

LOCAL AGENCY SEISMIC RETROFIT



**SEISMIC RETROFIT STRATEGY REPORT
MENDOCINO COUNTY
GARCIA RIVER BRIDGE @ EUREKA HILL ROAD
BRIDGE NO. 10C-0034
Fed. No. BRLO-5910(068)**

**PREPARED BY:
MARK THOMAS & COMPANY, INC.
7300 FOLSOM BLVD., SUITE 203
SACRAMENTO, CA 95826**

JUNE 11, 2015

SEISMIC RETROFIT STRATEGY REPORT
GARCIA RIVER BRIDGE @ EUREKA HILL ROAD
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TABLE OF CONTENTS

BRIDGE DESCRIPTION	1
SITE DESCRIPTION AND INITIAL INVESTIGATION	1
SEISMIC ANALYSIS	2
RETROFIT STRATEGY	3
RETROFITTED STRUCTURAL FINDINGS	3
CONSTRUCTION COST ESTIMATE	5
APPENDIX 1: SEISMIC STRATEGY MEETING MINUTES 2/12/2014	6
APPENDIX 2: MEETING WITH CALTRANS MINUTES 12/21/14	9
APPENDIX 3: GENERAL PLANS	12
APPENDIX 4: DUCTILITY DEMANDS & D/C RATIOS	16
APPENDIX 5: PRELIMINARY FOUNDATION REPORT 6/20/14	46
APPENDIX 6: GEOTECHNICAL RESPONSE TO COMMENTS 11/7/2014	108
APPENDIX 7: FAULT RUPTURE REVIEW 2/17/15	122
APPENDIX 8: DRAFT LOCATION HYDRAULIC STUDY REPORT	130
APPENDIX 9: AS-BUILT PLANS	169

BRIDGE DESCRIPTION

The Garcia River Bridge was built in 1969 by the Army Corps of Engineers. It is located in Mendocino County California on Eureka Hill Road at the crossing of the Garcia River along CR 505, at Mile Marker 4.92. The City of Point Arena is approximately five miles west of the project location. The surrounding land is primarily rural residential.

The structure is a 5' deep reinforced concrete box girder superstructure with integral bent caps. It is a four span structure on a 1000 foot radius curve with no hinges. The structure is 22.7 feet wide with 2.8' overhangs on each side. The interior spans are 90' each and the end spans 70.85', for a total length of 321.7'. The abutments and single column bents are oriented perpendicular to the centerline of the structure.

The abutments are seat type with 3.5' wide seats with elastomeric bearing pads. The abutments are supported on 12.75" cast in steel shell piles. The superstructure is unrestrained in the transverse direction.

The columns are oblong 2.5' deep and 8' wide and fixed at each end. The columns are lightly reinforced with #4 ties at 12" on center. Bent foundations are founded on 12.75" diameter pipe piles with concrete infill, with 45 ton maximum design loading. Bent pile caps lack shear reinforcement and top mats of reinforcement. Piles are doweled into pile caps with 2-#6 hooked bars and a pile embedment of 6".

The Caltrans Bridge Inspection Report does not indicate any history of scour at the bridge, and it has not been retrofitted to date.

SITE DESCRIPTION AND INITIAL INVESTIGATION

The Garcia River Bridge site has been reviewed and preliminary hydraulic study and foundation reports have been completed.

The project is located in the San Andres Fault Zone. Active traces of the fault are mapped approximately 400 ft east and 1000 ft west of the bridge. Preliminary foundation report shows:

- Maximum Moment Magnitude (Mmax) of 8.0 for the site
- Peak ground acceleration (PGA) of 0.83g
- **Horizontal ground movement due to lateral spreading:**
 - 24 inches could occur at the west bank, affecting abutment 1
 - 24 inches could occur in the channel area, affecting piers 2 & 3
 - **60 inches could occur at the east bank, affecting pier 4 and abutment 5**
- Post-liquefaction settlement of:
 - up to 2.5" at abutment 1
 - up to 1" at piers 2 & 3
 - up to 7" at pier 4 and abutment 5

- Potential for fault rupture displacement of 0.82 feet based on updated Foundation Memo by Blackburn after consultation with Caltrans

The preliminary hydraulic report notes that there will be negligible change to the water surface elevation and velocity from the 100 year storm due to the proposed retrofit changes at the bridge. There are no hydraulic capacity concerns at this site.

SEISMIC ANALYSIS

Linear, elastic, 3D models were developed using the SAP2000 structural analysis program for longitudinal and transverse seismic loading. Seismic cases considered are as follows:

- Seismic load case 1: 100% longitudinal + 30% transverse
- Seismic load case 2: 30% longitudinal + 100% transverse

Liquefied soil properties have been assumed in the LPILE soil properties, and the piles will be designed for the down-drag axial forces that result after a seismic event. Fault rupture was checked following MTD 20-8. Lateral spreading has not been considered in this analysis but will be considered during final design.

Results from these analyses were used to determine column displacement and shear/capacity ratios. Column shear demands were taken as the overstrength shear demand, and P-delta effects were checked.

The following assumptions and methods were used in modeling the structure:

- The ARS curve used is a combined envelope per Blackburn Consulting Inc. recommendations, using a PGA of 0.83g
- The abutment piles have been modeled in LPILE for lateral displacements considering the liquefiable soil layers
- P-multipliers have been used for the pile spacing to consider shadowing for lateral capacity
- Longitudinal and transverse abutment springs at the abutments were calculated using LPILE results and iterating SAP models.
- Columns are considered fixed at the base of column and pile caps are assumed adequate to transfer forces from columns to piles for retrofit condition
- Column moment capacities were calculated using XTRACT and considering material properties from Priestley
- Column effective section properties were used to determine column displacement demands
- Concrete unit weight = 150 pcf
- Concrete barriers and future utility load was included as an additional mass of 250 plf to the superstructure. Future wearing surface of 25 psf was also added.
- Reinforcement yield strength (existing) = 40 ksi
- Reinforcement yield strength (new) = 60 ksi

- Ultimate concrete compressive strength (existing) = 5.0 ksi
- Ultimate concrete compressive strength (new) = 5.0 ksi
- Ultimate unconfined concrete strain limit = 0.005
- Ultimate confined concrete strain limit = 0.02

RETROFIT STRATEGY

The bridge will follow a conventional retrofit strategy and will include the following:

- placing steel casings on columns 2, 3, and 4
- footing retrofits at piers 2, 3, and 4 consisting of adding 24" CISS piles and adding top mats of reinforcement to the enlarged pile cap
- retrofit of the bridge abutments: construction of two 72" cast-in-drilled-hole (CIDH) piles at each bridge abutment with pile cap and 2-12" steel pipe connectors from the pile cap to the superstructure

Seismic analysis shows that plastic hinges form at the bottom of the piers in the following order: 3, 2, and then 4. The displacement capacity of the structure is adequate without retrofitting pier 4 footing for earthquake demand only, however when considering lateral spread of 60 inches and post-liquefaction settlement of 7" this becomes a necessity.

In order to complete the needed seismic retrofit work, it is likely that the bridge will need to be limited to one lane traffic control through the project site for some construction activities. The drilling of CIDH piles behind the abutments may require limited closures of the bridge for construction. There are limited east-west connections between State Route 1 and 128 and closing Eureka Hill Road and detouring traffic would equate to a 125 mile detour around the project site.

RETROFITTED STRUCTURAL FINDINGS

Summary of retrofitted capacities:

- Column displacement demand/capacity ratios are between 0.43 to 0.51 in the longitudinal direction and 0.64 to 0.59 in the transverse direction.
- Column shear demand/capacity ratios are between 0.19 to 0.24 in the longitudinal direction and 0.26 to 0.32 in the transverse direction.
- Plastic hinges will form at the top and bottom of all columns in the longitudinal direction and the bottom of all columns in the transverse direction.
- Bent foundations are adequate to resist seismic forces due to the demand ARS, liquefied soil, fault rupture movement, and post-liquefaction settlement. More analysis during final design is required to solve the lateral spread movements at the supports.
- Traditional abutment seat extenders are not used in this seismic retrofit because the existing piles cannot support the superstructure under the demand EQ loading with

liquefied soils and lateral displacement demand. Instead the abutment will be tied to the 2 – 72” diameter abutment CIDH piles and cap for support. The superstructure will be tied to the cap of the new piles 2 – 12” diameter steel pipes.

Lateral spread design will be based on the “Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading”, October 2013.



BRIDGE GENERAL PLAN ESTIMATE



OR PLANNING ESTIMATE



STRUCTURE GARCIA RIVER BRIDGE		BR. NO. 10C-0034			
TYPE CIP P/S Concrete Box Girder		DIST. 1	CO. MEN	RTE. LOCAL	P.M.
LENGTH 321.8	x WIDTH 22.7	= AREA 7305		SQ FT	
QUANTITIES BY B. Osorio		DATE 7/9/2014		QUANTITIES CHCKD. BY M. Bianucci	
PRICED BY B. Osorio		DATE 7/9/2014		DATE 7/10/2014	

CONTRACT ITEMS			UNIT	QUANTITY	PRICE	AMOUNT
1	192003	STRUCTURE EXCAVATION (BRIDGE)	CY	688	\$200	\$137,600
2	193003	STRUCTURE BACKFILL (BRIDGE)	CY	476	\$120	\$57,120
3	490611	72" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	420	\$1,200	\$504,000
4	495115	FURNISH 24" CAST-IN-STEEL SHELL CONCRETE PILING	LF	2,000	\$170	\$340,000
5	495116	DRIVE 24" CAST-IN-STEEL SHELL CONCRETE PILE	EA	30	\$7,000	\$210,000
6	510051	STRUCTURAL CONCRETE, BRIDGE FOOTING	CY	214	\$600	\$128,400
7	510053	STRUCTURAL CONCRETE, BRIDGE	CY	6	\$3,000	\$18,000
8	511106	DRILL AND BOND DOWEL	LF	374	\$40	\$14,960
9	520102	BAR REINFORCING STEEL (BRIDGE)	LB	171,003	\$1.50	\$256,505
10	550110	COLUMN CASING	LB	67,337	\$8	\$538,696
11	750501	MISCELLANEOUS METAL (BRIDGE)	LB	4,016	\$24	\$96,384
12						
13						
14						
16						
16						
17						
18						
19						
20						
21						
22						
			SUBTOTAL			\$2,301,665
			MOBILIZATION (10 %)			255,741
			SUBTOTAL STRUCTURE ITEMS			\$ 2,557,405
			CONTINGENCIES (25 %)			639,351
			BRIDGE TOTAL (\$ 438 /SQ FT)			\$ 3,196,756
			GRAND TOTAL			\$ 3,196,756
			FOR BUDGET PURPOSES - USE			\$ 3,200,000

COMMENTS

APPENDIX 1

SEISMIC STRATEGY MEETING MINUTES 2/12/2014

File: 58-0242

By: MTCO

**SEISMIC RETROFIT STRATEGY MEETING MINUTES
EUREKA HILL ROAD BRIDGE REPLACEMENT
BRIDGE NO. 10C-0034
FED. NO. BRLO-5910(068)**

MEETING DATE: Wednesday, February 12, 2014 at 2:00pm

LOCATION: Caltrans Farmers Market #1, room 515

ATTENDEES: Gary Goff, Caltrans
Jaspal Gill, Caltrans
Fadel Alameddine, Caltrans
Dave Morrell, Blackburn
Po Chen, MTCO
Zach Siviglia, MTCO
Jason Hickey, MTCO
Julie Passalacqua, MTCO
Angela Priley, MTCO

MINUTES BY: Jason Hickey, MTCO & Angela Priley, MTCO

I. Introductions	
II. Project Description	
Zach Siviglia described the history of the project, including the Seismic Retrofit Strategy Report, which was completed in 1998. He also described the current decision for retrofit of the structure.	
III. Geotechnical Issues	
Dave Morrell described the subsurface conditions at the site. Up to 4.3 feet of fault rupture is possible at the site. Unlimited lateral spreading is possible at Pier 4 and Abutment 5.	
IV. Seismic Deficiencies	
Angela Priley described the structural seismic deficiencies. These included: • Pre-1971 construction • Minimal joint shear reinforcement • Lack of column confinement • Short extension of column reinforcement into bent cap • Lap splice slippage at bottom of column • No top mat reinforcing in footing • Insufficient pile connection to footing (inadequate tension capacity)	
V. Bridge Retrofit Approach	
Angela Priley described her approach to modeling the structure. She ran 3 models: • Model 1: Existing bridge under ARS only • Model 2: Retrofitted existing bridge under ARS only (assumed steel jacket, footing retrofit, joint shear fully transferable) • Model 3: Fault rupture on retrofitted structure ○ The retrofitted structure was unable to meet no-collapse criteria for this model ○ Alternative retrofit approaches were considered, including seismic isolation of superstructure or catcher bents ▪ Catcher bent simplest but orientation along fault is an issue ▪ Single column catcher bent not redundant and may have construction issues under bridge ▪ Two column catcher bent needs to account for sliding Fadel suggested to do a conventional retrofit that includes: • Foundation retrofit for lateral spread • Lowest level of retrofit for fault rupture • Not required to meet current SDC ductility criteria • Suggest rectangular casing with stiffener • Top mat reinforcing in footing with piling to resist vertical loads • Lateral resisting system at abutment to reduce D/C ratios and retain vertical support with CIDH piles Gary Goff stated that the strategy needs to be approved by Caltrans. He also mentioned that Caltrans will review plans to be sure strategy matches with 95% plans and will issue approval letter.	<i>MTCO will run analysis based on conventional retrofit strategy and prepare updated Strategy Report to Caltrans for approval before finalizing the PS&E</i>

APPENDIX 2

MEETING WITH CALTRANS MINUTES 12/21/2014

Eureka Hill Rd at Garcia River
Seismic Retrofit Strategy
Br. No. 10C0034
BRLRT-5910(041)

Notes from 12-21-14 meeting

SLA-Gary Goff
EQE-Fadel Alameddine, Mark Yashinsky

[DLAE-Mark Mueller (Eureka)]
[Mendocino Co-Alicia Meier]

2-12-14 Initial retrofit strategy meeting-MTCO/CT
7-10-14 Revised retrofit strategy report sent to CT
Oct/Nov 2014 CT geotech comments received and Blackburn responses
12-21-14 Revised retrofit strategy meeting-MTCO/CT
Feb 2015 Blackburn memo addressing CT Fault Rupture

- Built 1969
- R/C Box, L=322', 4-spans
- Seat abutments – 3.5' seat w/ bearing pads
- 2.5' x 8' columns, fix-fix
- CISS pipe piles w/ concrete
- San Andreas Fault Zone
- Mmax=8.0, PGA=0.83g
- Horizontal ground movement-2'
- Liquefaction settlement: 2.5", 1", 7"(AB5)
- **Fault Rupture = 0.82'**
- Lateral Spreading: 2' (P1-3) [piles will not fail with 2' displacement]; "unlimited" (P4,AB5)
- Fault Rupture and Lateral Spreading were NOT considered in Revised retrofit strategy
- Liquefied soil was considered
- P-Δ was considered
- Conc. Strain = 0.005 unconfined
- Conc. Strain = 0.02 confined
- STRATEGY: column casings, footing retrofit, abutment restraint w/ large diameter CIDH shafts
- NO road closures-Stage construction

Comments from Gary Goff, Fadel Alameddine, Mark Yashinsky, CT geotech comments:

Fault rupture analysis must agree with Caltrans in-house screening; must be consistent, no precedent.
Discuss with Ana Sojourner (CT geologist)

Site Specific strategy.

State Design Criteria for EACH hazard (fault rupture (Anna); lateral spreading (Tom Schantz guide)

Pinning effect-iteration between structure / geotech for piles & forces w/ sliding

Retrofit to satisfy EACH hazard

Confirm why 24" CISS pile footing at ALL footings? Must justify

Show collapse scenario-in order to accept retrofit

1-fault rupture

2-lateral spreading; include piles in slope

3-ground shaking

Steel piles are good system, not concrete

Abutment seat extender

Curvature capacity of piles-demand from hazard loading

Memo to Designer 20-10 (Norm Abrahms)

MINIMUM: ~~seat extender~~; pile cap; steel column casing

APPENDIX 3

GENERAL PLANS

REGISTERED STRUCTURAL ENGINEER DATE

PLANS APPROVAL DATE

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MENDOCINO COUNTY

DEPARTMENT OF TRANSPORTATION

340 LAKE MENDOCINO DRIVE

UKIAH, CA 95482

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7300 FOLSOM BLVD. SUITE 203

SACRAMENTO, CA 95826

REGISTERED PROFESSIONAL ENGINEER

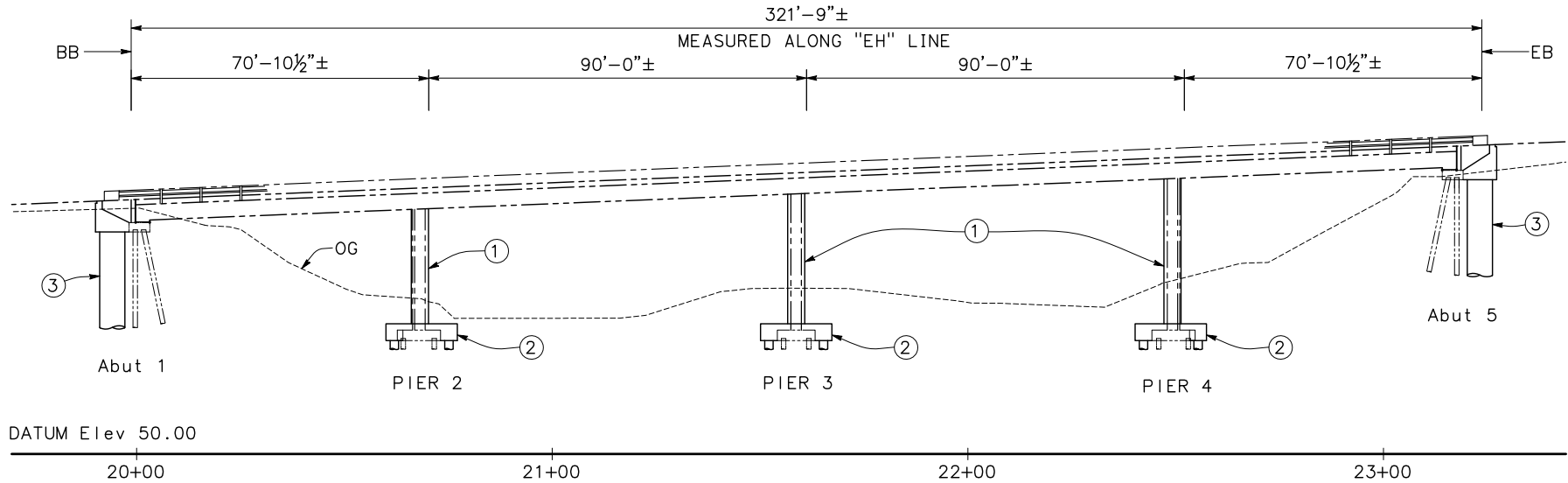
M. BIANUCCI

No. C52085

Exp. 12/31/14

CIVIL

STATE OF CALIFORNIA

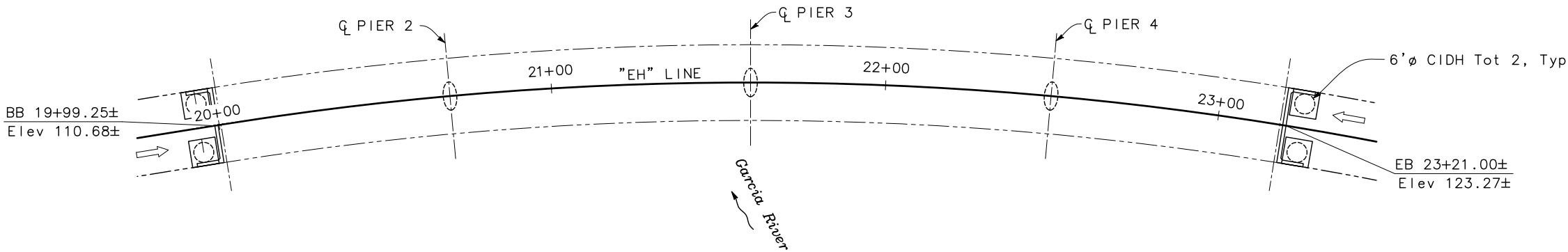


"EH" LINE

R = 1000.00'
Δ = 24°26'44"
L = 426.66'
T = 216.62'

ELEVATION

1" = 20'



PLAN

1" = 20'

INDEX TO PLANS

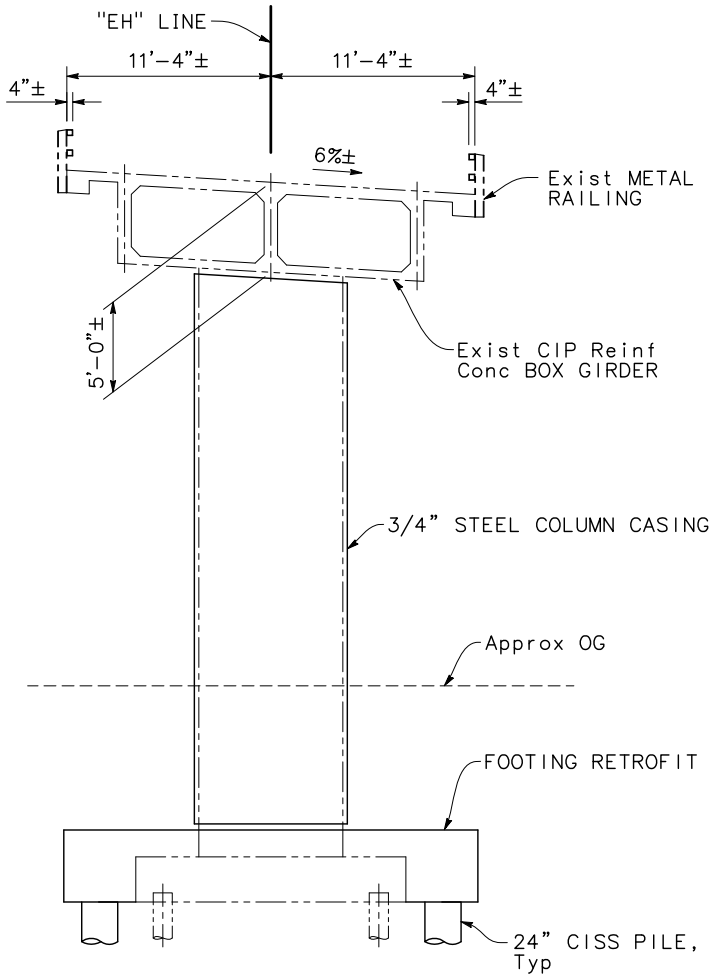
No.	TITLE
1	GENERAL PLAN
2	ABUTMENT DETAILS
3	FOOTING RETROFIT

NOTES:

- ① Column casing
- ② Footing retrofit
- ③ 6'Ø CIDH Piles with cap

LEGEND:

- Indicates Existing Structure
- Indicates New Construction
- ⇒ Direction of Traffic



TYPICAL SECTION

3/16" = 1'-0"

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD
DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

PROJ. No. C0205
FED. No. BRLO-5910(068)

DESIGN	BY P. CHEN	CHECKED X.
DETAILS	BY G. BOYKO	CHECKED X.
QUANTITIES	BY	CHECKED X.

PREPARED FOR
MENDOCINO COUNTY
DEPARTMENT OF TRANSPORTATION



MICHAEL BIANUCCI
PROJECT ENGINEER

BRIDGE NO.	10C-0034
POST MILES	

GARCIA RIVER BRIDGE (RETROFIT)
GENERAL PLAN

DISREGARD PRINTS BEARING EARLIER REVISION DATES	REVISION DATES (PRELIMINARY STAGE ONLY)	SHEET	OF
	5/18/14 7/10/14	1	3

ORIGINAL SCALE IN INCHES
FOR REDUCED PLANS



FILE => \$REQUEST

DATE PLOTTED => \$DATE
USERNAME => \$USER
TIME PLOTTED =>

REGISTERED STRUCTURAL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER

M. BIANUCCI

C52085

Exp. 12/31/14

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STATE OF CALIFORNIA

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MENDOCINO COUNTY

DEPARTMENT OF TRANSPORTATION

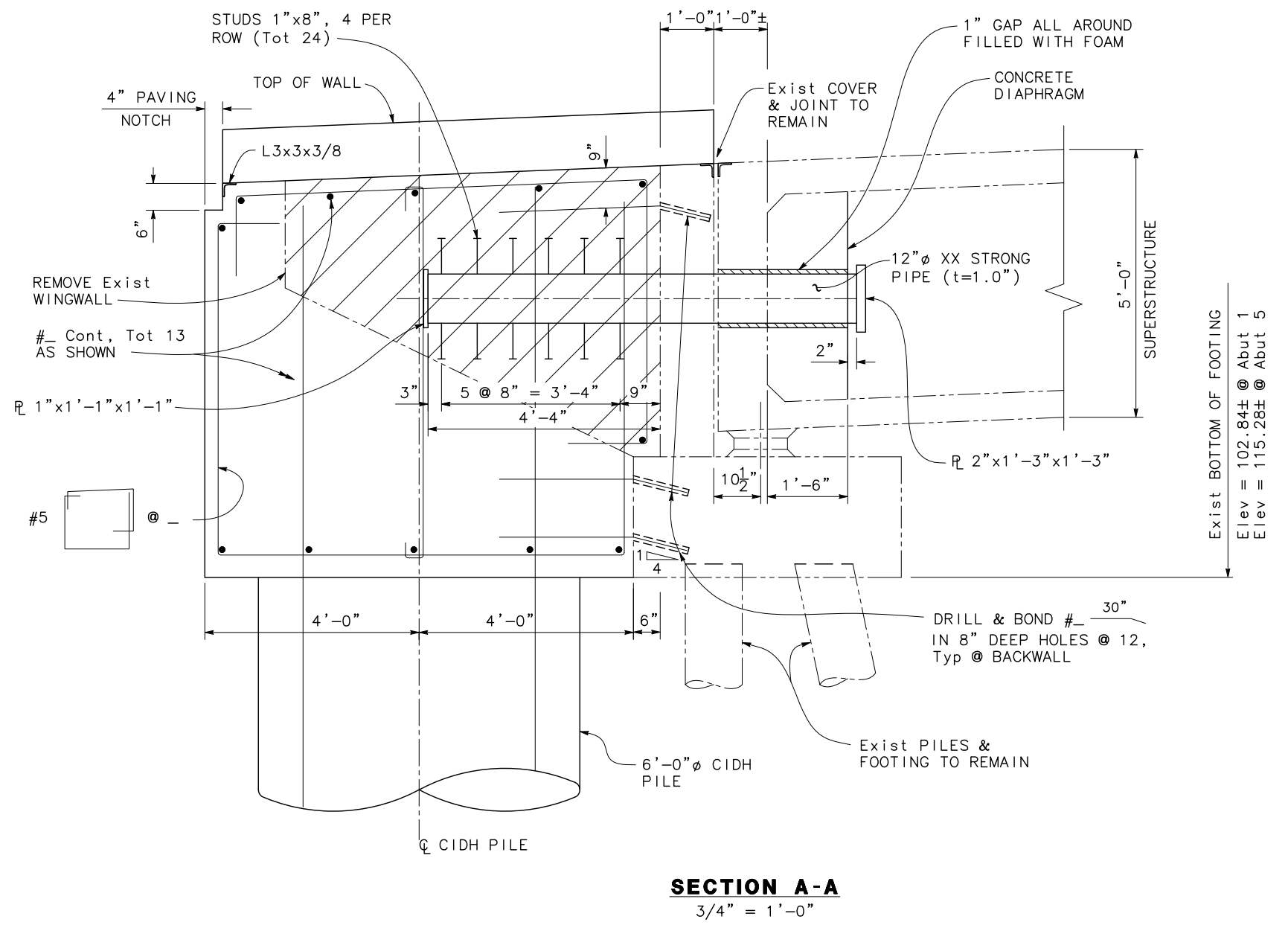
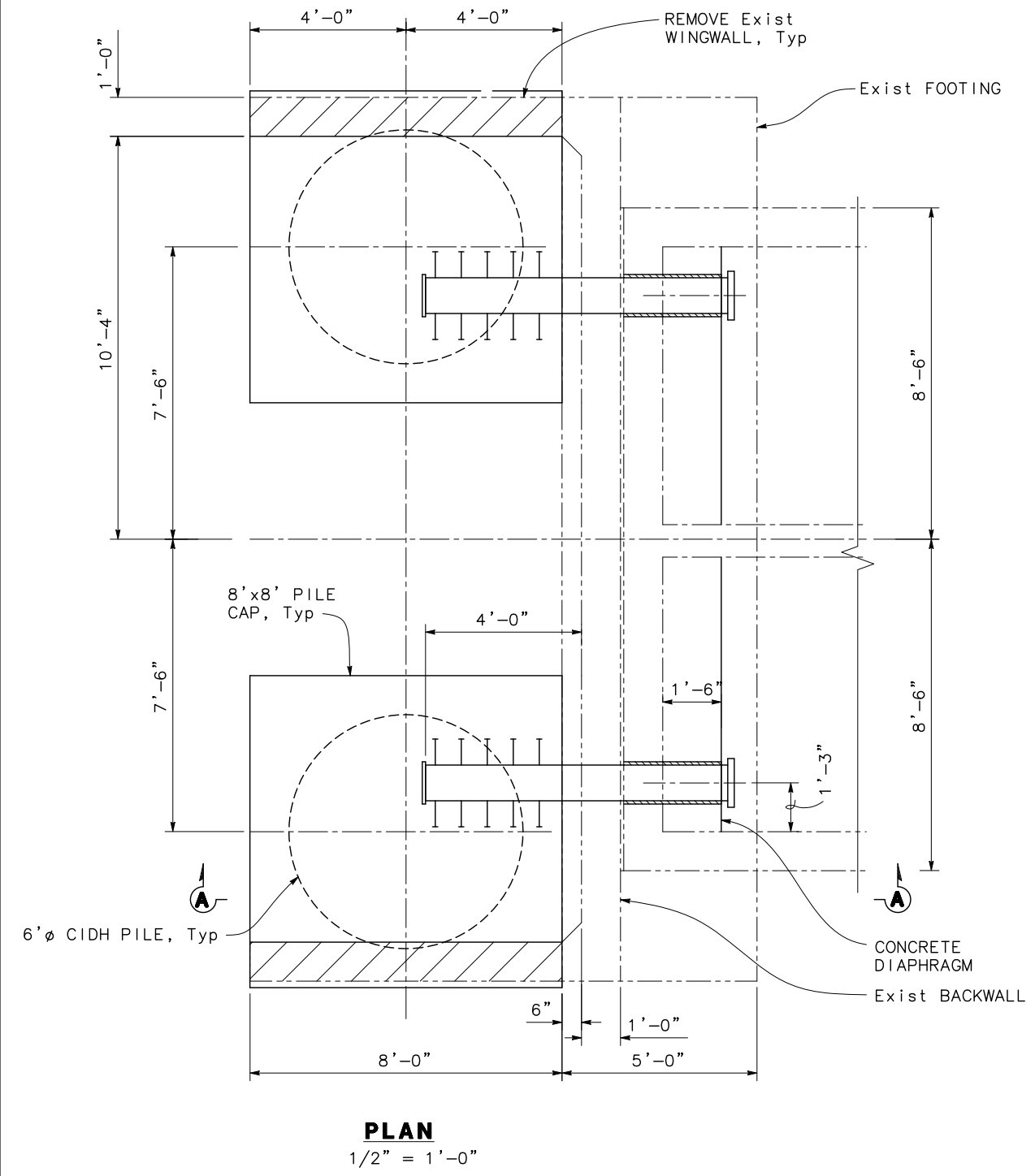
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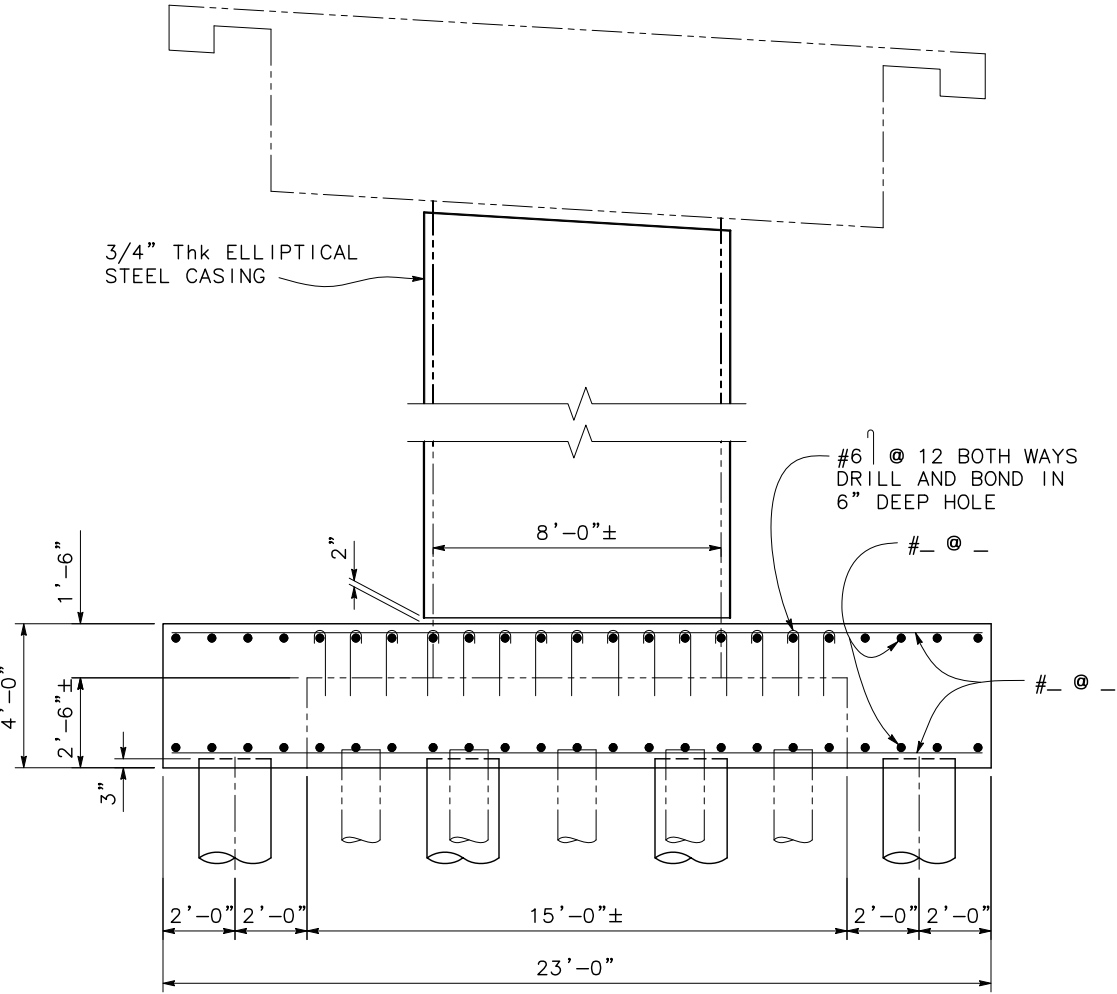
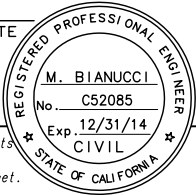
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PLANS APPROVAL DATE

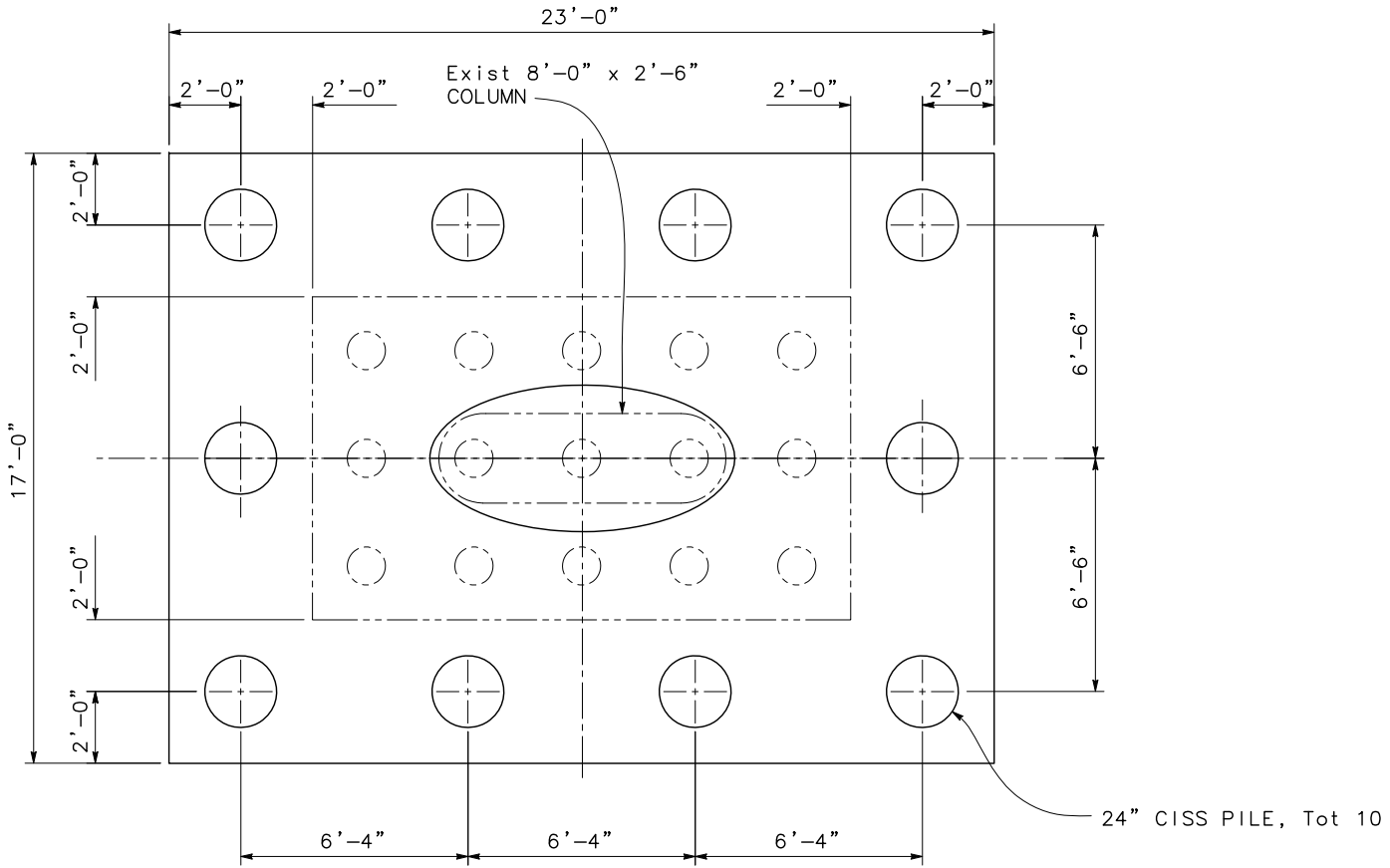
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MARK THOMAS & COMPANY, INC.
7300 FOLSOM BLVD. SUITE 203
SACRAMENTO, CA 95826



ELEVATION
3/8" = 1'-0"



PLAN
3/8" = 1'-0"

NOTE:
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DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

PROJ. No. C0205
FED. No. BRLO-5910(068)

DESIGN	BY H. MISTRY	CHECKED X.
DETAILS	BY G. BOYKO	CHECKED X.
QUANTITIES	BY	CHECKED X.

