontal pressure.
- Where train load is applicable, use q = live load (from train) + impact load (if considered due to train derailment) per AREMA, Chapter 8, Section 2.2.3.
- Earth pressures assumed no hydrostatic pressures. Hydrostatic pressures, if anticipated, should be added to lateral earth pressures.

Figure 5 - LATERAL EARTH PRESSURES FOR TEMPORARY STRUCTURES

The shoring system and adjacent buildings should be monitored using “real time” instrumentation and optical surveys to check for the lateral and vertical movements until the permanent structure is in place. If large deflections (greater than 0.25% of the shoring height) are noted, the bracing system should be checked and strengthened as needed. If tension cracks appear in the ground surface adjacent to the shoring, the cracks should be monitored and sealed to prevent infiltration of water, and the significance of the cracks should be evaluated immediately.

The type of shoring will depend on the contractor’s means and methods. The excavations should only remain open for very short periods of time.

In addition, the contractor should strictly adhere to any requirements of SCRRA (2021a) and applicable federal and state health and safety regulations such as those of the Occupational Safety and Health Administration (OSHA, 2020). In accordance with OSHA regulations, the near-surface on-site soils are classified as Type C.



POINT LOAD	STRIP LOAD
For $m > 0.4\,, \sigma'_h = \frac{1.77V}{H^2}\frac{m^2n^2}{\left(m^2+n^2\right)^3}$ For $m \le 0.4\,, \sigma'_h = \frac{0.28V}{H^2}\frac{n^2}{\left(0.16+n^2\right)^3}$	$\sigma'_h = \frac{2q}{\pi}(\beta - \sin\beta\cos2\alpha)$

Note(s):

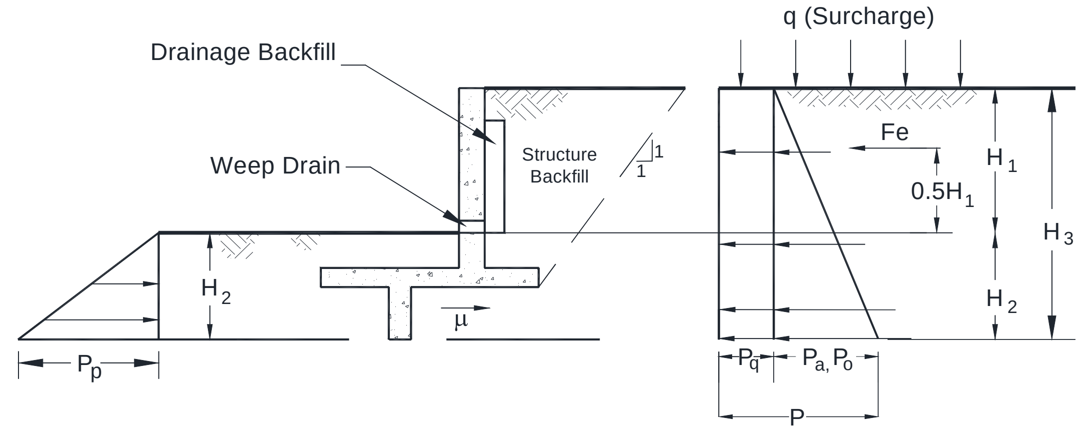
- σ'_h = horizontal stress (psf).
- V = vertical point load (pounds).
- q = vertical strip load (psf).
- H = height of wall (feet).
- α, β = angles (radians).
- m, n = dimensionless coefficients.

Figure 6 - SURCHARGE LATERAL PRESSURE DISTRIBUTION AGAINST A WALL

4.4.2 Permanent Structures

Lateral loads may be calculated per AREMA Chapter 8, Part 5, using trial wedge analysis with a soil friction angle of 32 degrees and soil density of 120 pcf. Lateral loads may also be calculated using Figure 7. Earth pressure coefficient calculations are provided in Appendix G.

The lateral resistance may be calculated using the following: 50% of passive resistance plus 50% of base friction, 100% passive resistance only, or 100% of the base friction only. Lateral loads can be resisted by an allowable passive soil pressure and base friction, as outlined on Figure 7for compacted fill, applied against below-grade walls and foundation elements.



$P_p = 390\ H_2 \leq 4{,}000$ psf	**CANTILEVER WALLS**	**RESTRAINED WALLS**
μ = 0.6 (nominal)	$P = P_a + P_q = 37\ H_3 + 0.3q$	$P = P_o + P_q = 56\ H_3 + 0.5q$
	INCREMENTAL SEISMIC FORCE	
	Serviceability[1]	$Fe = 4\ H_1^2$
	Ultimate[1]	$Fe = 9\ H_1^2$
	Survivability[1]	$Fe = 21\ H_1^2$
	Caltrans 975-year ARP[2]	$Fe = 17\ H_1^2$

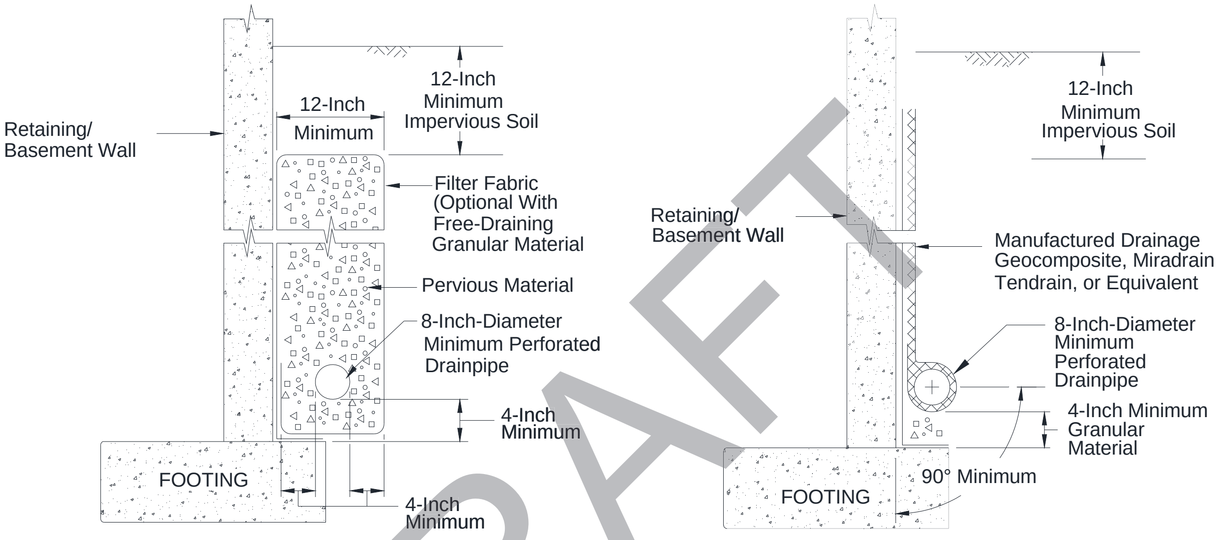
Note(s):

1. Per AREMA (2021) seismic design criteria, PGA_M = 0.193g, 0.439g, and 0.819g were used, respectively, for the Serviceability, Ultimate, and Survivability cases.
2. Per Caltrans ARS Online Tool V2 (Caltrans, 2023), PGA_M = 0.721g

- Lateral earth pressures were calculated using assumed abutment fill properties, including a unit weight of 120 pcf and a friction angle of 32 degrees.
- One-half of the PGA_M was used to calculate Fe.
- All values height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf), and force (F) in pounds.
- Where vehicular traffic from freeway is applicable, assume no less than a 240 psf uniform horizontal pressure.
- Where train load is applicable, use q = live load (from train) + impact load (if considered due to train derailment) per AREMA, Chapter 8, Section 2.2.3.
- P_p, P_a, and P_o are the passive, active, and at-rest earth pressures, respectively; Fe is the incremental seismic force.
- P_q is the incremental surcharge pressure; μ is the allowable friction coefficient applied to dead normal (buoyant) loads. Fe is in addition to the active and at-rest pressures. Below groundwater, in areas of potential pipeline rupture or areas of potential surface water infiltration, active and at-rest pressure should be reduced by 50% and hydrostatic pressure should be added to active and at-rest pressures. P_p should be reduced by 50% below the groundwater.
- For 2H:1V slopes above the wall, increase the active and at-rest pressures by 50%; for 1.5H:1V slope, increase the active and at-rest pressures by 100%.
- Neglect the upper 1 foot for passive pressure unless the surface is contained by a pavement or slab.

Figure 7 - LATERAL EARTH PRESSURES (PERMANENT STRUCTURES)

Retaining walls should be designed to resist lateral earth pressures with equivalent fluid pressures as illustrated on Figure 7. Lateral earth pressures are presented for walls free to rotate and restrained walls. At-rest earth pressures (restrained walls) should be used where the top of the wall is not expected to move laterally more than 0.001 H_1 (see Figure 7). The lateral earth pressures on Figure 7 are based on the structure backfill material noted in Section 4.2.1. The retaining walls should include a drain or be designed for hydrostatic pressure. See Figure 8 for typical sections of wall drains. The train surcharge pressures should be added to the lateral earth pressures on Figure 7 for the retaining wall for the total lateral pressure following the procedure discussed in Section 4.4.1. The seismic earth pressures provided on Figure 7 are based on the PGA_M based on ICC 2019 criteria discussed in Section 4.1.



MATERIAL	CALTRANS SPECIFICATIONS	GREENBOOK SPECIFICATIONS
Free-Draining Granular Material	68-2.02 (Class 2)	300-3.5.2
Geotextile Filter Fabric	88-1.02B	300-8
Perforated Pipe	68-2.02	207-13.4

Note(s):

- Drainpipe should drain to an outlet.
- Filter fabric wraps completely around perforated drainpipe and pervious materials.

Figure 8 - RETAINING WALL DRAINAGE

4.5 SOIL CORROSION POTENTIAL

One soil sample was tested for pH, soluble chloride and soluble sulfate, and soil electrical resistivity for corrosion potential. The test values are summarized in Table 11.

Analytical chemical test results indicated a value of 531.9 parts per million (ppm) soluble sulfate concentration in the near-surface soils. Based on these test results, we recommend that the concrete be designed for exposure class S1 from ACI 318 (ACI, 2011).

The corrosion potential test results are presented in Appendix E. Based on Caltrans Standards (2021) and the chemical test results, the on-site soils are classified as non-corrosive to buried metal pipes. In addition to the soil characteristics, external factors such as nearby active corrosion systems will greatly affect the need for an active corrosion protection system. The test data provided herein can be used by others to develop details of corrosion protection. Borrow soils imported to the Project site should be tested for corrosion potential.

Table 11 - CORROSION POTENTIAL

CONSTITUENT	CRITERIA FOR CORROSIVE MATERIALS	VALUE
pH	<5.5	7.2
Soluble sulfate content (ppm)[1]	>1,500	531.9
Soluble chloride content (ppm)	>500	7.9
Electrical resistivity (ohm-cm)	<1,500	1,541

Note(s):
- Caltrans Corrosion Guidelines (2021)
- ppm = parts per million.
- The lowest values for corrosive materials criteria are presented.

4.6 NOTES FOR CONSTRUCTION

The proposed CIDH piles will extend through gravel/cobble/boulder-rich alluvial dense sands. Additionally, the site is highly variable with layers of boulders, cobbles, and gravel, and those materials can be encountered at any depth. The subsurface cohesionless soils have the potential to slough, cave, and bottom heave during CIDH pile installation when subjected to vibration load from the adjacent traffic or if shallow groundwater is encountered. In addition, loss of drilling fluids was encountered during the subsurface field exploration. Therefore, “wet” construction methods and temporary casings should be considered for ease of construction and to reduce the potential for CIDH pile anomalies. The application of temporary casing may minimize loss of drilling fluid.

When "wet" construction methods are used, the integrity of concrete should be checked using downhole gamma-gamma and/or cross-hole sonic testing; PVC inspection pipes should be installed within the CIDH piles to facilitate the testing. Caltrans Standard Specifications for "Cast-in-Place Concrete Piling" should be followed. Difficult drilling conditions also should be anticipated to penetrate the very dense soils present at the site. In general, a minimum of 24 hours should be allowed between placing concrete in one pile shaft and drilling any nearby shafts or performing any other excavations within four pile diameters. It is the responsibility of the contractor to review all the pertaining boring records and LOTBs to understand the subsurface materials encountered in the borings, to select the appropriate drilling equipment, and to apply their means and methods to drill and install the CIDH piles.

Drilling and casing techniques, such as the oscillator casing method, can also be considered to help reduce construction-induced CIDH structural anomalies. Construction methods will have significant effects on the load-carrying capacity of the installed CIDH piles. Significant quality control and care must be exercised during construction including removal of temporary casing to ensure that the construction methods do not compromise the development of side friction. Selection of the CIDH pile construction contractor should be based on proven performance record on similar projects.

Hydraulics Report

Sespe Creek Overflow Channel Railroad Bridge Hydraulic Analysis at Fillmore, CA

Prepared for RailPros

October 24, 2023

→ The Power of Commitment

Project name	Reconstruction of Sespe Creek Overflow Railroad Bridge on the Santa Paula Branch Line (SPBL).
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S4	1	Seth Stevens	Greg Garrison	Greg Garrison	Erik Fanselau	Erik Fanselau	10/19/23
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[Status code]							

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1. Introduction

1.1 General Introduction

The Ventura County Transportation Commission (VCTC) is planning to reconstruct a portion of the existing railroad bridge over the Sespe Creek Overflow Channel in Fillmore, California. During high flows on January 10, 2023, the west end of the bridge was damaged, including washing out 2 bents and severe damage to the abutment, resulting in the loss of three bridge deck spans, as shown in Figure 1. The repair includes replacing the two washed out bents and one of the remaining bents with two new concrete bents, as well as replacing the existing west abutment with a new concrete abutment, resulting in two new bridge deck spans.

The design is currently at the 90% design level and this report and associated analysis is based on the information included in those design drawings.

Figure 1 Damaged Portion of Railroad Bridge (looking upstream)

1.2 Purpose of this Report

The purpose of this report is to present the methods and outcomes of a hydraulic analysis of Sespe Creek that was conducted to assist in the design of the repair of the damaged railroad bridge in Fillmore, California. The key objectives of the analysis are:

- Provide a hydraulic assessment to estimate the water surface profile, flow depth, and flow velocity in Sespe Creek in the vicinity of the Project for the pre-project and post-project conditions
- Provide an evaluation of bridge scour for the proposed improvements in the post-project conditions

1.3 Scope and Limitations

This report has been prepared by GHD for *RailPros* and may only be used and relied on by *RailPros* for the purpose agreed between GHD and *RailPros* as set out in Section 1.2 of this report.

GHD otherwise disclaims responsibility to any person other than *RailPros* arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions, and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report (refer Section 1.4 of this report). GHD disclaims liability arising from any of the assumptions being incorrect.

Accessibility of documents

If this report is required to be accessible in any other format, this can be provided by GHD upon request and at an additional cost if necessary.

1.4 Assumptions

It is assumed that the data provided for using in this study, including the hydraulic, topographic survey, and drawings represent the creek, hydraulic structures, and flows to a level of accuracy that is appropriate for this study.

2. Background

2.1 Study Area Description

The study area includes Sespe Creek in the vicinity of the railroad bridge. The land adjacent to the creek along the left bank (looking downstream) is developed with mostly single-family residences and the land adjacent to the creek along the right bank (looking downstream) consists of mostly agricultural land use. Sespe Creek flows from north to south within this reach and consists of a natural channel with a levee along the left bank (looking downstream). Approximately 2,500 feet upstream of the rail bridge, the creek splits into two channels, with the main channel to the west and the Sespe Creek Overflow Channel to the east. As a result, there are two rail bridges over Sespe Creek at this location, one over the main channel to the west and the other over the overflow channel to the east. The bridge over the overflow channel is the focus of this study.

2.2 Vertical Datum

The Project references the North American Vertical Datum of 1988 (NAVD88) in units of feet and all elevations presented herein are based on that datum.

2.3 Existing Conditions

The existing Sespe Creek Overflow Channel bridge was constructed in 1969 as a fifteen-span bridge and is approximately 450 feet long and 17 feet wide. The bridge superstructure consists of concrete box girders with ballast curbs and sidewalks that have a combined deck thickness of approximately 4 feet and 1 inch. Prior to the damage, the superstructure was supported on 14 bents (constructed of concrete bent caps with steel piles and concrete infill walls) and two abutments, as shown in Figure 2. The bridge deck slopes at approximately 1% from the west to the east from approximately elevation 451.0 feet to 447.0 feet. Adjacent to the railroad bridge and approximately 45 feet downstream is the Old Telegraph Road bridge.

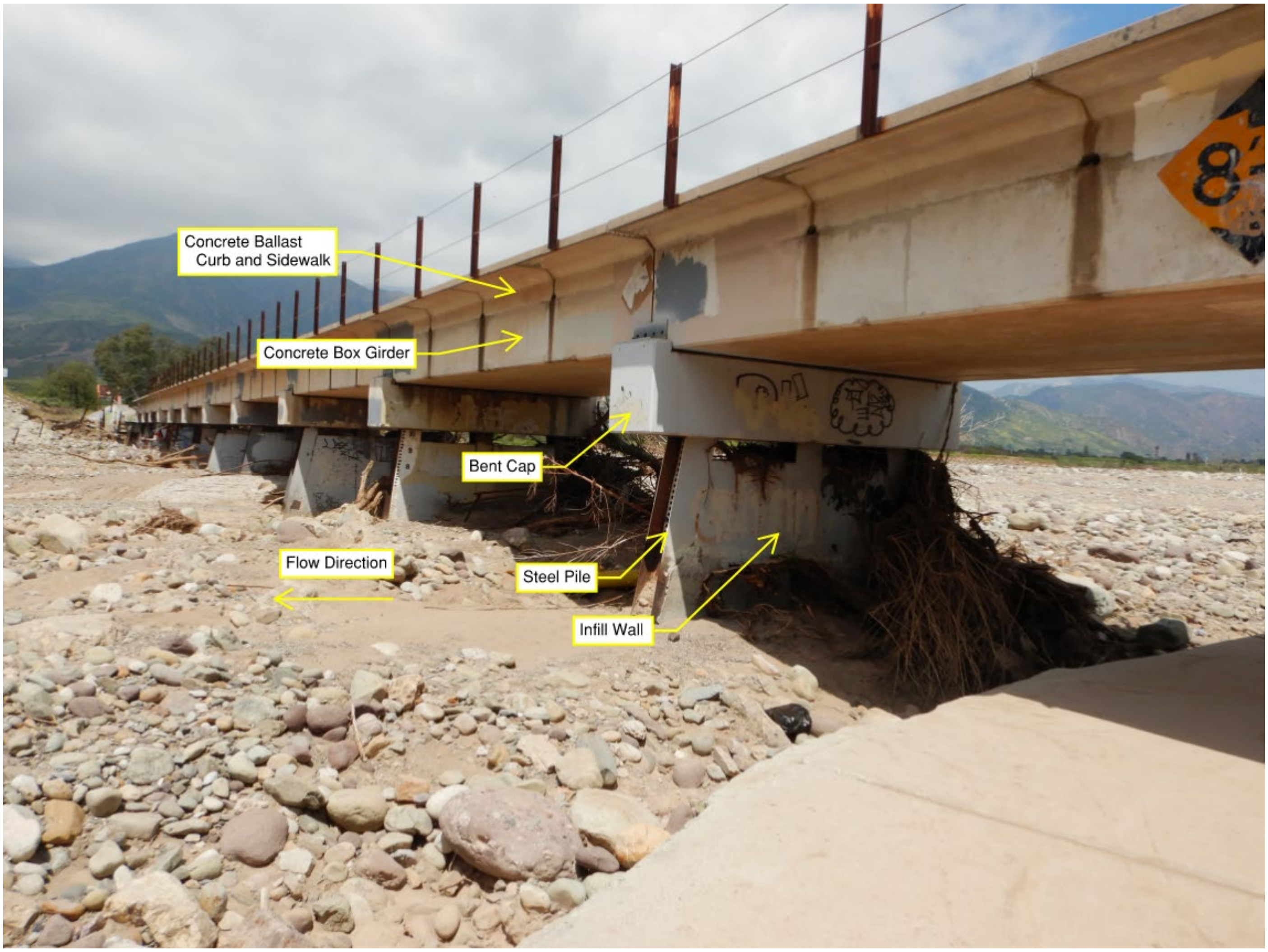


Figure 2 ***Downstream Side of Railroad bridge (East Side of Bridge)***

2.4 Proposed Improvements

The proposed improvements at the bridge include replacing the two washed out bents and one of the remaining bents with two new concrete bents, as well as replacing the existing west abutment with a new concrete abutment, resulting in two new bridge deck spans of 48 feet and 10 inches each and an overall bridge length that is approximately 6 feet and 9 inches longer than the existing bridge. The proposed bridge is 19 feet wide, so approximately 2 feet wider than the existing bridge. The proposed bridge superstructure is a concrete box girder with ballast curbs and sidewalks, like the existing bridge, however, the proposed bridge soffit will extend approximately 12 inches below the existing bridge soffit due to the increased thickness of the structure.

The two proposed bents consist of a 4-foot-thick bent cap on two 4-foot-diameter columns with a concrete infill wall between the columns. The two columns will each be supported by a 6-foot-diameter CIDH concrete pile. The proposed concrete abutment will be supported by a concrete pile cap and four 6-foot-diameter piles.

3. Hydraulic Design Standards and Criteria

3.1 Overview

This section summarizes the design standards and criteria that were considered for the hydraulic analysis, which include requirements from the Federal Emergency Management Association (FEMA) and the Southern California Regional Rail Authority (SCRRA).

3.2 FEMA

Sespe Creek at the project location is located within a FEMA Special Flood Hazard Area (SFHA) Zone A, as shown on Flood Insurance Rate Map (FIRM) panel numbers 06111C0641E and 06111C0643E, last revised on January 20, 2010 (Attachment 1). Zone A represents areas which are subject to inundation by the 1-percent-annual flood (100-yr flood), also known as the Base Flood. Detailed hydraulic analyses have not been conducted for these areas, and consequently Base Flood Elevations (BFEs) have not been determined. The areas of inundation in Zone A are generally determined using approximate methods.

The planned bridge repairs are proposed within a FEMA SFHA, and as such are subject to FEMA requirements which are intended to reduce flood loss and to protect resources. Typically, encroachments into a SFHA outside of a regulated floodway (which does not exist at the Project site), should not cause an increase in the BFE by more than one foot.

3.3 SCRRA

SCRRA criteria for the hydraulic design of bridges is specified in the SCRRA Design Criteria Manual (SCRRA, 2021) and includes the following for bridges conveying cross-track flood flows:

- the opening will be sized so that the water surface for a 50-year event will rise no higher than the lowest low chord of the bridge
- the opening will be sized so that the energy grade line for a 100-year event will not rise above the adjacent subgrade elevation (defined as 2.81 feet below top of rail elevation)

The existing (pre-disaster) condition of the bridge did not meet these criteria. A bridge design repair to meet these criteria would require the bridge and adjacent tracks to be raised substantially, thus a relocation of the railroad tracks would likely be more practical. Regardless, either of these efforts require design that exceeds the limitations of the rehabilitation which is to repair the bridge to its pre-disaster design, capacity, and function. As such, the proposed design will not meet the SCRRA criteria presented above.

4. Hydrologic Assessment

FEMA is currently in the process of updating the Flood Insurance Study (FIS) for Ventura County and the preliminary FIS (FEMA, 2022) that was developed as part of that effort includes the 50 and 100-year peak discharges for Sespe Creek that were used for this study and are shown in Table 1. These flows are at the confluence with the Santa Clara River and are based on a drainage area of 263 square miles. They were estimated by FEMA using the Hydrologic Simulation Program-FORTRAN (HSPF) model as described in the preliminary FIS (FEMA, 2022).

The peak discharge in the FIS is for the entire Sespe Creek, which includes the main channel and the overflow channel where the railroad bridge is located. As discussed in more detail in Section 5.2.3, the hydraulic model had to be truncated for the scour analysis to include only the Sespe Creek Overflow Channel, so the 100-year peak discharge within that section of the channel had to be estimated as part of this study. This was accomplished by using the data from the Preliminary FEMA Model described in Section 5.1. The model provides the flow through each of the two railroad bridges, which are shown in Table 1, and the flow at the bridge in the Sespe Creek Overflow Channel was used in the truncated hydraulic model for the scour analysis.

Table 1 Peak Discharge Rates

Annual Chance Exceedance	Sespe Creek (Entire Channel)	Sespe Creek Overflow Channel	Sespe Creek Main Channel
2% (50-year)	102,604 cfs	N/A	N/A
1% (100-year)	135,789 cfs	96,955 cfs	38,834 cfs

5. Hydraulic Assessment

5.1 Hydraulic Modeling Overview

Hydraulic modeling of Sespe Creek was conducted as part of this study to assess the effect of the proposed bridge repairs on the water surface profile, flow depth, and flow velocity in the channel. The modeling was performed using the USACE Hydrologic Engineering Center River Analysis System (HEC-RAS) version 6.3 software. The base model for this study was provided by Ventura County and is the model developed by FEMA as part of the recent FIS and FIRM updates that are currently preliminary FEMA products, referred to herein as the Preliminary FEMA Model. Although the model is preliminary is status, it is considered the best available model for this study area as it was recently developed and there are no previous FEMA models for this location. The model is a one-dimensional (1D) steady-flow model of Sespe Creek and extends from the confluence with the Santa Clara River to approximately 6 miles upstream.

5.2 Hydraulic Model Setup

5.2.1 Pre-Project Conditions

The Preliminary FEMA Model was assumed to represent the pre-project conditions, which, for purposes of this study, refers to the railroad bridge and channel prior to the damage that occurred in January of 2023. Accordingly, no updates were made to the model for the pre-project conditions.