PREPARED FOR
MENDOCINO COUNTY
DEPARTMENT OF TRANSPORTATION



MICHAEL BIANUCCI
PROJECT ENGINEER

BRIDGE NO.	10C-0034
POST MILES	

GARCIA RIVER BRIDGE (RETROFIT)
FOOTING RETROFIT

ORIGINAL SCALE IN INCHES
FOR REDUCED PLANS



FILE => \$REQUEST

DISREGARD PRINTS BEARING
EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

5-15-14 7-10-14

SHEET 3 OF 3

TIME PLOTTED => DATE PLOTTED => \$DATE USERNAME => \$USER

APPENDIX 4

DUCTILITY DEMANDS & D/C RATIOS



BY: BO DATE: 04/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

DEMAND/CAPACITY RATIOS - CASE 1

- 1) Calculations are based on 2-6' shafts at each abutment, steel jackets at all columns, and footing retrofits at Pier 2 & 3.
- 2) Lpile was used to obtain longitudinal and transverse abutment springs with liquefaction effects considered.
- 3) Pier foundation flexibility not considered, but will be considered at final design.
- 4) Displacement from ground shaking and fault offset combined.

1. DISPLACEMENT D/C RATIO (SDC 4.1.1)

LONGITUDINAL

	$\Delta_{d,GS}$ (in)	$\Delta_{d,FR}$ (in)	Δ_d (in)	Δ_c (in) Pushover	Δ_r (in)	Δ_c (in)	Δ_d/Δ_c Eq. 4.1	$\Delta_d/\Delta_c < 1.0$
PIER 2	5.02	4.14	9.16	11.16	0.00	11.16	0.82	OK
PIER 3	5.09	4.93	10.02	11.29	0.00	11.29	0.89	OK
PIER 4	5.31	5.66	10.97	11.27	0.00	11.27	0.97	OK

TRANSVERSE

	$\Delta_{d,GS}$ (in)	$\Delta_{d,FR}$ (in)	Δ_d (in)	Δ_c (in) Pushover	Δ_r (in)	Δ_c (in)	Δ_d/Δ_c Eq. 4.1	$\Delta_d/\Delta_c < 1.0$
PIER 2	4.49	1.57	6.06	9.24	0.00	9.24	0.66	OK
PIER 3	8.57	1.67	10.24	14.21	0.00	14.21	0.72	OK
PIER 4	8.41	2.25	10.66	10.68	0.00	10.68	1.00	OK

2. CHECK P-DELTA EFFECTS (SDC 4.2)

LONGITUDINAL

	P_{dl} (kips)	Δ_r (in)	M_{pCOL} (k-ft)	$P_{dl} \times \Delta_r$ (k-ft)	$0.2M_{pCOL}$ (k-ft)	Eq. 4.3
PIER 2	580	9.16	2834	443	567	OK
PIER 3	600	10.02	2858	501	572	OK
PIER 4	580	10.97	2144	530	429	NG

TRANSVERSE

	$P_{dl,max}$ (kips)	Δ_r (in)	M_{pCOL} (k-ft)	$P_{dl} \times \Delta_r$ (k-ft)	$0.2M_{pCOL}$ (k-ft)	Eq. 4.3
PIER 2	580	6.06	8237	293	1647	OK
PIER 3	600	10.24	8302	512	1660	OK
PIER 4	580	10.66	5524	515	1105	OK

3. SHEAR D/C RATIO

LONGITUDINAL

	$M_{p,top}$ (k-ft)	$M_{p,bot}$ (k-ft)	L (ft)	V_o (kips) SDC 4.3.1	ϕV_n (kip) (min)	$V_o/\phi V_n$ Eq. 3.14	$V_o/\phi V_n < 1.0$
PIER 2	2730	2834	27.45	243	1034	0.24	OK
PIER 3	2750	2858	30.97	217	1034	0.21	OK
PIER 4	2730	2858	35.99	186	1034	0.18	OK

TRANSVERSE

	$M_{p,top}$ (k-ft)	$M_{p,bot}$ (k-ft)	L (ft)	V_o (kips) SDC 4.3.1	ϕV_n (kip) (min)	$V_o/\phi V_n < 1.0$ Eq. 3.14	$V_o/\phi V_n < 1.0$
PIER 2	0	8237	30.25	327	1034	0.32	OK
PIER 3	0	8302	33.77	295	1034	0.29	OK
PIER 4	0	8302	38.79	257	1034	0.25	OK



BY: BO DATE: 04/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

DEMAND/CAPACITY RATIOS - CASE 2

- 1) Calculations are based on 2-6' shafts at each abutment, steel jackets at all columns, and footing retrofits at Pier 2, 3, & 4.
- 2) Lpile was used to obtain longitudinal and transverse abutment springs with liquefaction effects considered.
- 3) Pier foundation flexibility not considered, but will be considered at final design.
- 4) Displacement from ground shaking and fault offset combined.

1. DISPLACEMENT D/C RATIO (SDC 4.1.1)

LONGITUDINAL

	$\Delta_{d,GS}$ (in)	$\Delta_{d,FR}$ (in)	Δ_d (in)	Δ_c (in) Pushover	Δ_r (in)	Δ_c (in)	Δ_d/Δ_c Eq. 4.1	$\Delta_d/\Delta_c < 1.0$
PIER 2	4.76	4.12	8.87	20.43	0.00	20.43	0.43	OK
PIER 3	4.86	4.86	9.72	20.66	0.00	20.66	0.47	OK
PIER 4	5.00	5.53	10.54	20.61	0.00	20.61	0.51	OK

TRANSVERSE

	$\Delta_{d,GS}$ (in)	$\Delta_{d,FR}$ (in)	Δ_d (in)	Δ_c (in) Pushover	Δ_r (in)	Δ_c (in)	Δ_d/Δ_c Eq. 4.1	$\Delta_d/\Delta_c < 1.0$
PIER 2	4.49	1.59	6.07	9.50	0.00	9.50	0.64	OK
PIER 3	8.08	1.76	9.84	14.67	0.00	14.67	0.67	OK
PIER 4	6.76	1.94	8.69	10.20	0.00	10.20	0.85	OK

2. CHECK P-DELTA EFFECTS (SDC 4.2)

LONGITUDINAL

	P_{dl} (kips)	Δ_r (in)	M_{pCOL} (k-ft)	$P_{dl} \times \Delta_r$ (k-ft)	$0.2M_{pCOL}$ (k-ft)	Eq. 4.3
PIER 2	580	8.87	2834	429	567	OK
PIER 3	600	9.72	2858	486	572	OK
PIER 4	580	10.54	2858	509	572	OK

TRANSVERSE

	$P_{dl,max}$ (kips)	Δ_r (in)	M_{pCOL} (k-ft)	$P_{dl} \times \Delta_r$ (k-ft)	$0.2M_{pCOL}$ (k-ft)	Eq. 4.3
PIER 2	580	6.07	8237	294	1647	OK
PIER 3	600	9.84	8302	492	1660	OK
PIER 4	580	8.69	8302	420	1660	OK

3. SHEAR D/C RATIO

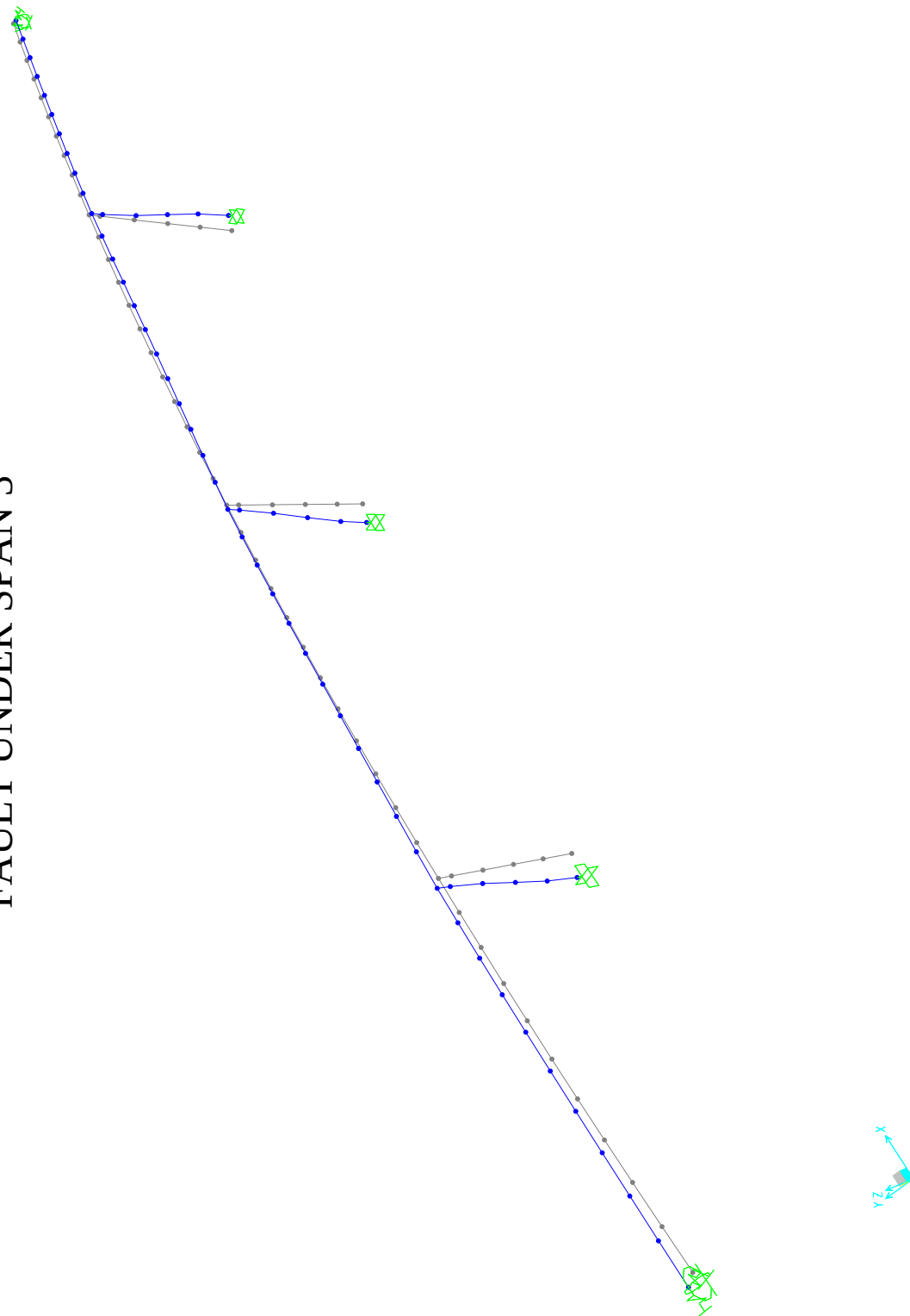
LONGITUDINAL

	$M_{p,top}$ (k-ft)	$M_{p,bot}$ (k-ft)	L (ft)	V_o (kips) SDC 4.3.1	ϕV_n (kip) (min)	$V_o/\phi V_n$ Eq. 3.14	$V_o/\phi V_n < 1.0$
PIER 2	2730	2834	27.45	243	1034	0.24	OK
PIER 3	2750	2858	30.97	217	1034	0.21	OK
PIER 4	2730	2858	34.49	194	1034	0.19	OK

TRANSVERSE

	$M_{p,top}$ (k-ft)	$M_{p,bot}$ (k-ft)	L (ft)	V_o (kips) SDC 4.3.1	ϕV_n (kip) (min)	$V_o/\phi V_n < 1.0$ Eq. 3.14	$V_o/\phi V_n < 1.0$
PIER 2	0	8237	30.25	327	1034	0.32	OK
PIER 3	0	8302	33.77	295	1034	0.29	OK
PIER 4	0	8302	37.29	267	1034	0.26	OK

EXAMPLE FAULT RUPTURE MODEL FAULT UNDER SPAN 3



**MARK THOMAS & COMPANY**

Providing Engineering, Surveying & Planning Services

BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF: CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.: **FAULT RUPTURE - CASE 1****MAXIMUM DISPLACEMENT DEMANDS DUE TO FAULT RUPTURE**

	Joint	U1 Long	U2 Trans
	Text	in	in
Abut	A1	1.20	2.82
	A5	1.18	1.99
PIERS	P2	4.14	1.57
	P3	4.93	1.67
	P4	5.66	2.25

Movement = 10.0 inches

Direction = 320.00 ° (true north azimuth)

Movement applied at half magnitude in opposite directions.

		Fault Location							
		Movement (Span 1)		Movement (Span 2)		Movement (Span 3)		Movement (Span 4)	
	Fault Skew	Long	Trans	Long	Trans	Long	Trans	Long	Trans
Abut 1	55.3944 °	-4.12	2.84	-4.12	2.84	-4.12	2.84	-4.12	2.84
Pier 2	59.3497 °	4.30	-2.55	-4.30	2.55	-4.30	2.55	-4.30	2.55
Pier 3	64.5064 °	4.51	-2.15	4.51	-2.15	-4.51	2.15	-4.51	2.15
Pier 4	69.6631 °	4.69	-1.74	4.69	-1.74	4.69	-1.74	-4.69	1.74
Abut 5	73.6139 °	4.80	-1.41	4.80	-1.41	4.80	-1.41	4.80	-1.41



BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

FAULT RUPTURE - CASE 1

DISPLACEMENT DEMANDS DUE TO FAULT RUPTURE (FAULT CROSSING SPAN 1)

		TOP		BOTTOM		RELATIVE	
	Joint	U1 Long	U2 Trans	U1 Long	U2 Trans	U1 Long	U2 Trans
	Text	in	in	in	in	in	in
Abut	A1	0.01	2.28			0.01	2.28
	A5	0.85	-1.92			0.85	-1.92
PIERS	P2	0.16	-1.28	4.30	-2.55	-4.14	1.27
	P3	0.39	-2.89	4.51	-2.15	-4.12	-0.74
	P4	0.62	-2.78	4.69	-1.74	-4.07	-1.04

DISPLACEMENT DEMANDS DUE TO FAULT RUPTURE (FAULT CROSSING SPAN 2)

		TOP		BOTTOM		RELATIVE	
	Joint	U1 Long	U2 Trans	U1 Long	U2 Trans	U1 Long	U2 Trans
	Text	in	in	in	in	in	in
Abut	A1	-0.56	2.78			-0.56	2.78
	A5	-0.05	-1.99			-0.05	-1.99
PIERS	P2	-0.75	0.98	-4.30	2.55	3.55	-1.57
	P3	-0.42	-1.70	4.51	-2.15	-4.93	0.45
	P4	-0.25	-2.37	4.69	-1.74	-4.94	-0.63

DISPLACEMENT DEMANDS DUE TO FAULT RUPTURE (FAULT CROSSING SPAN 3)

		TOP		BOTTOM		RELATIVE	
	Joint	U1 Long	U2 Trans	U1 Long	U2 Trans	U1 Long	U2 Trans
	Text	in	in	in	in	in	in
Abut	A1	-0.99	2.82			-0.99	2.82
	A5	-0.85	-1.82			-0.85	-1.82
PIERS	P2	-1.20	1.85	-4.30	2.55	3.10	-0.70
	P3	-1.19	0.48	-4.51	2.15	3.32	-1.67
	P4	-0.97	-1.23	4.69	-1.74	-5.66	0.51

DISPLACEMENT DEMANDS DUE TO FAULT RUPTURE (FAULT CROSSING SPAN 4)

		TOP		BOTTOM		RELATIVE	
	Joint	U1 Long	U2 Trans	U1 Long	U2 Trans	U1 Long	U2 Trans
	Text	in	in	in	in	in	in
Abut	A1	-1.20	2.81			-1.20	2.81
	A5	-1.18	-1.56			-1.18	-1.56
PIERS	P2	-1.41	1.98	-4.30	2.55	2.89	-0.57
	P3	-1.41	0.88	-4.51	2.15	3.10	-1.27
	P4	-1.33	-0.51	-4.69	1.74	3.36	-2.25


 BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:

 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

FAULT RUPTURE - CASE 2

MAXIMUM DISPLACEMENT DEMANDS DUE TO FAULT RUPTURE

	Joint	U1 Long	U2 Trans
	Text	in	in
Abut	A1	1.21	2.81
	A5	1.23	1.89
PIERS	P2	4.12	1.59
	P3	4.86	1.76
	P4	5.53	1.94

Movement = 10.0 inches

Direction = 320.00 ° (true north azimuth)

Movement applied at half magnitude in opposite directions.

		Fault Location							
		Movement (Span 1)		Movement (Span 2)		Movement (Span 3)		Movement (Span 4)	
	Fault Skew	Long	Trans	Long	Trans	Long	Trans	Long	Trans
Abut 1	55.3944 °	-4.12	2.84	-4.12	2.84	-4.12	2.84	-4.12	2.84
Pier 2	59.3497 °	4.30	-2.55	-4.30	2.55	-4.30	2.55	-4.30	2.55
Pier 3	64.5064 °	4.51	-2.15	4.51	-2.15	-4.51	2.15	-4.51	2.15
Pier 4	69.6631 °	4.69	-1.74	4.69	-1.74	4.69	-1.74	-4.69	1.74
Abut 5	73.6139 °	4.80	-1.41	4.80	-1.41	4.80	-1.41	4.80	-1.41



BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

FAULT RUPTURE - CASE 2

DISPLACEMENT DEMANDS DUE TO FAULT RUPTURE (FAULT CROSSING SPAN 1)

		TOP		BOTTOM		RELATIVE	
	Joint	U1 Long	U2 Trans	U1 Long	U2 Trans	U1 Long	U2 Trans
	Text	in	in	in	in	in	in
Abut	A1	0.03	2.35			0.03	2.35
	A5	0.86	-1.81			0.86	-1.81
PIERS	P2	0.18	-1.27	4.30	-2.55	-4.12	1.28
	P3	0.41	-2.85	4.51	-2.15	-4.10	-0.70
	P4	0.66	-2.63	4.69	-1.74	-4.03	-0.89

DISPLACEMENT DEMANDS DUE TO FAULT RUPTURE (FAULT CROSSING SPAN 2)

		TOP		BOTTOM		RELATIVE	
	Joint	U1 Long	U2 Trans	U1 Long	U2 Trans	U1 Long	U2 Trans
	Text	in	in	in	in	in	in
Abut	A1	-0.51	2.78			-0.51	2.78
	A5	0.01	-1.89			0.01	-1.89
PIERS	P2	-0.69	0.96	-4.30	2.55	3.61	-1.59
	P3	-0.35	-1.70	4.51	-2.15	-4.86	0.45
	P4	-0.16	-2.28	4.69	-1.74	-4.85	-0.54

DISPLACEMENT DEMANDS DUE TO FAULT RUPTURE (FAULT CROSSING SPAN 3)

		TOP		BOTTOM		RELATIVE	
	Joint	U1 Long	U2 Trans	U1 Long	U2 Trans	U1 Long	U2 Trans
	Text	in	in	in	in	in	in
Abut	A1	-0.92	2.81			-0.92	2.81
	A5	-0.74	-1.80			-0.74	-1.80
PIERS	P2	-1.12	1.80	-4.30	2.55	3.18	-0.75
	P3	-1.10	0.39	-4.51	2.15	3.41	-1.76
	P4	-0.84	-1.32	4.69	-1.74	-5.53	0.42

DISPLACEMENT DEMANDS DUE TO FAULT RUPTURE (FAULT CROSSING SPAN 4)

		TOP		BOTTOM		RELATIVE	
	Joint	U1 Long	U2 Trans	U1 Long	U2 Trans	U1 Long	U2 Trans
	Text	in	in	in	in	in	in
Abut	A1	-1.21	2.80			-1.21	2.80
	A5	-1.23	-1.43			-1.23	-1.43
PIERS	P2	-1.42	2.01	-4.30	2.55	2.88	-0.54
	P3	-1.43	1.02	-4.51	2.15	3.08	-1.13
	P4	-1.38	-0.20	-4.69	1.74	3.31	-1.94



BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

REDUCED MOMENT CAPACITY DUE TO JOINT SHEAR MODELING PROCEDURE - CASE 1

BDA 14-4 (AUG 2008)

Description of Retrofit: 2-6' CIDH shafts at each abutment (both long springs engaged 100%)
 Steel Jackets at all 3 piers
 Joint Retrofit at Pier 2 & 3 Footings only (resulting in 4 weak joints at column connections)

$$f_{ce} = 5000$$

$$v_n = 247.5 \text{ psi}$$

$$D_{cg} = 2.80 \text{ ft}$$

$$v_n = 3.5\sqrt{f_{ce}} \text{ (psi)}$$

$$D_{cg} = \text{Distance to c.g. of superstructure}$$

Joint Shear Strength, Weak Joint (Pre 1971)

T_c = Column tensile force in the column, obtained from moment-curvature analysis

M_p = Column plastic moment capacity

Column Forces (Pier 2):

				Long			Trans		
	D_{ftg} (ft)	H (ft)	P (k)	L_{contra} (ft)	T_c (k)	$M_{p,l}$ (k-ft)	L_{contra} (ft)	T_c (k)	$M_{p,t}$ (k-ft)
Top	4.00	27.45	580	13.73	1421	2730			
Bot			685	13.73	1421	2834	30.25	1612	8237

Column Forces (Pier 3):

				Long			Trans		
	D_{ftg} (ft)	H (ft)	P (k)	L_{contra} (ft)	T_c (k)	$M_{p,l}$ (k-ft)	L_{contra} (ft)	T_c (k)	$M_{p,t}$ (k-ft)
Top	4.00	30.97	600	15.49	1421	2750			
Bot			710	15.49	1420	2858	33.77	1601	8302

Column Forces (Pier 4):

				Long			Trans		
	D_{ftg} (ft)	H (ft)	P (k)	L_{contra} (ft)	T_c (k)	$M_{p,l}$ (k-ft)	L_{contra} (ft)	T_c (k)	$M_{p,t}$ (k-ft)
Top	2.50	35.99	580	18.00	1421	2730			
Bot			710	18.00	1420	2858	38.79	1601	8302

BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF: CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.: **REDUCED MOMENT CAPACITY DUE TO JOINT SHEAR MODELING PROCEDURE - CASE 1**

BDA 14-4 (AUG 2008)

SUPERSTRUCTURE/COLUMN MOMENT RESISTING JOINT

$$v_{jv} = T_c / A_{jv} \quad v_{jv} = \text{Joint vertical shear stress demand}$$

$$M_{pr} = (v_n / v_{jv}) \cdot 1.2M_p \quad M_{pr} = \text{Reduced moment capacity of hinge model}$$

$$(E_c \cdot I_{eff})_{mod} = 0.85 \cdot (E_c \cdot I_{eff}) \cdot (v_n / v_{jv})$$

$$\theta_{pj} = \theta_{yc} + 0.007 \quad \theta_{pj} = \text{Plastic rotation capacity of the joint}$$

$$\theta_{yc} = \Delta_y / L_{contra} \quad \theta_{yc} = \text{Column yield rotation capacity based on a rigid joint}$$

$$B_{cap} = 3.00 \text{ ft} \quad B_{cap} = \text{Bent cap width}$$

$$l_{ac} = 3.40 \text{ ft} \quad l_{ac} = \text{Length of column reinforcement embedded into the bent cap}$$

$$A_{jv} = 10.20 \text{ sq ft} \quad A_{jv} = l_{av} \cdot B_{cap}$$

	P (k)	Long			Trans		
		v_{jv} (psi)	v_n / v_{jv}	$M_{pr,t}$ (k-ft)	v_{jv} (psi)	v_n / v_{jv}	$M_{pr,t}$ (k-ft)
Pier 2	580	967.5	0.26	838			
Pier 3	600	967.5	0.26	844			
Pier 4	710	967.5	0.26	838			

	Long				Trans			
	Δ_y (in)	θ_{yc}	θ_{pj}	θ_{Tot}	Δ_y (in)	θ_{yc}	θ_{pj}	θ_{Tot}
Pier 2	1.14	0.007	0.007	0.014	0.00	0.000	0.000	0.000
Pier 3	1.45	0.008	0.007	0.015	0.00	0.000	0.000	0.000
Pier 4	1.79	0.008	0.007	0.015	0.00	0.000	0.000	0.000

FOOTING/COLUMN MOMENT RESISTING JOINT

$$v_{jv} = T_{jv} / (B_{eff}^{ftg} \cdot D_{ftg}) \quad v_{jv} = \text{Joint vertical shear stress demand}$$

$$M_{pr} = (v_n / v_{jv}) \cdot 1.2M_p$$

$$\theta_{pj} = \theta_{pj} + 0.007$$

$$\theta_{yc} = \Delta_y / L_{contra}$$

$$D_{ftg} = 2.50 \text{ ft} \quad D_{ftg} = \text{Footing Depth}$$

$$A_{col} = 18.66 \text{ sq ft} \quad A_{col} = \text{Column Area}$$

$$B_c = 2.50 \text{ ft} \quad B_c = \text{Column Depth}$$

$$D_c = 7.46 \text{ ft} \quad B_c = \text{Column Width, using equivalent length}$$

$$B_{eff}^{ftg} = 9.96 \text{ ft} \quad B_{eff}^{ftg} = B_c + D_c = \text{Effective Footing Width for Oblong Columns similar to Rectangular Columns}$$

	P (k)	D_{ftg} (ft)	Long			Trans		
			v_{jv} (psi)	v_n / v_{jv}	$M_{pr,t}$ (k-ft)	v_{jv} (psi)	v_n / v_{jv}	$M_{pr,t}$ (k-ft)
Pier 2	685	4.00	247.6	1.00	2834	280.9	1.00	8237
Pier 3	710	4.00	247.4	1.00	2858	279.0	1.00	8302
Pier 4	710	2.50	395.9	0.63	2144	446.3	0.55	5524

	Long				Trans			
	Δ_y (in)	θ_{yc}	θ_{pj}	θ_{Tot}	Δ_y (in)	θ_{yc}	θ_{pj}	θ_{Tot}
Pier 2	1.14	0.003	0.045	0.048	1.81	0.001	0.029	0.030
Pier 3	1.45	0.003	0.045	0.048	2.25	0.001	0.029	0.030
Pier 4	1.79	0.008	0.007	0.015	2.97	0.006	0.007	0.013



BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

REDUCED MOMENT CAPACITY DUE TO JOINT SHEAR MODELING PROCEDURE - CASE 1

BDA 14-4 (AUG 2008)

CAPACITY MODEL

	Long (Top)				Long (Bot)		
	EI (K-ft ²)	θ_{yc}	θ_{Tot}	$M_{pr,l}$ (k-ft)	θ_{yc}	θ_{Tot}	$M_{pr,l}$ (k-ft)
Pier 2	5.944E+06	0.007	0.014	838	0.003	0.048	2834
Pier 3	5.944E+06	0.008	0.015	844	0.003	0.048	2858
Pier 4	5.944E+06	0.008	0.015	838	0.008	0.015	2144

	Trans (Top)				Trans (Bot)		
	EI (K-ft ²)	θ_{yc}	θ_{Tot}	$M_{pr,l}$ (k-ft)	θ_{yc}	θ_{Tot}	$M_{pr,l}$ (k-ft)
Pier 2	5.600E+07	0.000	0.000	0	0.001	0.030	8237
Pier 3	5.600E+07	0.000	0.000	0	0.001	0.030	8302
Pier 4	5.600E+07	0.000	0.000	0	0.006	0.013	5524

DEMAND MODEL

Column EI:

$$(E_c \cdot I_{eff})_{jacket} = 1.4 \cdot (E_c \cdot I_{eff})$$

$$(E_c \cdot I_{eff})_{mod} : \text{Weak Joint} = 0.85 \cdot (E_c \cdot I_{eff})_{jacket} \cdot (v_n / v_{jv})$$

$$\text{Retrofit} = (E_c \cdot I_{eff})_{jacket}$$

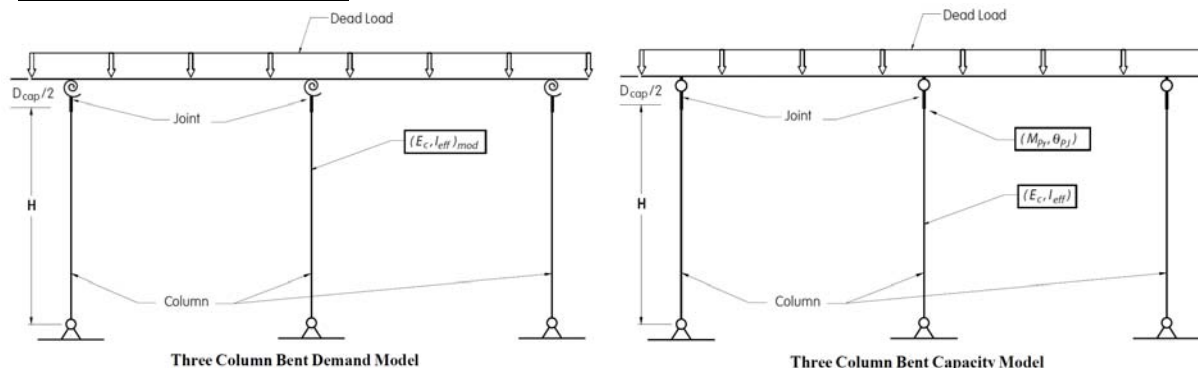
Columns are defined by gross properties, EI, and modified by the following factors.

	Longitudinal				Transverse				
	Top		Bottom		Top		Bottom		
	$0.85\ v_n / v_{jv}$	$0.85\ v_n / v_{jv}$	$(E_c \cdot I_{eff})$	$(E_c \cdot I_{eff})_{jacket}$	$0.85\ v_n / v_{jv}$	$0.85\ v_n / v_{jv}$	$(E_c \cdot I_{eff})$	$(E_c \cdot I_{eff})_{jacket}$	
Pier 2	0.22	1.00	0.310	0.433		1.00	0.298	0.418	Weak Joint @ Top
Pier 3	0.22	1.00	0.312	0.436		1.00	0.302	0.423	Weak Joint @ Top
Pier 4	0.22	0.53	0.311	0.435		0.47	0.302	0.423	Weak Joint @ Top & Bot

	Long	Trans	Long	Trans	Long	Trans
	EI (K-ft ²)	EI (K-ft ²)	EI _{eff} (K-ft ²)	EI _{eff} (K-ft ²)	$(E_c \cdot I_{eff})_{mod}$	$(E_c \cdot I_{eff})_{mod}$
Pier 2	5.944E+06	5.600E+07	1.841E+06	1.670E+07	0.26	0.42
Pier 3	5.944E+06	5.600E+07	1.852E+06	1.690E+07	0.27	0.42
Pier 4	5.944E+06	5.600E+07	1.847E+06	1.690E+07	0.16	0.20

Abutment Springs:

	Long Δ	Trans Δ	
Abut 1	5.10	0.65	4404 k/ft corresponding to 5.5" disp and 6764 k/ft corresponding to a 1" disp.
Abut 5	5.45	3.17	2960 k/ft corresponding to 5.5" disp and 3111 k/ft corresponding to a 3" disp.



BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF: CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.: **REDUCED MOMENT CAPACITY DUE TO JOINT SHEAR MODELING PROCEDURE - CASE 1**

BDA 14-4 (AUG 2008)

DISPLACEMENT DEMANDS

	Joint	U1 Long	U2 Trans
	Text	in	in
Abut	A1	5.10	0.61
	A5	5.38	3.20
PIERS	P2	5.02	4.49
	P3	5.09	8.57
	P4	5.31	8.41

GLOBAL PUSHOVER CAPACITY - TRANSVERSE LEFT PUSH

Abut 1	0.96	@ Step 3	6764 k/ft used for spring at Abut 1, corresponding to a 1" displacement
Abut 5	1.41	@ Step 3	4310 k/ft used for spring at Abut 5, corresponding to a 1.5" displacement

Abut 1	0.96	@ Step 3	6764 k/ft used for spring at Abut 1, corresponding to a 1" displacement
Abut 5	1.41	@ Step 3	4310 k/ft used for spring at Abut 5, corresponding to a 1.5" displacement

TABLE: Joint Displacements

StepType	Transverse Displacements			Transverse Shears			
	PIER 2	PIER 3	PIER 4	PIER 2	PIER 3	PIER 4	TOTAL
Text	in	in	in	Kip	Kip	Kip	Kip
Step 1	1.34	2.05	1.68	212	244	139	595
Step 2	1.73	2.84	2.31	272	244	141	658
Step 3	7.42	11.27	7.77	272	244	141	658
Step 4	7.21	11.12	7.98	239	226	0	466
Step 5	7.30	11.27	8.12	253	244	0	497
Step 6	9.24	14.21	10.68	272	244	0	516
Step 7	9.23	14.67	10.65	271	0	0	271
Step 8	12.62	19.69	14.17	272	0	0	272

P4 (Bot)

P3 (Bot)

P2 (Bot)



BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

REDUCED MOMENT CAPACITY DUE TO JOINT SHEAR MODELING PROCEDURE - CASE 1

BDA 14-4 (AUG 2008)

GLOBAL PUSHOVER CAPACITY - LONGITUDINAL LEFT PUSH (FULL SPRINGS AT EACH ABUTMENT)

Abut 1	6.29	@ Step 5	3817 k/ft used for spring at Abut 1, corresponding to a 7" displacement
Abut 5	6.40	@ Step 5	2583 k/ft used for spring at Abut 1, corresponding to a 7" displacement
Abut 1	6.30	@ Step 5	3999 k/ft used for spring at Abut 1, corresponding to a 6.5" displacement
Abut 5	6.41	@ Step 5	2699 k/ft used for spring at Abut 1, corresponding to a 6.5" displacement

TABLE: Joint Displacements							
StepType	Longitudinal Displacements			Longitudinal Shears			
	PIER 2	PIER 3	PIER 4	PIER 2	PIER 3	PIER 4	TOTAL
Text	in	in	in	Kip	Kip	Kip	Kip
Step 1	-0.64	-0.64	-0.64	-58	-46	-27	-131
Step 2	-2.31	-2.33	-2.32	-101	-88	-65	-254
Step 3	-3.60	-3.64	-3.63	-126	-116	-83	-325
Step 4	-4.94	-4.99	-4.96	-101	-105	-83	-289
Step 5	-6.41	-6.48	-6.45	-97	-92	-83	-272
Step 6	-8.59	-8.69	-8.67	-97	-92	-60	-249
Step 7	-9.88	-9.99	-9.97	-97	-92	-60	-249
Step 8	-11.16	-11.29	-11.27	-97	-92	-48	-237
Step 9	-12.50	-12.64	-12.63	-97	-92	-2	-192
Step 10	-13.75	-13.92	-13.90	-97	-92	0	-189
Step 11	-14.95	-15.13	-15.11	-97	-92	0	-189
Step 12	-16.15	-16.35	-16.32	-97	-92	0	-189
Step 13	-17.35	-17.56	-17.54	-97	-92	0	-189
Step 14	-18.55	-18.78	-18.75	-97	-92	0	-189
Step 15	-19.75	-19.99	-19.96	-97	-92	0	-189
Step 16	-21.04	-21.29	-21.26	-78	-92	0	-170
Step 17	-22.53	-22.80	-22.75	-25	-74	0	-99

P3 (Top) & P2 (Top)

P4 (Top)

P4 (Bot)

P2 (Bot)

P3 (Bot)

GLOBAL PUSHOVER CAPACITY - LONGITUDINAL RIGHT PUSH (FULL SPRINGS AT EACH ABUTMENT)

Abut 1	6.03	@ Step 5	3999 k/ft used for spring at Abut 1, corresponding to a 6.5" displacement
Abut 5	6.13	@ Step 5	2699 k/ft used for spring at Abut 1, corresponding to a 6.5" displacement

TABLE: Joint Displacements							
StepType	Longitudinal Displacements			Longitudinal Shears			
	PIER 2	PIER 3	PIER 4	PIER 2	PIER 3	PIER 4	TOTAL
Text	in	in	in	Kip	Kip	Kip	Kip
Step 1	0.76	0.78	0.79	60	56	40	156
Step 2	2.10	2.14	2.14	95	84	63	242
Step 3	3.55	3.61	3.62	127	111	80	317
Step 4	4.79	4.86	4.86	126	120	83	328
Step 5	6.13	6.21	6.20	98	92	83	273
Step 6	7.36	7.46	7.46	98	92	60	250
Step 7	8.65	8.76	8.75	98	92	60	250
Step 8	10.80	10.92	10.91	98	92	60	250
Step 9	12.06	12.20	12.20	98	92	48	238
Step 10	13.74	13.90	13.90	98	92	0	190
Step 11	14.94	15.12	15.11	98	92	0	190
Step 12	16.14	16.33	16.32	98	92	0	190
Step 13	17.34	17.55	17.54	98	92	0	190
Step 14	18.54	18.76	18.75	98	92	0	190
Step 15	19.74	19.98	19.96	98	92	0	190
Step 16	20.94	21.19	21.17	98	92	0	190
Step 17	22.52	22.80	22.77	78	92	0	171
Step 18	22.52	22.80	22.77	27	74	0	100

P2 (Top) & P3 (Top)

P4 (Top)

P4 (Bot)

P3 (Bot)

P2 (Bot)



MARK THOMAS & COMPANY
Providing Engineering, Surveying & Planning Services

BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

REDUCED MOMENT CAPACITY DUE TO JOINT SHEAR MODELING PROCEDURE - CASE 2

BDA 14-4 (AUG 2008)

Description of Retrofit: 2-6' CIDH shafts at each abutment (both long springs engaged 100%)
Steel Jackets at all 3 piers
Joint Retrofit at all Pier Footings only (resulting in 3 weak joints at column-superstructure connections)

$f_{ce} =$	5000	$v_n =$	$3.5\sqrt{f_{ce}}$ (psi)	Joint Shear Strength, Weak Joint (Pre 1971)
$v_n =$	247.5 psi	$D_{cg} =$	Distance to c.g. of superstructure	
$D_{cg} =$	2.80 ft	$D_{ftg, As-Built} =$	As-Built Footing Depth	
$D_{ftg, As-Built} =$	2.50 ft			

T_c = Column tensile force in the column, obtained from moment-curvature analysis

M_p = Column plastic moment capacity

Column Forces (Pier 2):

	D_{ftg} (ft)	H (ft)	P (k)	Long			Trans		
				L_{contra} (ft)	T_c (k)	$M_{p,l}$ (k-ft)	L_{contra} (ft)	T_c (k)	$M_{p,t}$ (k-ft)
Top	4.00	27.45	580	13.73	1421	2730			
Bot			685	13.73	1421	2834	30.25	1612	8237

Column Forces (Pier 3):

	D_{ftg} (ft)	H (ft)	P (k)	Long			Trans		
				L_{contra} (ft)	T_c (k)	$M_{p,l}$ (k-ft)	L_{contra} (ft)	T_c (k)	$M_{p,t}$ (k-ft)
Top	4.00	30.97	600	15.49	1421	2750			
Bot			710	15.49	1420	2858	33.77	1601	8302

Column Forces (Pier 4):

	D_{ftg} (ft)	H (ft)	P (k)	Long			Trans		
				L_{contra} (ft)	T_c (k)	$M_{p,l}$ (k-ft)	L_{contra} (ft)	T_c (k)	$M_{p,t}$ (k-ft)
Top	4.00	34.49	580	17.25	1421	2730			
Bot			710	17.25	1420	2858	37.29	1601	8302



BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

REDUCED MOMENT CAPACITY DUE TO JOINT SHEAR MODELING PROCEDURE - CASE 2

BDA 14-4 (AUG 2008)

SUPERSTRUCTURE/COLUMN MOMENT RESISTING JOINT

$$v_{jv} = T_c / A_{jv} \quad v_{jv} = \text{Joint vertical shear stress demand}$$

$$M_{pr} = (v_n / v_{jv}) \cdot 1.2M_p \quad M_{pr} = \text{Reduced moment capacity of hinge model}$$

$$(E_c \cdot I_{eff})_{mod} = 0.85 \cdot (E_c \cdot I_{eff}) \cdot (v_n / v_{jv})$$

$$\theta_{pj} = \theta_{yc} + 0.007 \quad \theta_{pj} = \text{Plastic rotation capacity of the joint}$$

$$\theta_{yc} = \Delta_y / L_{contra} \quad \theta_{yc} = \text{Column yield rotation capacity based on a rigid joint}$$

$$B_{cap} = 3.00 \text{ ft} \quad B_{cap} = \text{Bent cap width}$$

$$l_{ac} = 3.40 \text{ ft} \quad l_{ac} = \text{Length of column reinforcement embedded into the bent cap}$$

$$A_{jv} = 10.20 \text{ sq ft} \quad A_{jv} = l_{av} \cdot B_{cap}$$

	P (k)	Long			Trans		
		v_{jv} (psi)	v_n / v_{jv}	$M_{pr,l}$ (k-ft)	v_{jv} (psi)	v_n / v_{jv}	$M_{pr,t}$ (k-ft)
Pier 2	580	967.5	0.26	838			
Pier 3	600	967.5	0.26	844			
Pier 4	710	967.5	0.26	838			

	Long				Trans			
	Δ_y (in)	θ_{yc}	θ_{pj}	θ_{Tot}	Δ_y (in)	θ_{yc}	θ_{pj}	θ_{Tot}
Pier 2	1.14	0.007	0.007	0.014	0.00	0.000	0.000	0.000
Pier 3	1.45	0.008	0.007	0.015	0.00	0.000	0.000	0.000
Pier 4	1.79	0.009	0.007	0.016	0.00	0.000	0.000	0.000

FOOTING/COLUMN MOMENT RESISTING JOINT

$$v_{jv} = T_{jv} / (B_{eff}^{ftg} \cdot D_{ftg}) \quad v_{jv} = \text{Joint vertical shear stress demand}$$

$$M_{pr} = (v_n / v_{jv}) \cdot 1.2M_p$$

$$\theta_{pj} = \theta_{pj} + 0.007$$

$$\theta_{yc} = \Delta_y / L_{contra}$$

$$A_{col} = 18.66 \text{ sq ft} \quad A_{col} = \text{Column Area}$$

$$B_c = 2.50 \text{ ft} \quad B_c = \text{Column Depth}$$

$$D_c = 7.46 \text{ ft} \quad D_c = \text{Column Width, using equivalent length}$$

$$B_{eff}^{ftg} = 9.96 \text{ ft} \quad B_{eff}^{ftg} = B_c + D_c = \text{Effective Footing Width for Oblong Columns similar to Rectangular Columns}$$

	P (k)	D_{ftg} (ft)	Long			Trans		
			v_{jv} (psi)	v_n / v_{jv}	$M_{pr,l}$ (k-ft)	v_{jv} (psi)	v_n / v_{jv}	$M_{pr,t}$ (k-ft)
Pier 2	685	4.00	247.6	1.00	2834	280.9	1.00	8237
Pier 3	710	4.00	247.4	1.00	2858	279.0	1.00	8302
Pier 4	710	4.00	247.4	1.00	2858	279.0	1.00	8302

	Long				Trans			
	Δ_y (in)	θ_{yc}	θ_{pj}	θ_{Tot}	Δ_y (in)	θ_{yc}	θ_{pj}	θ_{Tot}
Pier 2	1.14	0.003	0.045	0.048	1.81	0.001	0.029	0.030
Pier 3	1.45	0.003	0.045	0.048	2.25	0.001	0.029	0.030
Pier 4	1.79	0.003	0.045	0.048	2.97	0.001	0.029	0.030



BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

REDUCED MOMENT CAPACITY DUE TO JOINT SHEAR MODELING PROCEDURE - CASE 2

BDA 14-4 (AUG 2008)

CAPACITY MODEL

		Long (Top)			Long (Bot)		
	EI (K-ft ²)	θ_{yc}	θ_{Tot}	$M_{pr,l}$ (k-ft)	θ_{yc}	θ_{Tot}	$M_{pr,l}$ (k-ft)
Pier 2	5.944E+06	0.007	0.014	838	0.003	0.048	2834
Pier 3	5.944E+06	0.008	0.015	844	0.003	0.048	2858
Pier 4	5.944E+06	0.009	0.016	838	0.003	0.048	2858

		Trans (Top)			Trans (Bot)		
	EI (K-ft ²)	θ_{yc}	θ_{Tot}	$M_{pr,l}$ (k-ft)	θ_{yc}	θ_{Tot}	$M_{pr,l}$ (k-ft)
Pier 2	5.600E+07	0.000	0.000	0	0.001	0.030	8237
Pier 3	5.600E+07	0.000	0.000	0	0.001	0.030	8302
Pier 4	5.600E+07	0.000	0.000	0	0.001	0.030	8302

DEMAND MODEL

Column EI:

$$(E_c \cdot I_{eff})_{jacket} = 1.4 \cdot (E_c \cdot I_{eff})$$

$$(E_c \cdot I_{eff})_{mod} : \text{Weak Joint} = 0.85 \cdot (E_c \cdot I_{eff})_{jacket} \cdot (v_n / v_{jv})$$

$$\text{Retrofit} = (E_c \cdot I_{eff})_{jacket}$$

Columns are defined by gross properties, EI, and modified by the following factors.