5.2.2 Post-Project Conditions

The post-project conditions geometry within the HEC-RAS model was developed by modifying the railroad bridge at the Sespe Creek Overflow Channel to reflect the proposed bridge repairs based on the 30% design, including the new bents, new bridge deck, new abutment, and rock slope protection at the abutment. In addition, the topography at the bridge crossing was updated based on the topography obtained for the design of the bridge repair.

5.3 Results

The following four scenarios were evaluated with the hydraulic model to assess the effect of the proposed bridge repairs on the water surface elevation in the channel:

1. Pre-project Conditions with 50-year peak discharge
2. Pre-project Conditions with 100-year peak discharge
3. Post-project Conditions with 50-year peak discharge
4. Post-project Conditions with 100-year peak discharge

Detailed model output for the entire model domains for the scenarios evaluated is included in Attachment 3. A comparison of the pre-project and post-project water surface elevations for both flow scenarios within the vicinity of the railroad bridge is shown in Table 2 and Table 3 and the water surface profiles are shown in Figure 3. For both flow rates, the modeling is showing minor decreases in water surface elevation for the post-project condition extending approximately 1,500 ft upstream of the bridge. The decreases in water surface elevation are less than 0.3 feet and 0.1 feet for the 50-year and 100-year peak discharges, respectively. The decreases are likely due to the removal of one bent and changes in topography at the bridge. In all other locations, there is no change in water surface elevation (WSE). The decreases in water surface elevations are in accordance with the FEMA requirements discussed in Section 3.

Table 2 Water Surface Elevation Summary for 50-Year Peak Discharge

Channel Station	Pre-Project Water Surface Elev. (ft)	Post-Project Water Surface Elev. (ft)	Change in Water Surface Elev. (ft)
15728	469.67	469.67	0.00
15144	462.80	462.80	0.00
14340	456.56	456.56	0.00
13782	452.15	451.97	-0.18
13104	451.79	451.59	-0.20
12892	451.62	451.39	-0.23
12852	Railroad Bridge		
12827	448.45	448.45	0.00
12807	448.44	448.44	0.00
12780	Old Telegraph Road Bridge		
12712	442.80	442.80	0.00
12238	437.85	437.85	0.00
11854	434.18	434.18	0.00
10652	427.37	427.37	0.00
10111	422.25	422.25	0.00

Table 3 Water Surface Elevation Summary for 100-Year Peak Discharge

Channel Station	Pre-Project Water Surface Elev. (ft)	Post-Project Water Surface Elev. (ft)	Change in Water Surface Elev. (ft)
15728	471.46	471.46	0.00
15144	464.53	464.53	0.00
14340	457.85	457.85	0.00
13782	455.46	455.41	-0.05
13104	455.23	455.18	-0.05
12892	455.13	455.07	-0.06
12852	Railroad Bridge		
12827	452.18	452.18	0.00
12807	451.45	451.45	0.00
12780	Old Telegraph Road Bridge		
12712	444.76	444.76	0.00
12238	439.53	439.53	0.00
11854	435.65	435.65	0.00
10652	428.75	428.75	0.00
10111	423.40	423.40	0.00

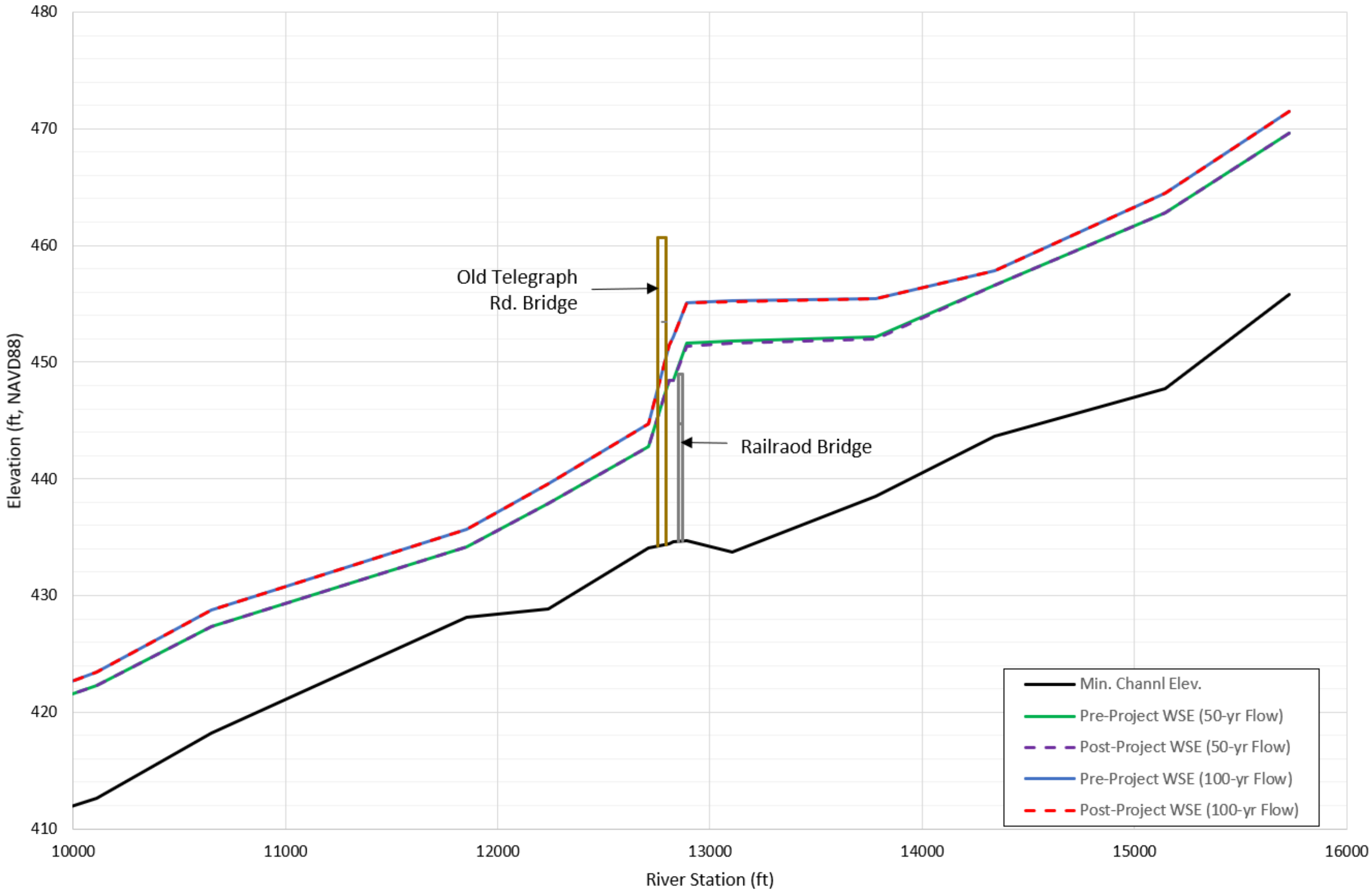


Figure 3 Water Surface Profiles for Sespe Creek in Project Vicinity

6. Bridge Scour Analysis

Scour analyses were conducted as part of this study to evaluate bridge scour for the post-project conditions. The scour analysis included four primary components: long-term degradation of the riverbed, contraction scour at the bridge, local scour at the proposed piers, and local scour at the proposed abutment. The sum of these components represents the total scour at the bridge.

6.1 Basis of Scour Analysis

The bridge scour analysis was conducted for the post-project condition based on the methods presented in the Federal Highway Administration (FHWA) Hydraulic Engineering Circular No. 18 *Evaluating Scour at Bridges* (FHWA, 2012), commonly referred to as HEC-18. The methods that are presented in HEC-18 are applicable to railroad structures, per the SCRRA Design Criteria Manual (SCRRA, 2021).

As discussed previously in this report, Sespe Creek is divided into two channels at the railroad crossing, the main channel and the overflow channel, and there are bridges at each channel. The Preliminary FEMA Model discussed in Section 5 represents the two bridges by using the Multiple Opening Bridge option, which allows the model to split flow between multiple openings within a crossing. While the Multiple Opening option is appropriate for predicting the hydraulic grade line through a structure, it prevents the use of the bridge scour tool within HEC-RAS. To use the bridge scour tool within HEC-RAS, GHD truncated the Preliminary FEMA model into a single channel with a single bridge over the Sespe Creek Overflow Channel only. The model truncation included the following process:

- created a new river reach for the Sespe Creek Overflow Channel from the split of Sespe Creek and extending downstream of the crossing (approximately 5,000 linear feet in total length)
- cross-section geometry and physical representation from the post-project conditions model were copied over to the new river reach (including bank stations, roughness coefficients, reach lengths, etc.)
- cross-section geometries were truncated at the right bank of the overflow channel to isolate it from the Sespe Creek main channel
- the railroad bridge and Old Telegraph Road bridge geometries were added to the model and truncated to the overflow channel, including overbank areas
- the upstream boundary condition was set to estimated 100-year peak discharge for the Sespe Creek Overflow Channel
- the downstream boundary condition was set to a water surface elevation from the post-project conditions model output for the 100-year peak discharge
- the resulting model output was compared to the post-project conditions model to ensure acceptable agreement between the two models.

The scour analysis was performed using the results of the truncated hydraulic model discussed above and the 100-yr peak discharge for the Sespe Creek Overflow Channel discussed in Section 4.

6.2 Geotechnical Data

The project geotechnical engineering consultant, Diaz Yourman & Associates, provided a particle size analysis from a soil sample taken from a boring at the project site which is included in Attachment 5. The soil sample was from a depth of 0 to 5 feet and had a median grain size diameter of approximately 5.1 mm, which was used for the scour analysis for this study.

6.3 Long-Term Bed Elevation Change

Long-term bed elevation change, as it relates to scour, is due to degradation of the channel bed as it tends toward an equilibrium slope. Historical channel elevation data at the project site could inform the potential for long-term degradation in the area, however, historical channel elevation data, including record drawings for the bridge were not available for this study. The only data provided for this study that relates to long-term bed elevation change were two bridge inspection reports, one from 11/30/2022 (Koppers Railroad Structures Inc., 2022) and one after the damage from 5/8/2023 (Wilson & Company, 2023). Both reports noted local scour at some of the bents and west abutment, neither indicated elevation change across the entire channel.

Most of the 263-square mile watershed for Sespe Creek is undeveloped so it is expected that the sediment supply and runoff from the watershed have historically remained consistent. Based on this, it was assumed that the lower reach of Sespe Creek was in equilibrium with respect to long-term bed elevation change and that estimating an equilibrium slope for the channel would provide an indication whether localized long-term bed elevation change may be expected. Using this approach, a channel profile was developed that extended from the Hwy. 126 bridge upstream through the Sespe Creek Overflow Channel and a “best-fit” equilibrium slope was drawn on the channel profile. This profile is included in Attachment 4. This approach indicated the potential for the channel to degrade at the railroad bridge by approximately 4 feet, so this is what was assumed for the long-term degradation at the bridge.

6.4 Clear-Water versus Live-Bed Scour

Clear-water scour and live-bed scour are two methods by which contraction and pier scour occur. Clear-water scour occurs where there is no transport of bed material from upstream of the bridge while live-bed scour occurs where there is transport of bed material from upstream. The type of scour which occurs is dependent on the bed material grain size, upstream average velocity, and upstream average depth of flow. The critical velocity, V_C, necessary for transport of the bed material median diameter, D_{50}, is used as an indicator for clear-water or live-bed scour conditions. Clear-water scour is assumed to occur when the average velocity, V, upstream of the bridge is less than or equal to V_C for the D_{50} of the bed material. Live-bed scour is assumed to occur if V is greater than V_C. The critical velocity was calculated using equation 6.1 from HEC-18:

$$V_C = K_u y^{1/6} D^{1/3}$$

Where:

$V_C = Critical\ velocity\ above\ which\ bed\ material\ of\ size\ D\ and\ smaller\ will\ be\ transported\ \left(\frac{ft}{s}\right)$
$y = Average\ depth\ of\ flow\ upstream\ of\ the\ bridge\ (ft)$
$D = Particle\ size\ for\ V_C\ (ft)$
$D_{50} = Particle\ size\ in\ a\ mixture\ of\ which\ 50\ percent\ are\ smaller\ (ft)$
$K_u = 11.17\ English\ units$

The critical velocity was calculated for the bed material (D_{50}=5.1 mm) at the bridge, and it was found that the channel velocity exceeded the critical velocity for particles of that size, so live-bed scour was used for the scour analysis. The calculation is included in Attachment 4.

6.5 Contraction Scour

Contraction scour occurs when the flow area of a stream is reduced due by either a bridge structure or a natural contraction of the channel. The reduction of flow area causes a corresponding increase in average velocity of the flow, resulting in increased erosion. The scour will reach maximum depth once the flow area is increased to the point at which there is no net sediment loss from the area.

Contraction scour calculations were performed as part of this study using the Hydraulic Design Function in the HEC-RAS model used for the hydraulic analysis. The Hydraulic Design Function uses the equation presented below to calculate live-bed contraction scour. The output from the calculations is included in Attachment 4 and the calculated scour depths are shown in Table 4.

Live-bed contraction scour was calculated using equation 6.2 from HEC-18, which is a modified version of Laursen's equation:

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1}$$

Where:

$y_s = y_2 - y_0 = Average\ contraction\ scour\ depth$
$y_1 = Average\ depth\ in\ the\ upstream\ main\ channel\ (ft)$
$y_2 = Average\ depth\ in\ the\ contracted\ section\ (ft)$
$y_0 = Existing\ depth\ in\ the\ contracted\ section\ before\ scour\ (ft)$
$Q_1 = Flow\ in\ the\ upstream\ channel\ transporting\ sediment\ (ft^3/s)$
$Q_2 = Flow\ in\ the\ contracted\ channel\ (ft^3/s)$
$W_1 = Bottom\ width\ of\ the\ upstream\ main\ channel\ that\ is\ transporting\ bed\ material\ (ft)$
$W_2 = Bottom\ width\ of\ main\ channel\ in\ contracted\ section\ less\ pier\ widths\ (ft)$
$k_1 = Mode\ of\ bed\ material\ transport\ exponent$

Table 4 Contraction Scour Depths

Location	Scour Depth (ft)
Left Overbank (looking downstream)	N/A
Channel	4.6
Right Overbank (looking downstream)	2.1

6.6 Pier Scour

Local scour at bridge piers occurs due to the formation of vortices at the base of the piers which causes the flow to accelerate in that area, resulting in increased sediment transport. The magnitude of pier scour is dependent on the flow velocity, flow depth, pier width, size and gradation of bed material, pier shape, and other factors. Pier scour increases with increased flow velocity, flow depth, and pier width.

Maximum pier scour depth can be predicted using the HEC-18 Pier Scour Equation, which is based on the Colorado State University (CSU) equation, for both clear-water and live-bed scour. The equation is based on the Colorado State University equation and was calculated as follows:

$$\frac{y_s}{y_1} = 2\ K_1 K_2 K_3 \left(\frac{a}{y_1}\right)^{0.65}\ Fr_1^{0.43}$$

Where:

$$y_s = Scour\ depth\ (ft)$$
$$y_1 = Flow\ depth\ depth\ directly\ upstream\ of\ the\ pier\ (ft)$$
$$K_1 = Correlation\ factor\ of\ pier\ nose\ shape$$
$$K_2 = Correlation\ factor\ of\ angle\ of\ attack\ of\ flow$$
$$K_3 = Correlation\ factor\ for\ bed\ condition$$
$$a = Pier\ width\ (ft)$$
$$Fr_1 = Froude\ Number\ directly\ upstream\ of\ the\ pier = \ V_1/(gy_1)^{1/2}$$
$$V_1 = Mean\ velocity\ of\ flow\ directly\ upstream\ of\ the\ pier\ (ft/s)$$
$$g = Acceleration\ due\ to\ gravity = 32.2\ (ft/s^2)$$

A Rule of Thumb for maximum scour depth for round nose piers aligned with flow is given in HEC-18 as:

$$y_s \leq 2.4a\ for\ Fr \leq 0.8$$

$$y_s \leq 3.0a\ for\ Fr > 0.8$$

Pier scour calculations were performed as part of this study for the two proposed bents using the Hydraulic Design Function in the HEC-RAS model used for the hydraulic analysis. The Hydraulic Design Function uses the equation presented above to calculate pier scour. The calculations were performed using a 5-foot diameter pier to represent an average of the 4-foot diameter of the columns and the 6-foot-diameter piles that would be exposed due to long term-degradation and contraction scour in the channel. The output from the calculations is included in Attachment 4.

The pier scour calculations performed in HEC-RAS do not account for debris loading on the piers which can increase scour at the piers. After the January 10, 2023 flow event that damaged the bridge, significant debris was observed on the upstream side of the bridge piers. To account for debris accumulation on the piers when considering scour, pier scour calculations were also performed as part of this study outside of HEC-RAS. Those calculations used the same method from HEC-18 described above but use an effective pier width that is calculated based on an assumed debris accumulation of 12 feet wide and 6 feet high. Those calculations were used as the basis for the design and are included in Attachment 4 and the calculated scour depths are shown in Table 5.

Table 5 Pier Scour Depths

Support Location	Scour Depth (ft)
Pier 2	15.8
Pier 3	15.8

6.7 Abutment Scour

Abutment scour occurs when the abutment and roadway embankment obstruct flow and cause contraction and turbulence of the flow at the abutment. Abutment scour was calculated using Equation 8.1 from HEC-18, which is known as the Froelich equation:

$$\frac{y_s}{y_a} = 2.27\, K_1 K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1$$

Where:

$y_s = Scour\ depth\ (ft)$

$K_1 = Coefficient\ for\ abutment\ shape$

$K_2 = Coefficient\ for\ angle\ of\ embankment\ to\ flow$

$L' = Length\ of\ active\ flow\ obstructed\ by\ the\ embankment\ (ft)$

$A_e = Flow\ area\ of\ the\ approach\ cross\ section\ obstructed\ by\ the\ embankment\ (ft^2)$

$Fr = Froude\ Number\ directly\ upstream\ of\ the\ pier = V_e/(gy_a)^{1/2}$

$V_e = Q_e/A_e\ (ft/s)$

$Q_e = Fow\ obstructed\ by\ the\ abutment\ and\ approach\ embankment\ (ft^3/s)$

$y_a = Average\ depth\ of\ flow\ on\ the\ floodplain\ (A_e/L)\ (ft)$

$L = Length\ of\ embankment\ projected\ normal\ to\ the\ flow\ (ft)$

$g = Acceleration\ due\ to\ gravity = 32.2\ (ft/s^2)$

Abutment scour calculations were performed as part of this study for the proposed abutment using the Hydraulic Design Function in the HEC-RAS models used for the hydraulic analyses. The Hydraulic Design Function uses the equation presented above to calculate abutment scour and the output from the calculations is included in Attachment 4 and the calculated scour depths are shown in Table 6.

Table 6 Abutment Scour Depth

Support Location	Scour Depth (ft)
Abutment 1	23.5

6.8 Total Scour

Total scour was calculated as part of this study as the sum of the estimated long-term degradation, contraction, abutment, and pier scour. The results are shown in Table 7.

Table 7 Total Estimated Scour Depths

Support Location	Long-Term Scour Depth (ft)		Local Scour Depth (ft)		Total Scour Depth (ft)
	Long-Term Degradation	Contraction	Pier	Abutment	
Abutment 1	4.0	4.6	N/A	23.5	32.1
Pier 2	4.0	4.6	15.8	N/A	24.4
Pier 3	4.0	4.6	15.8	N/A	24.4

Table 8 Total Estimated Scour Elevation

Support Location	Approx. Existing Ground Elev. at Upstream Side of Pier or Face of Abutment (ft)	Total Scour Depth (ft)	Total Scour Elev. (ft)
Abutment 1	444.0	32.1	411.9
Pier 2	430.7	24.4	406.3
Pier 3	431.0	24.4	406.6

7. Scour Countermeasures

The scour analysis presented in Section 6 assumes that no scour countermeasures are in place. At a minimum, scour countermeasures should be installed at the railroad embankment to protect the embankment from scouring and/or breaching behind the abutment. In addition, scour countermeasures could be installed at the bents and abutment to protect against local (pier and abutment) scour. If used, these scour countermeasures should be designed in accordance with allowable methods, such as those presented in FHWA HEC No. 23 (FHWA, 2009). If scour countermeasures are incorporated into the design to protect against local scour, the design of the bridge and scour countermeasures should take into account the long-term and contraction scour of the channel.

Evaluation of the scour potential at the Old Telegraph Road bridge immediately downstream of the railroad bridge was outside of the scope of this study. Based on photographs provided, it appears that scour has damaged the grouted rock slope protection (RSP) at the western abutment of that bridge. The design of the railroad bridge and any associated scour countermeasures should be coordinated with any planned repairs at the Old Telegraph Road bridge.

8. References

FEMA. (2022). *Flood Insurance Study for Ventra County, California (Revised Preliminary: August 19, 2022).*
FHWA. (2009). *Bridge Scour and Sream Instability Countermeasures: Experience, Selection, and Design Guidance - Third Edition.*
FHWA. (2012). *Hydraulic Engineering Circular No. 18 Evaluating Scour at Bridges, Fifth Edition.*
Koppers Railroad Structures Inc. (2022). *Inspection Summary (Milepost 423.44).*
SCRRA. (2021, January). SCRRA Design Criteria Manual.
Wilson & Company. (2023). *Bridge Inspetion Report (Milepost 423.44).*

Attachments

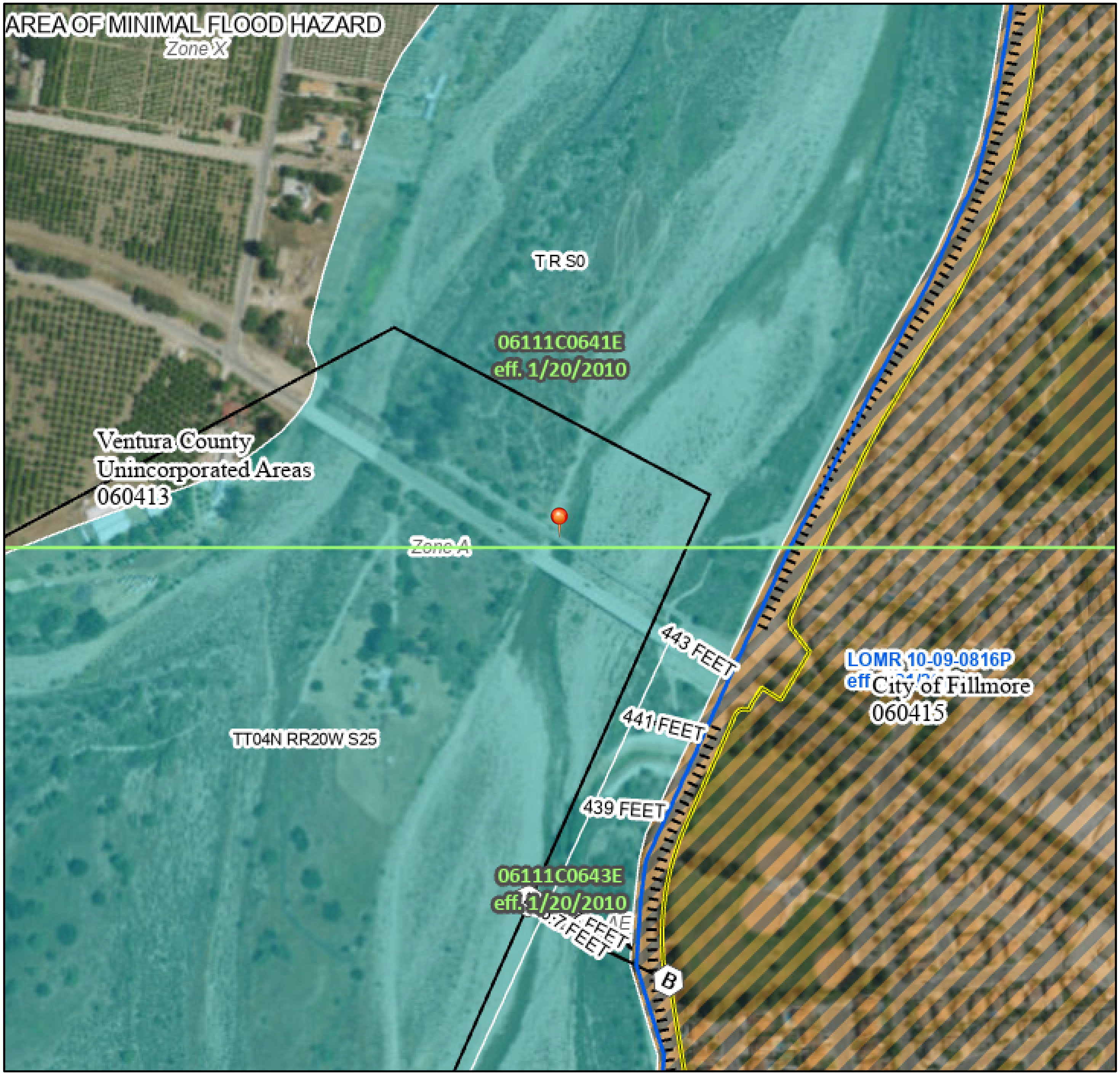
Attachment 1

FEMA FIRMette

National Flood Hazard Layer FIRMette

FEMA

118°56'14"W 34°24'38"N



0 250 500 1,000 1,500 2,000 Feet

1:6,000

118°55'36"W 34°24'8"N

Basemap Imagery Source: USGS National Map 2023

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS

- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD

- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS

- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES

- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES

- B 20.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- 8 Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS

- Digital Data Available
- No Digital Data Available
- Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

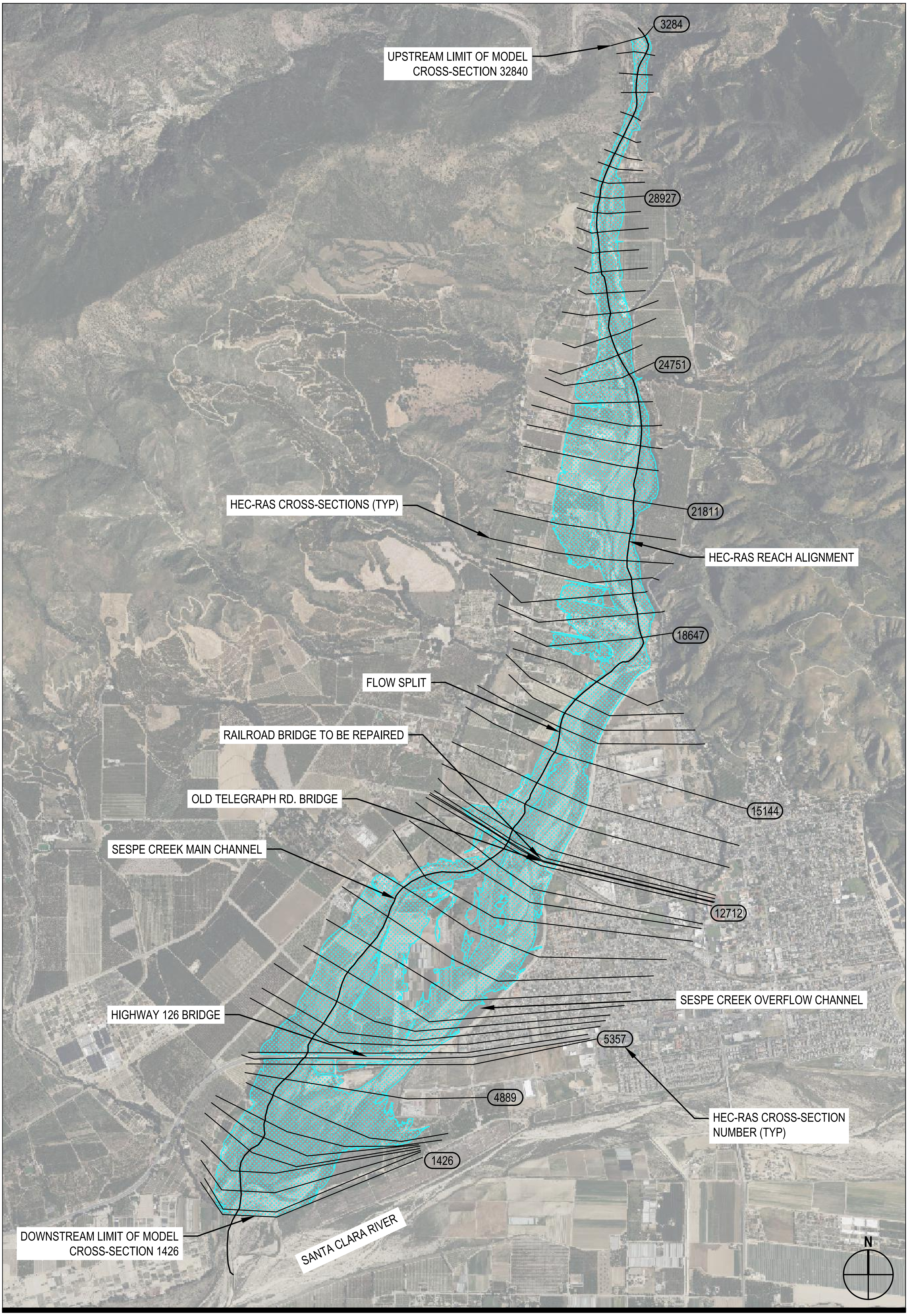
This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 9/18/2023 at 7:52 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

Attachment 2

Hydraulic Workmap



▨ APPROXIMATE LIMIT OF 100-YEAR FLOOD INUNDATION

0 2,000' 4,000'

RAILPROS
SESPE CREEK OVERFLOW CHANNEL
RAILROAD BRIDGE

HYDRAULIC WORKMAP
OVERVIEW

Project No. **12611830**
Date **10/13/23**

ATTACHMENT 2-A

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Plot Date: 17 October 2023 9:17 AM

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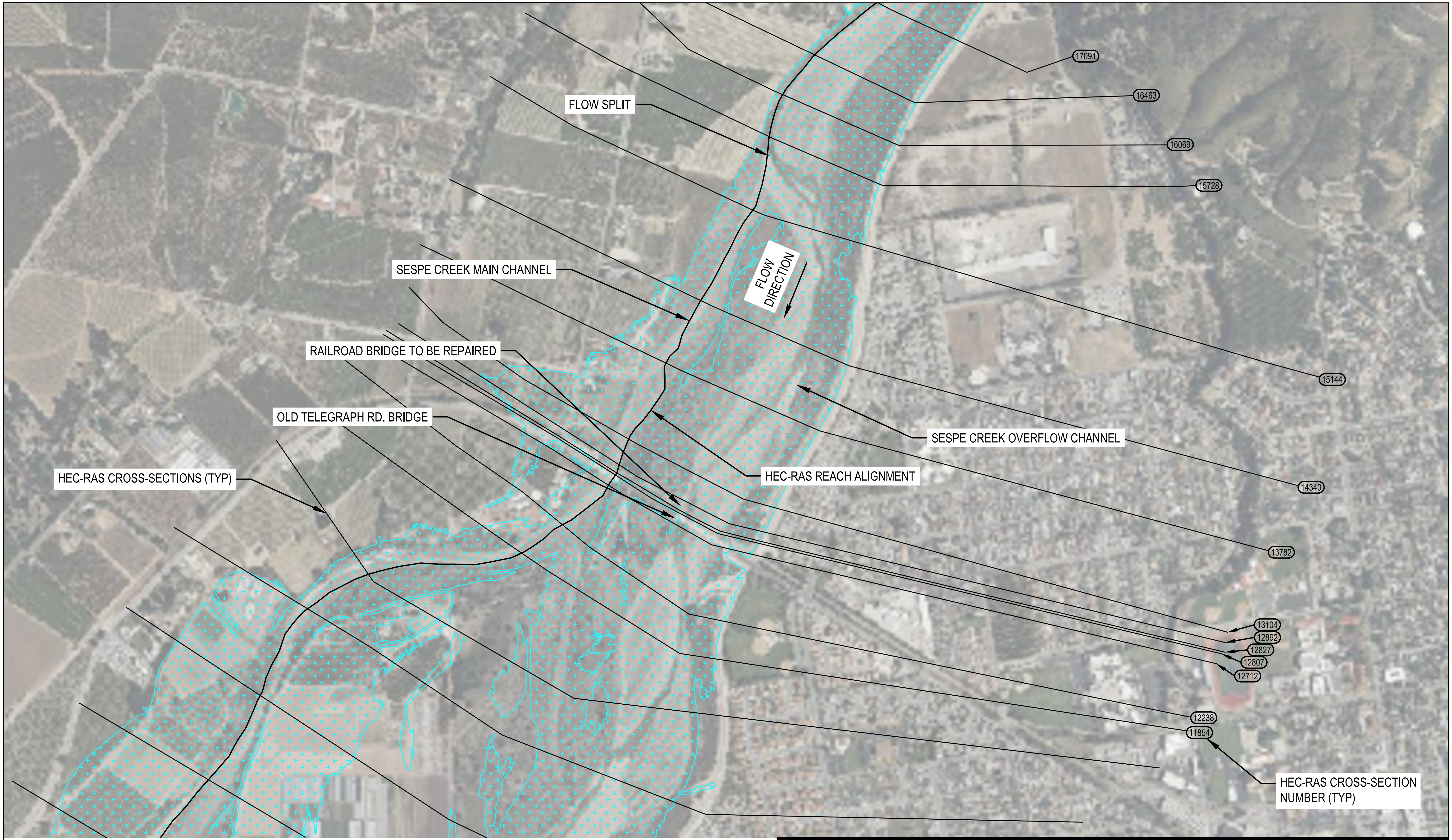

▨ APPROXIMATE LIMIT OF 100-YEAR FLOOD INUNDATION

RAILPROS
SESPE CREEK OVERFLOW RAILROAD BRIDGE

HYDRAULIC WORKMAP
PROJECT AREA

Project No. **12611830**
Date **10/13/23**

ATTACHMENT 2B

Filename: \\ghdnet\ghd\US\Roseville\Projects\561\12611830\Digital_Design\ACAD\Figures\12611830-FIG-HYDRAULIC WORKMAP.dwg
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Data Source

Attachment 3

Hydraulic Model Results

Hydraulic Model Output

Pre-project Conditions with 50-year Peak Discharge

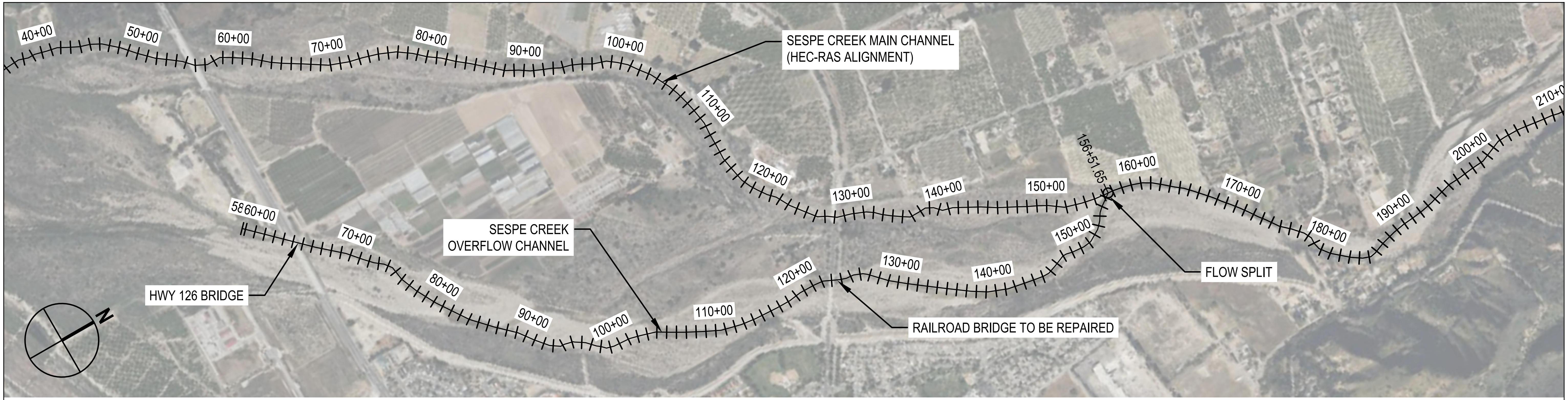
NOTE: Model Calculations omitted from the online published version of Hydrology Report due to File Size. Available upon request.