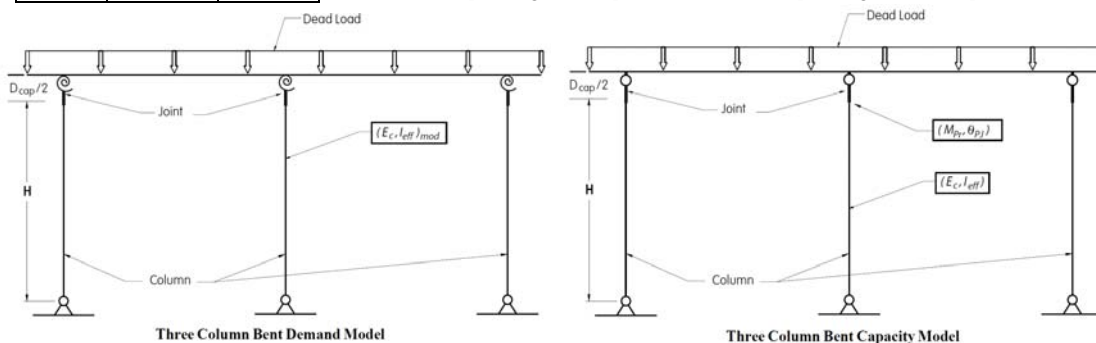
	Longitudinal				Transverse				
	Top	Bottom			Top	Bottom			
	$0.85 v_n / v_{jv}$	$0.85 v_n / v_{jv}$	$(E_c \cdot I_{eff})$	$(E_c \cdot I_{eff})_{jacket}$	$0.85 v_n / v_{jv}$	$0.85 v_n / v_{jv}$	$(E_c \cdot I_{eff})$	$(E_c \cdot I_{eff})_{jacket}$	
Pier 2	0.22	1.00	0.310	0.433		1.00	0.298	0.418	Weak Joint @ Top
Pier 3	0.22	1.00	0.312	0.436		1.00	0.302	0.423	Weak Joint @ Top
Pier 4	0.22	1.00	0.311	0.435		1.00	0.302	0.423	Weak Joint @ Top

	Long	Trans	Long	Trans	Long	Trans
	EI (K-ft ²)	EI (K-ft ²)	EI_{eff} (K-ft ²)	EI_{eff} (K-ft ²)	$(E_c \cdot I_{eff})_{mod}$	$(E_c \cdot I_{eff})_{mod}$
Pier 2	5.944E+06	5.600E+07	1.841E+06	1.670E+07	0.26	0.42
Pier 3	5.944E+06	5.600E+07	1.852E+06	1.690E+07	0.27	0.42
Pier 4	5.944E+06	5.600E+07	1.847E+06	1.690E+07	0.26	0.42

Abutment Springs:

	Long Δ	Trans Δ
Abut 1	4.82	0.65
Abut 5	5.45	3.17

4627 k/ft corresponding to 5" disp and 7947 k/ft corresponding to a .5" disp.
 3104 k/ft corresponding to 5" disp and 3667 k/ft corresponding to a 2.5" disp.





BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

REDUCED MOMENT CAPACITY DUE TO JOINT SHEAR MODELING PROCEDURE - CASE 2

BDA 14-4 (AUG 2008)

DISPLACEMENT DEMANDS

	Joint	U1 Long	U2 Trans
	Text	in	in
Abut	A1	4.82	0.56
	A5	5.07	2.12
PIERS	P2	4.76	4.49
	P3	4.86	8.08
	P4	5.00	6.76

GLOBAL PUSHOVER CAPACITY - TRANSVERSE LEFT PUSH

Abut 1	0.96	@ Step 3	6764 k/ft used for spring at Abut 1, corresponding to a 1" displacement
Abut 5	1.41	@ Step 3	4310 k/ft used for spring at Abut 5, corresponding to a 1.5" displacement

Abut 1	0.96	@ Step 3	6118 k/ft used for spring at Abut 1, corresponding to a 1.5" displacement
Abut 5	1.41	@ Step 3	3667 k/ft used for spring at Abut 5, corresponding to a 2" displacement

TABLE: Joint Displacements

StepType	Transverse Displacements			Transverse Shears			
	PIER 2	PIER 3	PIER 4	PIER 2	PIER 3	PIER 4	TOTAL
Text	in	in	in	Kip	Kip	Kip	Kip
Step 1	1.34	2.05	1.70	212	244	140	596
Step 2	2.22	3.45	2.58	272	244	212	729
Step 3	9.51	14.21	10.23	272	244	212	729
Step 4	9.50	14.67	10.20	270	0	210	480
Step 5	9.51	14.70	10.22	272	0	211	484
Step 6	12.62	19.29	13.48	272	0	211	484
Step 7	13.53	19.61	13.38	0	0	203	203
Step 8	13.67	19.80	13.49	0	0	212	212

P3 (Bot)

P2 (Bot)

P4 (Bot)



BY: BO DATE: 4/2015 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

REDUCED MOMENT CAPACITY DUE TO JOINT SHEAR MODELING PROCEDURE - CASE 2

BDA 14-4 (AUG 2008)

GLOBAL PUSHOVER CAPACITY - LONGITUDINAL LEFT PUSH (FULL SPRINGS AT EACH ABUTMENT)

Abut 1	6.14	@ Step 5	3999 k/ft used for spring at Abut 1, corresponding to a 6.5" displacement
Abut 5	6.24	@ Step 5	2699 k/ft used for spring at Abut 1, corresponding to a 6.5" displacement
Abut 1	6.14	@ Step 5	3999 k/ft used for spring at Abut 1, corresponding to a 6.5" displacement
Abut 5	6.24	@ Step 5	2699 k/ft used for spring at Abut 1, corresponding to a 6.5" displacement

TABLE: Joint Displacements

StepType	Longitudinal Displacements			Longitudinal Shears			
	PIER 2	PIER 3	PIER 4	PIER 2	PIER 3	PIER 4	TOTAL
Text	in	in	in	Kip	Kip	Kip	Kip
Step 1	-0.64	-0.64	-0.64	-58	-46	-27	-131
Step 2	-2.31	-2.33	-2.32	-101	-88	-65	-254
Step 3	-3.78	-3.83	-3.81	-126	-120	-85	-331
Step 4	-4.99	-5.04	-5.01	-99	-103	-101	-304
Step 5	-6.25	-6.32	-6.29	-97	-92	-82	-271
Step 6	-8.35	-8.45	-8.42	-97	-92	-79	-269
Step 7	-9.55	-9.66	-9.63	-97	-92	-79	-269
Step 8	-10.75	-10.87	-10.84	-97	-92	-79	-269
Step 9	-12.15	-12.29	-12.25	-97	-92	-79	-269
Step 10	-13.35	-13.50	-13.46	-97	-92	-79	-269
Step 11	-15.63	-15.80	-15.76	-97	-92	-79	-269
Step 12	-16.83	-17.02	-16.97	-97	-92	-79	-269
Step 13	-18.03	-18.23	-18.19	-97	-92	-79	-269
Step 14	-19.23	-19.45	-19.40	-97	-92	-79	-269
Step 15	-20.43	-20.66	-20.61	-97	-92	-79	-269
Step 16	-22.01	-22.25	-22.20	-42	-92	-79	-214
Step 17	-23.22	-23.47	-23.40	0	-51	-79	-130
Step 18	-24.68	-24.96	-24.88	0	0	-79	-79
Step 19	-25.88	-26.18	-26.09	0	0	-79	-79
Step 20	-27.35	-27.67	-27.59	0	0	-48	-48

P2 (Top) & P3 (Top)
 P4 (Top)
 P2 (Bot)
 P3 (Bot)
 P4 (Bot)

GLOBAL PUSHOVER CAPACITY - LONGITUDINAL RIGHT PUSH (FULL SPRINGS AT EACH ABUTMENT)

Abut 1	5.72	@ Step 5	4181 k/ft used for spring at Abut 1, corresponding to a 6" displacement
Abut 5	5.82	@ Step 5	2197 k/ft used for spring at Abut 1, corresponding to a 6" displacement

TABLE: Joint Displacements

StepType	Longitudinal Displacements			Longitudinal Shears			
	PIER 2	PIER 3	PIER 4	PIER 2	PIER 3	PIER 4	TOTAL
Text	in	in	in	Kip	Kip	Kip	Kip
Step 1	0.76	0.78	0.79	1	1	1	2
Step 2	2.09	2.14	2.15	2	2	2	6
Step 3	3.34	3.40	3.42	3	3	3	10
Step 4	4.66	4.74	4.76	5	5	5	14
Step 5	5.92	6.02	6.03	6	6	6	18
Step 6	8.28	8.41	8.43	8	8	8	25
Step 7	9.48	9.62	9.65	9	10	10	29
Step 8	10.68	10.84	10.86	11	11	11	32
Step 9	12.18	12.37	12.39	12	12	12	37
Step 10	13.38	13.58	13.61	13	14	14	41
Step 11	15.51	15.74	15.77	16	16	16	47
Step 12	16.71	16.96	16.99	17	17	17	51
Step 13	17.91	18.18	18.21	18	18	18	54
Step 14	19.11	19.39	19.42	19	19	19	58
Step 15	20.31	20.61	20.64	20	21	21	62
Step 16	21.62	21.93	21.97	22	22	22	66
Step 17	23.01	23.34	23.36	23	23	23	70
Step 18	24.34	24.69	24.71	24	25	25	74
Step 19	25.80	26.18	26.20	26	26	26	78
Step 20	27.16	27.56	27.59	27	28	28	82

P2 (Top) & P3 (Top)
 P4 (Top)
 P2 (Bot)
 P3 (Bot)



BY: BO DATE: 05/2014 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

ABUTMENT SPRINGS

- 1) Calculations are based on 2-6' shafts at each abutment and steel jackets at the columns.
- 2) Lpile was used to obtain longitudinal and transverse abutment springs with liquefaction effects considered.

Abut 1

Displacement	Values per 1 Shaft				2 shafts			
	Long		Trans		Long		Trans	
	Shear (kips)	Stiffness (kips/in)	Shear (kips)	Stiffness (kips/in)	Shear (kips)	Stiffness (kips/ft)	Shear (kips)	Stiffness (kips/ft)
0.25	108.9	435.6	85.7	342.7	217.8	10453.2	171.3	8224.6
0.5	210.1	420.2	165.6	331.1	420.2	10085.0	331.1	7946.9
1	357.1	357.1	281.8	281.8	714.2	8570.0	563.7	6764.2
2	576.0	288.0	456.0	228.0	1152.1	6912.4	912.0	5472.2
3	743.1	247.7	586.6	195.5	1486.2	5944.9	1173.1	4692.5
4	865.7	216.4	687.4	171.8	1731.4	5194.1	1374.7	4124.2
5	963.9	192.8	769.3	153.9	1927.9	4626.9	1538.5	3692.4
6	1045.3	174.2	837.7	139.6	2090.6	4181.1	1675.5	3350.9
7	1113.4	159.1	896.5	128.1	2226.8	3817.4	1793.1	3073.9
8	1172.9	146.6	949.4	118.7	2345.7	3518.6	1898.7	2848.1
9	1226.5	136.3	998.1	110.9	2453.0	3270.7	1996.3	2661.7
10	1276.0	127.6	1043.9	104.4	2552.0	3062.4	2087.9	2505.5

Abut 5

Displacement	Values per 1 Shaft				2 shafts			
	Long		Trans		Long		Trans	
	Shear (kips)	Stiffness (kips/in)	Shear (kips)	Stiffness (kips/in)	Shear (kips)	Stiffness (kips/ft)	Shear (kips)	Stiffness (kips/ft)
0.25	87.5	349.9	69.2	276.8	174.9	8397.0	138.4	6643.8
0.5	164.0	327.9	129.9	259.8	327.9	7870.3	259.8	6234.3
1	262.5	262.5	206.4	206.4	525.0	6300.1	412.8	4953.3
2	391.3	195.6	305.6	152.8	782.5	4695.2	611.2	3667.2
3	497.1	165.7	388.9	129.6	994.3	3977.0	777.7	3110.8
4	579.3	144.8	453.3	113.3	1158.5	3475.6	906.5	2719.6
5	646.7	129.3	505.5	101.1	1293.4	3104.2	1011.0	2426.4
6	704.0	117.3	549.2	91.5	1408.1	2816.1	1098.3	2196.7
7	753.4	107.6	585.8	83.7	1506.8	2583.0	1171.7	2008.6
8	796.0	99.5	616.4	77.1	1592.0	2388.0	1232.9	1849.3
9	832.7	92.5	641.6	71.3	1665.5	2220.6	1283.3	1711.1
10	863.8	86.4	662.4	66.2	1727.7	2073.2	1324.8	1589.7



MARK THOMAS & COMPANY
Providing Engineering, Surveying & Planning Services

BY: BO DATE: 05/2014 CLIENT: MENDOCINO COUNTY SHEET NO.: OF:
CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.:

LPILE INPUT

- Calculations are based on 2-6' shafts at each abutment and steel jackets at the columns.
- Lpile was used to obtain longitudinal and transverse abutment springs with liquefaction effects considered.

Axial Load on Shaft = 48 kips + weight of additional end diaphragm. Assume 48.6 kips for now

Pile Head Elevation

Abut 1 = 102.83 feet
Abut 5 = 115.25 feet

Top of Soil Layer taken at Top of Existing Footing:

Abut 1 = 105.08 feet
Abut 5 = 117.50 feet

P-Multipliers for Multiple Row Shading

Pile Spacing =	2.0	3.0	5.0	7.0
1st Row =	0.60	0.75	1.00	1.00
2nd Row =	0.35	0.55	0.85	1.00
3rd or Higher =	0.25	0.40	0.70	0.90

P-Multipliers for loading perpendicular to a single row of piles

Pile Spacing =	2.5	3.0	4.0
1st Row =	0.80	0.90	1.00

Abutment P-multipliers

Alternative (Shaft to the sides of existing structure)

	Direction	Pile Diameter (ft)	Pile Spacing (ft)	Pile Spacing (Diameters)	Pile Rows	Piles per Row	Total Piles	GRF (average)
72" CIDH	Long	6.0	27	4.5	1	2	2	1.00
72" CIDH	Trans	6.0	27	4.5	2	1	2	0.86

Alternative (Shafts behind backwall)

	Direction	Pile Diameter (ft)	Pile Spacing (ft)	Pile Spacing (Diameters)	Pile Rows	Piles per Row	Total Piles	GRF (average)
72" CIDH	Long	6.0	15	2.5	1	2	2	0.80
72" CIDH	Trans	6.0	15	2.5	2	1	2	0.56

Current alternative is to use 2-6' shafts behind each abutment.
Placing the shafts to the sides of the existing structures would yield better results and will be evaluated.



BY: BO DATE: 05/2014 CLIENT: MENDOCINO COUNTY SHEET NO.: OF: _____
 CHECKED: DATE: JOB: GARCIA RIVER BRIDGE RETROFIT JOB NO.: _____

LPILE INPUT

Soil Springs

	Shafts on the side		Shafts behind Abutment	
	A1	A5	A1	A5
mp top liquified layer	0.28	0.10	0.28	0.10
mp lower liquified layer	0.28	0.26	0.28	0.26
GRF (Non Liquified) - Long	1.00	1.00	0.80	0.80
GRF (Non Liquified) - Trans	0.86	0.86	0.56	0.56

Vary P-multipliers in layers neighboring the liquefied layer to their full capacity as follows:

Vary Over (ft) = 9.00 1.5 pile diameters
 In 3 Increments of (ft) = 3.00

Alternative (Shaft to the sides of existing structure)

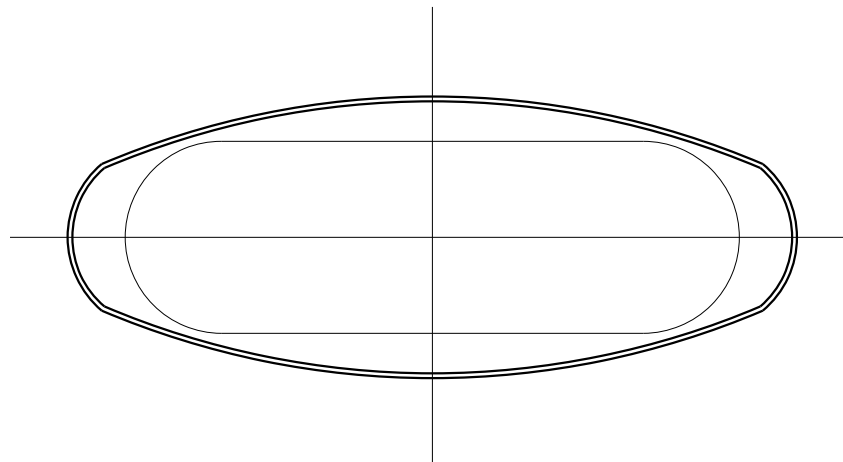
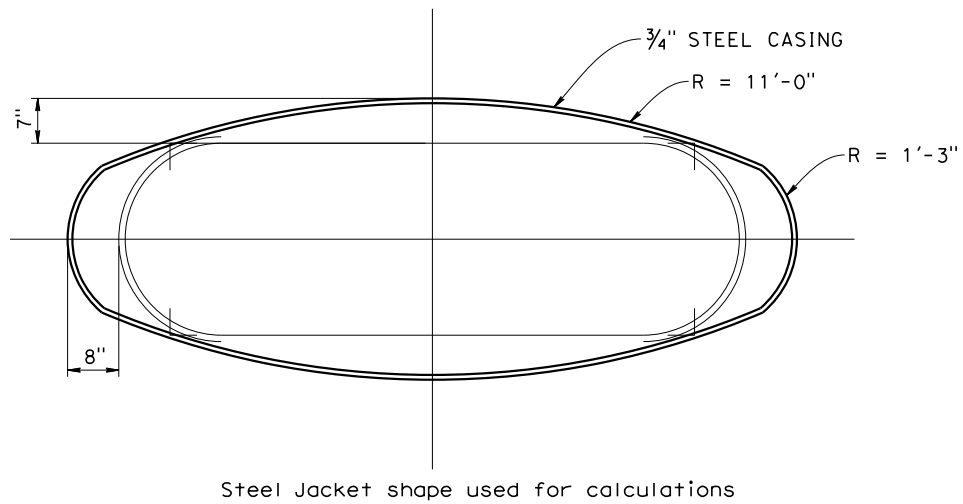
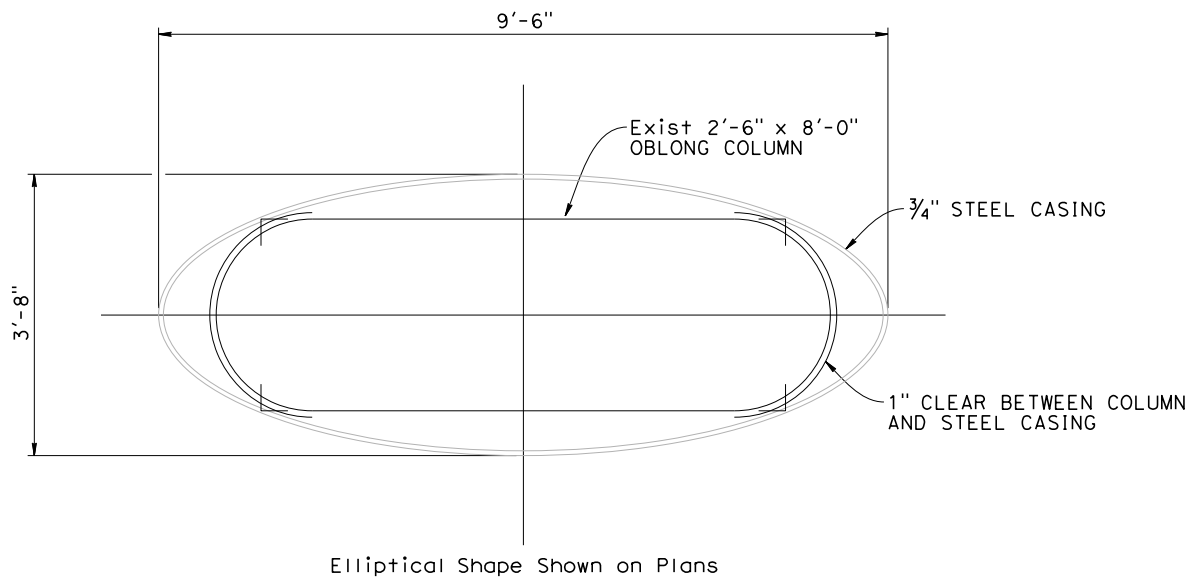
Longitudinal Direction		Dist. from Liquified Layer		
		3.00	6.00	9.00
A1	Layer above liq. layer	0.52	0.76	1.00
	Layer below liq. layer	0.52	0.76	1.00
A5	Layer above liq. layer	0.40	0.70	1.00
	Layer below liq. layer	0.51	0.75	1.00

Transverse Direction		Dist. from Liquified Layer		
		3.00	6.00	9.00
A1	Layer above liq. layer	0.45	0.65	0.86
	Layer below liq. layer	0.45	0.65	0.86
A5	Layer above liq. layer	0.34	0.60	0.86
	Layer below liq. layer	0.43	0.65	0.86

Alternative (Shafts behind backwall)

Longitudinal Direction		Dist. from Liquified Layer		
		3.00	6.00	9.00
A1	Layer above liq. layer	0.42	0.61	0.80
	Layer below liq. layer	0.42	0.61	0.80
A5	Layer above liq. layer	0.32	0.56	0.80
	Layer below liq. layer	0.41	0.60	0.80

Transverse Direction		Dist. from Liquified Layer		
		3.00	6.00	9.00
A1	Layer above liq. layer	0.29	0.43	0.56
	Layer below liq. layer	0.29	0.43	0.56
A5	Layer above liq. layer	0.23	0.39	0.56
	Layer below liq. layer	0.29	0.42	0.56



COLUMN CASING

XTRACT Analysis Report

Section Name: Shaft #14 bars
Loading Name: P69
Analysis Type: Moment Curvature

Section Details:

X Centroid: -76.95E-9 ft
Y Centroid: 2.826E-6 ft
Section Area: 28.26 ft²

Loading Details:

Constant Load - P: 69.00 kips
Incrementing Loads: Mxx Only
Number of Points: 30
Analysis Strategy: Displacement Control

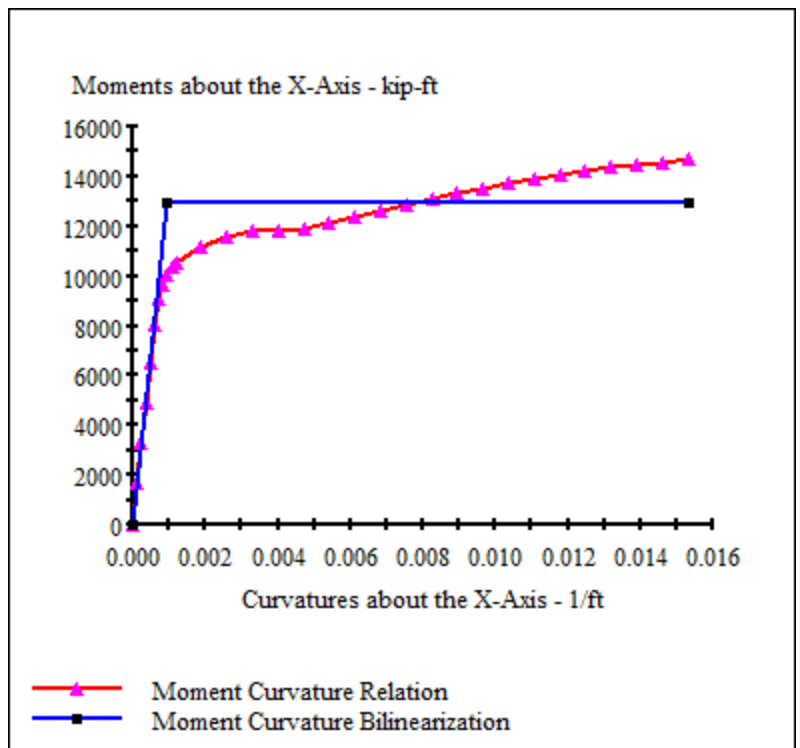
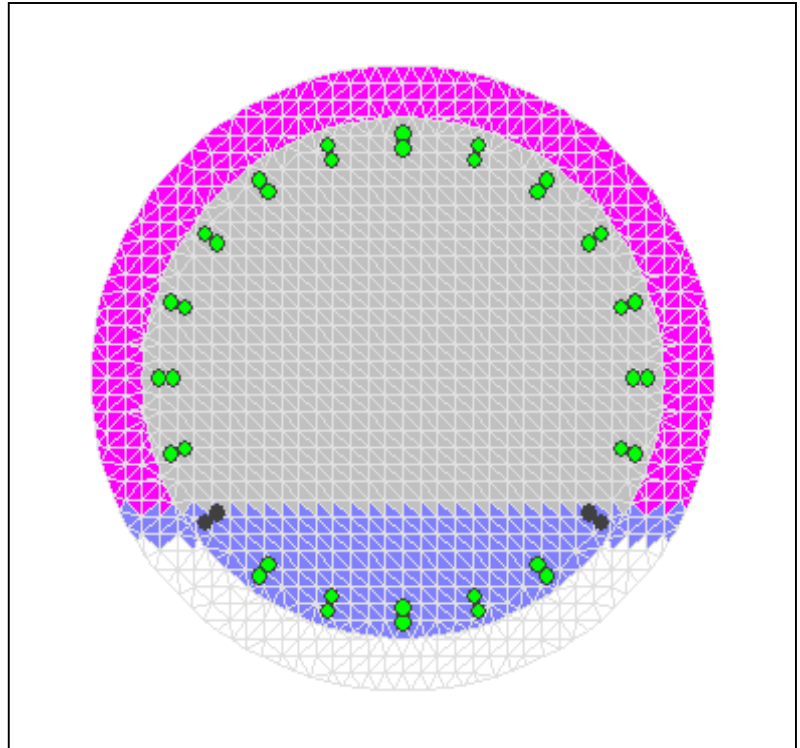
Analysis Results:

Failing Material: Confined #14
Failure Strain: 18.13E-3 Compression
Curvature at Initial Load: .9900E-22 1/ft
Curvature at First Yield: .5984E-3 1/ft
Ultimate Curvature: 15.32E-3 1/ft
Moment at First Yield: 8020 kip-ft
Ultimate Moment: 14.69E+3 kip-ft
Centroid Strain at Yield: .6650E-3 Ten
Centroid Strain at Ultimate: 19.30E-3 Ten
N.A. at First Yield: 1.111 ft
N.A. at Ultimate: 1.260 ft
Energy per Length: 191.9 kips
Effective Yield Curvature: .9654E-3 1/ft
Effective Yield Moment: 12.94E+3 kip-ft
Over Strength Factor: 1.135
EI Effective: 1.34E+7 kip-ft²
Yield EI Effective: 0 kip-ft²
Bilinear Harding Slope: 0 %
Curvature Ductility: 15.87

Comments:

User Comments

Moment Demand based on seismic shear/deflections per shaft = 11,715 kip-ft



XTRACT Analysis Report

Section Name: Retrofit
Loading Name: P580Mx
Analysis Type: Moment Curvature

Strategy Report page 39

AP

Mark Thomas

4/15/2015

Garcia River Bridge Retrofit

EXIST

Page __ of __

Section Details:

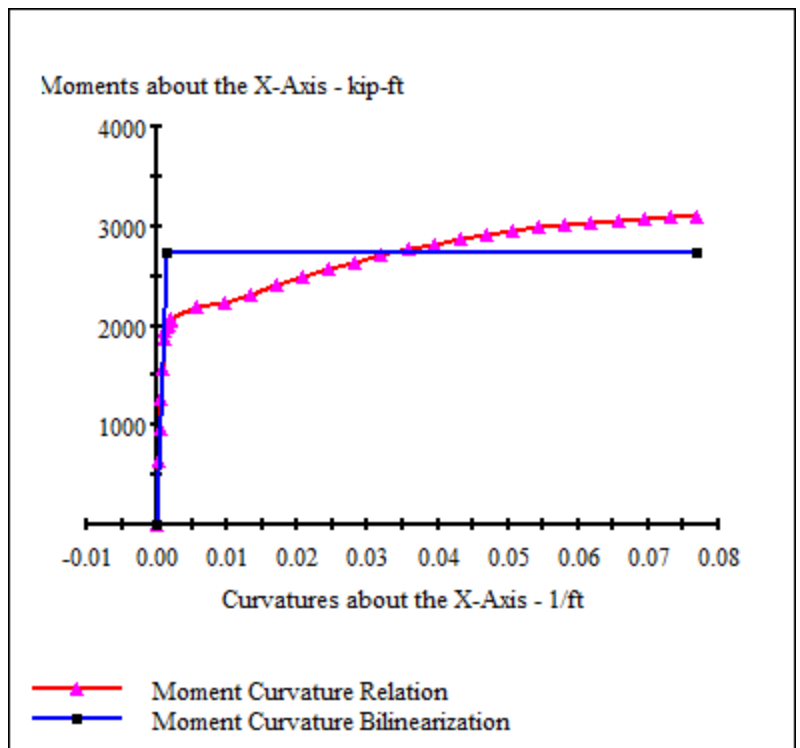
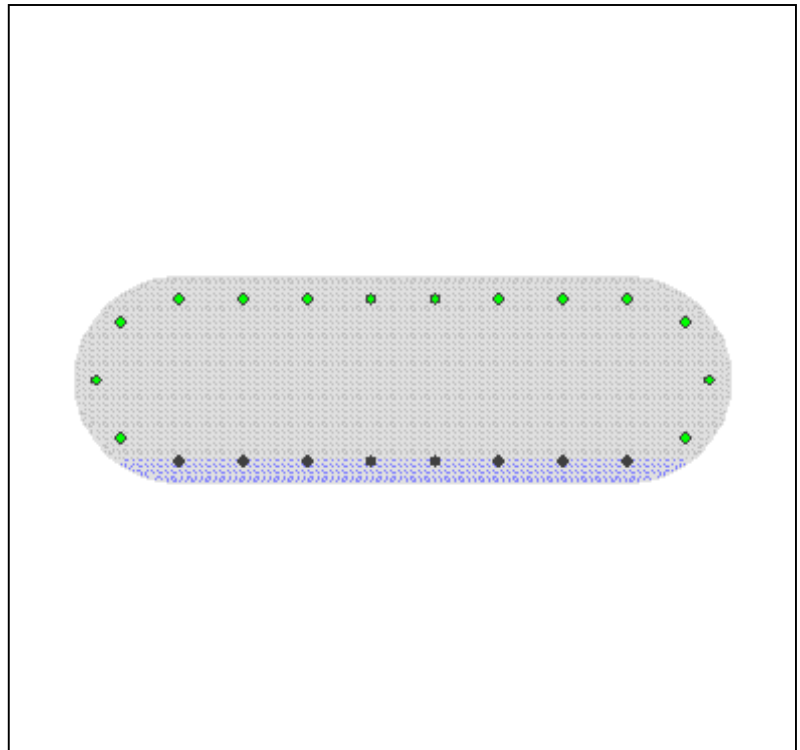
X Centroid: .2026E-14 ft
Y Centroid: -.3241E-17 ft
Section Area: 18.66 ft²

Loading Details:

Constant Load - P: 580.0 kips
Incrementing Loads: Mxx Only
Number of Points: 30
Analysis Strategy: Displacement Control

Analysis Results:

Failing Material: Steel1
Failure Strain: .1500 Tension
Curvature at Initial Load: -.5535E-20 1/ft
Curvature at First Yield: 1.032E-3 1/ft
Ultimate Curvature: 76.87E-3 1/ft
Moment at First Yield: 1870 kip-ft
Ultimate Moment: 3092 kip-ft
Centroid Strain at Yield: .5029E-3 Ten
Centroid Strain at Ultimate: 74.42E-3 Ten
N.A. at First Yield: .4875 ft
N.A. at Ultimate: .9682 ft
Energy per Length: 207.8 kips
Effective Yield Curvature: 1.506E-3 1/ft
Effective Yield Moment: 2730 kip-ft
Over Strength Factor: 1.132
EI Effective: 1.813E+6 kip-ft²
Yield EI Effective: 0 kip-ft²
Bilinear Harding Slope: 0 %
Curvature Ductility: 51.04



Comments:

User Comments

XTRACT Analysis Report

Section Name: Retrofit
 Loading Name: P580My
 Analysis Type: Moment Curvature

Section Details:

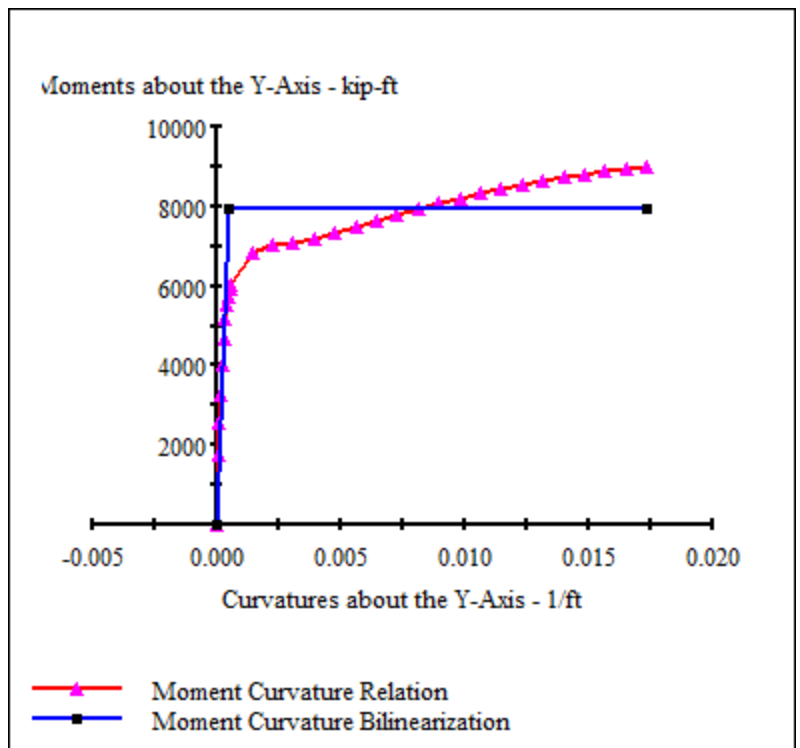
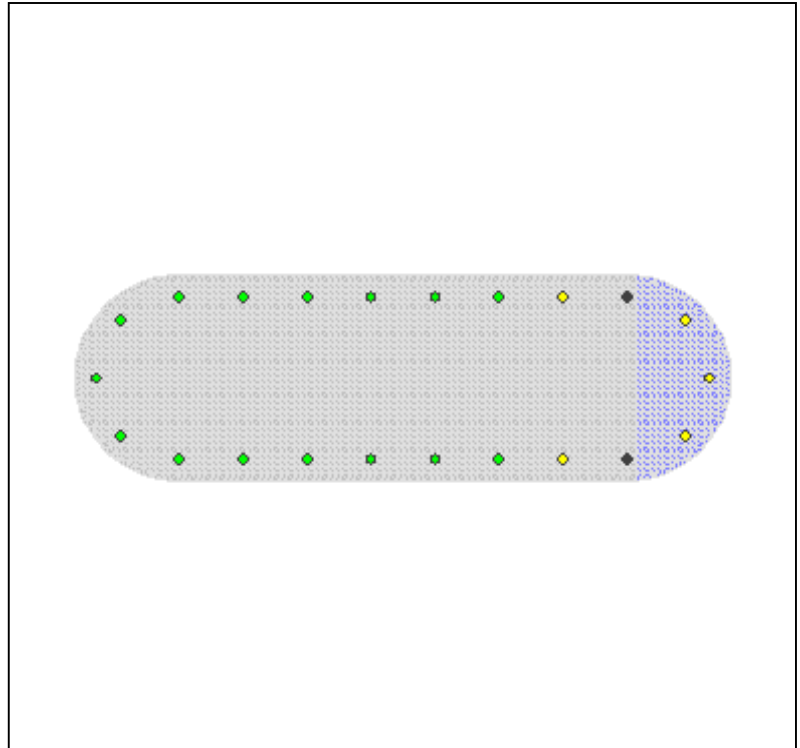
X Centroid: .2026E-14 ft
 Y Centroid: -.3241E-17 ft
 Section Area: 18.66 ft²

Loading Details:

Constant Load - P: 580.0 kips
 Incrementing Loads: Myy Only
 Number of Points: 30
 Analysis Strategy: Displacement Control

Analysis Results:

Failing Material: Confined2
 Failure Strain: 20.00E-3 Compression
 Curvature at Initial Load: -.2314E-19 1/ft
 Curvature at First Yield: .2914E-3 1/ft
 Ultimate Curvature: 17.34E-3 1/ft
 Moment at First Yield: 4696 kip-ft
 Ultimate Moment: 9020 kip-ft
 Centroid Strain at Yield: .4294E-3 Ten
 Centroid Strain at Ultimate: 48.88E-3 Ten
 N.A. at First Yield: 1.474 ft
 N.A. at Ultimate: 2.818 ft
 Energy per Length: 136.1 kips
 Effective Yield Curvature: .4940E-3 1/ft
 Effective Yield Moment: 7960 kip-ft
 Over Strength Factor: 1.133
 EI Effective: 1.61E+7 kip-ft²
 Yield EI Effective: 0 kip-ft²
 Bilinear Harding Slope: 0 %
 Curvature Ductility: 35.11



Comments:

User Comments

XTRACT Analysis Report

Section Name: Retrofit
Loading Name: P600Mx
Analysis Type: Moment Curvature

Section Details:

X Centroid: .2026E-14 ft
Y Centroid: -.3241E-17 ft
Section Area: 18.66 ft²

Loading Details:

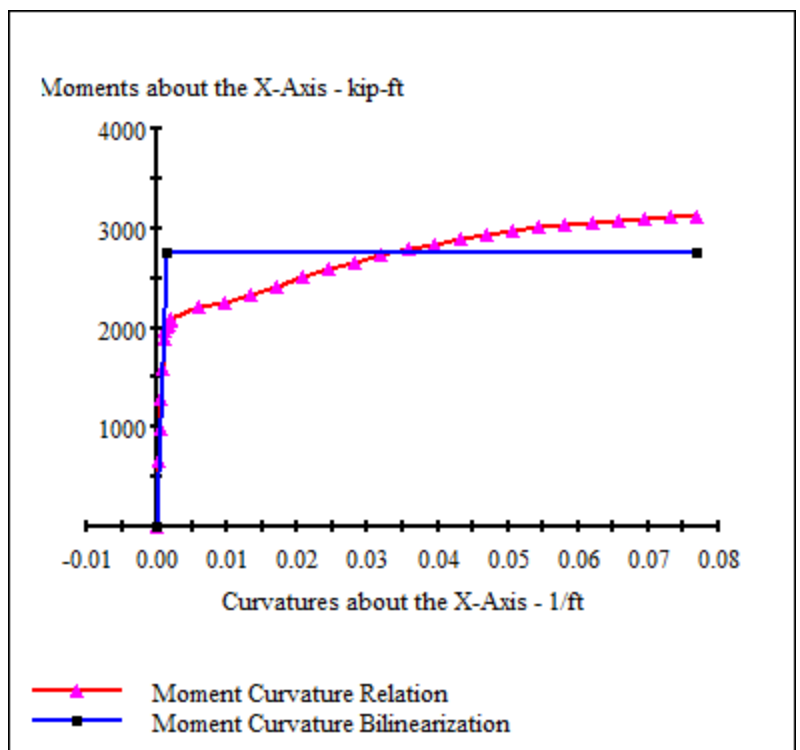
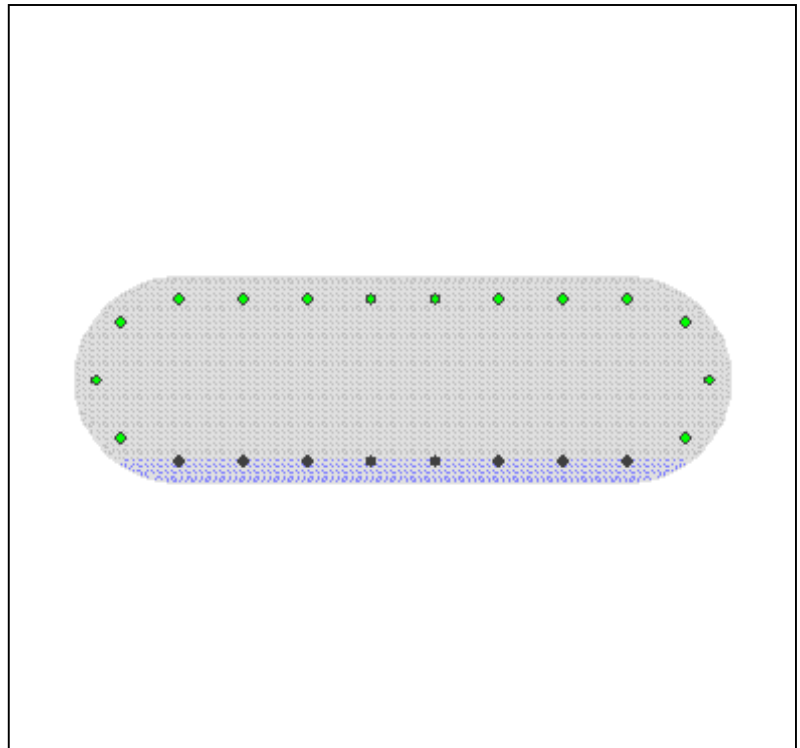
Constant Load - P: 600.0 kips
Incrementing Loads: Mxx Only
Number of Points: 31
Analysis Strategy: Displacement Control

Analysis Results:

Failing Material: Steel1
Failure Strain: .1500 Tension
Curvature at Initial Load: -.5186E-20 1/ft
Curvature at First Yield: 1.035E-3 1/ft
Ultimate Curvature: 76.89E-3 1/ft
Moment at First Yield: 1888 kip-ft
Ultimate Moment: 3112 kip-ft
Centroid Strain at Yield: .4994E-3 Ten
Centroid Strain at Ultimate: 74.40E-3 Ten
N.A. at First Yield: .4824 ft
N.A. at Ultimate: .9676 ft
Energy per Length: 209.4 kips
Effective Yield Curvature: 1.508E-3 1/ft
Effective Yield Moment: 2750 kip-ft
Over Strength Factor: 1.132
EI Effective: 1.824E+6 kip-ft²
Yield EI Effective: 0 kip-ft²
Bilinear Harding Slope: 0 %
Curvature Ductility: 50.99

Comments:

User Comments



XTRACT Analysis Report

Section Name: Retrofit
Loading Name: P600My
Analysis Type: Moment Curvature

Strategy Report page 42

AP

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5/20/2015

Garcia River Bridge Retrofit

EXIST

Page __ of __

Section Details:

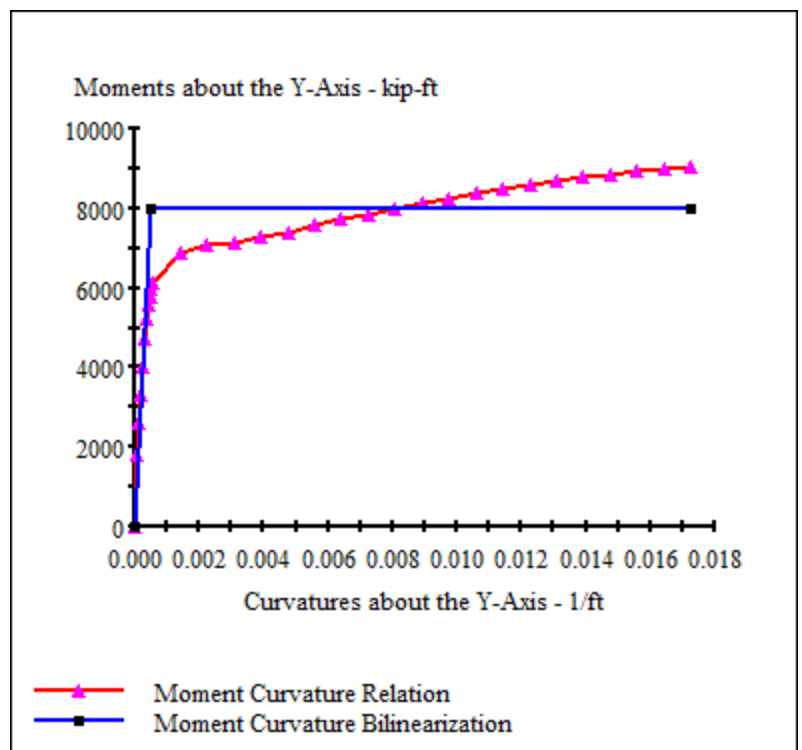
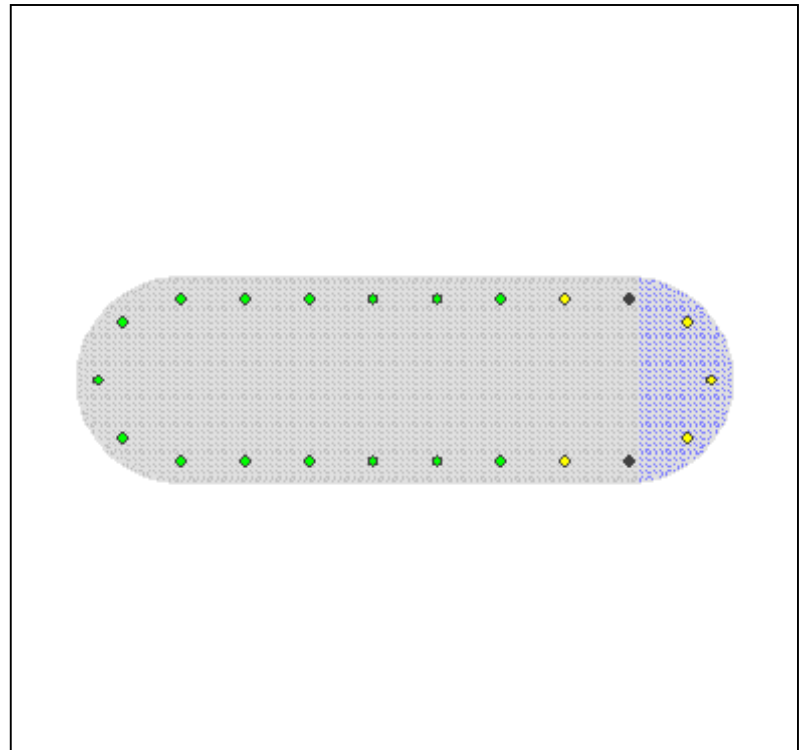
X Centroid: .2026E-14 ft
Y Centroid: -.3241E-17 ft
Section Area: 18.66 ft²

Loading Details:

Constant Load - P: 600.0 kips
Incrementing Loads: Myy Only
Number of Points: 31
Analysis Strategy: Displacement Control

Analysis Results:

Failing Material: Confined2
Failure Strain: 20.00E-3 Compression
Curvature at Initial Load: .3575E-19 1/ft
Curvature at First Yield: .2923E-3 1/ft
Ultimate Curvature: 17.27E-3 1/ft
Moment at First Yield: 4744 kip-ft
Ultimate Moment: 9067 kip-ft
Centroid Strain at Yield: .4261E-3 Ten
Centroid Strain at Ultimate: 48.58E-3 Ten
N.A. at First Yield: 1.458 ft
N.A. at Ultimate: 2.813 ft
Energy per Length: 136.4 kips
Effective Yield Curvature: .4937E-3 1/ft
Effective Yield Moment: 8013 kip-ft
Over Strength Factor: 1.132
EI Effective: 1.62E+7 kip-ft²
Yield EI Effective: 0 kip-ft²
Bilinear Harding Slope: 0 %
Curvature Ductility: 34.98



Comments:

User Comments

XTRACT Section Report

Section Name: Retrofit

Strategy Report page 43

AP

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4/15/2015

Garcia River Bridge Retrofit

EXIST

Page __ of __

Section Details:

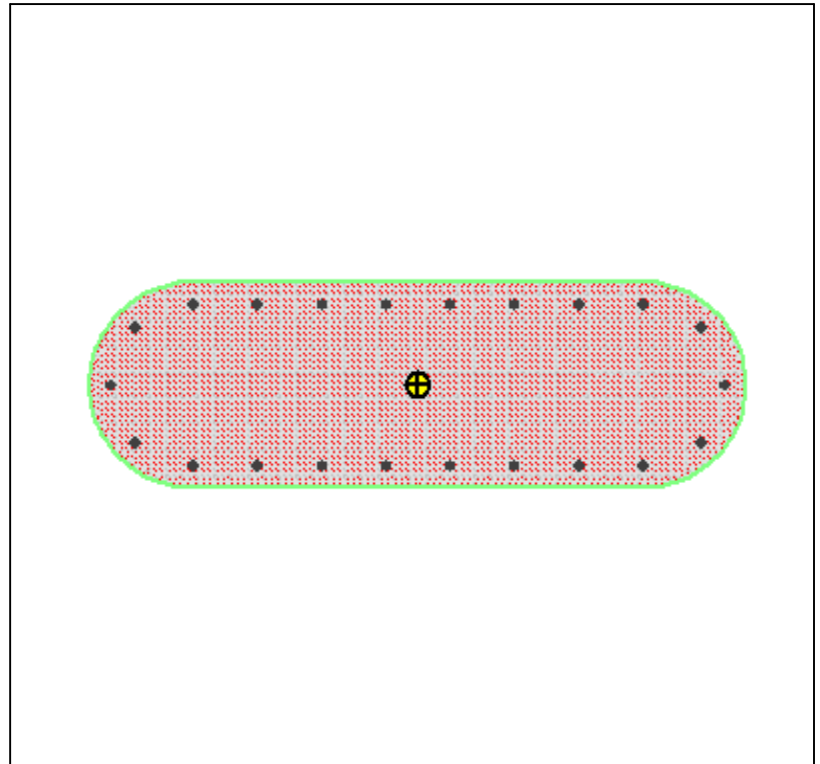
X Centroid:	.2026E-14 ft
Y Centroid:	-.3241E-17 ft
Section Area:	18.66 ft^2
EI gross about X:	5.944E+6 kip-ft^2
EI gross about Y:	5.60E+7 kip-ft^2
I trans (Confined2) about X:	10.24 ft^4
I trans (Confined2) about Y:	96.46 ft^4
Reinforcing Bar Area:	.2387 ft^2
Percent Longitudinal Steel:	1.280 %
Overall Width:	7.999 ft
Overall Height:	2.500 ft
Number of Fibers:	5398
Number of Bars:	22
Number of Materials:	2

Material Types and Names:

Strain Hardening Steel:	■ Steel1
Confined Concrete:	■ Confined2

Comments:

User Comments



XTRACT Material Report

Material Name: Confined
Material Type: Confined Concrete

Strategy Report page 44

AP

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4/15/2015

Garcia River Bridge Retrofit

EXIST

Page __ of __

Input Parameters:

Tension Strength: 0 ksi
28 Day Strength: 5.000 ksi
Confined Concrete Strength: 7.232 ksi
Tension Strain Capacity: 0 Ten
Strain at Peak Stress: 6.464E-3
Crushing Strain: 20.00E-3 Comp
Elastic Modulus: 4031 ksi
Secant Modulus: 1119 ksi

Model Details:

For Strain - $\varepsilon < 2 \cdot \varepsilon_t$ $f_c = 0$

For Strain - $\varepsilon < 0$ $f_c = \varepsilon \cdot E_c$

For Strain - $\varepsilon < \varepsilon_{cu}$ $f_c = \frac{f_{cc} \cdot x \cdot r}{r - 1 + x^r}$

$$x = \frac{\varepsilon}{\varepsilon_{cc}}$$

$$\varepsilon_{cc} = .002 \cdot \left[1 + 5 \cdot \left(\frac{f_{cc}}{f_c} - 1 \right) \right]$$

$$r = \frac{E_c}{E_c - E_{sec}}$$

$$E_{sec} = \frac{f_{cc}}{\varepsilon_{cc}}$$

ε = Concrete Strain

f_c = Concrete Stress

E_c = Elastic Modulus

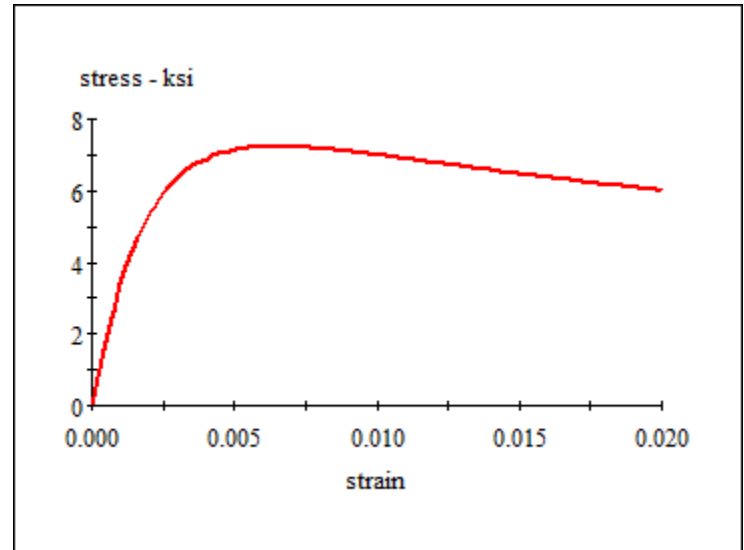
ε_t = Tension Strain Capacity

ε_{cu} = Ultimate Concrete Strain

ε_{cc} = Strain at Peak Stress

f_c = 28 Day Compressive Strength

f_{cc} = Confined Concrete Strength



Material Color States:

- ☐ Tension strain after tension capacity
- ☐ Tension strain before tension capacity
- ☐ Initial state
- ☐ Compression before crushing strain

Reference:

Mander, J.B., Priestley, M. J. N., "Observed Stress-Strain Behavior of Confined Concrete", Journal of Structural Engineering, ASCE, Vol. 114, No. 8, August 1988, pp. 1827-1849

XTRACT Material Report

Material Name: Steel1
Material Type: Strain Hardening Steel

Strategy Report page 45

AP

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4/15/2015

Garcia River Bridge Retrofit

EXIST

Page __ of __

Input Parameters:

Yield Stress: 44.00 ksi
Fracture Stress: 70.00 ksi
Yield Strain: 1.517E-3
Strain at Strain Hardening: 20.00E-3
Failure Strain: .1500
Elastic Modulus: 29.00E+3 ksi
Additional Information: Symetric Tension and Comp.

Model Details:

For Strain - $\varepsilon < \varepsilon_y$ $f_s = E \cdot \varepsilon$

For Strain - $\varepsilon < \varepsilon_{sh}$ $f_s = f_y$

For Strain - $\varepsilon < \varepsilon_{su}$ $f_s = f_u - (f_u - f_y) \cdot \left(\frac{\varepsilon_{su} - \varepsilon}{\varepsilon_{su} - \varepsilon_{sh}} \right)^2$

ε = Steel Strain

f_s = Steel Stress

f_y = Yield Stress

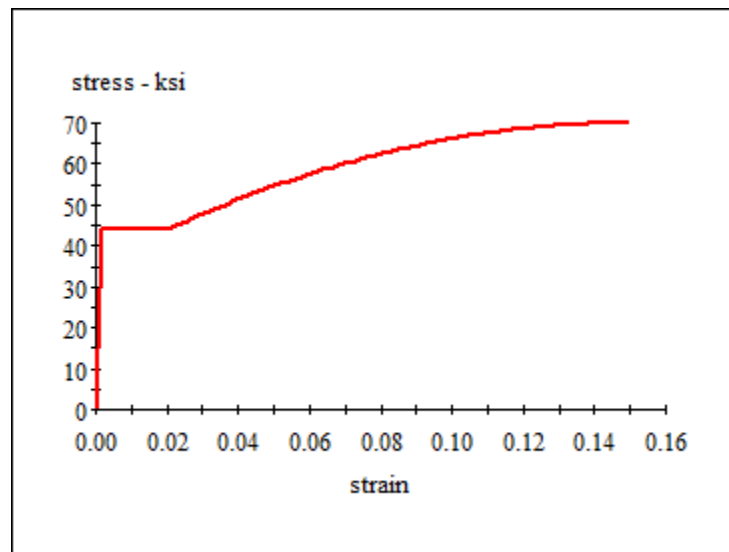
f_u = Fracture Stress

ε_y = Yield Strain

ε_{sh} = Strain at Strain Hardening

ε_{su} = Failure Strain

E = Elastic Modulus



Material Color States:

■ Tension force after onset of strain hardening

■ Tension force after yield

■ Initial state

■ Compression force after yield

■ Compression force after onset of strain hardening

APPENDIX 5

PRELIMINARY FOUNDATION REPORT

PRELIMINARY FOUNDATION REPORT

Garcia River Bridge (Retrofit)
Bridge No. 10C-0034
Mendocino County, California

Prepared by:

BLACKBURN CONSULTING
2491 Boatman Avenue
West Sacramento, California 95691
(916) 375-8706

June 27, 2014

Prepared for:

Mark Thomas & Company, Inc.
Sacramento, California

West Sacramento Office:
2491 Boatman Ave. ▪ West Sacramento, CA 95691
(916) 375-8706 ▪ Fax (916) 375-8709



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Fresno Office: (559) 438-8411
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Geotechnical ▪ Geo-Environmental ▪ Construction Services ▪ Forensics

File No. 1615.1
June 27, 2014

Mr. Zach Siviglia, P.E.
Mark Thomas & Company, Inc.
7300 Folsom Boulevard, Suite 203
Sacramento, CA 95826

Subject: **PRELIMINARY FOUNDATION REPORT**
Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California

Dear Mr. Siviglia:

Blackburn Consulting (BCI) prepared this Preliminary Foundation Report for use by Mark Thomas & Company, Inc. (MTCO) to develop an updated seismic retrofit strategy for the Garcia River Bridge at Eureka Hill Road in Mendocino County, California. BCI prepared this report under our April 16, 2010 Agreement and June 6, 2011 Amendment No. 1 to the Agreement between MTCO and BCI. Patrick Fischer is the responsible Certified Engineering Geologist for the site specific response spectrum and fault rupture hazard evaluations. David Morrell is the responsible engineer related to remainder of this report.

Thank you for the opportunity to be part of your design team. Please call if you have questions or require additional information.

Sincerely,

BLACKBURN CONSULTING


David J. Morrell, C.E., G.E.
Senior Project Manager




Patrick Fischer, P.G., C.E.G., C.E.
Principal



Copies: PDF copy to addressee

PRELIMINARY FOUNDATION REPORT

Garcia River Bridge (Retrofit)
 Bridge No. 10C-0034
 Mendocino County, California

TABLE OF CONTENTS

1	INTRODUCTION.....	1
1.1	Purpose	1
1.2	Scope of Services	1
2	PROJECT LOCATION.....	2
3	SITE DESCRIPTION.....	2
4	PROPOSED SEISMIC RETROFIT	2
5	SITE GEOLOGY.....	2
6	SUBSURFACE CONDITIONS	3
6.1	Subsurface Investigation	3
6.2	Soil and Rock	3
6.3	Ground Water	4
7	LABORATORY TESTING	4
8	PRELIMINARY SEISMIC DATA AND EVALUATION	5
8.1	Ground Motions	5
8.1.1	Soil Profile.....	5
8.1.2	Selection of Input Ground Motions.....	6
8.1.3	Response Analysis.....	7
8.1.4	Site Specific Acceleration Response	8
8.2	Fault Rupture	9
8.3	Liquefaction.....	10
8.4	Seismic Settlement	11
8.5	Seismic Slope Stability and Displacement.....	11
9	PRELIMINARY FOUNDATION RECOMMENDATIONS	12
9.1	Retrofit Pile Strategy Feasibility	12
9.2	Preliminary LPILE Parameters	12
10	PRELIMINARY CONSTRUCTION CONSIDERATIONS	15
10.1	CIDH Retrofit Piling (Abutments).....	15
10.2	CISS Retrofit Piling (Piers).....	15
11	LIMITATIONS	16

PRELIMINARY FOUNDATION REPORT

Garcia River Bridge (Retrofit)

Bridge No. 10C-0034

Mendocino County, California

TABLE OF CONTENTS (Continued)

FIGURES:

Figure 1 – Vicinity Map

Figure 2 – Regional Geologic Map

Figure 3 – Earthquake Fault Zone Map

Figure 4 – Preliminary Combined Design Response Spectrum

APPENDIX A:

Draft Log of Test Borings (2011, Blackburn Consulting)

Log of Test Borings (1997, Taber Consultants)

Log of Test Borings (1968, Clair A. Hill & Associates)

APPENDIX B:

Laboratory Test Results

APPENDIX C:

Liquefaction Triggering Calculations

APPENDIX D:

SLOPE/W 2007 Version Output Graphs

PRELIMINARY FOUNDATION REPORT

*Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California*

BCI File No. 1615.1

June 27, 2014

1 INTRODUCTION

1.1 Purpose

Blackburn Consulting (BCI) prepared this Preliminary Foundation Report for the Garcia River Bridge (Retrofit) project at Eureka Hill Road in Mendocino County, California. This report provides seismic hazard evaluation, seismic design criteria consistent with current Caltrans procedures, preliminary foundation recommendations (retrofit pile strategy feasibility and L-Pile parameters for lateral pile analysis by MTCO), and retrofit pile construction considerations.

This report is for Mark Thomas & Co. (MTCO) and the County of Mendocino to use to develop an updated seismic retrofit strategy for the bridge. It shall not be used or relied upon by others, or for different locations or improvements without the written consent of BCI.

1.2 Scope of Services

To prepare this report, BCI:

- Discussed the project with Zach Siviglia, Jason Hickey and Victor Sherby with MTCO.
- Reviewed the following documents:
 - *Seismic Retrofit Strategy Report, Garcia River Bridge @ Eureka Hill Rd, 10C-0034, Holmes & Narver, Inc., June 30, 1998.*
 - *Foundation Investigation, Garcia River Bridge at Eureka Hill Road, Taber Consultants, September 23, 1997.*
 - *Construction Plans, Garcia River Bridge, Point Arena Air Force Station Access Road, U.S. Department of Transportation, Federal Highway Administration, October 1968.*
 - *Geology and Geomorphic Features Related to Landsliding, Point Arena NE (Eureka Hill) 7.5-Minute Quadrangle, Mendocino County, CA, California DMG Open-File Report 84-47, 1984.*
 - *Earthquake Fault Zone Map, CD 2001-04, California Geological Survey (CGS).*
 - *Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading, California Department of Transportation, October 2013.*
 - 1997 Log of Test Borings prepared by Taber Consultants.
 - 1968 Log of Test Borings prepared by Clair A. Hill & Associates
- Reviewed published maps and literature related to site geologic and seismic conditions.
- Observed, logged and sampled three exploratory borings between October 3-6, 2011 to depths of 99.0 to 110.7 feet below existing ground surface; one in the vicinity of each existing abutment and one approximately mid-span.
- Conducted a field review to evaluate existing site conditions.
- Performed laboratory tests on soil samples obtained from the exploratory borings.
- Performed preliminary engineering analysis and calculations to develop our conclusions and recommendations.

PRELIMINARY FOUNDATION REPORT*Garcia River Bridge (Retrofit), Bridge No. 10C-0034**Mendocino County, California**BCI File No. 1615.1**June 27, 2014***2 PROJECT LOCATION**

The project is located on Eureka Hill Road at the Garcia River crossing, about 4.5 miles southeast of the City of Point Arena in Mendocino County, California. Site coordinates are approximately latitude 38.903272° N and longitude 123.607583°W. We show the project location on Figure 1.

3 SITE DESCRIPTION

The Garcia River Bridge, built in 1969 by the U.S. Army Corps of Engineers, is a 4-span cast-in-place reinforced concrete box girder structure, 22.7 feet wide and 321.8 feet long on a 1000-foot radius curve. Abutments are seat type. Bent columns are pier-type columns measuring about 2.5 feet in depth and 8 feet in width. Both abutment and bents are supported on 12.75-inch diameter steel pipe piles with a 0.375-inch wall thickness. Existing approach fills (8 to 28 feet in height) are constructed on natural terrace deposits. The west approach (Abutment 1) extends about 30 feet above the adjacent channel bottom while the east approach extends about 40 feet above the adjacent channel bottom.

At the bridge location, the Garcia River flows northerly within a channel area that is approximately 180 feet wide. From Summer to early Fall, river flows are generally confined to the low flow channel, approximately 60 feet wide, located on the western side of the riverbed.

4 PROPOSED SEISMIC RETROFIT

The project consists of seismic retrofit of the existing bridge to correct structural deficiencies based on a “no collapse” criterion. The improvements will strengthen the structural components to resist seismic loads on the structure.

The proposed retrofit strategy includes:

- Placing steel casings on columns for Piers 2, 3, and 4.
- Footing retrofits at Piers 2, 3, and 4 that consists of driving 24-inch diameter Cast-In-Steel-Shell (CISS) piles, and constructing top mats to reinforce the widened footing.
- Retrofit of the bridge abutments by construction of two 72” diameter Cast-In-Drilled-Hole (CIDH) piles at each bridge abutment with a pile cap, and designing abutment catcher blocks and shear keys.

5 SITE GEOLOGY

The project is located in the Coast Ranges geomorphic province of California. This geomorphic province is characterized by a series of discontinuous northwest-trending mountain ranges extending from the Klamath Mountains along the north coast of California to the Transverse Ranges in Southern California. The Coast Ranges are composed of thick Mesozoic and Cenozoic sedimentary strata that have a complex structure due to intense folding, faulting, and metamorphism.

PRELIMINARY FOUNDATION REPORT

*Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California*

BCI File No. 1615.1

June 27, 2014

Published geologic mapping¹ shows surface materials at the project site as Paleocene and Eocene marine sedimentary rocks (sandstone and mudstone) of the German Rancho Formation. Tertiary-Cretaceous Coastal Belt Rocks of the Franciscan Complex (marine sandstone, shale and conglomerate) are mapped just east of the site. No rock outcrops were observed within the channel at the existing bridge and surface materials within the stream channel and banks consist of sand, gravel and cobble (unconsolidated recent alluvium) in the immediate vicinity of the bridge.

The site is located within the San Andreas Fault Zone (North Coast Section) in an Alquist-Priolo Special Studies Zone. Active traces of this fault are mapped approximately 400 feet east of the bridge and 830 feet to the west. Refer to Section 8.2 for further discussion of the surface fault rupture hazard at the site.

The Mendocino County Air Quality Management District map entitled “Map of Likely Areas to Contain Naturally Occurring Asbestos in Mendocino County” shows the project site to be outside the “area of concern” for naturally occurring asbestos (NOA).

6 SUBSURFACE CONDITIONS

6.1 Subsurface Investigation

BCI observed, logged and sampled three exploratory borings drilled with a CME 55 track-mounted rig between October 3-6, 2011 to depths of 99.0 to 110.7 feet below existing ground surface; one in the vicinity of each existing abutment and one approximately mid-span. Taber Drilling advanced Borings BCI-11-03 and BCI-11-02 using 4-inch O.D. solid flight auger, switching to 3-inch O.D. rotary wash methods at or just below the ground water level. Taber advance Boring BCI-11-01 from the existing bridge deck using 6-inch O.D. steel casing and 3-inch O.D. rotary wash drilling methods. Taber obtained relatively undisturbed samples using both Modified California Samplers (equipped with 2.4-inch I.D. brass liners), and Standard Penetration Test samplers (1.5-inch I.D.). Samplers were driven into the ground with a 140 pound automatic trip hammer falling 30 inches.

BCI’s project engineer, Mr. Rob Pickard logged the borings generally consistent with the Caltrans Soil and Rock Logging, Classification and Presentation Manual (2010 Edition).

6.2 Soil and Rock

The existing pavement section at the bridge approaches consists of 6 inches of hot mix asphalt over 12 inches of aggregate base.

At the west abutment area, BCI encountered approach fill to a depth of about 13½ feet, which consisted of medium dense to dense silty sand. Beneath the fill, we observed a 5-foot thick layer of loose silty, clayey sand followed by medium dense, silty gravel with sand to a depth of about 39 feet, then dense to very dense, silty gravel with sand to the 100.5-foot depth explored.

¹ Wagner, D.L., Bortugno, E.J., 1982, Geologic Map of the Santa Rosa Quadrangle, California Geological Survey, Regional Geologic Map No. 2A, Scale 1:250,000.

PRELIMINARY FOUNDATION REPORT

*Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California*

BCI File No. 1615.1

June 27, 2014

At the east abutment area, BCI encountered approach fill to a depth of about 28 feet, which consisted of interlayered loose silty gravel with sand, dense clayey sand, loose silty sand with gravel and loose clayey sand in that order. Beneath the fill, we observed loose silty, clayey sand stiff silt and very stiff lean clay to a depth of 41 feet, underlain by medium dense silty sand and medium dense well-graded sand with silt and gravel to a depth of about 54 feet. Below a depth of 54 feet, BCI encountered very dense, well graded sand with silt and gravel to a depth of about 83 feet followed by soft, massive, intensely weathered to decomposed Sandstone (greywacke) to the 110.7-foot depth explored.

In the channel area, we observed medium dense to dense, well-graded gravel with sand to a depth of about 17 feet below the channel bottom, followed by very dense, well graded gravel with silt and clay to a depth of about 74.5 feet below the channel bottom. Beneath the very dense gravel, we observed soft to moderately hard, massive, moderately to intensely weathered sandstone (greywacke) to the 99-foot depth explored below the channel bottom.

The 1997 Log of Test Borings prepared by Taber Consultants display similar soil conditions to those described above. The 1968 Log of Test Borings prepared by Clair A. Hill & Associates also display similar conditions, except the classification terminology is different and the sandstone (greywacke) was likely classified as dense to very dense, claybound sand, and gravel.

We include BCI's Draft Log of Test Borings in Appendix A. We also include the 1968 and 1997 Log of Test Borings for reference.

6.3 Ground Water

BCI encountered ground water at a depth of 27.5 feet (elev. 83.3 ft.) in Boring BCI-11-03 (west bank area), and at a depth of 32 feet (elev. 91.7 ft.) in Boring BCI-11-02 during drilling in October 2011. In Boring BCI-11-01 (channel), we encountered ground water at the existing surface water elevation (elev. 85.0 ft.) These ground water levels are generally consistent with groundwater elevations shown on the 1968 and 1997 Log of Test Borings.

Ground water levels can fluctuate due to changes in precipitation, river water levels, pumping of wells, and other factors. BCI expects that ground water levels will be near the river surface water level in the vicinity of the bridge, although slightly higher ground water levels should be expected in the east bank area based on levels measured in Boring BCI-11-02 and in Taber's 1997 Boring B-3.

7 LABORATORY TESTING

To classify the subsurface soil and obtain parameters for analysis, BCI performed laboratory tests on some of the samples obtained from our exploratory borings. Tests included:

- Moisture content
- Density
- Grain size
- Plasticity Index
- Resistance Value

BCI performed laboratory tests in substantial conformance with current ASTM and Caltrans test procedures. Test results are presented in Appendix B. We display the moisture content and dry density test results on the Log of Test Borings in Appendix A.

8 PRELIMINARY SEISMIC DATA AND EVALUATION

8.1 Ground Motions

Considering the known presence of loose to medium dense soils at the site and the potential for liquefaction (see Section 8.3), BCI reviewed dynamic site response for consideration in liquefaction analysis and development of the preliminary design response spectrum for the project structure.

Our preliminary review and analysis consists of the following:

1. Develop a general soil profile
2. Select input ground motions for site response analysis
3. Perform site response analysis

8.1.1 Soil Profile

In general, the soil profile at the abutments consists of 40 to 54 feet of loose to medium dense, silty and clayey sands, and silty gravel, underlain by dense to very dense, silty sand and gravel that are underlain by sandstone (greywacke). Some stiff to very stiff, lean clay is also present in the upper 50 feet. Without consideration for liquefaction potential, the soil profile would be classified as Soil Profile Type D (Caltrans Seismic Design Criteria, V1.7, 2013, Figure B.12).

For preliminary site specific response analysis, we used a soil profile based primarily on the Log of Boring B-11-02 by BCI (2011). We consider this to be a conservative soil profile (from a ground motion amplification standpoint) for the alignment. The soil profile extends to a depth of 85 feet where we interpret it to be underlain by Soil Profile Type C (shear wave velocity of 1,200 to 2,500 feet per second, fps). We show the general profile used in our analysis in Table 1.

PRELIMINARY FOUNDATION REPORT

Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California

BCI File No. 1615.1

June 27, 2014

Table 1: General Soil Profile Used in Analysis			
Soil Layer (USCS Classification)	Depth to Top of Layer (ft)	Total Unit Weight (pcf)	Shear Wave Velocity (fps)
1) Loose, Silty Gravel (GM)	0	110	590
2) Medium Dense, Clayey Sand (SC)	8	130-135	725
3) Loose, Silty Sand (SM)	18	133	690
4) Loose, Clayey Sand (SC)	23	127	675
5) Loose, Silty, Clayey Sand (SC-SM)	28	120	755
6) Stiff to Very Stiff, Lean Clay (CL)	34	122	640
7) Medium Dense, Silty Sand (SM)	45	140	980
8) Very Dense, Silty Sand (SM)	53	144	1115
9) Dense, Silty Sand (SM)	68	138	1000
10) Very Dense, Silty Sand (SM)	77	145	1130
11) Type C Soil Column (sandstone)	85	---	1,840

Total unit weights for the soils are estimates and/or are based on laboratory testing. The shear wave velocity is an estimate based on soil/rock type and Standard Penetration Test (N_{60} -value) correlations presented in Caltrans Geotechnical Services Design Manual (Version 1.0, 2009).

8.1.2 Selection of Input Ground Motions

For selection of input ground motions, we used the Caltrans ARS Online (Version 2.2.06) website to determine the initial response spectrum and controlling faults for ground motion at the site. For use in ARS Online, ground conditions at the site are represented as Soil Profile Type D with an approximate shear wave velocity [VS_{30}] of 270 fps.