HEC-RAS Plan: Multi   River: SespeCreek   Reach: Reach1   Profile: 50Yr

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach1	32840	50Yr	102000.00	613.47	636.50	636.50	643.64	0.007742	21.44	4780.33	350.78	0.99
Reach1	32411	50Yr	102000.00	609.99	630.57	630.57	637.58	0.007584	21.26	4838.79	362.36	0.99
Reach1	31894	50Yr	102000.00	597.39	624.46	624.46	632.96	0.007102	23.46	4439.68	283.87	0.99
Reach1	31482	50Yr	102000.00	595.13	620.92	620.92	628.49	0.007631	22.07	4630.70	313.03	1.00
Reach1	30924	50Yr	102000.00	590.05	614.60	613.67	622.38	0.006251	22.39	4582.33	265.17	0.92
Reach1	30411	50Yr	102000.00	584.48	609.84	609.84	618.87	0.006990	24.15	4281.21	249.81	0.99
Reach1	29986	50Yr	102000.00	580.62	604.55	604.55	613.11	0.007419	23.48	4358.91	263.80	1.00
Reach1	29672	50Yr	102000.00	579.41	600.95	600.95	608.89	0.007988	22.61	4516.91	290.72	1.00
Reach1	29284	50Yr	102000.00	575.87	597.91	597.91	604.80	0.006384	21.16	5084.58	450.74	0.93
Reach1	28927	50Yr	102000.00	574.82	595.73	595.73	601.67	0.005352	20.15	6006.02	593.44	0.85
Reach1	28571	50Yr	102000.00	573.87	591.53	591.53	597.48	0.005854	19.91	5634.34	604.69	0.89
Reach1	28122	50Yr	102000.00	567.66	587.62	587.62	594.24	0.006327	21.05	5168.36	411.41	0.93
Reach1	27668	50Yr	102000.00	562.66	584.29	584.29	591.10	0.007020	21.04	4990.71	420.10	0.97
Reach1	27222	50Yr	102000.00	561.98	581.38		585.06	0.005730	15.49	6809.87	631.93	0.74
Reach1	26715	50Yr	102604.00	556.60	578.14		581.03	0.010160	13.63	7560.96	506.44	0.61
Reach1	26242	50Yr	102604.00	554.12	570.11	570.11	575.83	0.010644	19.17	5354.84	476.48	1.00
Reach1	25713	50Yr	102604.00	546.47	564.33	564.33	569.14	0.007984	17.67	6025.20	698.38	0.97
Reach1	25258	50Yr	102604.00	544.62	561.98	560.95	565.43	0.006169	14.92	6924.36	765.23	0.85
Reach1	24751	50Yr	102604.00	541.72	558.67	558.02	561.97	0.007391	14.60	7234.60	1224.52	0.88
Reach1	24081	50Yr	102604.00	536.87	552.36	552.36	556.38	0.009222	16.09	6376.04	805.38	1.01
Reach1	23531	50Yr	102604.00	531.05	546.82	546.82	550.59	0.006874	15.63	7000.15	1460.74	0.92
Reach1	23080	50Yr	102604.00	527.22	543.89	543.89	546.55	0.006218	14.21	9807.25	1934.20	0.84
Reach1	22567	50Yr	102604.00	520.15	535.86	535.86	538.28	0.005957	13.70	9455.00	2033.58	0.75
Reach1	21811	50Yr	102604.00	511.42	525.86	525.86	529.23	0.008057	15.12	7462.23	1247.09	0.94
Reach1	21055	50Yr	102604.00	501.73	517.64	517.64	521.09	0.008089	14.93	7158.14	1456.78	0.94
Reach1	20433	50Yr	102604.00	497.18	511.17	511.17	514.76	0.008463	15.28	6873.12	1011.94	0.97
Reach1	19991	50Yr	102604.00	489.17	505.89	505.89	509.94	0.011713	16.15	6353.25	792.90	1.01
Reach1	19675	50Yr	102604.00	485.77	501.97	501.42	505.59	0.008152	15.26	6726.58	786.06	0.92
Reach1	19191	50Yr	102604.00	482.46	499.53		502.37	0.004816	13.52	7629.70	727.12	0.71
Reach1	18647	50Yr	102604.00	478.75	494.33	494.33	498.78	0.008586	16.92	6077.58	1318.82	1.00
Reach1	17091	50Yr	102604.00	467.11	482.74	482.74	487.37	0.005720	17.71	6461.67	738.78	0.86
Reach1	16463	50Yr	102604.00	463.81	477.19	477.19	480.83	0.011882	15.30	6705.55	920.83	1.00
Reach1	16069	50Yr	102604.00	460.12	472.92	472.87	476.46	0.008664	15.14	6856.27	980.80	0.97
Reach1	15728	50Yr	102604.00	455.80	469.67	469.67	473.53	0.008348	15.78	6584.26	911.62	0.97
Reach1	15144	50Yr	102604.00	447.74	462.80	462.80	466.64	0.007607	15.72	6526.92	847.31	1.00
Reach1	14340	50Yr	102604.00	443.62	456.56	456.56	459.53	0.008122	14.12	7959.79	1376.90	0.95
Reach1	13782	50Yr	102604.00	438.49	452.15	450.51	453.91	0.005208	10.70	9723.88	1405.45	0.71
Reach1	13104	50Yr	102604.00	433.71	451.79	444.84	452.39	0.000848	6.31	17001.11	1712.84	0.31
Reach1	12892	50Yr	102604.00	434.72	451.62	445.12	452.20	0.000743	6.67	16758.66	1622.19	0.31
Reach1	12852		Mult Open									
Reach1	12827	50Yr	102604.00	434.59	448.45	444.15	450.02	0.001934	9.52	10217.87	1497.92	0.49
Reach1	12807	50Yr	102604.00	434.42	448.44	444.07	449.95	0.001880	9.56	10547.23	1157.32	0.48
Reach1	12780		Mult Open									
Reach1	12712	50Yr	102604.00	434.07	442.80	442.80	446.92	0.008639	14.42	6342.99	3808.91	0.96
Reach1	12238	50Yr	102604.00	428.86	437.85	437.85	441.08	0.005768	10.38	7363.93	3016.71	0.76
Reach1	11854	50Yr	102604.00	428.14	434.18	434.18	436.82	0.005845	10.07	8197.13	3236.58	0.76
Reach1	10652	50Yr	102604.00	418.20	427.37	427.37	429.87	0.007163	13.08	8119.27	1657.86	0.87
Reach1	10111	50Yr	102604.00	412.64	422.25	422.25	424.64	0.006065	12.63	8295.56	1743.66	0.81
Reach1	9338	50Yr	102604.00	408.13	417.40	417.40	419.04	0.005342	10.99	10318.84	2940.90	0.75
Reach1	8571	50Yr	102604.00	400.71	411.65	411.65	413.40	0.005235	11.32	10213.88	2853.94	0.75
Reach1	7774	50Yr	102604.00	397.88	404.99	404.99	406.71	0.009050	13.66	9951.39	2901.84	0.96
Reach1	7062	50Yr	102604.00	393.83	402.18	401.63	403.39	0.005100	9.45	11689.83	3232.46	0.78
Reach1	6474	50Yr	102604.00	388.98	401.70	398.28	402.16	0.001295	6.50	20405.12	3438.08	0.39
Reach1	6159	50Yr	102604.00	388.55	401.52	397.09	401.84	0.000698	5.76	24941.21	3492.51	0.30
Reach1	5697	50Yr	102604.00	384.34	400.09	395.51	400.83	0.001151	8.07	15557.92	3619.39	0.39
Reach1	5552		Mult Open									
Reach1	5357	50Yr	102604.00	383.52	394.34	394.15	396.85	0.006711	14.01	8486.65	2858.30	0.86
Reach1	4899	50Yr	102604.00	381.58	391.64	391.45	393.36	0.006817	14.99	11199.36	2433.98	0.89
Reach1	4441	50Yr	102604.00	377.19	388.70		389.64	0.005708	13.99	15171.22	3872.39	0.78
Reach1	3829	50Yr	102604.00	373.71	384.57	383.95	386.04	0.006941	11.79	11056.22	2832.15	0.84
Reach1	3433	50Yr	102604.00	371.89	380.66	380.66	382.71	0.009249	10.81	8989.49	2182.83	0.93
Reach1	2991	50Yr	102604.00	369.65	379.29	377.74	380.25	0.003523	8.34	13190.33	2628.60	0.60
Reach1	2449	50Yr	102604.00	365.18	376.73	376.73	378.49	0.007478	12.43	10529.02	2743.64	0.86
Reach1	2002	50Yr	102604.00	360.59	376.21	372.42	376.78	0.001252	6.73	18357.01	2749.30	0.36
Reach1	1562	50Yr	102604.00	359.68	374.33	372.65	375.82	0.003195	10.82	12153.92	2691.99	0.61
Reach1	1426	50Yr	102604.00	359.40	374.12	372.43	375.33	0.002999	10.02	13100.40	3028.28	0.57

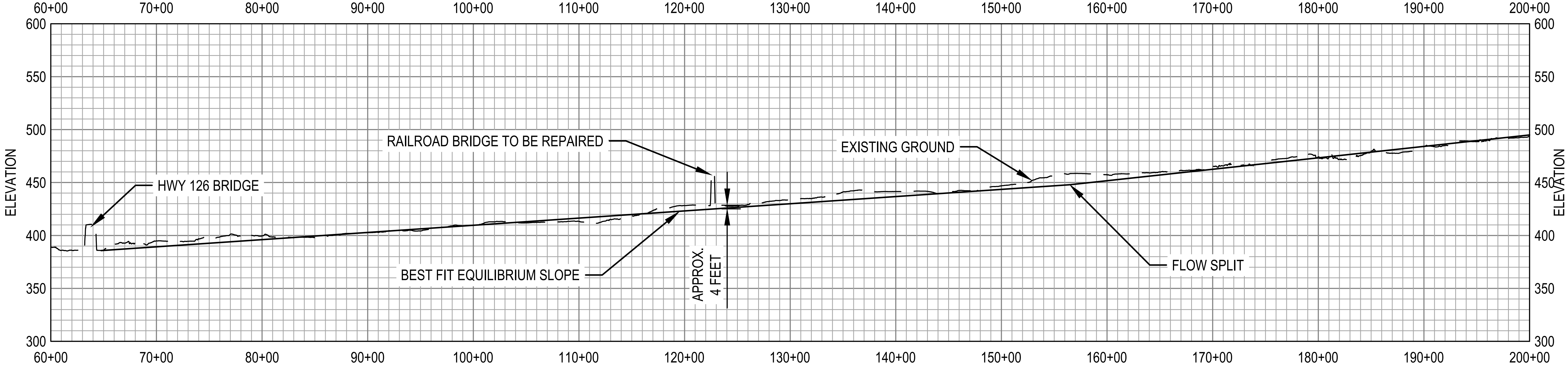
Attachment 4

Bridge Scour Calculations



1 PLAN VIEW

1" = 1,000'



2 PROFILE VIEW

H: 1" = 1,000'; V: 1" = 100'

NOTE: THE EXISTING GROUND PROFILE WAS DEVELOPED USING A SURFACE CREATED FROM THE 2018 USGS LIDAR: SOUTHERN CA WILDFIRES DATA SET.

0 1,000' 2,000'

GHD

RAILPROS
SESPE CREEK OVERFLOW RAILROAD BRIDGE

CHANNEL PROFILE FOR
LONG-TERM DEGRADATION ESTIMATION

Project No. **12611830**
Date **10/16/23**

ATTACHMENT 4A

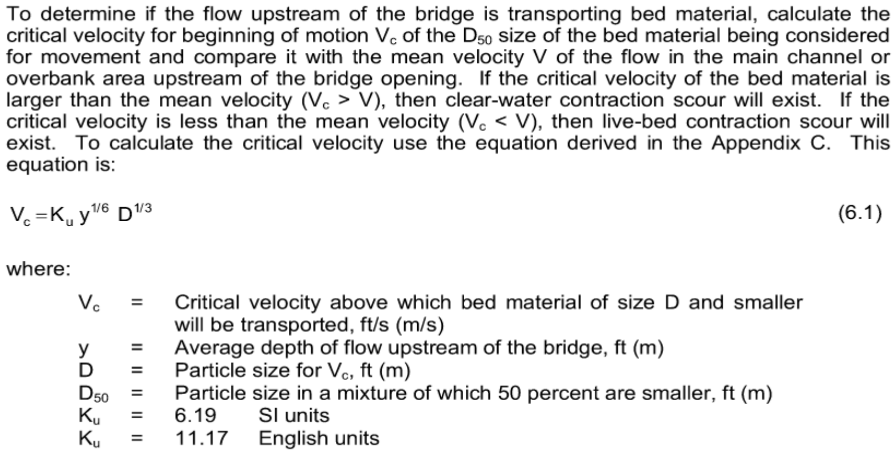
Filename: \\ghdnet\ghd\US\Roseville\Projects\561\12611830\Digital_Design\ACAD\Figures\12611830-FIG-CHANNEL PROFILE.dwg
Plot Date: 16 October 2023 1:56 PM

Data Source

Live-Bed vs Clear-Water Scour Determination

Project Name: Sespe Creek Bridge
Project No.: 12611830
Updated: 9/25/2023
Calc By: STS

The following calculations are based on the methods presented in FHWA HEC No. 18, Fifth Edition for calculating critical velocity.



To determine if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c of the D_{50} size of the bed material being considered for movement and compare it with the mean velocity V of the flow in the main channel or overbank area upstream of the bridge opening. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V$), then clear-water contraction scour will exist. If the critical velocity is less than the mean velocity ($V_c < V$), then live-bed contraction scour will exist. To calculate the critical velocity use the equation derived in the Appendix C. This equation is:

$$V_c = K_u y^{1/6} D^{1/3} \tag{6.1}$$

where:

V_c = Critical velocity above which bed material of size D and smaller will be transported, ft/s (m/s)
y = Average depth of flow upstream of the bridge, ft (m)
D = Particle size for V_c, ft (m)
D_{50} = Particle size in a mixture of which 50 percent are smaller, ft (m)
K_u = 6.19 SI units
K_u = 11.17 English units

Critical Velocity Calculation

Input Parameters

Ku: 11.17
D_{50}: 0.0167 ft (5.1 mm)

Flow Scenario	Flow* (cfs)	Avg. Flow Depth*, y (ft)	Critical Velocity, V_c (ft/s)	Channel Velocity*, V (ft/s)	Contraction Scour Type
100-yr Sespe Creek Overflow	88,957	16.25	4.55	8.81	Live-Bed

*Channel Flow, Velocity and Avg. Flow Depth are from HEC-RAS output.