Caltrans ARS Online includes spectrum based on both deterministic and probabilistic methods. The deterministic spectrum is controlled by the following:

- San Andreas (North Coast) at a distance (R_{jb}) of approximately 0.09 miles (0.15 km) with a maximum magnitude (M_{max}) of 8.0.
- San Andreas (Offshore) at a distance (R_{jb}) of approximately 19.64 miles (31.63 km) with a maximum magnitude (M_{max}) of 8.0.
- Maacama Fault Zone (North Section) at a distance (R_{jb}) of approximately 28.02 miles (45.11 km) with a maximum magnitude of 7.4.

However, for this site the probabilistic spectrum controls (has the greatest spectral values) over all site periods. The probabilistic spectrum is based on a probability of exceedance equal to 5% in 50 years (975 year return period).

PRELIMINARY FOUNDATION REPORT

Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California

BCI File No. 1615.1

June 27, 2014

To determine the modal (most likely) earthquake magnitude and distance for the event controlling the 975 year return period, we used the USGS 2008 Interactive Deaggregation (<http://geohazards.usgs.gov/deaggint/2008/>) website. Based on deaggregation, the modal earthquake magnitude and distance for the return period of interest is:

- Magnitude 8.0
- Distance of less than 0.6 miles (1.0 km)

Based on the modal earthquake magnitude, distance, Site Class/Geology, style of controlling faults (strike-slip), and ground accelerations, we selected the following earthquake time histories (pairs) from the Pacific Earthquake Engineering Research Center (PEER) Ground Motion Database website (<http://peer.berkeley.edu>) for use in our preliminary site response analysis:

- Kocaeli, Turkey, 1999, M = 7.51 (NGA #1165)
- Kocaeli, Turkey, 1999, M = 7.51 (NGA #1148)
- Landers, 1992, M = 7.28 (NGA #879)
- Chi-Chi, Taiwan 1999, M = 6.2 (NGA #2739)
- Chi-Chi, Taiwan 1999, M = 6.2 (NGA #2734)
- Hector Mine, 1999, M = 7.13 (NGA #1787)
- Imperial Valley, 1979, M = 6.53 (NGA #164)
- Kobe, Japan, 1995, M = 6.9 (NGA #1111)
- Victoria, Mexico, 1980, M = 6.33 (NGA #265)
- Morgan Hill, 1984, M = 6.19 (NGA #448)
- Morgan Hill, 1984, M = 6.19 (NGA #451)
- Morgan Hill, 1984, M = 6.19 (NGA #459)
- Parkfield, 1966, M = 6.19 (NGA #33)
- Duzce, Turkey, 1999, M = 7.14 (NGA #1617)

8.1.3 Response Analysis

For preliminary site response analysis, we used the SHAKE2000 program (v 9.1.0, 2012). This program computes the response in a system of homogeneous, viscous-elastic layers of infinite horizontal extent subjected to vertically traveling shear waves. The nonlinearity of the shear modulus and damping is accounted for by the use of equivalent linear soil properties using an iterative procedure to obtain values for modulus and damping compatible with the effective strains in each layer.

In the analysis, we use the soil profile and ground motions indicated above. We scaled ground motions to approximate a Target Spectrum. The Target Spectrum represents ground motions for Soil Profile Type C that underlies the analyzed soil column, and is obtained from the Caltrans ARS Online spectrum (975 year return period) with an approximate shear wave velocity [VS_{30}] of 1840 fps.

PRELIMINARY FOUNDATION REPORT

Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California

BCI File No. 1615.1

June 27, 2014

We use the Ratio of Response Spectra (RRS) to develop the design spectra at the ground surface. Response spectra at the ground surface are calculated by the program for the soil profile and each input ground motion. The RRS for ground surface motions to input outcropping motion (for Site Class C at a depth of 85 feet) are then determined by the program. We applied the mean plus one standard deviation of the RRS to the Target Spectrum and then obtained modified spectrum for the site. Due to the presence of a variable soil column with various depths of liquefiable soil, and in accordance with MTCO's request, we also considered the standard Caltrans ARS for a non-liquefied soil profile (shear wave velocity $[V_{S30}]$ of 1033 fps) and combined the envelope of the two spectrum for preliminary design consideration.

8.1.4 Site Specific Acceleration Response

Our review of the preliminary site specific ground response indicates the following:

- A preliminary Peak Ground Acceleration value of 0.83g is appropriate in assessment of liquefaction at the site.
- We assign the site a Maximum Moment Magnitude (M_{max}) of 8.0.
- For preliminary design considerations, use the envelope spectrum to account for affects of the site soil profile and potential liquefaction.
- For ground motion analysis, BCI used the equivalent-linear method for this preliminary site specific evaluation. Further analysis, possibly including non-linear analysis, should be considered for final design due to variable soil profiles, significant liquefiable soil layers, high ground motions, and the potential for fault rupture at the site. Caltrans should be consulted to determine if further analysis is necessary for final design.

Table 2 presents our Preliminary Combined Design Response Spectrum Data and Figure 4 shows the curve.

Table 2: Preliminary Combined Design Response Spectrum Data	
Period (seconds)	Acceleration Response Spectrum Data (g)
0.00	0.83
0.01	0.83
0.02	0.97
0.03	1.05
0.05	1.18
0.075	1.28
0.10	1.36
0.20	1.70
0.25	1.77
0.30	1.83
0.40	1.87
0.50	1.90
0.75	1.90
1.00	1.87
1.50	1.49
2.00	1.09
3.00	0.72
4.00	0.53
5.00	0.42

PRELIMINARY FOUNDATION REPORT

*Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California*

BCI File No. 1615.1

June 27, 2014

8.2 Fault Rupture

The site is crossed by the northwest trending San Andreas Fault (North Coast Section) and is located within an Alquist-Priolo Earthquake Fault Zone (EFZ) with mapped fault traces located west and east of the bridge (See Figure 3, Earthquake Fault Zone Map). The fault is historically active with the most recent rupture in 1906. The maximum moment magnitude for this section of the San Andreas Fault is 8.0 based on the 2013 Caltrans Fault Database.

Existing Alquist-Priolo and published Lidar mapping² shows:

- The 1906 rupture is located approximately 830 feet west of the existing bridge.
- A historically active trace is shown approximately 400 feet east of the bridge.
- Lidar imagery indicates a linear feature consistent with faulting approximately 265 feet east of the bridge.

Measurements of site specific fault displacements are not available for the site. Trenching to identify/evaluate traces of the San Andreas Fault at the bridge site would be difficult due to dense vegetation and locally steep slopes. The closest possible access would be 90 feet north of the bridge abutments at both the west and east banks. The Garcia River drainage contains thick fluvial channel and bank deposits too recent to allow for observation of active fault traces in the channel area.

To calculate expected displacement at the bridge, we used Caltrans methods outlined in Memo to Designers 20-10 and the Caltrans spreadsheet for Strike Slip offset which is available at http://www.dot.ca.gov/hq/esc/geotech/geo_support/geo_instrumentation/fault_rupture/. We used the following fault parameters based on the 2013 Caltrans Fault Database:

- Mmax-8.0
- Slip rate- 24mm/yr
- Aseismic slip factor- 0

Displacements at the main trace of the San Andreas Fault are calculated to be:

- 25.0 feet deterministically
- 31.3 feet probabilistically (return period of 5% in 50 years)

Displacement is most likely to occur along the 1906 rupture, but displacement could occur at other locations. To calculate potential offset at the bridge, we divide the displacement as follows:

- 1906 main trace, weighted at 70%
- Secondary trace 265 feet east of the bridge (as observed in Lidar imagery), weighted at 15%
- An inferred tertiary trace at the bridge, weighted at 15%

² Zachariasen, J. & Prentice, C., Final Technical Report, Detailed Mapping of the Northern San Andreas Fault Using Lidar Imagery, U.S. Geological Survey National Earthquake Hazards Reduction Program Award Number 05HQGR0069, June 2008.

PRELIMINARY FOUNDATION REPORT

*Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California*

BCI File No. 1615.1

June 27, 2014

Considering the lack of site specific data, a probabilistic displacement value should be used. Based on the above parameters and the Caltrans Strike-Slip Offset (Excel spreadsheet), bridge design should accommodate a probabilistic right-lateral offset of 4.3 feet which could potentially occur between any of the supports. The trend of the movement could range from 310 degrees to 330 degrees (true north azimuth) based on the trend of other mapped fault traces within the San Andreas Fault (North Coast Section) zone.

A field investigation could be completed to further evaluate the location of active traces and/or displacement magnitude; however, detection of active traces by trenching or other geophysical techniques would be difficult. Additional work would most likely not eliminate or significantly reduce the expected displacement at the bridge site.

8.3 Liquefaction

Soil liquefaction can occur when saturated, relatively loose sand and specific soft, fine-grained saturated soils are subject to ground shaking strong enough to create soil particle separation that results from increased pore pressure. This separation and subsequent pore pressure dissipation can lead to decreased soil shear strength and settlement.

Liquefaction susceptibility of a soil deposit is a function of the soil grain size, relative density, percent fines, plasticity of the fines, degree of saturation, age of deposit, and earthquake ground motion. BCI used the simplified procedure outlined by Youd et al. (2001)³ to evaluate the potential for soil liquefaction to occur at the site.

We evaluated soil liquefaction potential using data from our 2011 borings and from Taber Consultant's 1997 borings. We used a site-to-fault distance of 350 feet, Maximum Moment Magnitude (M_{max}) of 8.0, and a Peak Ground Acceleration (PGA) of 0.83g. For our analysis, WRECO provided mean winter and mean summer river water elevations that we generally used to determine the depth to ground water in our analysis. Where the ground water levels measured in the borings were higher than both the mean winter (elev. 88.8 ft.) and mean summer levels (elev. 84.8 ft.), we used the higher elevation to determine the depth to ground water in our analysis.

Based on our analysis of the existing boring data, potentially liquefiable soils are present at the site. We identify potentially critical liquefiable soil zones in Table 3 based on our analysis results and engineering judgment given the potential for soil variability between the borings.

Table 3: Potentially Liquefiable Soil Zones	
Location	Potentially Liquefiable Soil Zone (elevation, feet)
Abut 1	82.0 to 70.0
Pier 2	82.0 to 70.0
Pier 3	85.0 to 70.0
Pier 4	89.0 to 70.0
Abut 5	89.0 to 70.0

³ Youd, T. L., et al, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, October 2001, pp. 817-833

PRELIMINARY FOUNDATION REPORT

Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California

BCI File No. 1615.1

June 27, 2014

Appendix C contains our calculation results for liquefaction triggering along with estimated residual shear strength values and reduced p-multipliers (for lateral pile analysis) for the liquefiable material.

8.4 Seismic Settlement

BCI used methods outlined by Idriss and Boulanger (2008)⁴ to estimate post-liquefaction settlement. The method follows the approach developed by Ishihara and Yoshimine (1992) that relates volumetric strain, SPT $(N_1)_{60cs}$, and the factor of safety against liquefaction to estimate the post-liquefaction settlement of a liquefied layer.

Based on our analysis of the existing boring data, potential post-liquefaction settlement could be up to 2.5 inches at Abutment 1, up to 1 inch at Piers 2 and 3, and up to 7 inches at Pier 4 and Abutment 5.

Post-liquefaction settlement will apply permanent downdrag loads (negative skin friction) on the existing piles and proposed retrofit piles, the magnitude of which should be considered during final retrofit design.

8.5 Seismic Slope Stability and Displacement

BCI evaluated seismic slope stability and deformation for the west channel bank (including approach fill), channel area and east channel bank (including approach fill).

To evaluate post-earthquake slope stability and determine yield acceleration (k_y) values, BCI used SLOPE/W 2007 Version limit equilibrium slope stability software by GEO-SLOPE International Ltd. We analyzed cross-sections for bank areas using the Spencer method of slices, which satisfies both force and moment equilibrium and compared both circular and non-circular search algorithms. The yield acceleration is the horizontal acceleration acting upon a sliding mass above which permanent deformations occur and corresponds to a Factor of Safety (FS) equal to one.

We performed slope stability analysis without a horizontal seismic coefficient to determine the post-earthquake slope stability Factor of Safety, using residual undrained shear strength as the post-earthquake shear strength of liquefied soils. A post-earthquake FS less than or equal to 1.0 indicates an unstable slope condition (i.e., capable of flow-type failure with unlimited movement) and no further analysis is needed. Where the post-earthquake FS is greater than 1.0, we performed pseudo-static slope stability analysis to determine the yield acceleration (k_y). The yield acceleration is then used to estimate the seismic slope displacement.

Based on our analysis there is potential for significant ground displacement during the design seismic event ($PGA = 0.83g$). Table 4 shows the results of our seismic slope stability and displacement analysis.

⁴ Idriss, I. M. and Boulanger, R. W., "Soil Liquefaction During Earthquakes," Earthquake Engineering Research Institute, pages 140-142 and 152-158, 2008.

PRELIMINARY FOUNDATION REPORT

Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California

BCI File No. 1615.1

June 27, 2014

Table 4: Seismic Slope Stability and Displacement					
Location	Bridge Supports	Post-Earthquake Slope Stability Factor of Safety	Yield Acceleration (k_y)	Horizontal Displacement	Base of Movement (elev.)
West Bank	Abutment 1	1.67	0.21	2 feet	70.0 feet
Channel Area	Piers 2 and 3	*	*	2 feet	70.0 feet
East Bank	Pier 4 and Abutment 5	0.76	N/A	Unlimited Movement (flow slide)	70.0 feet

*BCI did not evaluate slope stability within the channel area but instead, estimated horizontal displacement using Faris et al. (2006) method⁵.

To estimate seismic slope displacement for the west bank area, BCI used both the Faris (2006) method discussed above and the Bray and Travasarou (2007) method⁵.

Appendix D contains selected graphical SLOPE/W 2007 Version output that shows the results of our analysis, including the input soil parameters.

9 PRELIMINARY FOUNDATION RECOMMENDATIONS

9.1 Retrofit Pile Strategy Feasibility

The proposed retrofit strategy of using large diameter Cast-In-Drilled-Hole (CIDH) Piles to retrofit abutment foundations and 24-inch diameter Cast-In-Steel-Shell (CISS) piles to retrofit the existing pier foundations is feasible provided the piles are designed to consider the anticipated seismic demand, fault rupture offset, lateral spreading and liquefaction-induced downdrag based on a “no bridge collapse” criterion.

Foundation loading and deformation due to lateral spreading should be evaluated in accordance with Caltrans *Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading* (October 2013), using the “Unrestrained Case” method.

Once the proposed retrofit strategy has been approved, BCI will prepare a Seismic Retrofit Foundation Report for the project that will include updated conclusions and final design recommendations, including evaluation of existing pile/retrofit pile axial compression, axial tension capacity, estimated pile downdrag loading and pile settlement estimates.

9.2 Preliminary LPILE Parameters

Use the LPILE parameters shown in Tables 5-9 to evaluate the lateral pile capacity for the existing piles and seismic retrofit foundation elements. Use the P-Multiplier (m_p) to generate p-y curves for liquefied soil. For evaluation of foundation loading and deformation due to

⁵ Per method outlined in the Caltrans *Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading* (October 2013)

PRELIMINARY FOUNDATION REPORT

Garcia River Bridge (Retrofit), Bridge No. 10C-0034

Mendocino County, California

BCI File No. 1615.1

June 27, 2014

liquefaction induced lateral spreading, the tables include our recommended lateral spread soil movements for each support and reduced p-multipliers for the liquefiable layers.

**Table 5: Preliminary LPILE Parameters
(Abutment 1)**

Elevation (feet)	L-Pile Soil Type (p-y curve model)	Unit Weight (lb/ft ³)	Friction Angle (degrees)	Cohesion (psf)	E ₅₀	**Modulus k (lb/in ³)	Reduced P-Multiplier m _p *** (lb/in ³)	Lateral Spread Soil Movement (in.)
111.0 to 97.5	Sand (Reese)	125	34	--	--	90	--	24
97.5 to 92.5	Sand (Reese)	112	29	--	--	25	--	24
92.5 to 85.0	Sand (Reese)	125	34	--	--	90	--	24
85.0 to 82.0	Sand (Reese)	*63	34	--	--	60	--	24
82.0 to 70.0	Sand (Reese)	*58	35	--	--	60	0.28	24 (layer top) to zero (layer bottom)
70.0 to -12.0	Sand (Reese)	*78	40	--	--	125	--	--

*Buoyant Unit Weight

**LPILE 5.0 does not report the k value for Clay Without Free Water soil type. The k value is internally computed using the cohesion and E50 input parameters.

*** Use reduced p-multiplier to generate p-y curves for Liquefied Soil: P-Multiplier Method, page 10 of Caltrans "Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading" (October 2013).

**Table 6: Preliminary LPILE Parameters
(Pier 2)**

Elevation (feet)	L-Pile Soil Type (p-y curve model)	Unit Weight (lb/ft ³)	Friction Angle (degrees)	Cohesion (psf)	E ₅₀	**Modulus k (lb/in ³)	Reduced P-Multiplier m _p *** (lb/in ³)	Lateral Spread Soil Movement (in.)
82.0 to 70.0	Sand (Reese)	*68	35	--	--	125	0.28	24 (layer top) to zero (layer bottom)
70.0 to -12.0	Sand (Reese)	*78	40	--	--	125	--	--

*Buoyant Unit Weight

**LPILE 5.0 does not report the k value for Clay Without Free Water soil type. The k value is internally computed using the cohesion and E50 input parameters.

*** Use reduced p-multiplier to generate p-y curves for Liquefied Soil: P-Multiplier Method, page 10 of Caltrans "Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading" (October 2013).

PRELIMINARY FOUNDATION REPORT

Garcia River Bridge (Retrofit), Bridge No. 10C-0034

Mendocino County, California

BCI File No. 1615.1

June 27, 2014

**Table 7: Preliminary LPILE Parameters
(Pier 3)**

Elevation (feet)	L-Pile Soil Type (p-y curve model)	Unit Weight (lb/ft ³)	Friction Angle (degrees)	Cohesion (psf)	ϵ_{50}	**Modulus k (lb/in ³)	Reduced P-Multiplier m_p *** (lb/in ³)	Lateral Spread Soil Movement (in.)
89.0 to 85.0	Sand (Reese)	120	35	--	--	90	--	24
85.0 to 70	Sand (Reese)	*68	35	--	--	60	0.24	24 (layer top) to zero (layer bottom)
70.0 to 12.0	Sand (Reese)	*78	40	--	--	125	--	--
12.0 to -12.0	Weak Rock (Reese)	*78	Young's Modulus (Er) = 29,000 psi, Uniaxial Compressive Strength = 500 psi, RQD (%) = 50, k_rm (stiffness coefficient) = 0.00005				--	--

*Buoyant Unit Weight

**LPILE 5.0 does not report the k value for Clay Without Free Water soil type. The k value is internally computed using the cohesion and E50 input parameters.

*** Use reduced p-multiplier to generate p-y curves for Liquefied Soil: P-Multiplier Method, page 10 of Caltrans "Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading" (October 2013).

**Table 8: Preliminary LPILE Parameters
(Pier 4)**

Elevation (feet)	L-Pile Soil Type (p-y curve model)	Unit Weight (lb/ft ³)	Friction Angle (degrees)	Cohesion (psf)	ϵ_{50}	**Modulus k (lb/in ³)	Reduced P-Multiplier m_p *** (lb/in ³)	Lateral Spread Soil Movement (in.)
95.0 to 89.0	Sand (Reese)	117	25	--	--	25	--	60
89.0 to 85.0	Soft Clay (Matlock)	*55	--	800	0.01	--	0.10	60 (elev. 89.0) to zero (elev. 70.0)
85.0 to 78.0	Stiff Clay with Free Water (Reese)	*58	--	1,500	0.007	500	0.13	
78.0 to 70.0	Sand (Reese)	*68	35	--	--	60	0.26	
70.0 to 30.0	Sand (Reese)	*78	40	--	--	125	--	--
30.0 to -12.0	Weak Rock (Reese)	*78	Young's Modulus (Er) = 29,000 psi, Uniaxial Compressive Strength = 500 psi, RQD (%) = 50, k_rm (stiffness coefficient) = 0.00005				--	--

*Buoyant Unit Weight

**LPILE 5.0 does not report the k value for Clay Without Free Water soil type. The k value is internally computed using the cohesion and E50 input parameters.

*** Use reduced p-multiplier to generate p-y curves for Liquefied Soil: P-Multiplier Method, page 10 of Caltrans "Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading" (October 2013).

PRELIMINARY FOUNDATION REPORT

Garcia River Bridge (Retrofit), Bridge No. 10C-0034

Mendocino County, California

BCI File No. 1615.1

June 27, 2014

**Table 9: Preliminary LPILE Parameters
(Abutment 5)**

Elevation (feet)	L-Pile Soil Type (p-y curve model)	Unit Weight (lb/ft ³)	Friction Angle (degrees)	Cohesion (psf)	ϵ_{50}	***Modulus k (lb/in ³)	Reduced P- Multiplier m_p *** (lb/in ³)	Lateral Spread Soil Movement (in.)
124.0 to 110.0	Sand (Reese)	120	33	--	--	80	--	60
110.0 to 95.0	Sand (Reese)	120	28	--	--	50	--	60
95.0 to 89.0	Sand (Reese)	117	25	--	--	25	--	60
89.0 to 85.0	Soft Clay (Matlock)	*55	--	800	0.01	--	0.10	60 (elev. 89.0) to zero (elev. 70.0)
85.0 to 78.0	Stiff Clay with Free Water (Reese)	*58	--	1,500	0.007	500	0.13	
78.0 to 70.0	Sand (Reese)	*68	35	--	--	60	0.26	
70.0 to 30.0	Sand (Reese)	*78	40	--	--	125	--	--
30.0 to -12.0	Weak Rock (Reese)	*78	Young's Modulus (E_r) = 29,000 psi, Uniaxial Compressive Strength = 500 psi, RQD (%) = 50, k_{rm} (stiffness coefficient) = 0.00005				--	--

*Buoyant Unit Weight

**LPILE 5.0 does not report the k value for Clay Without Free Water soil type. The k value is internally computed using the cohesion and E_{50} input parameters.

*** Use reduced p-multiplier to generate p-y curves for Liquefied Soil: P-Multiplier Method, page 10 of Caltrans "Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading" (October 2013).

10 PRELIMINARY CONSTRUCTION CONSIDERATIONS**10.1 CIDH Retrofit Piling (Abutments)**

Construction of CIDH piles will penetrate loose/soft soils below ground water and surface water, and require special installation measures, including temporary casing and slurry drilling methods with gamma-gamma inspection tubes.

10.2 CISS Retrofit Piling (Piers)

Difficult pile installation is anticipated due to the presence of dense to very dense sand and gravel layers (and possibly soft to moderately hard sandstone) above anticipated pile tip elevations. Interior relief drilling to assist pile driving will likely be necessary to achieve the pile tip elevations.

PRELIMINARY FOUNDATION REPORT

*Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California*

BCI File No. 1615.1

June 27, 2014

11 LIMITATIONS

This Preliminary Foundation Report should not be used for final design. Once the proposed retrofit strategy has been approved, BCI will prepare a Seismic Retrofit Foundation Report that will include additional analysis, conclusions and recommendations for final design.

BCI performed these services in accordance with generally accepted geotechnical engineering principles and practices currently used in this area. Do not use or rely on this report for different locations or improvements without the written consent of BCI.

FIGURES

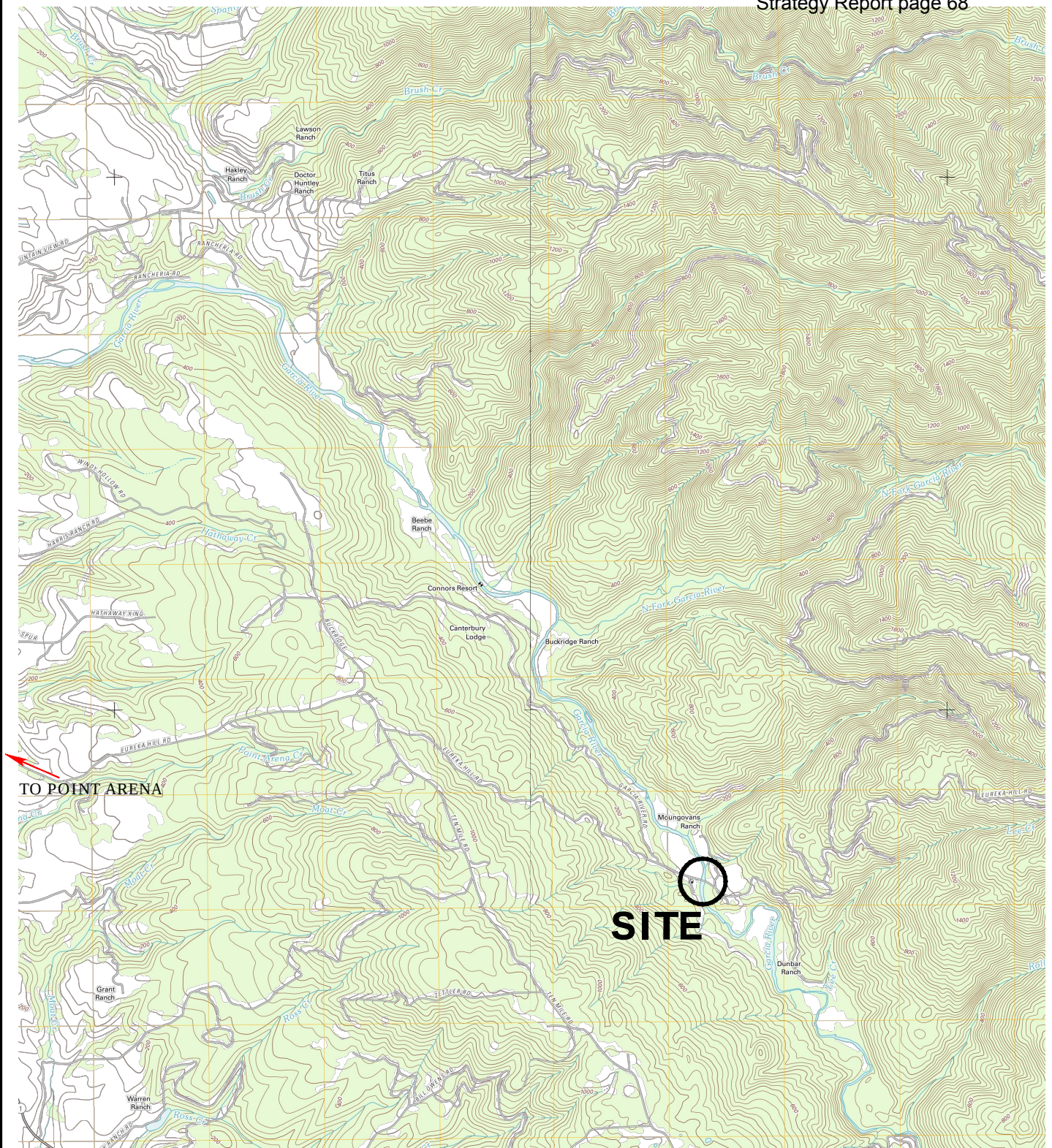
Figure 1 – Vicinity Map

Figure 2 – Regional Geologic Map

Figure 3 – Earthquake Fault Zone Map

Figure 4 – Preliminary Combined Design Response Spectrum





SOURCE: USGS 7.5 Minute Quadrangle, topographic.
Point Arena and Eureka Hill, 1:24000 dated 2012.



SCALE 1:48,000

VICINITY MAP

Garcia River Bridge (Retrofit)
Bridge No. 10C-0034
Mendocino County, California

File No. 1615.1

June 2014

Figure 1



2491 Boatman Avenue
West Sacramento, CA 95691
Phone: (916) 375-8706
Fax: (916) 375-8709
www.blackburnconsulting.com



LEGEND



Gallaway - Skooner Gulch Formations (Marine sandstone and mudstone)



German Rancho Formation (Marine sandstone and mudstone)



Coastal Belt Franciscan (Marine sandstone, shale, and conglomerate)



NO SCALE

Contact

Observed or approximately located; queried where gradational or inferred.

SOURCE: Geologic Map of the Santa Rosa Quadrangle, California
Geological Survey, Regional Geologic Map No. 2A, 1:250,000
scale. Compilation by; D.L. Wagner and E. J. Bortugno, 1982.



2491 Boatman Avenue
West Sacramento, CA 95691
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Fax: (916) 375-8709
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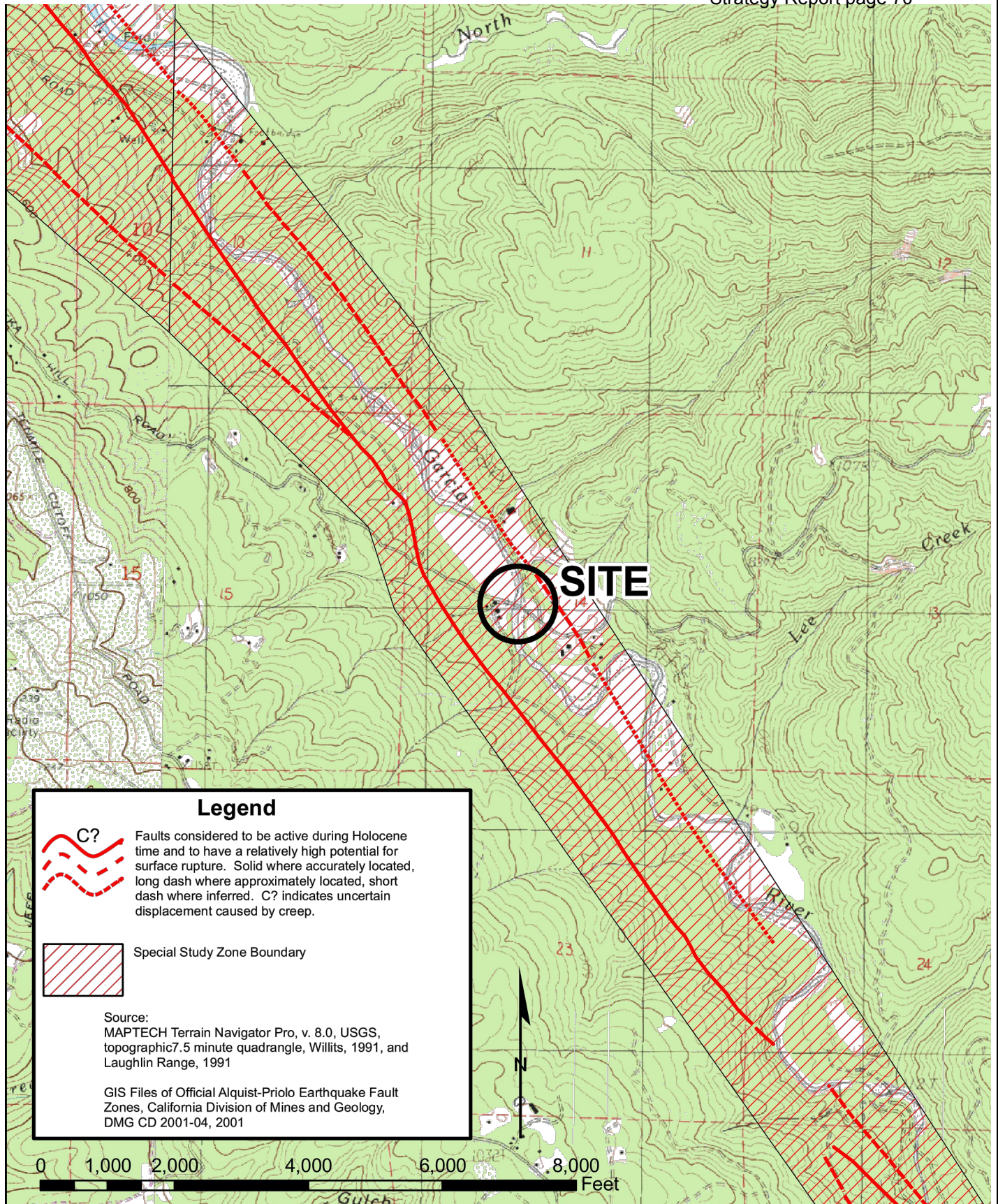
REGIONAL GEOLOGIC MAP

Garcia River Bridge (Retrofit)
Bridge No. 10C-0034
Mendocino County, California

File No. 1615.1

June 2014

Figure 2



EARTHQUAKE FAULT ZONE MAP

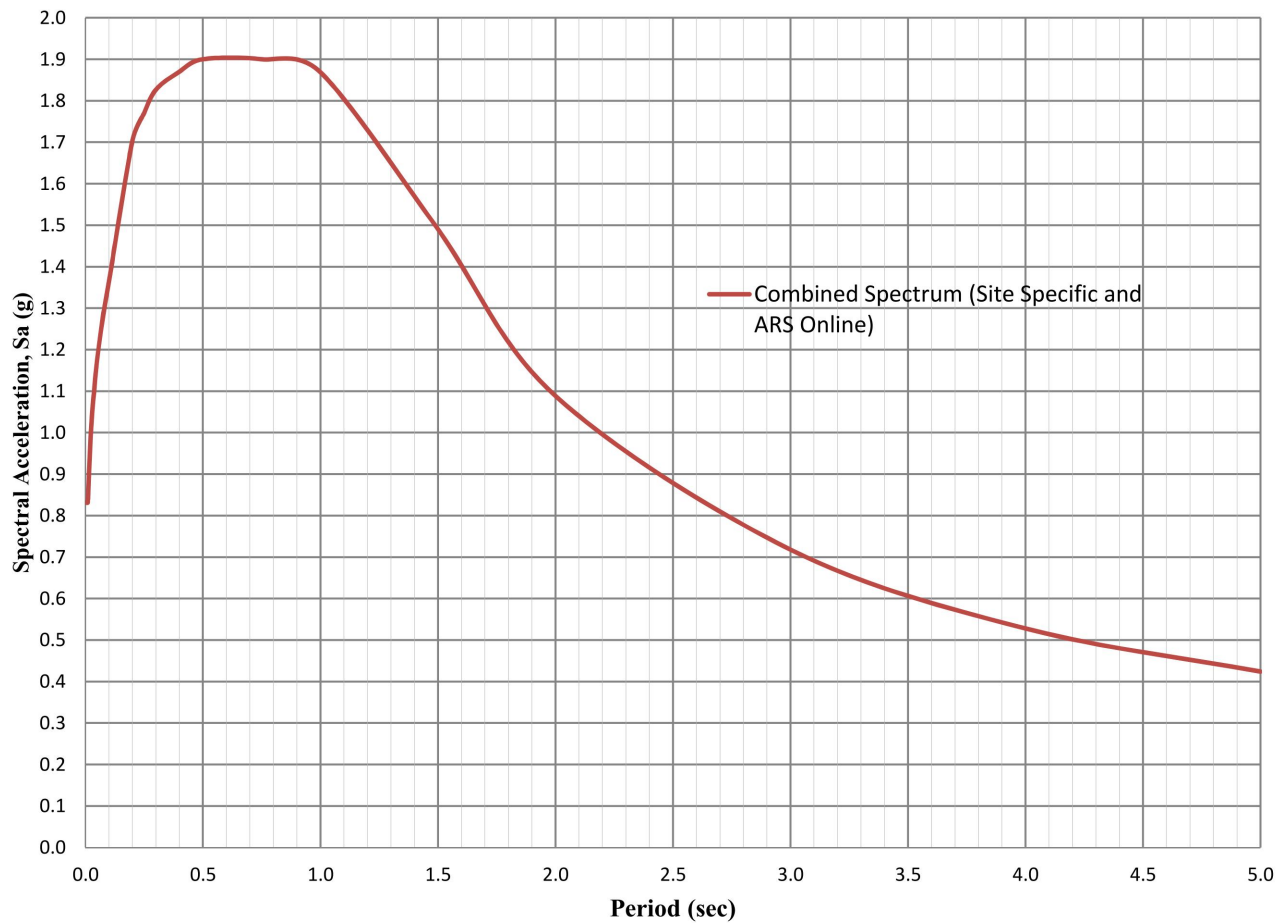
Garcia River Bridge (Retrofit)
Bridge No. 10C-0034
Mendocino County, California

File No. 1615.1

June 2014

Figure 3

Preliminary Combined Design Spectrum (5% Damping)
Eureka Hill Road Bridge over Garcia River



APPENDIX A

Draft Log of Test Borings (2011, Blackburn Consulting)

Log of Test Borings (1997, Taber Consultants)

Log of Test Borings (1968, Clair A. Hill & Associates)



DIST	COUNTY	ROUTE	POST MILES	SHEET	TOTAL
01	MEN		TOTAL PROJECT	No	SHEETS
REGISTERED ENGINEER			DATE		
PLANS APPROVAL DATE			CIVIL		
The County of Mendocino or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.					
MENDOCINO COUNTY DEPARTMENT OF TRANSPORTATION 340 LAKE MENDOCINO DRIVE UKIAH, CA 95482					
BLACKBURN CONSULTING 2491 BOATMAN AVENUE WEST SACRAMENTO, CA 95691					
FILE No. 1615.1					

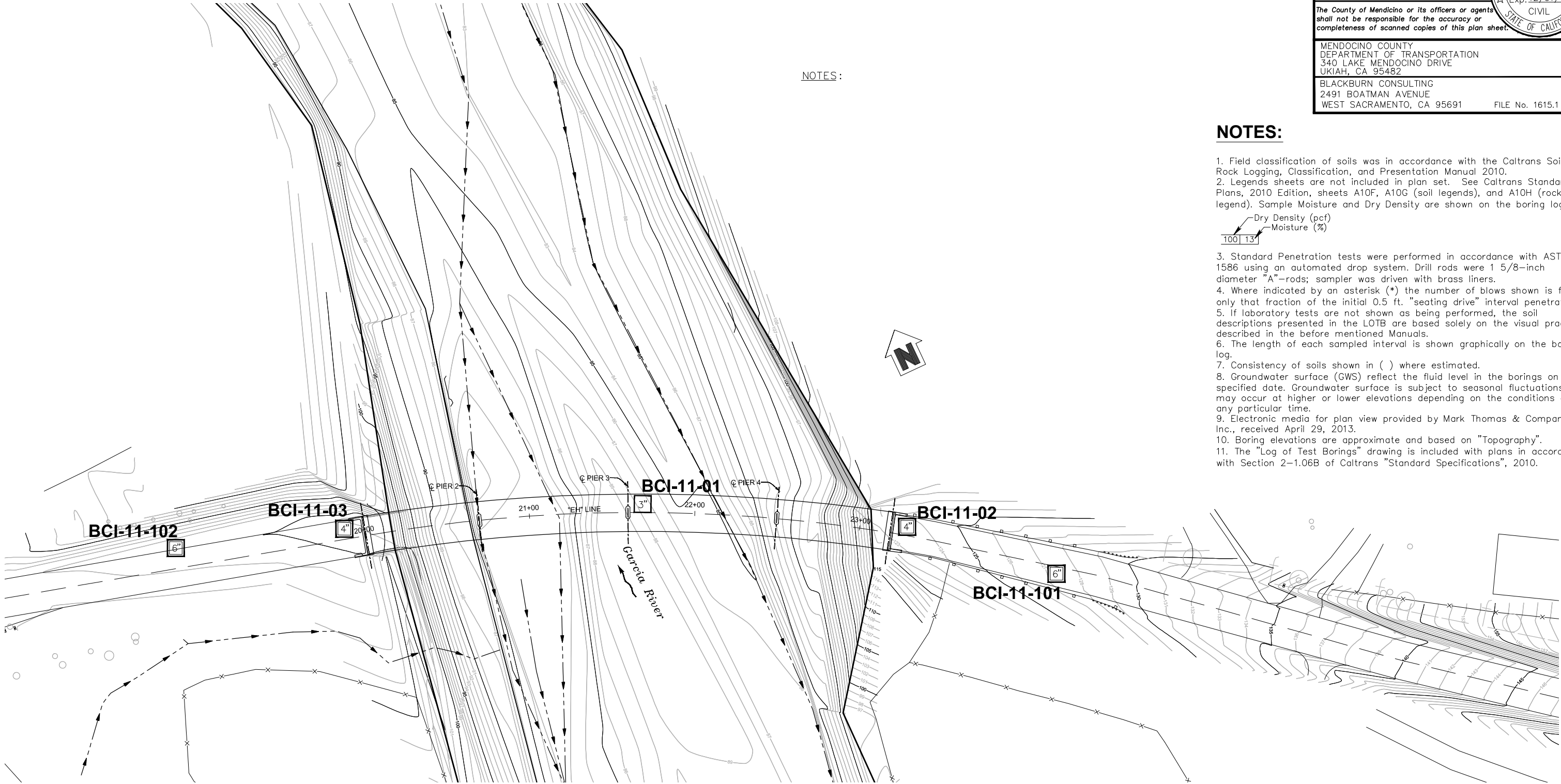
BENCHMARKS
BENCHMARK# MH-230 ELEV. XX.XX NAVD XX
DESCRIPTION: X, NORTHING XX, EASTING XX.

NOTES:

NOTES:

- Field classification of soils was in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual 2010.
- Legends sheets are not included in plan set. See Caltrans Standard Plans, 2010 Edition, sheets A10F, A10G (soil legends), and A10H (rock legend). Sample Moisture and Dry Density are shown on the boring logs as;
- Standard Penetration tests were performed in accordance with ASTM D 1586 using an automated drop system. Drill rods were 1 5/8-inch diameter "A"-rods; sampler was driven with brass liners.
- Where indicated by an asterisk (*) the number of blows shown is for only that fraction of the initial 0.5 ft. "seating drive" interval penetrated.
- If laboratory tests are not shown as being performed, the soil descriptions presented in the LOTB are based solely on the visual practices described in the before mentioned Manuals.
- The length of each sampled interval is shown graphically on the boring log.
- Consistency of soils shown in () where estimated.
- Groundwater surface (GWS) reflect the fluid level in the borings on the specified date. Groundwater surface is subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
- Electronic media for plan view provided by Mark Thomas & Company, Inc., received April 29, 2013.
- Boring elevations are approximate and based on "Topography".
- The "Log of Test Borings" drawing is included with plans in accordance with Section 2-1.06B of Caltrans "Standard Specifications", 2010.

Dry Density (pcf)
Moisture (%)



DRAFT
PLAN
1" = 30'

DESIGN OVERSIGHT	DRAWN BY M. ROBERTSON	R. C. PICKARD FIELD INVESTIGATION BY:	PREPARED FOR THE COUNTY OF MENDOCINO DEPARTMENT OF PUBLIC WORKS	MICHAEL BIANUCCI PROJECT ENGINEER	BRIDGE NO. 10C-0034	GARCIA RIVER BRIDGE (RETROFIT)		
SIGN OFF DATE	CHECKED BY D. MORRELL	DATE: OCTOBER 2011			POST MILE XX.X	LOG OF TEST BORINGS 1 OF 2		
GS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)				UNIT: PROJECT NUMBER & PHASE: C0205	CONTRACT NO.: X	DISREGARD PRINTS BEARING EARLIER REVISION DATES		
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS				0 1 2 3	FILE => \$REQUEST	REVISION DATES 06/19/14		
						SHEET OF		

6/23/2014 1615.1 Garcia River Bridge LOTB.dwg

TIME PLOTTED => \$DATE
USERNAME => \$USER

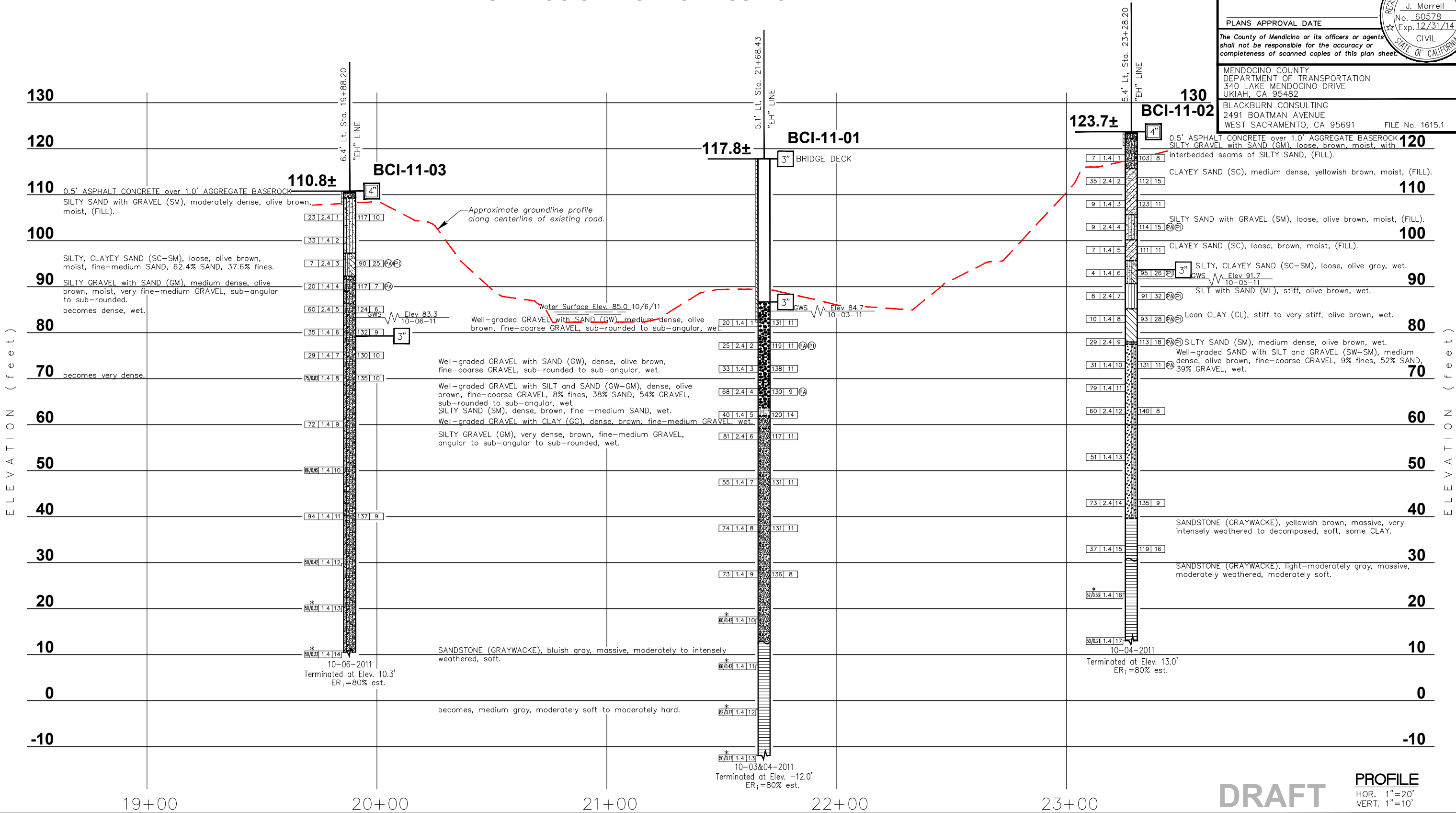
6/23/2014 1615.1 Garcia River Bridge LOTB.dwg

FOR PLAN VIEW AND BORING LOG NOTES
SEE LOG OF TEST BORINGS 1 OF 2

DIST	COUNTY	ROUTE	POST MILES	SHEET	TOTAL
01	MEN		Strategy Report page 74	No	SHEETS

REGISTERED ENGINEER	DATE
David J. Morrell No. 60578 Exp. 12/31/14 CIVIL STATE OF CALIFORNIA	
PLANS APPROVAL DATE	
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MENDOCINO COUNTY
DEPARTMENT OF TRANSPORTATION
340 LAKE MENDOCINO DRIVE
UKIAH, CA 95482
BLACKBURN CONSULTING
2491 BOATMAN AVENUE
WEST SACRAMENTO, CA 95691
FILE No. 1615.1



DRAFT
PROFILE
HOR. 1"=20'
VERT. 1"=10'

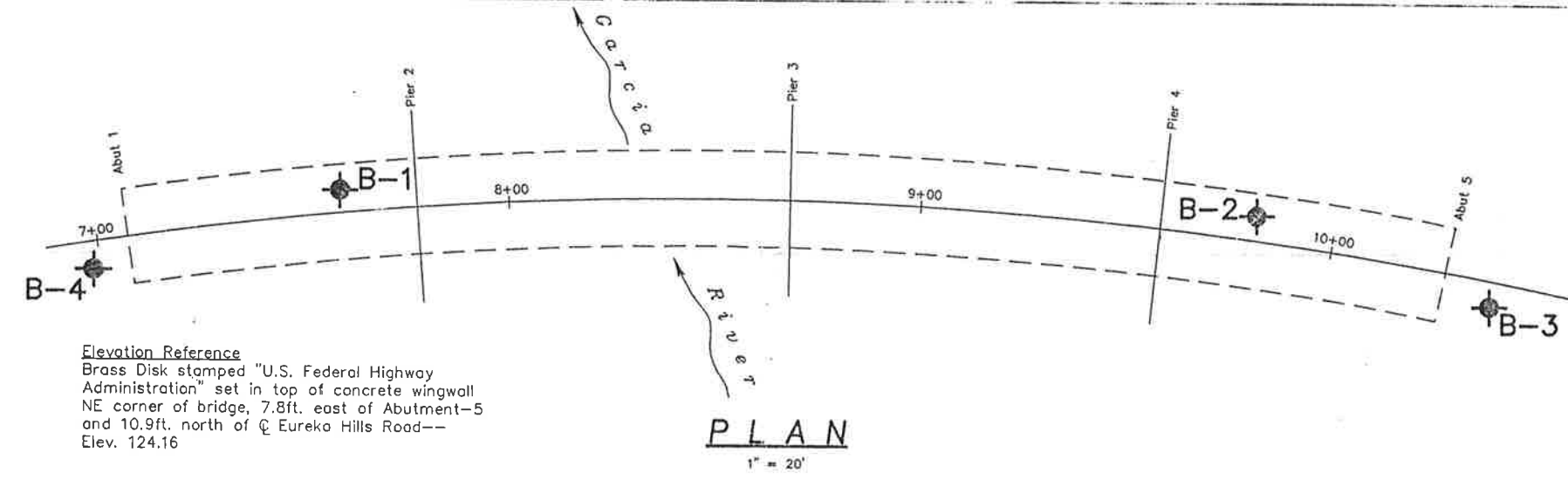
DESIGN OVERSIGHT	DRAWN BY M. ROBERTSON	R. C. PICKARD FIELD INVESTIGATION BY: DATE: OCTOBER 2011	PREPARED FOR THE COUNTY OF MENDOCINO DEPARTMENT OF PUBLIC WORKS	MICHAEL BIANUCCI PROJECT ENGINEER	BRIDGE NO. 10C-0034 POST MILE XX.X	GARCIA RIVER BRIDGE (RETROFIT)	
SIGN OFF DATE	CHECKED BY D. MORRELL					LOG OF TEST BORINGS 2 OF 2	
GS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)				UNIT: PROJECT NUMBER & PHASE: C0205		CONTRACT NO.: X	
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS				DISREGARD PRINTS BEARING EARLIER REVISION DATES		REVISION DATES 06/19/14	

TIME PLOTTED => \$DATE
USERNAME => \$USER

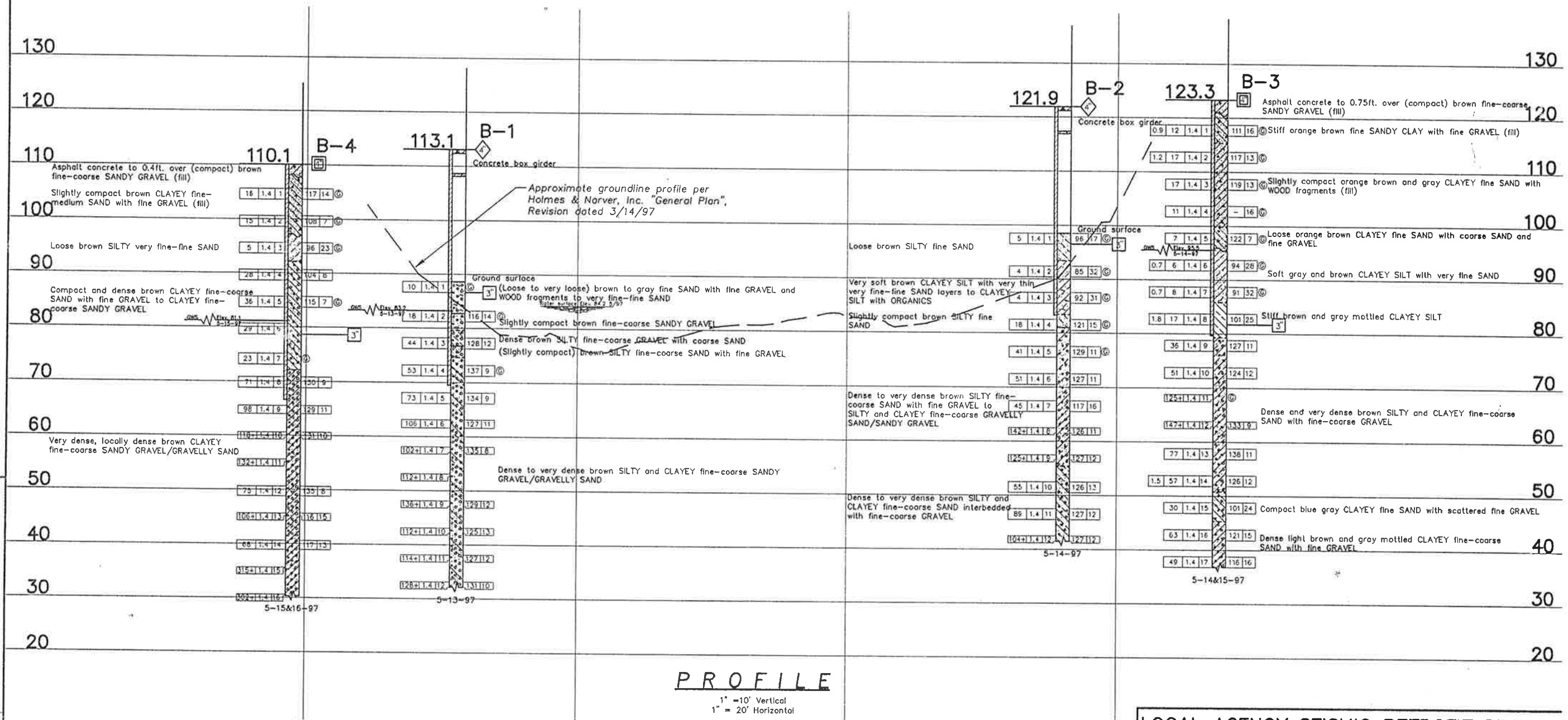
REGISTERED GEOTECHNICAL ENGINEER
FRANKLIN P. TABER
No. 816
EXP. 3-31-00
PLANS APPROVAL DATE

TABER CONSULTANTS
536 GALVESTON STREET
WEST SACRAMENTO, CA 95691
JOB No. 1R2/398/63-2 (Task)
LOCATION: 38123-H5:104N:30:

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.



Elevation Reference
Brass Disk stamped "U.S. Federal Highway Administration" set in top of concrete wingwall NE corner of bridge, 7.8ft. east of Abutment-5 and 10.9ft. north of Eureka Hills Road-- Elev. 124.16



LEGEND OF BORING OPERATIONS
PLAN OF ANY BORING
2 1/2" CONE PENETROMETER
SAMPLE BORING (DRY)
ROTARY SAMPLE BORING (WET)
AUGER BORING (DRY)
TEST PIT
DIAMOND CORE BORING
LET BORING
ELECTRONIC CONE PENETROMETER
1" CONE PENETROMETER
Retraction marker line

LEGEND OF EARTH MATERIALS
GRAVEL
SAND
SILT
CLAY
SANDY CLAY or CLAYEY SAND
SANDY SILT or SILTY SAND
SILT CLAY

CONSISTENCY CLASSIFICATION FOR SOILS
According to the Standard Penetration Test
Cohesive
Very loose
Loose
Slightly compact
Compact
Very dense
Very stiff
Very hard

NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.

DESIGN OVERSIGHT	DRAWN BY M.D.R. 6/97	W. E. Nichols FIELD INVESTIGATOR
SIGN OFF DATE	CHECKED BY H.C.V. 7/97	DATE May 1997

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

PROJECT ENGINEER	BRIDGE NO. 10C-0034
	POST MILE

LOCAL AGENCY SEISMIC RETROFIT PROJECT
Garcia River Bridge @ Eureka Hills Road
LOG OF TEST BORINGS (1 of 1)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS

DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES (PRELIMINARY STAGE ONLY)
SHEET 40

LOCAL AGENCY SEISMIC RETROFIT PROJECT

DESIGN OVERSIGHT	DRAWN BY	FIELD INVESTIGATOR	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	PROJECT ENGINEER	BRIDGE NO. 10C-0034	Garcia River Bridge @ Eureka Hills Road LOG OF TEST BORINGS (2 of 3)
SIGN OFF DATE	CHECKED BY	DATE			POST MILE	
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS			0 1 2 3	CU EA	DISREGARD PRINTS BEARING EARLIER REVISION DATES	REVISION DATES (PRELIMINARY STAGE ONLY) SHEET OF



LEGEND OF DRILLING, SAMPLING & TESTING OPERATIONS

ROTARY BORING

PENETRATION BORING

THE UNIFIED SOIL CLASSIFICATION SYSTEM

Major	Minor	Symbol	Name
COARSE GRAINED	GW	Well-sorted gravel or gravel-sand	
	GP	POORLY-SORTED GRAVEL OR SAND	
	GM	Silty gravel, gravel-sand-silt	
	GC	CLAYEY GRAVEL, GRAVEL-SAND-CLAY	
FINE GRAINED	SW	Well-sorted sand or sand-silt	
	SP	POORLY-SORTED SAND OR SAND-SILT	
	SM	Silty sand, sand-silt	
	SC	CLAYEY SAND, SAND-SILT	

ROCK CLASSIFICATION

Symbol	Name
MR	METAMORPHIC ROCK
SR	SEDIMENTARY ROCK
IR	IGNEOUS ROCK

SOIL CONSISTENCY CLASSIFICATION

Consistency	Symbol	Blows per foot
VERY LOOSE	0 TO 5	0 TO 5
LOOSE	5 TO 10	5 TO 10
MEDIUM DENSITY	10 TO 20	10 TO 20
DENSE	20 TO 30	20 TO 30
VERY DENSE	30 TO 40	30 TO 40

LOG OF TEST BORINGS

GARCIA RIVER BRIDGE

POINT ARENA AIR FORCE STATION ACCESS ROAD

CALIF. R-AD PROJ. 30 (2)

CLARK A. HILL & ASSOCIATES

FOUNDATION ENGINEERING & TESTING LABORATORY

1500 COUNTY STREET, REDDING, CALIFORNIA

Drawn by: *RG*

Scale: *1" = 10'*

Sheet: *8 of 8*

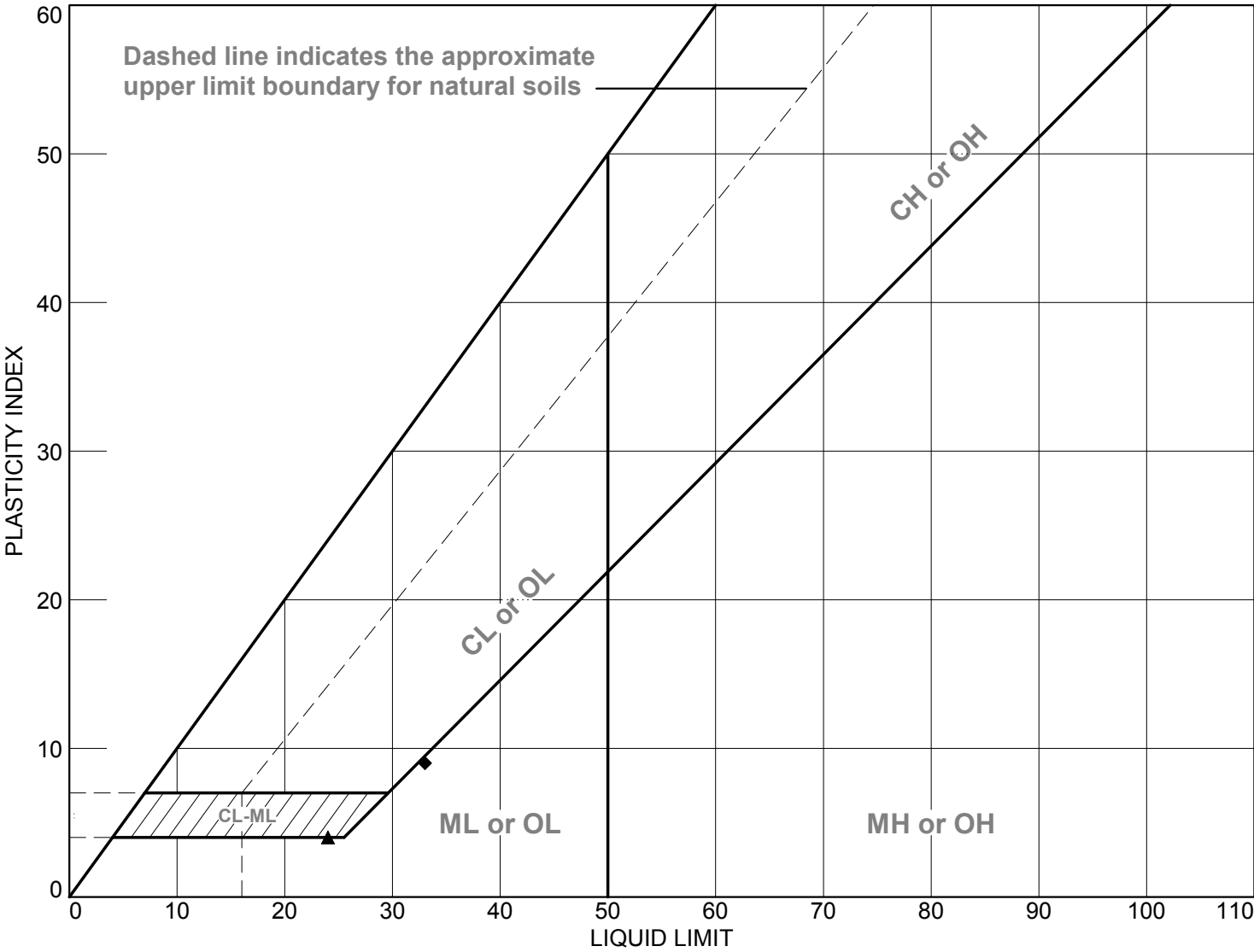
LOCAL AGENCY SEISMIC RETROFIT PROJECT
Garcia River Bridge @ Eureka Hills Road
LOG OF TEST BORINGS (3 of 3)

APPENDIX B

Laboratory Test Results



LIQUID AND PLASTIC LIMITS TEST REPORT



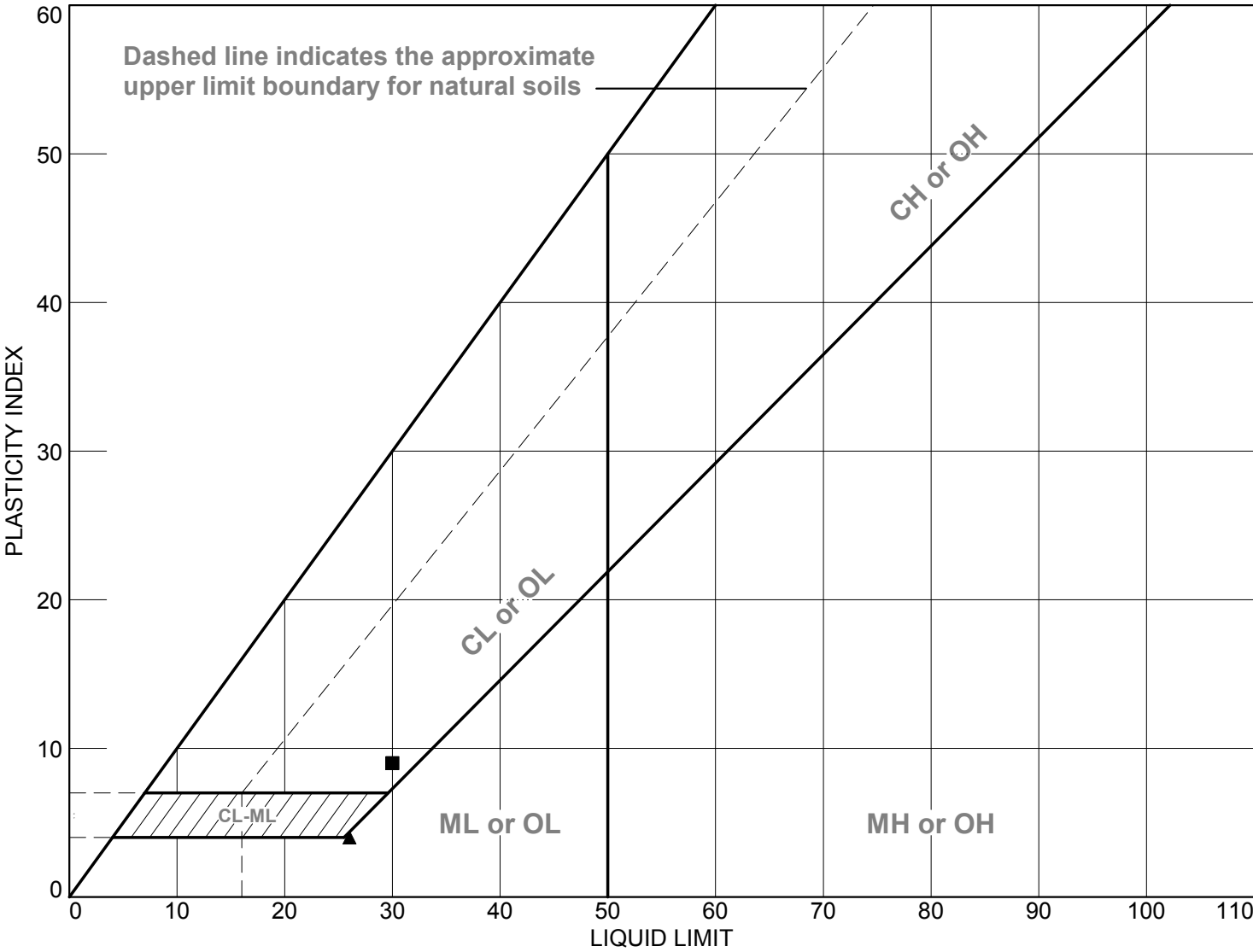
SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BCI 11-01 #1 #2 & #3	3.8'-15.3'		NP	NP	NP	GW
■		BCI 11-02 #4	20.5'-21.5'		NP	NP	NP	SM
▲		BCI 11-02 #6	31.00'-31.5'		20	24	4	CL-ML
◆		BCI 11-02 #7	35.5'-36.0'		24	33	9	ML

Blackburn Consulting
W. Sacramento, CA

Client: Mark Thomas Company
Project: Garcia River Bridge Retrofit
Project No.: 1615.X

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BCI 11-02 #9	45.5'-46.0'		NP	NP	NP	SM
■		BCI 11-02 #8	40.5'-41.0'		21	30	9	CL
▲		BCI 11-03 #3	15.5'-16.0'		22	26	4	SC-SM

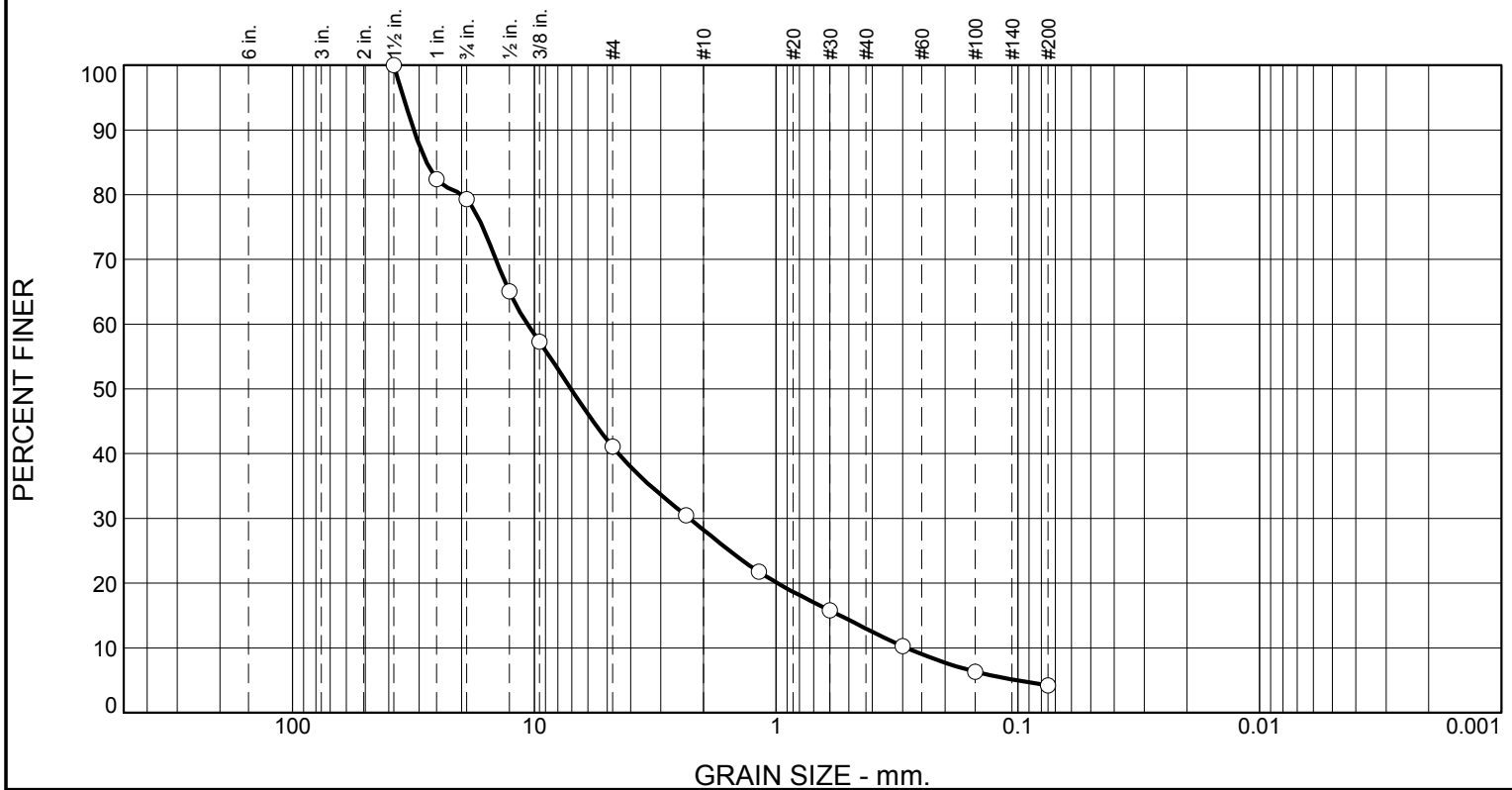
Blackburn Consulting
W. Sacramento, CA

Client: Mark Thomas Company
Project: Garcia River Bridge Retrofit
Project No.: 1615.X

Figure

Particle Size Distribution Report

Category Report page 81



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	20.7	38.2	12.8	15.3	8.8	4.2	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1.0"	82.4		
3/4"	79.3		
1/2"	65.1		
3/8"	57.3		
#4	41.1		
#8	30.5		
#16	21.8		
#30	15.8		
#50	10.3		
#100	6.3		
#200	4.2		

* (no specification provided)

Material Description Olive Brown Well Graded GRAVEL with Sand	
Atterberg Limits (ASTM D 4318) PL= NP LL= NP PI= NP	
Classification USCS (D 2487)= GW AASHTO (M 145)= A-1-a	
Coefficients D ₉₀ = 31.5305 D ₈₅ = 27.9591 D ₆₀ = 10.6696 D ₅₀ = 7.0579 D ₃₀ = 2.2796 D ₁₅ = 0.5435 D ₁₀ = 0.2886 C _u = 36.96 C _c = 1.69	
Remarks	
Date Received: _____ Tested By: KLC _____ Checked By: KLC _____ Title: _____	Date Tested: _____

Sample Number: BCI 11-01 #1 #2 & #3

Depth: 3.8'-15.3'

Date Sampled: 11-21-2011

Blackburn Consulting

W. Sacramento, CA

Client: Mark Thomas Company

Project: Garcia River Bridge Retrofit

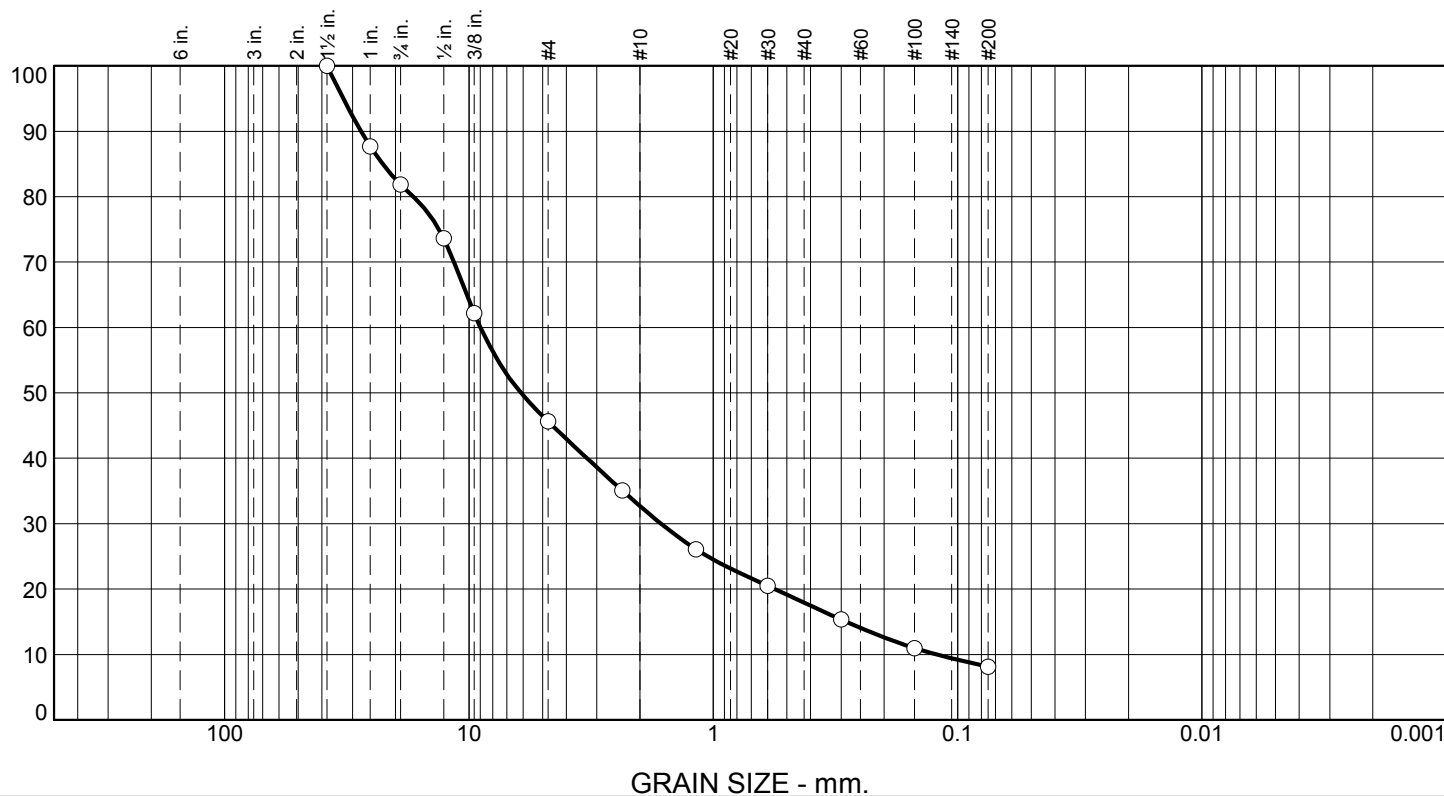
Project No: 1615.X

Figure

Particle Size Distribution Report

Category Report page 82

PERCENT FINER



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	18.1	36.3	12.9	14.8	9.8	8.1	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1.0"	87.7		
3/4"	81.9		
1/2"	73.6		
3/8"	62.2		
#4	45.6		
#8	35.1		
#16	26.1		
#30	20.5		
#50	15.4		
#100	11.0		
#200	8.1		

* (no specification provided)

Material Description

Olive Brown Well Graded GRAVEL with Silt and Sand

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= GW-GM AASHTO (M 145)=

Coefficients

D₉₀= 27.6987 D₈₅= 22.5925 D₆₀= 8.9814
D₅₀= 6.1329 D₃₀= 1.6380 D₁₅= 0.2854
D₁₀= 0.1218 C_u= 73.73 C_c= 2.45

Remarks

Date Received:

Date Tested:

Tested By: KLC

Checked By: KLC

Title:

Sample Number: BCI 11-01 #4

Depth: 19.8'-20.3'

Date Sampled: 11-21-2011

Blackburn Consulting

W. Sacramento, CA

Client: Mark Thomas Company

Project: Garcia River Bridge Retrofit

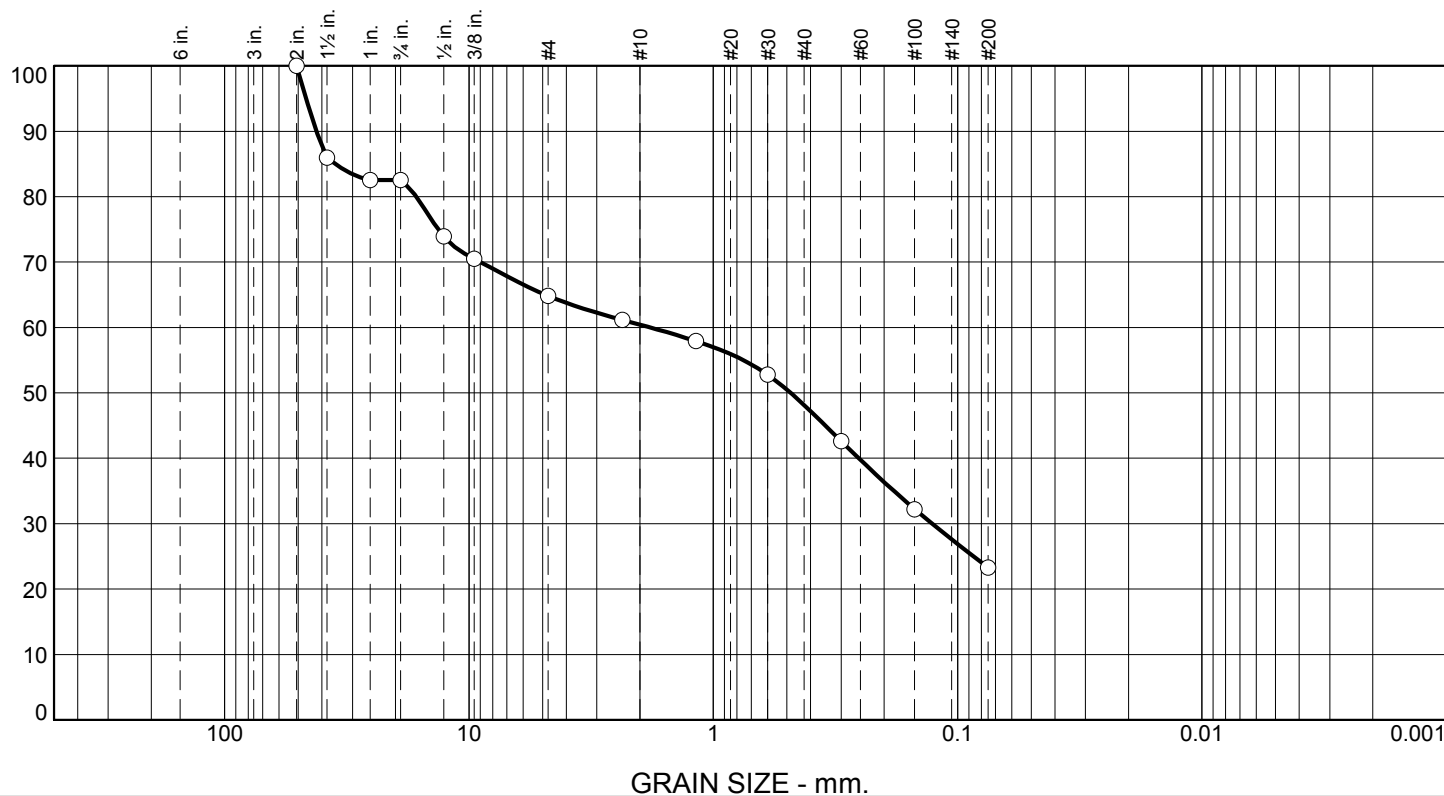
Project No: 1615.X

Figure

Particle Size Distribution Report

Category Report page 83

PERCENT FINER



GRAIN SIZE - mm.

% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	17.5	17.7	4.4	12.3	24.8	23.3	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2.0"	100.0		
1.5	86.0		
1.0"	82.5		
3/4"	82.5		
1/2"	73.9		
3/8"	70.5		
#4	64.8		
#8	61.2		
#16	57.9		
#30	52.8		
#50	42.6		
#100	32.2		
#200	23.3		

* (no specification provided)

Material Description

Olive Brown Silty SAND with Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NP PI= NP

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 42.1453 D₈₅= 35.2711 D₆₀= 1.8149
D₅₀= 0.4830 D₃₀= 0.1273 D₁₅=
D₁₀= C_u= C_c=

Remarks

BCI 11-02 #4C @ 21.0'-21.5'

Date Received:

Date Tested:

Tested By: KLC

Checked By: KLC

Title:

Sample Number: BCI 11-02 #4

Depth: 20.5'-21.5'

Date Sampled: 11-21-2011

Blackburn Consulting

W. Sacramento, CA

Client: Mark Thomas Company

Project: Garcia River Bridge Retrofit

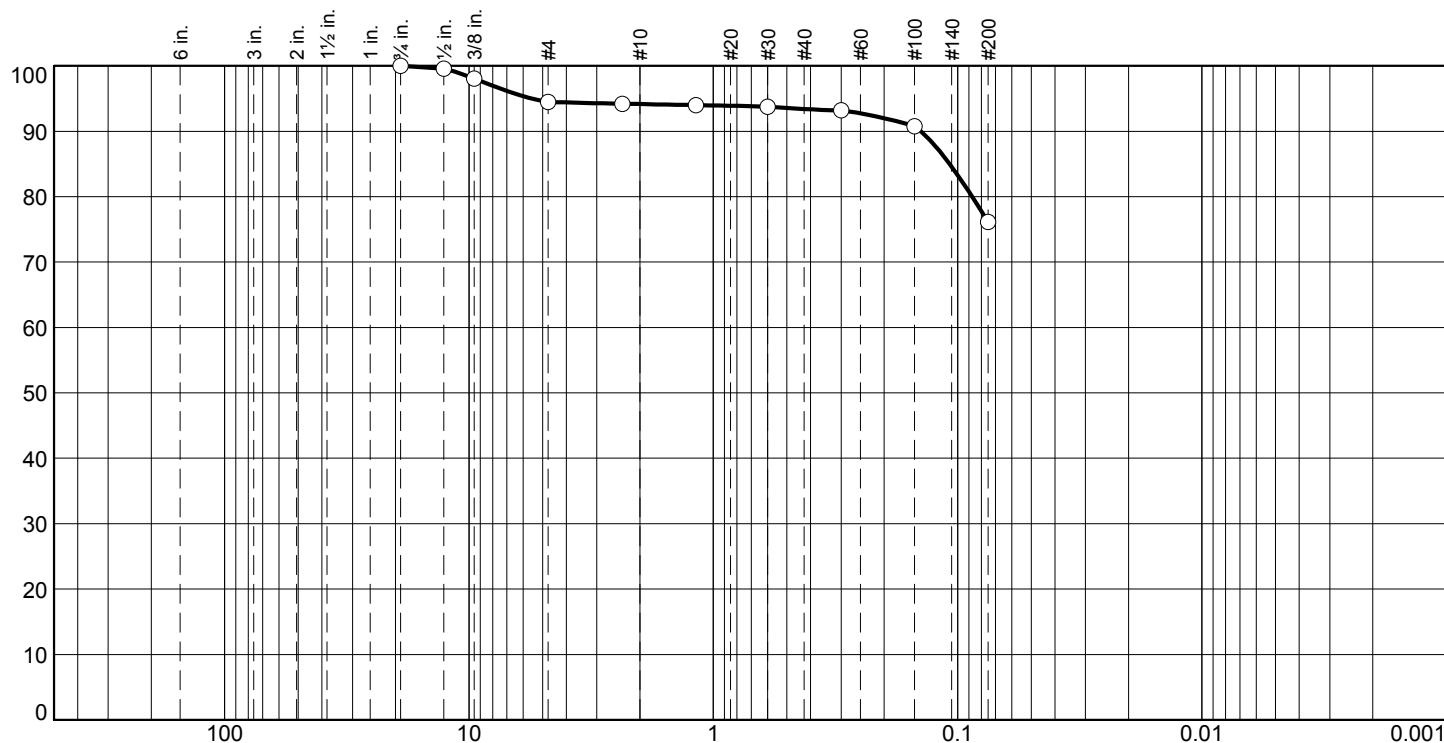
Project No: 1615.X

Figure

Particle Size Distribution Report

Category Report page 84

PERCENT FINER



GRAIN SIZE - mm.

% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	5.5	0.3	0.8	17.3	76.1	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/4"	100.0		
1/2"	99.6		
3/8"	98.0		
#4	94.5		
#8	94.2		
#16	94.0		
#30	93.7		
#50	93.2		
#100	90.8		
#200	76.1		

* (no specification provided)

Material Description

SILT with SAND, olive brown

Atterberg Limits (ASTM D 4318)

PL= 24 LL= 33 PI= 9

Classification

USCS (D 2487)= CL AASHTO (M 145)= A-4(6)

Coefficients

D₉₀= 0.1415 D₈₅= 0.1079 D₆₀=
D₅₀= D₃₀= D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: _____ Date Tested: _____
Tested By: KLC
Checked By: KLC
Title: _____

Sample Number: BCI 11-02 #7

Depth: 35.5'-36.0'

Date Sampled: 11-21-2011

Blackburn Consulting

W. Sacramento, CA

Client: Mark Thomas Company

Project: Garcia River Bridge Retrofit

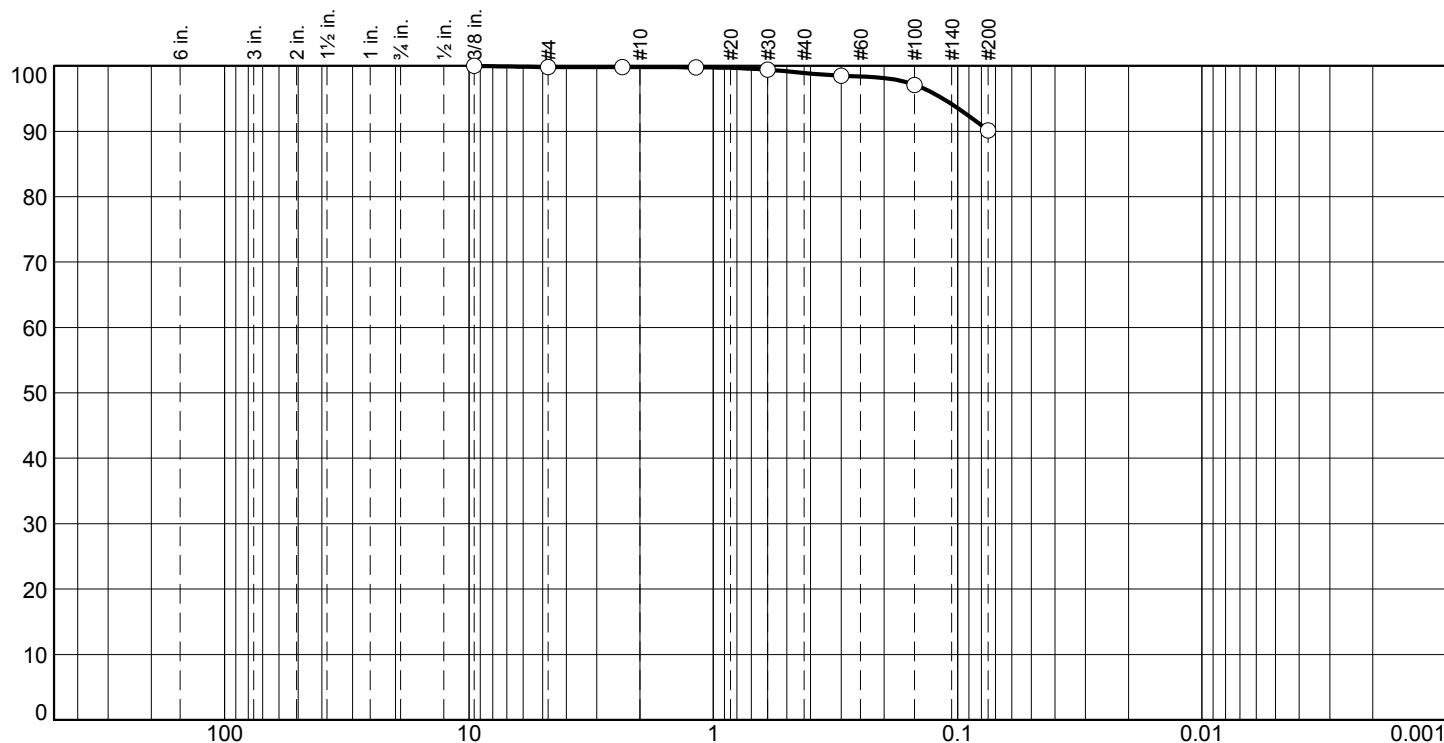
Project No: 1615.X

Figure

Particle Size Distribution Report

Category Report page 85

PERCENT FINER



GRAIN SIZE - mm.

% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.2	0.0	0.9	8.8	90.1	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/8"	100.0		
#4	99.8		
#8	99.8		
#16	99.8		
#30	99.4		
#50	98.5		
#100	97.1		
#200	90.1		

* (no specification provided)

Material Description

Brown Lean CLAY

Atterberg Limits (ASTM D 4318)

PL= 21 LL= 30 PI= 9

Classification

USCS (D 2487)= CL AASHTO (M 145)= A-4(8)

Coefficients

D₉₀= D₈₅= D₆₀=
D₅₀= D₃₀= D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received:

Date Tested:

Tested By: KLC

Checked By: KLC

Title:

Sample Number: BCI 11-02 #8

Depth: 40.5'-41.0'

Date Sampled: 11-21-2011

Blackburn Consulting

W. Sacramento, CA

Client: Mark Thomas Company

Project: Garcia River Bridge Retrofit

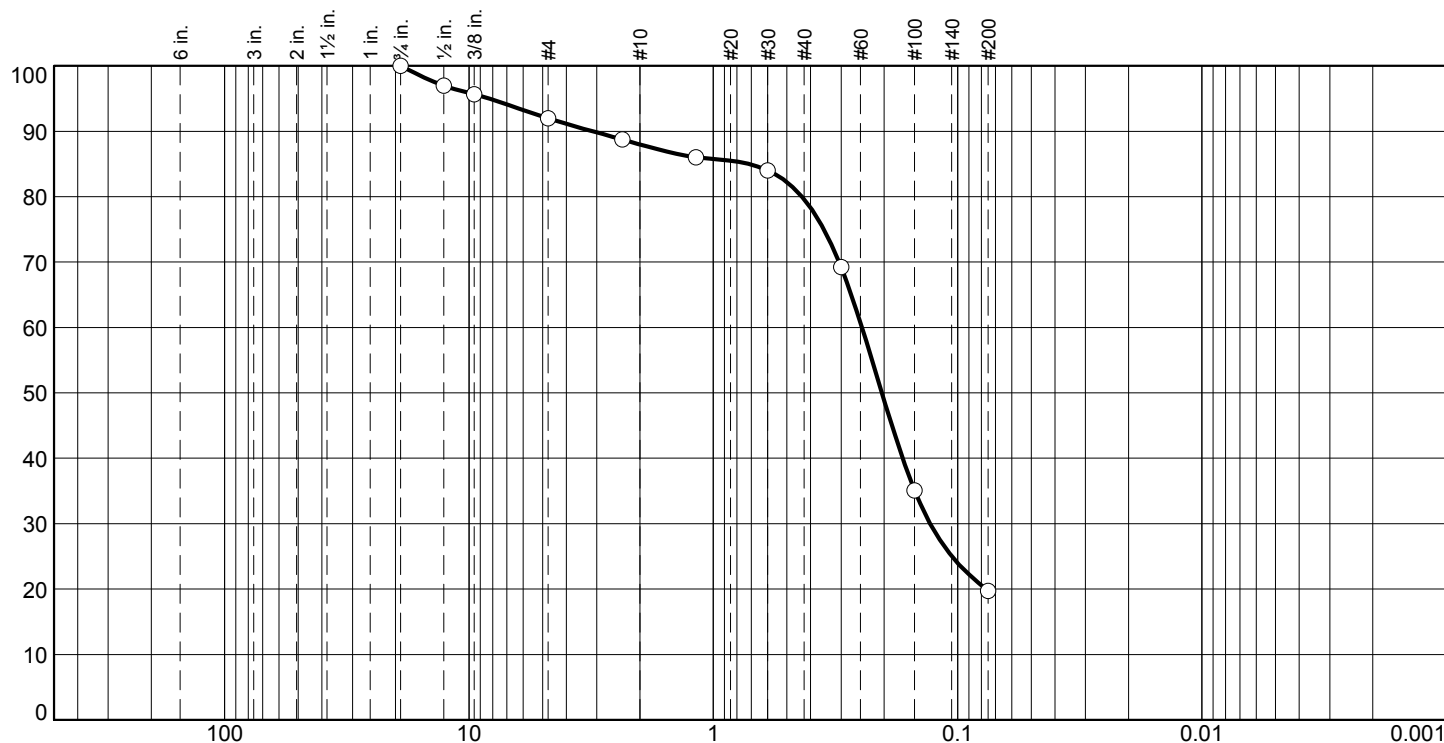
Project No: 1615.X

Figure

Particle Size Distribution Report

Category Report page 86

PERCENT FINER



GRAIN SIZE - mm.

% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	8.0	4.0	8.4	59.9	19.7	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/4"	100.0		
1/2"	97.0		
3/8"	95.7		
#4	92.0		
#8	88.8		
#16	86.0		
#30	84.0		
#50	69.2		
#100	35.1		
#200	19.7		

* (no specification provided)

Material Description
Olive Brown Silty SAND

Atterberg Limits (ASTM D 4318)
PL= NP LL= NP PI= NP

Classification
USCS (D 2487)= SM AASHTO (M 145)= A-2-4(0)

Coefficients
D₉₀= 3.1225 D₈₅= 0.7161 D₆₀= 0.2465
D₅₀= 0.2042 D₃₀= 0.1297 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: Date Tested:
Tested By: KLC
Checked By: KLC
Title:

Sample Number: BCI 11-02 #9

Depth: 45.5'-46.0'

Date Sampled: 11-22-2011

Blackburn Consulting

W. Sacramento, CA

Client: Mark Thomas Company

Project: Garcia River Bridge Retrofit

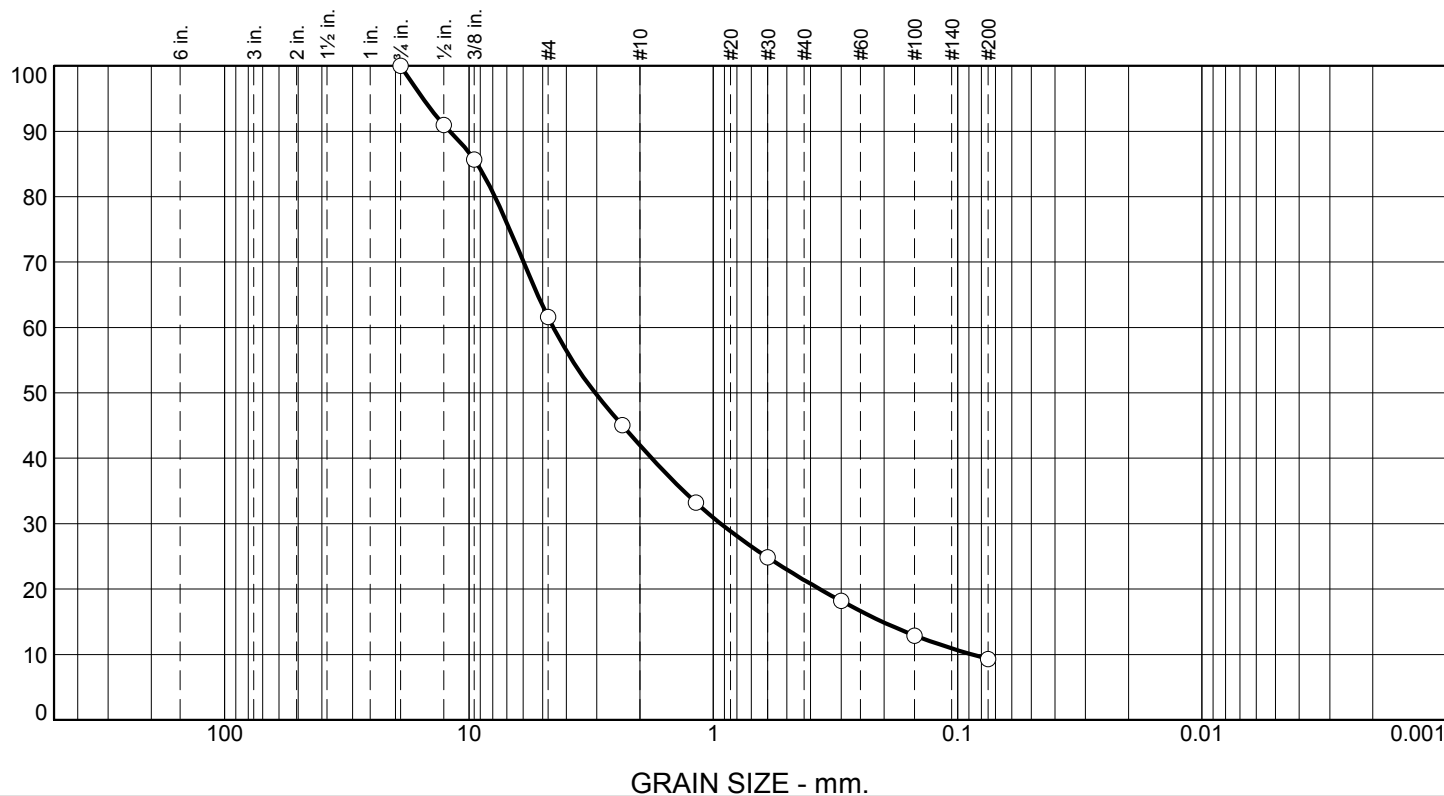
Project No: 1615.X

Figure

Particle Size Distribution Report

Category Report page 87

PERCENT FINER



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	38.4	19.6	20.6	12.1	9.3	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/4"	100.0		
1/2"	91.0		
3/8"	85.7		
#4	61.6		
#8	45.1		
#16	33.2		
#30	24.9		
#50	18.2		
#100	12.9		
#200	9.3		

* (no specification provided)

Material Description

Olive Brown Well Graded SAND with Silt and Gravel

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D₉₀= 12.0051 D₈₅= 9.2654 D₆₀= 4.5183
D₅₀= 3.0458 D₃₀= 0.9330 D₁₅= 0.2031
D₁₀= 0.0872 C_u= 51.80 C_c= 2.21

Remarks

Date Received:

Date Tested:

Tested By: KLC

Checked By: KLC

Title:

Sample Number: BCI 11-02 #10

Depth: 50.75'-51.25'

Date Sampled: 11-22-2011

Blackburn Consulting

W. Sacramento, CA

Client: Mark Thomas Company

Project: Garcia River Bridge Retrofit

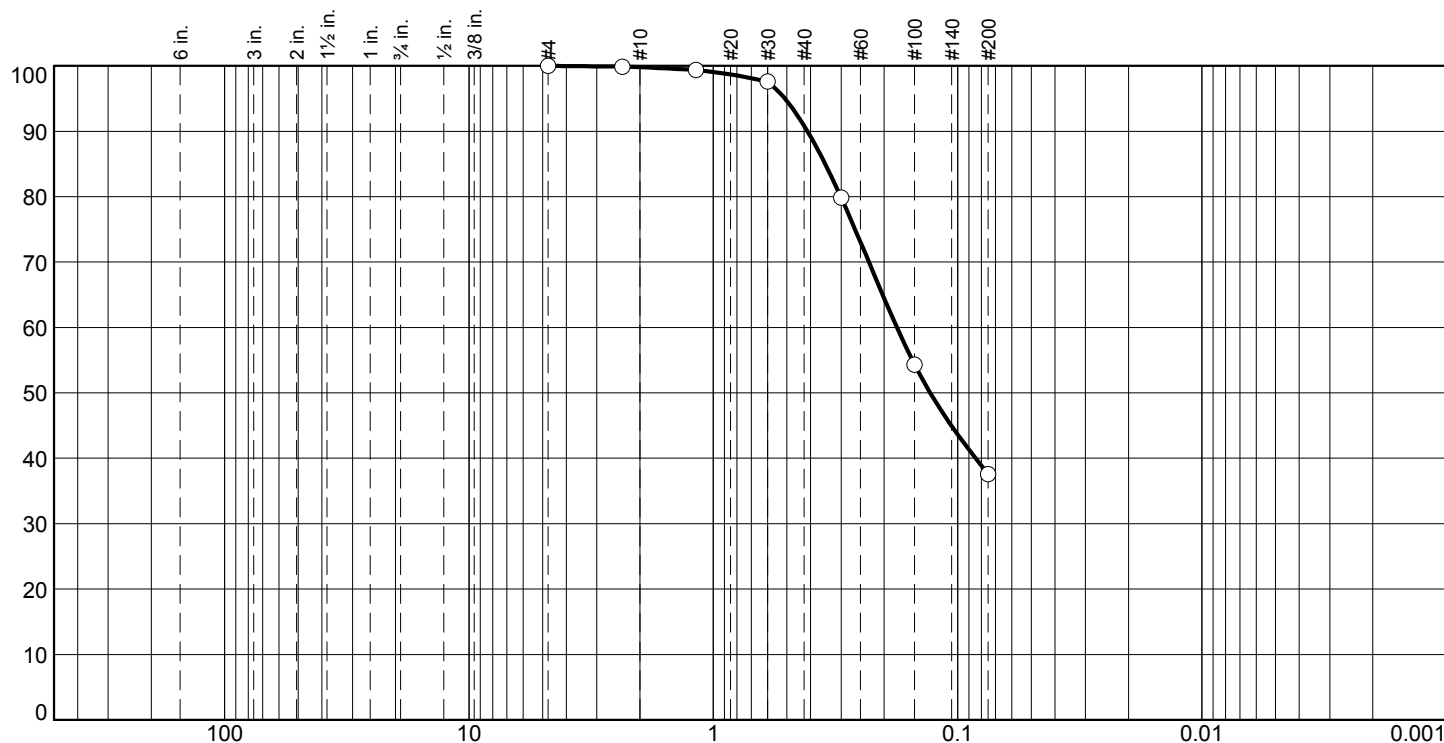
Project No: 1615.X

Figure

Particle Size Distribution Report

Category Report page 88

PERCENT FINER



GRAIN SIZE - mm.

% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.2	9.0	53.2	37.6	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#8	99.9		
#16	99.4		
#30	97.6		
#50	79.9		
#100	54.3		
#200	37.6		

* (no specification provided)

Material Description

SILTY CLAYEY SAND, olive brown

Atterberg Limits (ASTM D 4318)

PL= 22 LL= 26 PI= 4

Classification

USCS (D 2487)= SC-SM AASHTO (M 145)= A-4(0)

Coefficients

D₉₀= 0.4122 D₈₅= 0.3486 D₆₀= 0.1772
D₅₀= 0.1296 D₃₀= C_u= D₁₅= C_c=

Remarks

Date Received:

Date Tested:

Tested By: KLC

Checked By: KLC

Title:

Sample Number: BCI 11-03 #3

Depth: 15.5'-16.0'

Date Sampled: 11-22-2011

Blackburn Consulting

W. Sacramento, CA

Client: Mark Thomas Company

Project: Garcia River Bridge Retrofit

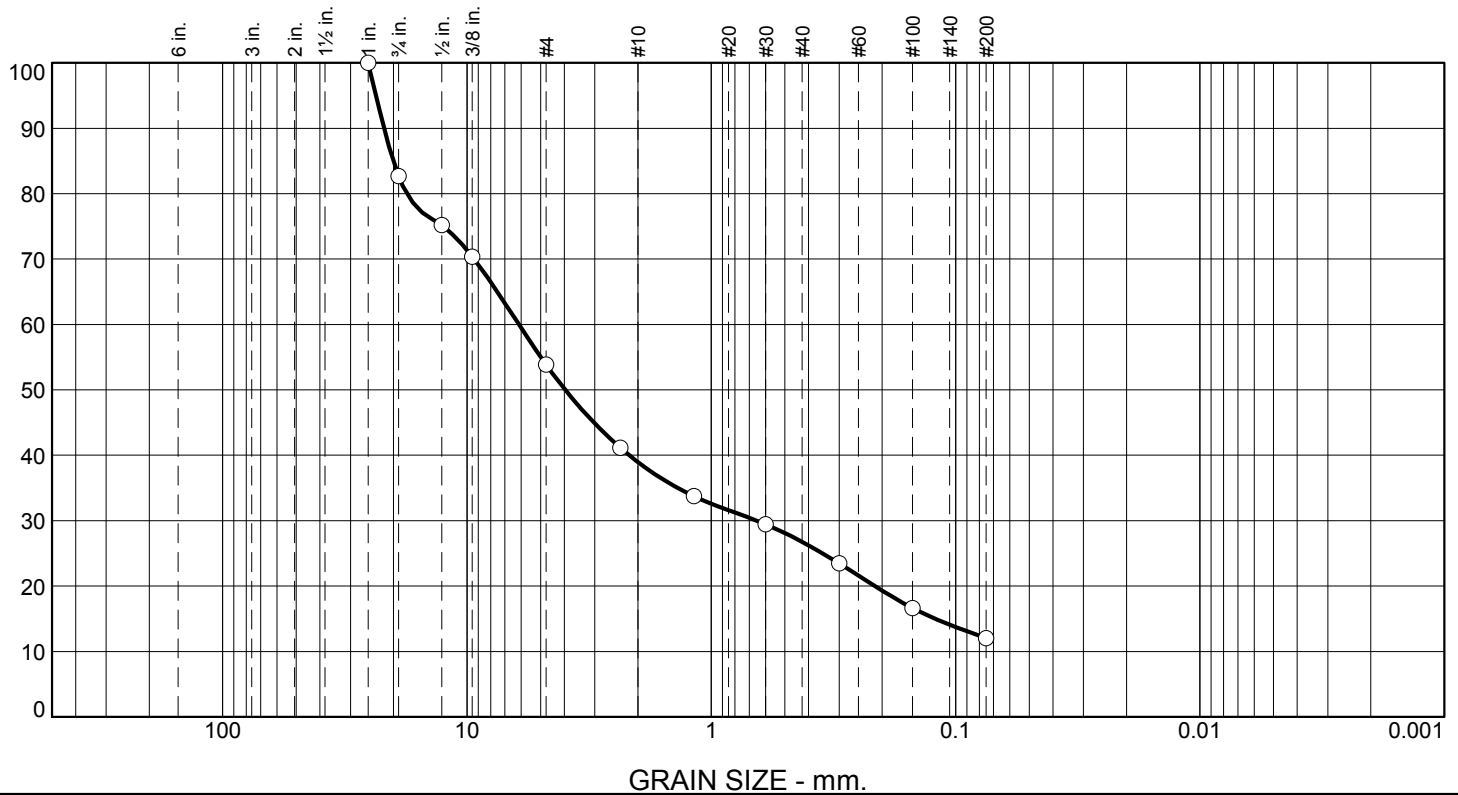
Project No: 1615.X

Figure

Particle Size Distribution Report

Category Report page 89

PERCENT FINER



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	17.3	28.8	14.9	12.3	14.6	12.1	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.0"	100.0		
3/4"	82.7		
1/2"	75.2		
3/8"	70.3		
#4	53.9		
#8	41.2		
#16	33.7		
#30	29.4		
#50	23.5		
#100	16.6		
#200	12.1		

* (no specification provided)

Material Description

Olive Brown Silty GRAVEL with Sand

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D₉₀= 21.8809 D₈₅= 20.0351 D₆₀= 6.1368
D₅₀= 3.9632 D₃₀= 0.6533 D₁₅= 0.1212
D₁₀= C_u= C_c=

Remarks

Date Received:

Date Tested:

Tested By: KLC

Checked By: KLC

Title:

Sample Number: BCI 11-03 #4

Depth: 20.7521.25

Date Sampled: 11-22-2011

Blackburn Consulting

W. Sacramento, CA

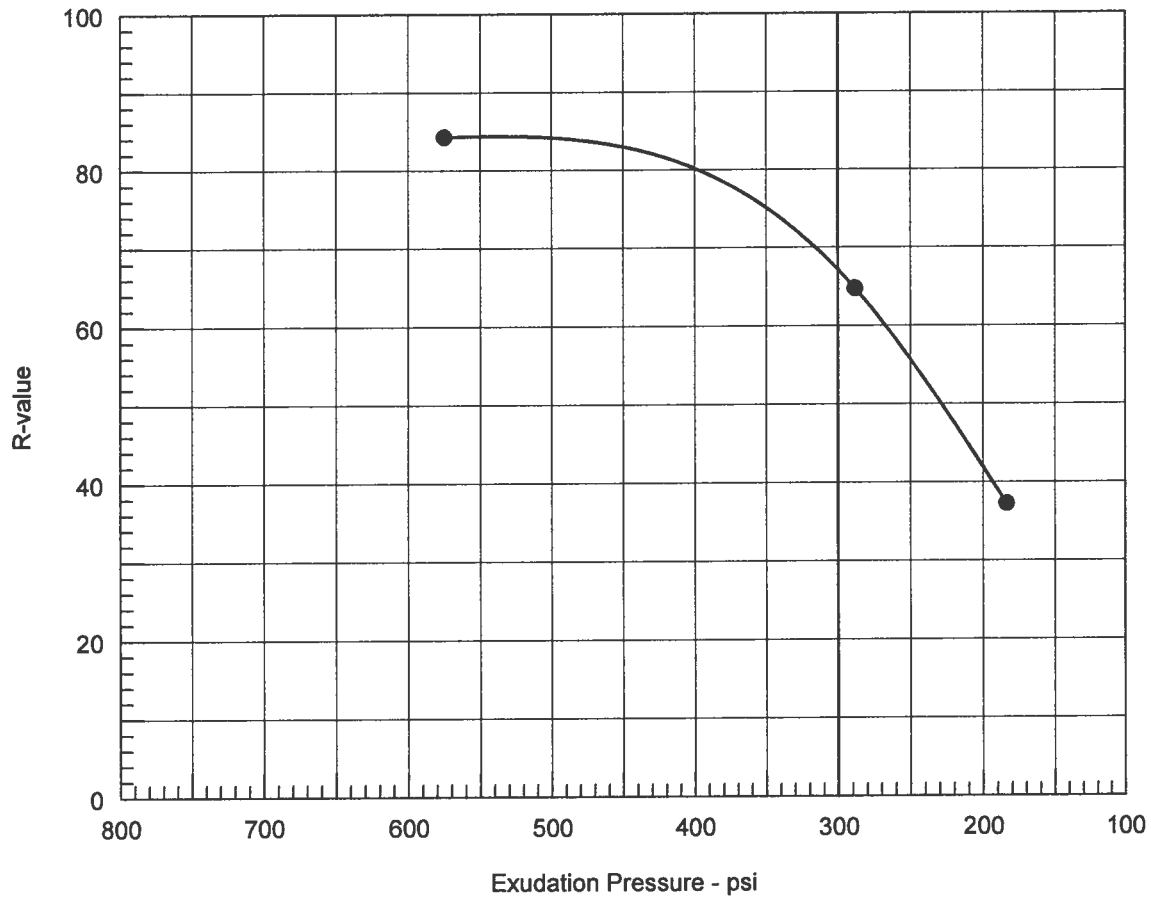
Client: Mark Thomas Company

Project: Garcia River Bridge Retrofit

Project No: 1615.X

Figure

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	17	124.2	13.1	0	18	2.53	575	84	84
2	10	122.5	13.7	0	45	2.54	288	65	65
3	7	120.1	14.7	0	84	2.55	184	37	37

Test Results						Material Description			
R-value at 300 psi exudation pressure = 67						Lean CLAYEY SAND with GRAVEL, dark olive brown			
Project No.: 1615.X Project: Eureka Hill Rd. Bridge Retrofit Sample Number: BCI-11-101 Bag A Depth: 1-5.5' Date: 11/16/2011						Tested by: MDR/BWM Checked by: Remarks: 39.4% retained on No. 4 sieve, sample batched			
R-VALUE TEST REPORT Blackburn Consulting						Figure _____			

APPENDIX C

Liquefaction Triggering Calculations



note applicable to sloping ground surfaces greater than about 6%

Project: Eureka Hill Road Bridge (WINTER CASE)

BCI No.: 1615.1

Date: 4/25/2013

Location: BCI-11-01

Moment Magnitude, $M_w = 8.0$
PGA = 0.83 g

Ground Water Depth: feet (For Liquefaction Analysis)

[illegible]
$$CSR = 0.65(a_{\max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \quad [\text{Cyclic Stress Ratio}] \text{ -- Seed and Idriss 1971}$$

Where: $r_d = 1.0 - 0.00233z$ for $z \leq 30$ feet

$$r_d = 1.174 - 0.00814z \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$r_d = 0.744 - 0.00244z \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$r_d = 0.5 \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$(N_1)_{60CS} = \text{clean-sand equivalent } (N_1)_{60} = \alpha + \beta(NI)_{60}$$
 $\alpha = 0; \text{FC} \leq 5\%$
$$\alpha = \exp [1.76 - 190/FC^2]; 5\% < FC < 35\%$$
 $\alpha = 5.0; FC \geq 35\%$ $\beta = 1.0; \text{FC} \leq 5\%$
$$\beta = [0.99 + (FC^{1.5}/1,000)]; 5\% < FC < 35\%$$
 $\beta = 1.2; FC \geq 35\%$
$$D_r^1 = [(N_1)_{60cs}/46]^{0.5} \text{ where } (N_1)_{60} \leq 46 \text{ (Idriss and Boulanger)}$$
$$D_r^2 = 14 \cdot [(N_l)_{60cs}]^{0.5} \text{ where } (N_l)_{60} \leq 42 \text{ [modified version of Myerhof (1957)]}$$
$$CRR_{7.5} = [1/(34 - (N_1)_{60cs})] + [(N_1)_{60cs}/135] + [50/(10 * (N_1)_{60cs} + 45)^2] - [1/200] \text{ for } (N_1)_{60} < 30$$
$$\text{MSF} = 10^{2.24} / \text{Mw}^{2.56}$$
$$K\sigma = (\sigma'v/Pa)^{(f-1)}$$

where: $f = 1 - 0.005Dr$ for $40\% < Dr^1 < 80\%$ (Overburden Correction)

$$f = 0.8 \text{ for } Dr^1 \leq 40\%$$
$$f = 0.6 \text{ for } Dr^1 \geq 80\%$$
$$\text{Corrected CRR} = (\text{CRR}_{7.5})(\text{MSF})(K_{\sigma})$$
$$FS = \text{Corrected CRR/CSR}$$

Ground Water Depth: feet (For Liquefaction Analysis)

$$FS = \text{Corrected CRR/CSR}$$

note applicable to sloping ground surfaces greater than about 6%

Project: Eureka Hill Road Bridge (SUMMER CASE)

BCI No.: 1615.1

Date: 4/25/2013

Location: BCI-11-03

Moment Magnitude, $M_w = 8.0$
PGA = 0.83 g

Ground Water Depth: feet (For Liquefaction Analysis)

[illegible]
$$CSR = 0.65(a_{\max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \quad [\text{Cyclic Stress Ratio}] \text{ -- Seed and Idriss 1971}$$

Where: $r_d = 1.0 - 0.00233z$ for $z \leq 30$ feet

$$r_d = 1.174 - 0.00814z \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$r_d = 0.744 - 0.00244z \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$r_d = 0.5 \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$(N_1)_{60CS} = \text{clean-sand equivalent } (N_1)_{60} = \alpha + \beta(N_1)_{60}$$
 $\alpha = 0; \text{FC} \leq 5\%$
$$\alpha = \exp [1.76 - 190/FC^2]; 5\% < FC < 35\%$$
 $\alpha = 5.0; FC \geq 35\%$ $\beta = 1.0; FC \leq 5\%$
$$\beta = [0.99 + (FC^{1.5}/1,000)]; 5\% < FC < 35\%$$
 $\beta = 1.2; FC \geq 35\%$
$$D_r^1 = [(N_1)_{60cs}/46]^{0.5} \text{ where } (N_1)_{60} \leq 46 \text{ (Idriss and Boulanger)}$$
$$D_r^2 = 14 \cdot [(N_1)_{60cs}]^{0.5} \text{ where } (N_1)_{60} \leq 42 \text{ [modified version of Myerhof (1957)]}$$
$$CRR_{7.5} = [1/(34 - (N_1)_{60cs})] + [(N_1)_{60cs}/135] + [50/(10 * (N_1)_{60cs} + 45)^2] - [1/200] \text{ for } (N_1)_{60} < 30$$
$$\text{MSF} = 10^{2.24} / \text{Mw}^{2.56}$$
$$K\sigma = (\sigma'v/Pa)^{(f-1)}$$

where: $f = 1 - 0.005Dr$ for $40\% < Dr^1 < 80\%$ (Overburden Correction)

$$f = 0.8 \text{ for } Dr^1 \leq 40\%$$
$$f = 0.6 \text{ for } Dr^1 \geq 80\%$$
$$\text{Corrected CRR} = (\text{CRR}_{7.5})(\text{MSF})(K_{\sigma})$$
$$FS = \text{Corrected CRR/CSR}$$

$$f = 0.6 \text{ for } Dr^1 \geq 80\%$$

$$f = 0,6 \text{ for } Dr^1 \geq 80\%$$

note applicable to sloping ground surfaces greater than about 6%

Project: Eureka Hill Road Bridge (WINTER CASE)

BCI No.: 1615.1

Date: 4/26/2013

Location: Taber Boring B1

Moment Magnitude, $M_w = 8.0$
PGA = 0.83 g

Ground Water Depth: feet (For Liquefaction Analysis)