HEC-RAS Hydraulic Design Function Scour Calculations

Contraction Scour

	Left	Channel	Right
Input Data			
Average Depth (ft):	8.16	16.25	11.09
Approach Velocity (ft/s):	4.27	8.81	5.23
Br Average Depth (ft):	8.44	15.04	0.99
BR Opening Flow (cfs):		93840.61	3171.73
BR Top WD (ft):	167.73	497.10	422.78
Grain Size D50 (mm):	5.10	5.10 5.10	
Approach Flow (cfs):	3954.72	88957.41	4043.09
Approach Top WD (ft):	113.59	620.90	69.70
K1 Coefficient:	0.590	0.640	0.590
Results			
Scour Depth Ys (ft):		4.57	2.12
Critical Velocity (ft/s):	8.79	4.56	4.28
Equation:		Live	Live

Pier: #2 (CL = 4109)

Input Data

Pier Shape:	Round nose
Pier Width (ft):	5.00
Grain Size D50 (mm):	5.10000
Depth Upstream (ft):	24.52
Velocity Upstream (ft/s):	9.24
K1 Nose Shape:	1.00
Pier Angle:	0.00
Pier Length (ft):	19.50
K2 Angle Coef:	1.00
K3 Bed Cond Coef:	1.10
Grain Size D90 (mm):	
K4 Armouring Coef:	1.00

Results

Scour Depth Ys (ft):	11.89 (without debris)
Froude #:	0.33
Equation:	CSU equation

Pier: #3 (CL = 4060)

Input Data

Pier Shape:	Round nose
Pier Width (ft):	5.00
Grain Size D50 (mm):	5.10000
Depth Upstream (ft):	24.52
Velocity Upstream (ft/s):	9.24
K1 Nose Shape:	1.00
Pier Angle:	0.00
Pier Length (ft):	19.50
K2 Angle Coef:	1.00
K3 Bed Cond Coef:	1.10
Grain Size D90 (mm):	
K4 Armouring Coef:	1.00

Results

Scour Depth Ys (ft):	11.89 (without debris)
Froude #:	0.33
Equation:	CSU equation

Abutment #1 Scour

Input Data

Station at Toe (ft): 4158.00
Toe Sta at appr (ft): 4477.90
Abutment Length (ft): 52.95
Depth at Toe (ft): 13.00
K1 Shape Coef: 0.82 - Vert. with wing walls
Degree of Skew (degrees): 90.00
K2 Skew Coef: 1.00
Projected Length L' (ft): 52.95
Avg Depth Obstructed Ya (ft): 7.00
Flow Obstructed Qe (cfs): 4217.99
Area Obstructed Ae (sq ft): 795.29

Results

Scour Depth Ys (ft): 23.49
Qe/Ae = Ve: 5.30
Froude #: 0.35
Equation: Froehlich

Pier Scour - Effective Pier Width with Debris

Project Name: Sespe Creek Bridge
Project No.: 12611830
Updated: 10/17/2023
Calc By: STS

The following calculations are based on the methods presented in FHWA HEC No. 18, Fifth Edition for calculating the effective pier width with debris.

Equation 7.32 (HEC-18 Equation):

$$a^*_d = \frac{K_1(HW) + (y - K_1H)a}{y} \qquad (7.32)$$

where:

a^*_d = Effective width of pier when debris is present, ft (m)
a = Width of pier perpendicular to flow, ft (m)
K_1 = 0.79 for rectangular debris, 0.21 for triangular debris
H = Height (thickness) of the debris, ft (m)
W = Width of debris perpendicular to the flow direction, ft (m)
y = Depth of approach flow, ft (m)

a: 5.00 ft
K_1: 0.79
H: 6.00 ft (assumed from 5/8/23 Bridge Inspection Report)
W: 12.00 ft (assumed from 5/8/23 Bridge Inspection Report)
y: 12.22 ft (from HEC-RAS model)

a^*_d: 7.72 ft

Pier Scour - Pier 2

Project Name: Sespe Creek Bridge
Project No.: 12611830
Updated: 10/17/2023
Calc By: STS

The following calculations are based on the methods presented in FHWA HEC No. 18, Fifth Edition for calculating pier scour.

Location: Pier 2

Equation 7.1 (HEC-18 Equation):

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ K_3 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}$$

As a Rule of Thumb, the maximum scour depth for round nose piers aligned with the flow is:

$y_s \le 2.4$ times the pier width (a) for $Fr \le 0.8$ (7.2)
$y_s \le 3.0$ times the pier width (a) for $Fr > 0.8$

where:

y_s = Scour depth, ft (m)
y_1 = Flow depth directly upstream of the pier, ft (m)
K_1 = Correction factor for pier nose shape from Figure 7.3 and Table 7.1
K_2 = Correction factor for angle of attack of flow from Table 7.2 or Equation 7.4
K_3 = Correction factor for bed condition from Table 7.3
a = Pier width, ft (m)
L = Length of pier, ft (m)
Fr_1 = Froude Number directly upstream of the pier = $V_1/(gy_1)^{1/2}$
V_1 = Mean velocity of flow directly upstream of the pier, ft/s (m/s)
g = Acceleration of gravity (32.2 ft/s^2) (9.81 m/s^2)

Shape:	Round Nose
y_1:	24.52 ft
K_1:	1.0
K_2:	1.0
K_3:	1.1
a:	7.72 ft (using effective pier width with debris accumulation)
L:	19.5 ft
Fr_1:	0.33
V_1:	9.2 ft/s
g:	32.2 ft/s^2
θ:	0 degrees
Scour Depth y_s:	**15.8 ft**

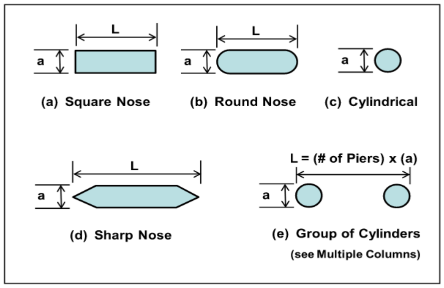


Figure 7.3. Common pier shapes.

Equation 7.4:

$$K_2 = (\cos\theta + \frac{L}{a}\sin\theta)^{0.65}$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 7.4 and Table 7.2. Table 7.2 illustrates the magnitude of the effect of the angle of attack on local pier scour.

Table 7.1. Correction Factor, K_1, for Pier Nose Shape.

Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group of cylinders	1.0
(e) Sharp nose	0.9

Table 7.2. Correction Factor, K_2, for Angle of Attack, 2, of the Flow.

Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Angle = skew angle of flow
L = length of pier

Table 7.3. Increase in Equilibrium Pier Scour Depths, K_3, for Bed Condition.

Bed Condition	Dune Height ft	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	$10 > H \ge 2$	1.1
Medium Dunes	$30 > H \ge 10$	1.2 to 1.1
Large Dunes	$H \ge 30$	1.3

Pier Scour - Pier 3

Project Name: Sespe Creek Bridge
Project No.: 12611830
Updated: 10/17/2023
Calc By: STS

The following calculations are based on the methods presented in FHWA HEC No. 18, Fifth Edition for calculating pier scour.

Location: Pier 3

Equation 7.1 (HEC-18 Equation):

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ K_3 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}$$

As a Rule of Thumb, the maximum scour depth for round nose piers aligned with the flow is:

$y_s \leq 2.4$ times the pier width (a) for $Fr \leq 0.8$ (7.2)
$y_s \leq 3.0$ times the pier width (a) for $Fr > 0.8$

where:

y_s = Scour depth, ft (m)
y_1 = Flow depth directly upstream of the pier, ft (m)
K_1 = Correction factor for pier nose shape from Figure 7.3 and Table 7.1
K_2 = Correction factor for angle of attack of flow from Table 7.2 or Equation 7.4
K_3 = Correction factor for bed condition from Table 7.3
a = Pier width, ft (m)
L = Length of pier, ft (m)
Fr_1 = Froude Number directly upstream of the pier = $V_1/(gy_1)^{1/2}$
V_1 = Mean velocity of flow directly upstream of the pier, ft/s (m/s)
g = Acceleration of gravity (32.2 ft/s^2) (9.81 m/s^2)

Shape: Round Nose
y_1: 24.52 ft
K_1: 1.0
K_2: 1.0
K_3: 1.1
a: 7.72 ft (using effective pier width with debris accumulation)
L: 19.5 ft
Fr_1: 0.33
V_1: 9.2 ft/s
g: 32.2 ft/s^2
θ: 0 degrees

Scour Depth y_s: 15.8 ft

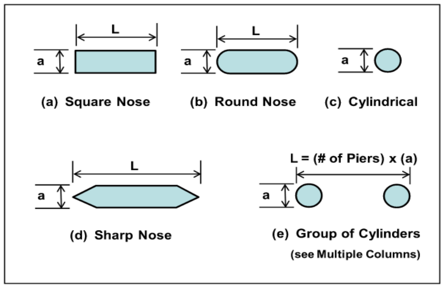


Figure 7.3. Common pier shapes.

Equation 7.4:

$$K_2 = (\cos\theta + \frac{L}{a}\sin\theta)^{0.65}$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 7.4 and Table 7.2. Table 7.2 illustrates the magnitude of the effect of the angle of attack on local pier scour.

Table 7.1. Correction Factor, K_1, for Pier Nose Shape.

Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group of cylinders	1.0
(e) Sharp nose	0.9

Table 7.2. Correction Factor, K_2, for Angle of Attack, 2, of the Flow.

Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Angle = skew angle of flow
L = length of pier

Table 7.3. Increase in Equilibrium Pier Scour Depths, K_3, for Bed Condition.

Bed Condition	Dune Height ft	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	$10 > H \geq 2$	1.1
Medium Dunes	$30 > H \geq 10$	1.2 to 1.1
Large Dunes	$H \geq 30$	1.3

Attachment 5

Geotechnical Data

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D6913

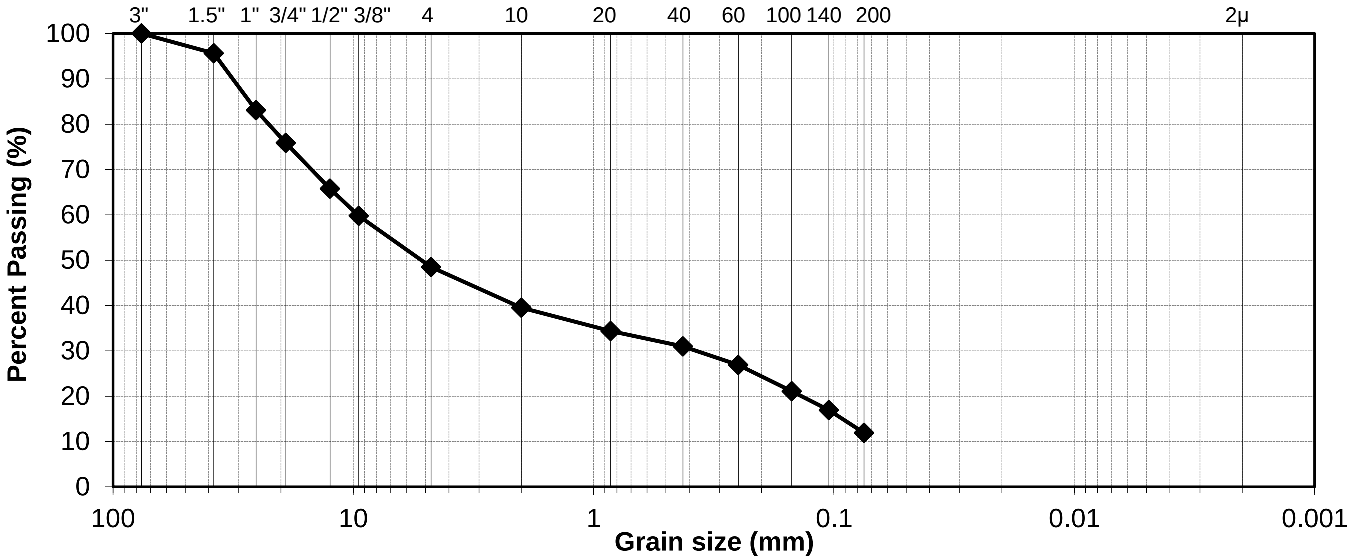
Client:	Diaz Yourman	*HAI Project No.:*	DYAL-23-008-2
Project Name:	VCTC Sespe Creek Bridge	*Tested by:*	GA
Project No.:	2023-010	*Checked by:*	KL
Boring No.:	DYB23-01	*Date:*	08/01/23
Sample No.:	0		
Depth (ft):	0-5		
Sample Description:	Light Brown, Poorly Graded Gravel with Silt and Sand (GP-GM)		

Dry Weight (g) **16097.7**

Sieve Size	Aperture	Weight Retained	% Retained	**(Accumulative) % Passing**	**Project Specification**
	mm	g	%	**%**	**%**
3"	76.2	0.00	0.0	**100.0**	**-**
1.5"	38.1	700.01	4.3	**95.7**	**-**
1"	25.4	2018.62	12.5	**83.1**	**-**
3/4 "	19.1	1166.06	7.2	**75.9**	**-**
1/2 "	12.5	1619.76	10.1	**65.8**	**-**
3/8 "	9.5	972.52	6.0	**59.8**	**-**
# 4	4.75	1817.43	11.3	**48.5**	**-**

Dry Weight (g) **680.2**

# 10	2.00	125.72	18.5	**39.5**	**-**
# 20	0.85	71.79	10.6	**34.4**	**-**
# 40	0.425	48.28	7.1	**31.0**	**-**
# 60	0.250	57.25	8.4	**26.9**	**-**
# 100	0.150	81.18	11.9	**21.1**	**-**
# 140	0.105	58.54	8.6	**16.9**	**-**
# 200	0.075	70.41	10.4	**11.9**	**-**
Pan		167.06	24.6	**0.0**	**-**



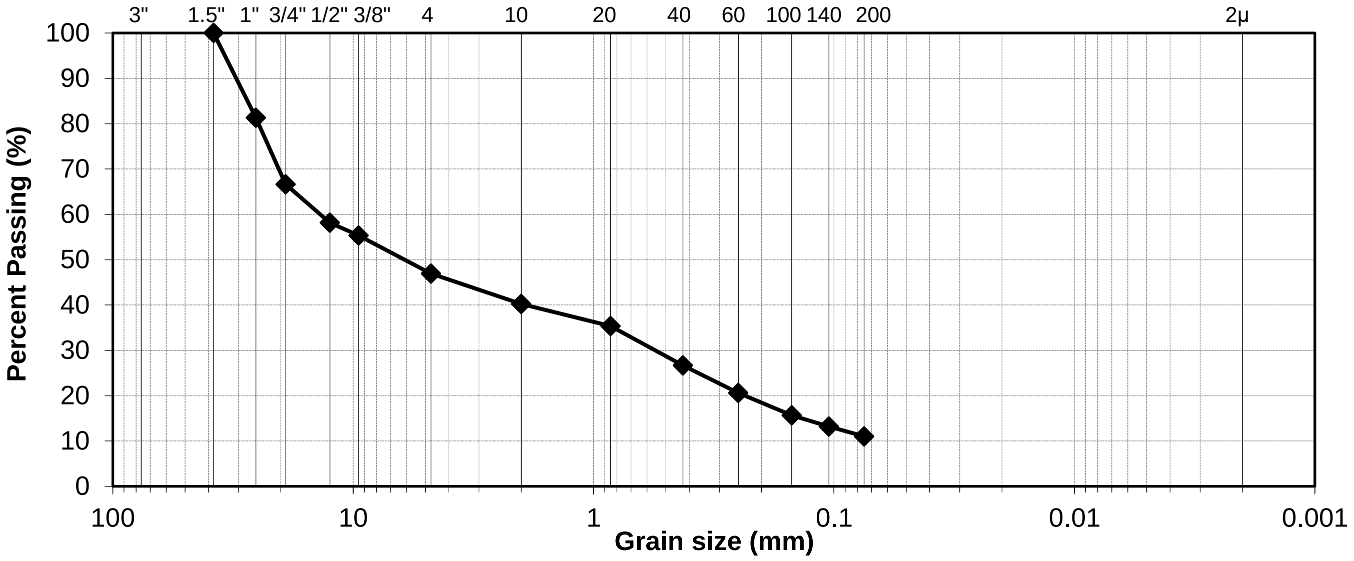
Particle-Size Analysis	D_{10}	0.07	% Gravel	% Sand	% Fines
	D_{30}	0.38	51.5	36.6	11.9
	D_{60}	9.62	Sample Description / USCS Classification		
	C_u	137.38	Light Brown, Poorly Graded Gravel with Silt and Sand (GP-GM)		
	C_c	0.22			

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM D6913

Client:	Diaz Yourman	*HAI Project No.:*	DYAL-23-008-2
Project Name:	VCTC Sespe Creek Bridge	*Tested by:*	GA
Project No.:	2023-010	*Checked by:*	KL
Boring No.:	DYB23-01	*Date:*	08/01/23
Sample No.:	2		
Depth (ft):	10		
Sample Description:	Olive Brown, Poorly Graded Gravel with Silt and Sand (GP-GM)		

Dry Weight (g) **824.4**

Sieve Size	Aperture	Weight Retained	% Retained	**% Passing**	**Project Specification**
	mm	g	%	**%**	**%**
3"	76.2	0.00	0.0	**100.0**	-
1.5"	38.1	0.00	0.0	**100.0**	-
1"	25.4	154.16	18.7	**81.3**	-
3/4 "	19.1	121.21	14.7	**66.6**	-
1/2 "	12.5	69.63	8.4	**58.2**	-
3/8 "	9.5	23.15	2.8	**55.3**	-
# 4	4.75	69.19	8.4	**47.0**	-
# 10	2.00	55.27	6.7	**40.2**	-
# 20	0.85	40.54	4.9	**35.3**	-
# 40	0.425	71.48	8.7	**26.7**	-
# 60	0.250	49.80	6.0	**20.6**	-
# 100	0.150	40.92	5.0	**15.7**	-
# 140	0.105	20.46	2.5	**13.2**	-
# 200	0.075	17.70	2.1	**11.0**	-
Pan		90.91	11.0	**0.0**	-



Particle-Size Analysis	D_{10}	0.07	% Gravel	% Sand	% Fines
	D_{30}	0.59	53.0	35.9	11.0
	D_{60}	13.94	Sample Description / USCS Classification		
	C_u	199.20	Olive Brown, Poorly Graded Gravel with Silt and Sand (GP-GM)		
	C_c	0.36			

RAILPROS

QUALITY FORM

QUALITY ASSURANCE STATEMENT

CLIENT: VCTC

CONTRACT NO: N/A

PROJECT: Sespe Creek Bridge Overflow

DATE: 11/1/23

SUBMITTAL: 90 % Design Submittal

CONTRACT TITLE: AGREEMENT BETWEEN VENTUR A COUNTY TRANSPORTATION COMMISSION AND RAILPROS, INC. FOR PLANNING, DESIGN, AND ENVIRONMENTAL COMPLIANCE TO RECONSTRUCT THE SESPE CREEK OVERFLOW BRIDGE ON THE S ANTA PAULA BR ANCH LINE

This submittal contains the following design documents (check all that apply)	**X**
1. Sespe Creek Bridge 90 pct Plan_20231031 (drawings)	x
2. Calculations_ Sespe_90pct	x

Signatures below confirms that design documents included in this submittal have been reviewed in accordance with the QA/QC requirements for this project.

Quality Manager signature: ______

Comments: ______

Project Manager signature: ______

Comments: ______

ENGINEER'S ESTIMATE

Project Name: Sespe Creek Bridge Overflow Emergency Repair
Design Level: Interim Design (90%)
Last Updated: 12/13/2023

ITEM NO.	WORK DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST	NOTES
SCHEDULE XX-BASE BID						
DIVISION 01	**GENERAL REQUIREMENTS**					
01 55 26.01	Traffic Control	LS	1	$25,000.00	$25,000.00	
01 71 13.01	Mobilization, Demobilization, and Controls (Maximum of -- % of Total Bid)	LS	1	$782,444.00	$782,444.00	accelerated schedule
01 57 19.01	Erosion Control Compliance (SWPPP waiver for under 5 acres)	LS	1	$30,000.00	$30,000.00	
		GENERAL REQUIREMENTS SUBTOTAL			**$807,444.00**	
DIVISION 03	**CONCRETE**					
03 21 00.01	Reinforcing Steel	LBS	309,484	$2.00	$618,968.00	
03 31 00.01	Cast-in-place Concrete - Pier Caps and Column Infill Walls (2 Piers)	CY	79	$2,850.00	$225,007.50	
03 31 00.02	Cast-in-place Concrete - Columns 4'-0" Diameter (4 - Columns)	CY	25	$2,250.00	$56,160.00	
03 31 00.03	Cast-in-place Concrete- Abutment 1, Wingwalls, Footing	CY	241	$1,460.00	$351,276.00	
		CONCRETE SUBTOTAL			**$1,251,411.50**	
DIVISION 05	**METALS**					
05 12 23.01	Miscellaneous iron and steel	LS	1,797	$4.50	$8,086.50	
		METALS SUBTOTAL			**$8,086.50**	
DIVISION 31	**EARTHWORK**					
31 11 00.01	Site Clearing and grubbing/ shrub removal for staging area and access	CY	4,000	$211.00	$844,000.00	
31 11 50.01	Track Excavation	TON	23	$40.00	$920.00	
31 11 50.02	Remove and Dispose Track (Salvage Rail Only)	TF	200	$102.00	$20,400.00	
31 11 50.07	Demolition of Damage Bridge Portion -Concrete	CY	100	$537.90	$53,790.00	
31 11 50.07	Remove and Dispose- Damaged bridge portion and Washed out Bridge components within 500 ft radius	LS	1	$144,000.00	$144,000.00	
31 20 00.02	Positively Locate Utilities (Utility verification)	LS	1	$5,000.00	$5,000.00	
31 20 00.03	Structural Excavation (Bridge and Wingwall)	CY	692	$150.00	$103,800.00	
31 20 00.05	Structural Backfill (Bridge and Wingwall)	CY	492	$200.00	$98,400.00	
31 20 50.01	Removal and Disposal of Hazardous Materials (Category 2, 3 and 4) Allowance	ALL		$25,000.00		assuming n/a
31 11 50.09	Remove existing rock slope protection at Abutment 1	CY	5	$243.00	$1,215.00	
		EARTHWORK SUBTOTAL			**$1,271,525.00**	
DIVISION 32	**EXTERIOR IMPROVEMENTS**					
32 91 00.01	Furnish and Install Cofferdam/temporary dike	LF	250	$156.00	$39,000.00	
		EXTERIOR IMPROVEMENTS SUBTOTAL			**$39,000.00**	
DIVISION 34	**TRANSPORTATION (HIGHWAY-RAIL GRADE CROSSINGS)**					
34 71 50.01	Temporary grade crossing installation and removal	LS	1	$15,000.00	$15,000.00	
34 71 50.02	Temporary k-rail installation and removal					
		TRANS. GRADE CROSSINGS SUBTOTAL			**$15,000.00**	
DIVISION 34	**TRANSPORTATION (TRACK CONSTRUCTION)**					
34 72 00.02	Furnish 115 worn to 115 taper rail (80' segments)	LF	400	$54.40	$21,760.00	
34 72 00.02	Install new track (Timber ties) including ballast, cut spikes, plates, anchors	TF	200	$400.00	$80,000.00	

ENGINEER'S ESTIMATE

Project Name: Sespe Creek Bridge Overflow Emergency Repair

Design Level: Interim Design (90%)

Last Updated: 12/13/2023

ITEM NO.	WORK DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST	NOTES
34 72 00.02	Install sub-ballast	TON	42	$127.00	$5,334.00	
34 11 15	Furnish and install Insulated Joints	EA	2	$8,500.00	$17,000.00	
TRANS. TRACK CONSTRUCTION SUBTOTAL					**$124,094.00**	
DIVISION 34	**TRANSPORTATION (RAILROAD BRIDGES)**					
34 80 11.01	Furnish and Place unrouted Class I Riprap (Rock slope protection)	CY	618	$230.00	$142,140.00	
34 80 11.02	Furnish and Install -Rock slope protection Fabric	SQYD	408	$6.00	$2,448.00	
34 80 22.01	6'-0" Diameter Cast-In-Drilled Hole (CIDH) Pier Piles - Total 8	LF	694	$4,225.00	$2,932,150.00	
34 80 33.01	Hot Mix Asphalt (HMA) for Bridge Deck	TON	46	$842.40	$38,750.40	
34 80 32.01	Adhered Bridge Deck Waterproofing	LS	1	$114,285.00	$114,285.00	
34 80 43.01	Furnish and Erect Precast Prestressed Concrete Girder Bridge Superstructure with walkway	EA	4	$64,100.00	$256,400.00	
35 80 43.02	Finish and Erect Precast Concrete Catcher Block	EA	1	$14,000.00	$14,000.00	
34 80 51.01	Elastomeric Bearings	EA	8	$4,749.68	$37,997.44	
34 80 53.01	Cable Handrail (Structural Steel)	LF	234	$330.72	$77,388.48	
TRANS. RAILROAD BRIDGES SUBTOTAL					**$3,615,559.32**	
SCHEDULE XX - BASE BID TOTAL CONSTRUCTION COST:					**$7,132,120.32**	

ENGINEER'S ESTIMATE

Project Name: Sespe Creek Bridge Overflow Emergency Repair

Design Level: Interim Design (90%)

Last Updated: 12/13/2023

ITEM NO.	WORK DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST	NOTES
DIVISION 09	FINISHES					
09 96 23.01	Graffiti-Resistant Coating Abutment 1 and wingwalls	LS	1	$53,029.20	$53,029.20	
				FINISHES SUBTOTAL	$53,029.20	
SCHEDULE XX - BID OPTION 1 TOTAL CONSTRUCTION COST:					$53,029.20	