[illegible]
$$CSR = 0.65(a_{\max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \quad [\text{Cyclic Stress Ratio}] \text{ -- Seed and Idriss 1971}$$

Where: $r_d = 1.0 - 0.00233z$ for $z \leq 30$ feet

$$r_d = 1.174 - 0.00814z \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$r_d = 0.744 - 0.00244z \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$r_d = 0.5 \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$(N_1)_{60CS} = \text{clean-sand equivalent } (N_1)_{60} = \alpha + \beta(N_1)_{60}$$
 $\alpha = 0; \text{FC} \leq 5\%$
$$\alpha = \exp [1.76 - 190/FC^2]; 5\% < FC < 35\%$$
 $\alpha = 5.0; FC \geq 35\%$ $\beta = 1.0; \text{FC} \leq 5\%$
$$\beta = [0.99 + (FC^{1.5}/1,000)]; 5\% < FC < 35\%$$
 $\beta = 1.2; FC \geq 35\%$
$$D_r^1 = [(N_1)_{60cs}/46]^{0.5} \text{ where } (N_1)_{60} \leq 46 \text{ (Idriss and Boulanger)}$$
$$D_r^2 = 14 \cdot [(N_1)_{60cs}]^{0.5} \text{ where } (N_1)_{60} \leq 42 \text{ [modified version of Myerhof (1957)]}$$
$$CRR_{7.5} = [1/(34 - (N_1)_{60cs})] + [(N_1)_{60cs}/135] + [50/(10 * ((N_1)_{60cs} + 45)^2)] - [1/200] \text{ for } (N_1)_{60} < 30$$
$$\text{MSF} = 10^{2.24} / \text{Mw}^{2.56}$$
$$K\sigma = (\sigma'v/Pa)^{(f-1)}$$

where: $f = 1 - 0.005Dr$ for $40\% < Dr^1 < 80\%$ (Overburden Correction)

$$f = 0.8 \text{ for } Dr^1 \leq 40\%$$
$$f = 0.6 \text{ for } Dr^1 \geq 80\%$$
$$\text{Corrected CRR} = (\text{CRR}_{7.5})(\text{MSF})(K\sigma)$$
$$FS = \text{Corrected CRR/CSR}$$

note applicable to sloping ground surfaces greater than about 6%

Project: Eureka Hill Road Bridge (SUMMER CASE)

BCI No.: 1615.1

Date: 4/26/2013

Location: Taber Boring B2

Moment Magnitude, $M_w = 8.0$
PGA = 0.83 g

Ground Water Depth: feet (For Liquefaction Analysis)

[illegible]
$$CSR = 0.65(a_{\max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \quad [\text{Cyclic Stress Ratio}] \text{ -- Seed and Idriss 1971}$$

Where: $r_d = 1.0 - 0.00233z$ for $z \leq 30$ feet

$$r_d = 1.174 - 0.00814z \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$r_d = 0.744 - 0.00244z \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$r_d = 0.5 \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$(N_I)_{60CS} = \text{clean-sand equivalent } (N_I)_{60} = \alpha + \beta(NI)_{60}$$
 $\alpha = 0; \text{FC} \leq 5\%$
$$\alpha = \exp [1.76 - 190/FC^2]; 5\% < FC < 35\%$$
 $\alpha = 5.0; FC \geq 35\%$ $\beta = 1.0; FC \leq 5\%$
$$\beta = [0.99 + (FC^{1.5}/1,000)]; 5\% < FC < 35\%$$
 $\beta = 1.2; FC \geq 35\%$
$$D_r^1 = [(N_1)_{60cs}/46]^{0.5} \text{ where } (N_1)_{60} \leq 46 \text{ (Idriss and Boulanger)}$$
$$D_r^2 = 14 \cdot [(N_1)_{60cs}]^{0.5} \text{ where } (N_1)_{60} \leq 42 \text{ [modified version of Myerhof (1957)]}$$
$$CRR_{7.5} = [1/(34 - (N_1)_{60cs})] + [(N_1)_{60cs}/135] + [50/(10 * (N_1)_{60cs} + 45)^2] - [1/200] \text{ for } (N_1)_{60} < 30$$
$$\text{MSF} = 10^{2.24}/\text{Mw}^{2.56}$$
$$K\sigma = (\sigma'v/Pa)^{(f-1)}$$

where: $f = 1 - 0.005Dr$ for $40\% < Dr^1 < 80\%$ (Overburden Correction)

$$f = 0.8 \text{ for } Dr^1 \leq 40\%$$
$$f = 0.6 \text{ for } Dr^1 \geq 80\%$$
$$\text{Corrected CRR} = (\text{CRR}_{7.5})(\text{MSF})(K_{\sigma})$$
$$FS = \text{Corrected CRR/CSR}$$

note applicible to sloping ground surfaces greater than about 6%

Project: Eureka Hill Road Bridge (WINTER CASE)

BCI No.: 1615.1

Date: 4/26/2013

Location: Taber Boring B2

Moment Magnitude, $M_w = 8.0$
PGA = 0.83 g

Ground Water Depth: feet (For Liquefaction Analysis)

[illegible]
$$CSR = 0.65(a_{\max}/g)(\sigma_{vo}'/\sigma_{vo}')r_d \quad [\text{Cyclic Stress Ratio}] \text{ -- Seed and Idriss 1971}$$

Where: $r_d = 1.0 - 0.00233z$ for $z \leq 30$ feet

$$r_d = 1.174 - 0.00814z \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$r_d = 0.744 - 0.00244z \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$r_d = 0.5 \quad \text{for } 30 < z \leq 75 \text{ feet}$$
$$(N_1)_{60CS} = \text{clean-sand equivalent } (N_1)_{60} = \alpha + \beta(N_1)_{60}$$
 $\alpha = 0; \text{FC} \leq 5\%$
$$\alpha = \exp [1.76 - 190/FC^2]; 5\% < FC < 35\%$$
 $\alpha = 5.0; FC \geq 35\%$ $\beta = 1.0; \text{FC} \leq 5\%$
$$\beta = [0.99 + (FC^{1.5}/1,000)]; 5\% < FC < 35\%$$
 $\beta = 1.2; FC \geq 35\%$
$$D_r^1 = [(N_1)_{60cs}/46]^{0.5} \text{ where } (N_1)_{60} \leq 46 \text{ (Idriss and Boulanger)}$$
$$D_r^2 = 14 \cdot [(N_1)_{60cs}]^{0.5} \text{ where } (N_1)_{60} \leq 42 \text{ [modified version of Myerhof (1957)]}$$
$$CRR_{7.5} = [1/(34 \cdot (N_1)_{60cs})] + [(N_1)_{60cs}/135] + [50/(10 \cdot (N_1)_{60cs} + 45)^2] - [1/200] \text{ for } (N_1)_{60} < 30$$
$$\text{MSF} = 10^{2.24} / \text{Mw}^{2.56}$$
$$K\sigma = (\sigma'v/Pa)^{(f-1)}$$

where: $f = 1 - 0.005Dr$ for $40\% < Dr^1 < 80\%$ (Overburden Correction)

$$f = 0.8 \text{ for } Dr^1 \leq 40\%$$
$$f = 0.6 \text{ for } Dr^1 \geq 80\%$$
$$\text{Corrected CRR} = (\text{CRR}_{7.5})(\text{MSF})(K\sigma)$$
$$FS = \text{Corrected CRR/CSR}$$

Liquefaction Spreadsheet - Based on 1996 NCEER and 1998 NCEER/NSF Workshops
Simplified Procedure - Standard Penetration Test Analysis and
Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework (Report No. EERC 2003-06)

Eureka Hill Road Bridge
Taber Boring B3

Project: Eureka Hill Road Bridge
BCI No.: 1615.1
Date: 4/26/2013
Location: Taber Boring B3

Moment Magnitude, $M_w = 8.0$
PGA = 0.83 g

Ground Water Depth: 27.8 feet (For Liquefaction Analysis)

										Analysis based on SPT Blow Counts											Reduced P-Multiplier for LPILE Lateral Analysis	Residual Shear Strength - psf Seed & Harder 1990	Residual Shear Strength - psf Kramer & Wang 2007		
Sample Number	Sample Depth	Depth to Bottom of Layer	Soil Type	Total Unit Weight	Field N	Fines	PI	Total Stress	Analysis Effective Stress	r _d	CSR	(N ₁) ₆₀	(N ₁) _{60cs}	D _r ¹	D _r ²	CRR _{7.5}	MSF	K _σ	Corrected CRR	Factor of Safety					
												(bpf)	(bpf)	(%)	(%)					FS				FS ≤ 1.2	
1	6.0	8.0	CL	128	12		6	768	768	0.986	0.532	15	15	57.1	54.2	0.160	0.85	1.000	0.136	unsaturated		N/A	--	--	
2	11.0	13.0	SC	132	17	26	4	1420	1420	0.974	0.526	17	23	71.4	67.8	0.264	0.85	1.000	0.224	unsaturated		N/A	--	--	
3	16.0	18.0	SC	134	17	26	4	2086	2086	0.963	0.519	16	22	69.7	66.2	0.247	0.85	1.005	0.210	unsaturated		N/A	--	--	
4	21.0	23.0	SC	134	11	22	5	2756	2756	0.951	0.513	9	14	54.7	51.9	0.148	0.85	0.930	0.117	unsaturated		N/A	--	--	
5	26.0	28.0	SC	131	7	28		3417	3417	0.939	0.507	5	10	47.2	44.8	0.115	0.85	0.893	0.087	unsaturated		N/A	--	--	
6	31.0	33.0	CL	120	6	60	7	4039	3839	0.922	0.523	4	10	46.2	43.8	0.111	0.85	0.872	0.082	0.16	x	0.06	156	212	
7	36.0	38.0	CL	120	8	60	12	4639	4127	0.881	0.534	6	12	51.5	48.9	0.133	0.85	0.842	0.095	0.18	x	0.09	239	275	
8	41.0	43.0	CL	126	17	60	12	5257	4433	0.840	0.538	11	18	62.9	59.7	0.194	0.85	0.792	0.130	NL		N/A	--	--	
9	46.0	48.0	SC	141	35			5932	4796	0.800	0.534	23	23	70.7	67.1	0.257	0.85	0.749	0.163	0.31	x	0.25	1496	1916	
10	51.0	53.0	SC	139	51			6631	5183	0.759	0.524	32	32	83.4	79.2	N/A	0.85	0.699	N/A	NL		N/A	--	--	
11	56.0	58.0	SM	139	125	14		7326	5566	0.718	0.510	75	80	100.0	90.7	N/A	0.85	0.679	N/A	NL		N/A	--	--	
12	61.0	63.0	SC	145	145	12		8039	5967	0.677	0.492	84	88	100.0	90.7	N/A	0.85	0.661	N/A	NL		N/A	--	--	
13	66.0	68.0	SC	135	77	12		8734	6350	0.637	0.472	43	46	99.9	90.7	N/A	0.85	0.644	N/A	NL		N/A	--	--	
14	71.0	73.0	SC	135	57	12		9409	6713	0.596	0.451	31	34	85.4	81.1	N/A	0.85	0.630	N/A	NL		N/A	--	--	
15	76.0	78.0	SC	125	30	12		10054	7046	0.559	0.430	16	18	62.7	59.5	0.192	0.85	0.686	0.112	0.26	x	0.17	813	1123	
16	81.0	83.0	SC	139	63	12		10721	7401	0.546	0.427	33	36	88.0	83.5	N/A	0.85	0.606	N/A	NL		N/A	--	--	
17	86.0	86.0	SC	135	49	12		11404	7772	0.534	0.423	25	27	77.1	73.2	0.348	0.85	0.606	0.179	0.42	x	0.34	--	3180	
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--

$CSR = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$ [Cyclic Stress Ratio] -- Seed and Idriss 1971

Where: $r_d = 1.0 - 0.00233z$ for $z \leq 30$ feet
 $r_d = 1.174 - 0.00814z$ for $30 < z \leq 75$ feet
 $r_d = 0.744 - 0.00244z$ for $30 < z \leq 75$ feet
 $r_d = 0.5$ for $30 < z \leq 75$ feet

$(N_1)_{60cs} = \text{clean-sand equivalent } (N_1)_{60} = \alpha + \beta(N_1)_{60}$

$\alpha = 0$; FC $\leq 5\%$ $\beta = 1.0$; FC $\leq 5\%$
 $\alpha = \exp [1.76 - 190/FC^2]$; $5\% < FC < 35\%$ $\beta = [0.99 + (FC^{1.5}/1,000)]$; $5\% < FC < 35\%$
 $\alpha = 5.0$; FC $\geq 35\%$ $\beta = 1.2$; FC $\geq 35\%$

$D_r^1 = [(N_1)_{60cs}/46]^{0.5}$ where $(N_1)_{60} \leq 46$ (Idriss and Boulanger)

$D_r^2 = 14[(N_1)_{60cs}]^{0.5}$ where $(N_1)_{60} \leq 42$ [modified version of Myerhof (1957)]

$CRR_{7.5} = [1/(34 - (N_1)_{60cs})] + [(N_1)_{60cs}/135] + [50/(10 * (N_1)_{60cs} + 45)^2] - [1/200]$ for $(N_1)_{60} < 30$

$MSF = 10^{2.24/Mw^{2.56}}$

$K_\sigma = (\sigma'v/Pa)^{(f-1)}$ where: $f = 1 - 0.005Dr$ for $40\% < Dr^1 < 80\%$ (Overburden Correction)

$f = 0.8$ for $Dr^1 \leq 40\%$

$f = 0.6$ for $Dr^1 \geq 80\%$

Corrected CRR = $(CRR_{7.5})(MSF)(K_\sigma)$

$FS = \text{Corrected CRR}/CSR$

Liquefaction Spreadsheet - Based on 1996 NCEER and 1998 NCEER/NSF Workshops
Simplified Procedure - Standard Penetration Test Analysis and
Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework (Report No. EERC 2003-06)

Eureka Hill Road Bridge (WINTER CASE)
Taber Boring B4

Project: Eureka Hill Road Bridge (WINTER CASE)
BCI No.: 1615.1
Date: 4/26/2013
Location: Taber Boring B4

Moment Magnitude, $M_w = 8.0$
PGA = 0.83 g

Ground Water Depth: 21.3 feet (For Liquefaction Analysis)

										Analysis based on SPT Blow Counts										Reduced P-Multiplier for LPILE Lateral Analysis	Residual Shear Strength - psf Seed & Harder 1990	Residual Shear Strength - psf Kramer & Wang 2007			
Sample Number	Sample Depth	Depth to Bottom of Layer	Soil Type	Total Unit Weight	Field N	Fines	PI	Total Stress	Analysis Effective Stress	r _d	CSR	(N ₁) ₆₀	(N ₁) _{60cs}	D _r ¹	D _r ²	CRR _{7.5}	MSF	K _σ	Corrected CRR				Factor of Safety		
												(bpf)	(bpf)	(%)	(%)								FS	FS ≤ 1.2	
1	6.0	8.5	SP	133	15	21.5		798	798	0.986	0.532	19	25	73.1	69.4	0.283	0.85	1.000	0.240	unsaturated		N/A	--	--	
2	11.0	13.0	SP	116	15	15		1421	1421	0.974	0.526	15	18	62.9	59.8	0.194	0.85	1.000	0.165	unsaturated		N/A	--	--	
3	16.0	18.0	SM	118	5	48		2007	2007	0.963	0.519	5	11	48.9	46.4	0.122	0.85	1.013	0.105	unsaturated		N/A	--	--	
4	21.0	23.5	SC	110	28			2573	2573	0.951	0.513	23	23	70.7	67.1	0.257	0.85	0.933	0.203	unsaturated		N/A	--	--	
5	26.0	28.5	SC	123	36	17.5	10	3155	2862	0.939	0.559	27	32	83.2	79.0	N/A	0.85	0.886	N/A	NL		N/A	--	--	
6	31.0	32.5	SC	110	29	12		3738	3132	0.922	0.593	22	24	72.6	68.9	0.278	0.85	0.867	0.204	0.34	x	0.28	1496	1347	
7	36.0	37.5	SC	110	23	7.5		4288	3370	0.881	0.605	17	17	61.5	58.4	0.185	0.85	0.867	0.136	0.22	x	0.16	813	814	
8	41.0	43.5	GP	142	71			4950	3720	0.840	0.603	49	49	100.0	90.7	N/A	0.85	0.798	N/A	NL		N/A	--	--	
9	46.0	48.5	GP	143	98			5662	4121	0.800	0.593	65	65	100.0	90.7	N/A	0.85	0.766	N/A	NL		N/A	--	--	
10	51.0	53.5	GP	144	110			6380	4526	0.759	0.577	70	70	100.0	90.7	N/A	0.85	0.738	N/A	NL		N/A	--	--	
11	56.0	58.5	GP	144	132			7100	4934	0.718	0.557	80	80	100.0	90.7	N/A	0.85	0.713	N/A	NL		N/A	--	--	
12	61.0	63.5	GP	145	75			7822	5345	0.677	0.535	44	44	97.8	90.7	N/A	0.85	0.690	N/A	NL		N/A	--	--	
13	66.0	68.5	GP	133	106			8517	5728	0.637	0.511	60	60	100.0	90.7	N/A	0.85	0.671	N/A	NL		N/A	--	--	
14	71.0	73.5	GP	132	88			9180	6078	0.596	0.486	49	49	100.0	90.7	N/A	0.85	0.656	N/A	NL		N/A	--	--	
15	76.0	78.5	GP	132	100			9840	6426	0.559	0.461	54	54	100.0	90.7	N/A	0.85	0.641	N/A	NL		N/A	--	--	
16	81.0	81.5	GP	132	100			10500	6774	0.546	0.457	53	53	100.0	90.7	N/A	0.85	0.628	N/A	NL		N/A	--	--	
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--
																							N/A	--	--

$CSR = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$ [Cyclic Stress Ratio] -- Seed and Idriss 1971

Where: $r_d = 1.0 - 0.00233z$ for $z \leq 30$ feet
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$\alpha = 0$; FC $\leq 5\%$ $\beta = 1.0$; FC $\leq 5\%$
 $\alpha = \exp [1.76 - 190/FC^2]$; $5\% < FC < 35\%$ $\beta = [0.99 + (FC^{1.5}/1,000)]$; $5\% < FC < 35\%$
 $\alpha = 5.0$; FC $\geq 35\%$ $\beta = 1.2$; FC $\geq 35\%$

$D_r^1 = [(N_1)_{60cs}/46]^{0.5}$ where $(N_1)_{60} \leq 46$ (Idriss and Boulanger)

$D_r^2 = 14 \cdot [(N_1)_{60cs}]^{0.5}$ where $(N_1)_{60} \leq 42$ [modified version of Myerhof (1957)]

$CRR_{7.5} = [1/(34 - (N_1)_{60cs})] + [(N_1)_{60cs}/135] + [50/(10 \cdot (N_1)_{60cs} + 45)^2] - [1/200]$ for $(N_1)_{60} < 30$

$MSF = 10^{2.24/M_w^{2.56}}$

$K_\sigma = (\sigma'v/Pa)^{(f-1)}$

where: $f = 1 - 0.005Dr$ for $40\% < Dr^1 < 80\%$ (Overburden Correction)

$f = 0.8$ for $Dr^1 \leq 40\%$

$f = 0.6$ for $Dr^1 \geq 80\%$

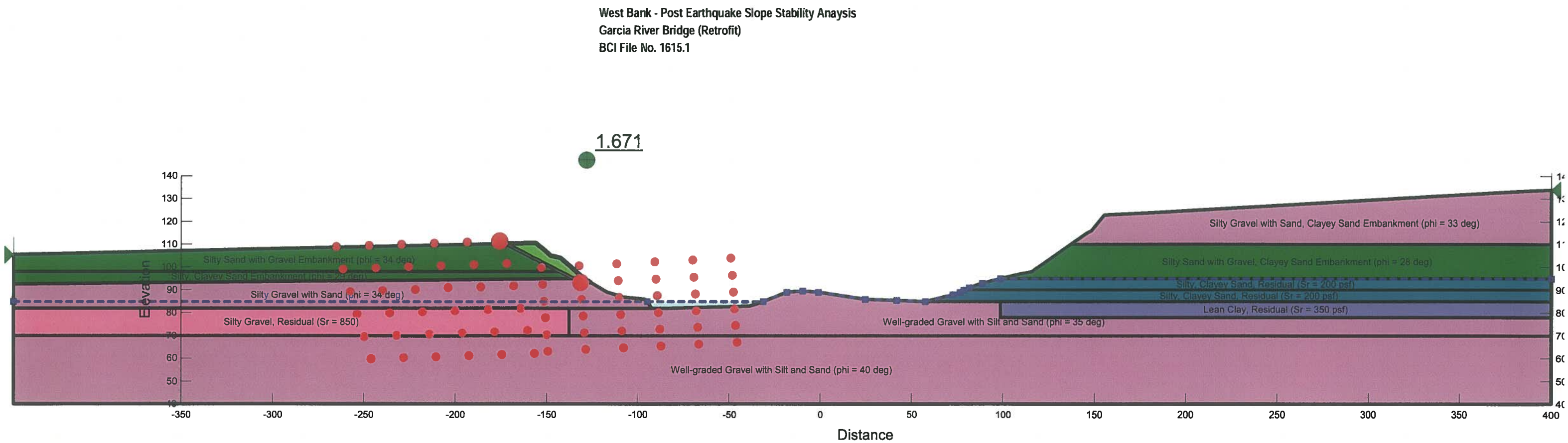
Corrected CRR = (CRR7.5)(MSF)(K_σ)

$FS = \text{Corrected CRR}/CSR$

APPENDIX D

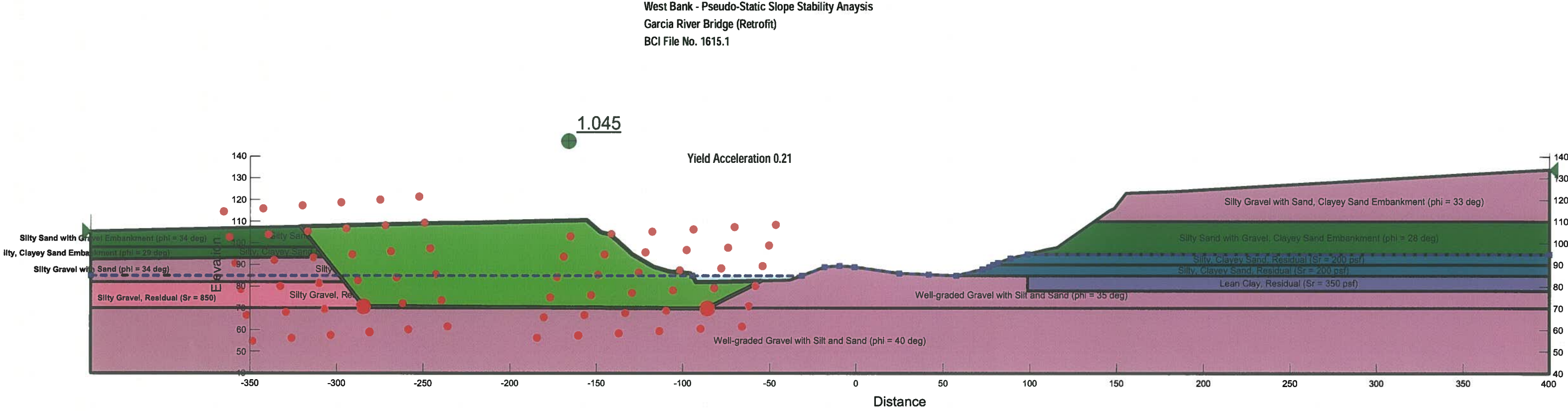
SLOPE/W 2007 Version Output Graphs





Material ID and Type, Unit Weight, Strength (Effective/Residual), Friction Angle

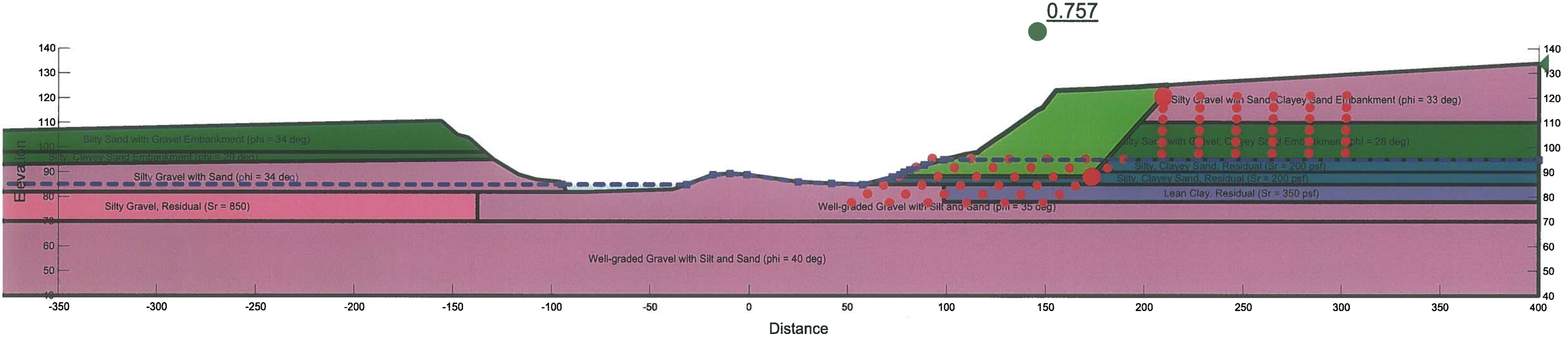
Silty Sand with Gravel Embankment (phi = 34 deg)	125 pcf	0 psf	34 °
Silty, Clayey Sand Embankment (phi = 29 deg)	112 pcf	0 psf	29 °
Silty Gravel with Sand (phi = 34 deg)	125 pcf	0 psf	34 °
Well-graded Gravel with Silt and Sand (phi = 40 deg)	140 pcf	0 psf	40 °
Well-graded Gravel with Silt and Sand (phi = 35 deg)	130 pcf	0 psf	35 °
Lean Clay, Residual (Sr = 350 psf)	120 pcf	350 psf	
Silty, Clayey Sand, Residual (Sr = 200 psf)	117 pcf	200 psf	
Silty Sand with Gravel, Clayey Sand Embankment (phi = 28 deg)	120 pcf	0 psf	28 °
Silty Gravel with Sand, Clayey Sand Embankment (phi = 33 deg)	120 pcf	0 psf	33 °
Silty Gravel, Residual (Sr = 850)	130 pcf	850 psf	



Material ID and Type, Unit Weight, Strength (Effective/Residual), Friction Angle

Silty Sand with Gravel Embankment (phi = 34 deg)	125 pcf	0 psf	34 °
Silty, Clayey Sand Embankment (phi = 29 deg)	112 pcf	0 psf	29 °
Silty Gravel with Sand (phi = 34 deg)	125 pcf	0 psf	34 °
Well-graded Gravel with Silt and Sand (phi = 40 deg)	140 pcf	0 psf	40 °
Well-graded Gravel with Silt and Sand (phi = 35 deg)	130 pcf	0 psf	35 °
Lean Clay, Residual (Sr = 350 psf)	120 pcf	350 psf	
Silty, Clayey Sand, Residual (Sr = 200 psf)	117 pcf	200 psf	
Silty Sand with Gravel, Clayey Sand Embankment (phi = 28 deg)	120 pcf	0 psf	28 °
Silty Gravel with Sand, Clayey Sand Embankment (phi = 33 deg)	120 pcf	0 psf	33 °
Silty Gravel, Residual (Sr = 850)	130 pcf	850 psf	

East Bank - Post Earthquake Slope Stability Analysis
Garcia River Bridge (Retrofit)
BCI File No. 1615.1



Material ID and Type, Unit Weight, Strength (Effective/Residual), Friction Angle

Silty Sand with Gravel Embankment (phi = 34 deg)	125 pcf	0 psf	34 °
Silty, Clayey Sand Embankment (phi = 29 deg)	112 pcf	0 psf	29 °
Silty Gravel with Sand (phi = 34 deg)	125 pcf	0 psf	34 °
Well-graded Gravel with Silt and Sand (phi = 40 deg)	140 pcf	0 psf	40 °
Well-graded Gravel with Silt and Sand (phi = 35 deg)	130 pcf	0 psf	35 °
Lean Clay, Residual (Sr = 350 psf)	120 pcf	350 psf	
Silty, Clayey Sand, Residual (Sr = 200 psf)	117 pcf	200 psf	
Silty Sand with Gravel, Clayey Sand Embankment (phi = 28 deg)	120 pcf	0 psf	28 °
Silty Gravel with Sand, Clayey Sand Embankment (phi = 33 deg)	120 pcf	0 psf	33 °
Silty Gravel, Residual (Sr = 850)	130 pcf	850 psf	

APPENDIX 6

GEOTECHNICAL RESPONSE TO COMMENTS 11/7/2014

West Sacramento Office:
2491 Boatman Ave. ▪ West Sacramento, CA 95691
(916) 375-8706 ▪ Fax (916) 375-8709



Main Auburn Office: (530) 887-1494
Fresno Office: (559) 438-8411
Modesto Office: (209) 522-6273

Geotechnical ▪ Geo-Environmental ▪ Construction Services ▪ Forensics

File No. 1615.1
November 7, 2014

Mr. Eric Fredrickson
Mark Thomas & Company, Inc.
7300 Folsom Boulevard, Suite 203
Sacramento, CA 95826

Subject: RESPONSE TO COMMENTS
Preliminary Foundation Report
Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California

Dear Mr. Fredrickson,

Blackburn Consulting (BCI) prepared a Preliminary Foundation Report (PFR) dated June 27, 2014 for use by Mark Thomas & Company, Inc. (MTCO) to develop an updated seismic retrofit strategy for the Garcia River Bridge at Eureka Hill Road in Mendocino County, California.

BCI received Caltrans' draft review comments on the PFR. We attach the Caltrans comments and our response.

Please call if you have questions or require additional information.

Sincerely,

BLACKBURN CONSULTING

Patrick Fischer, C.E.G., C.E.
Engineering Geologist, Principal

David J. Morrell, C.E., G.E.
Senior Project Manager

Attached: Caltrans Review Comments with BCI Response
Design Spectrum Comparison Graph
Table 1 - Fault Traces Used in Rupture Potential Analysis
Map of 1906 Fault Trace (Figure 6c, Zachariasen, 2008)
Alquist Priolo Special Studies Zone Map
USGS LiDAR Image with Fault Traces
Probabilistic Offset Graph

DRAFT REVIEW COMMENTS FROM GEOTECHNICAL

Caltrans Comment

Section/Page 8:

Please clarify how $V_{s30} = 1033$ fbs was derived.

BCI Response

A shear wave velocity (V_{s30} value) of 886 fps (270 m/s) was actually used to develop the standard Caltrans ARS curve, not the reported 1033 fps (315 m/s) value. BCI estimated V_{s30} values for each boring (BCI 2011 borings and Taber 1997 borings) using Caltrans "Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations", Appendix A, pgs 15-18, November 2012. The individual boring V_{s30} values ranged from 262 m/s to 415 m/s.

Caltrans Comment

Figure 4:

Please provide comparison graphs showing how the spectrum was combined, it seems that the SHAKE ARS controls the short periods in general, but the difference between SHAKE and ARS online with deaggregation consideration could be within 5-10%.

BCI Response

A graph that shows each spectra and the envelope provided is attached.

Caltrans Comment

Section/Page 8:

Many soil models exist and the research on this area is still developing. Non-linear free field analysis requires detailed and reliable soils parameters to support, which is hard to get, therefore, the results of such analysis may not produce a refined and more accurate ARS. If the SHAKE and ARS online has around less than 10% difference, the current ARS may be used for design

BCI Response

The attached graph shows the ARS Online and SHAKE derived data. MTCO provided preliminary structure period data below.

CASE	PREDOMINANT STRUCTURE PERIOD (seconds)	
	Longitudinal Direction	Transverse Direction
Existing Bridge	1.81	1.18
Bridge Retrofit	0.69	0.61

The difference between the ARS online and SHAKE derived data is slightly greater than 10% for the existing bridge case and bridge retrofit case in the transverse direction.

Caltrans Comment

Section/Page 8:

Our fault rupture specialist will review this section, but one question I have is that any vertical offset could occur?

BCI Response

Vertical offset can occur. We are not aware of accepted methods to estimate the potential vertical offset but have seen estimates equal to 10% of the horizontal displacement used on other Caltrans projects.

Caltrans Comment

Section/Page 12:

In terms of retrofit strategy, SD could consider "floating deck" concept to allow the foundations move without collapsing the deck/bridge?

BCI Response

Response to this comment is deferred to MTCO.

Caltrans Comment

Section/Page 12:

Please clarify how the 2 ft lateral movement was calculated for West Bank.

BCI Response

We calculated the horizontal displacement for the west bank using both the Bray and Travarasrou (2007) and Faris (2006) methods referenced in the footnote. The Faris (2006) method generated horizontal displacements between 3 and 17.5 inches depending on the boring data used. The Bray and Travarasrou method produced a horizontal displacement of about 9 inches for a yield acceleration (k_y) of 0.21g. We recommended a horizontal displacement value of 2 feet to be conservative.

Note that for a previous Foundation Investigation (Taber, 1997) for this structure, horizontal displacement of the west bank was estimated at 1-2 feet for a k_y of 0.22g.

DRAFT REVIEW COMMENTS ON FAULT RUPTURE HAZARD ANALYSIS

Caltrans Comment #1

You list three traces of the fault mapped near the bridge

1. 1906 rupture (253 m west of bridge)
2. "historically active trace" (122m east of bridge)
3. LiDAR feature (31 m east of bridge). *[BCI note - error in listed distance]*

In the spreadsheet input, you have three different traces:

1. 1906 rupture, 830 feet (253 m) west of the bridge, weighted 70%
2. Secondary trace from LiDAR imagery 256 feet (81 m) east of the bridge, weighted 15%
3. Inferred tertiary trace at the bridge site (15%)

These do not match up.

Each trace that is used as input should be documented on figures and with field work. For each trace, describe:

1. Quality of mapping. Accurate, Approximate, Concealed, Inferred is the CGS standard, and applies to the spreadsheet. This also can be described qualitatively, by knowing the method by which trace was mapped (such as subsurface investigation, historical surface rupture, air photo interpretation).
2. Scale at which the fault was originally mapped, and a qualitative assessment of whether the mapped fault is likely to reflect the real fault accurately. For instance, the USGS Quaternary fault and fold database and map is a national map, and may or may not represent faults accurately at the scale of your bridge.
3. Whether fault features are visible in field immediately at the bridge, and how these were used to find an accurate site-to-source distance. Use air photos and LiDAR and field observations.
4. Relative amount of slip that you believe has occurred on each strand. The weight of each strand should be assigned based on the percentage of slip that you believe occurs on each strand. Use geomorphology, quality of mapping, and any paleoseismic studies to make this determination. See Petersen et al. (2011) for a brief summary of rupture distribution in this model.

BCI Response

To clarify our measurements and interpretation, we attach a table with the requested information. Hard field measurements were not made. This is expected to have no impact on the calculated hazard for the 1906 trace (west of the bridge) but may increase or decrease the hazard slightly for the trace located east of the bridge.

Caltrans Comment #2

How did you measure site-to-source distance? You visited the site, did you pace off or measure the distance from the closest point of the bridge to some of the closer features you describe? Usually the features can be located and measured in the field or on air photos. Be aware that Google Earth is very unreliable for measuring distances in a small area. Once a fault strand lies within 100 meters or so of this very active fault, the distance will affect the calculated hazard, so the most accurate possible distance is desirable.

BCI Response

Acknowledged. As indicated above, hard field measurements were not made. Final measurements between the bridge and the fault trace are from LiDAR data. An additional site visit would be required to field measure the distances.

Caltrans Comment #3

For the trace of the fault you identified on LiDAR, provide a figure documenting it, and discuss the geomorphology of the features. Is this trace the same as another trace mapped with less accuracy?

BCI Response

See the attached LiDAR map with the fault traces plotted. We interpret the eastern trace to be the same as the approximately located eastern trace shown on the Special Studies Zone map; the LiDAR data provides more detail (scale and geomorphic expression) for accuracy in fault placement. Geomorphic expression of the fault includes a linear channel portion and possible degraded scarps on the higher bench east of the bridge and further downstream.

Caltrans Comment 4

Your report states: "Considering the lack of site specific data, a probabilistic displacement should be used."

The purpose of calculating probabilistic displacement is not to account for location uncertainty. Location uncertainty is accounted for elsewhere in the spreadsheet. Probabilistic displacement calculation incorporates slip rates on the fault into predicted offset within the design life of the structure. Since the SAF has a very high slip rate, the probabilistic displacement will be higher and will be more accurately reflect what could happen at the bridge in the design life.

Account for location uncertainty in the spreadsheet inputs. The map ratings accurate (1), approximate (2), concealed (3) or inferred (4) should be used as inputs.

BCI Response

Acknowledged. Location uncertainty was accounted for as indicated.

Caltrans Comment #5

You assign a 15% weight to a random fault strand located within the bridge footprint. Read Petersen, et al., for a discussion of the methodology. The methodology produces a probability density function for displacement on and near mapped faults, depending on the mapping accuracy. The methodology already accounts for the probability of off-fault displacement. If you have reason to believe there is a concealed fault strand within the bridge footprint, document this with figures and a discussion of geomorphology or whatever methods were used to define this potential strand.

While I can't replicate your calculated offset because I'm not sure exactly which faults and distances you used, it is clear that adding this inferred trace raises the calculated rupture hazard at the site significantly. The site lies far away enough from both the 1906 trace and the secondary trace on the AP map that calculated offset at the site from just these strands could be much less than you report. It's possible that documenting the fault strands you use for input, and working to refine the site-to-source distances could result in a very different, and much lower, offset hazard at the bridge.

BCI Response

The inferred fault is a separate strand considered to have a high potential to occur at the bridge site. Its location through the bridge site is based on:

- LiDAR lineaments both upstream and downstream of the site (primarily linear channel bank; see the attached figure).
- The location between two mapped active strands within the San Andreas Fault Zone (and, therefore, a high potential for an active trace in between)
- Overall linear river channel in this area.
- A significant change in depth to rock across the channel (see BCI Borings 11-01 and 11-02). While not indicative of faulting on its own, it does support the potential for another strand at the bridge.
- Consideration that the active channel will obscure recent fault features.

While the inferred fault strand is supported by geomorphology both upstream and downstream, inference of a somewhat random strand adjacent to a highly active fault (such as the San Andreas) is supported by other Caltrans Fault Rupture Potential evaluations and appears to be the accepted practice (for examples see: Frazier Mountain UC, Bridge No. 53-1776L/R, 2013; Highland Avenue OC, Bridge No. 54-1086, 2013; Gateway Drive UC, Bridge No. 35-0192, 2013; Alder Creek Bridge, Bridge No. 10-0116, 2013).

Based on the available site information and the bridge location, it is prudent to include the inferred fault through the bridge site. The distribution of hazard between the various fault traces is very qualitative since there are no available fault studies in this area. Considering the complex nature of the San Andreas Fault and the potential for several active breaks, the potential for a break other than the 1906 trace is high. We reviewed our previous preliminary weighting

contributions listed the PFR and made some weighting adjustments. We assigned a 15% contribution from the AP/LiDAR trace east of the bridge, a 10% contribution from the inferred trace at the bridge, and a 75% contribution from the 1906 trace. With the adjusted weighting, the revised combined fault displacement was calculated at 4.0 feet (previously 4.3 feet in PFR). We attach the probabilistic offset graph for reference.

Preliminary Combined Design Spectrum (5% Damping)

Eureka Hill Road Bridge over Garcia River

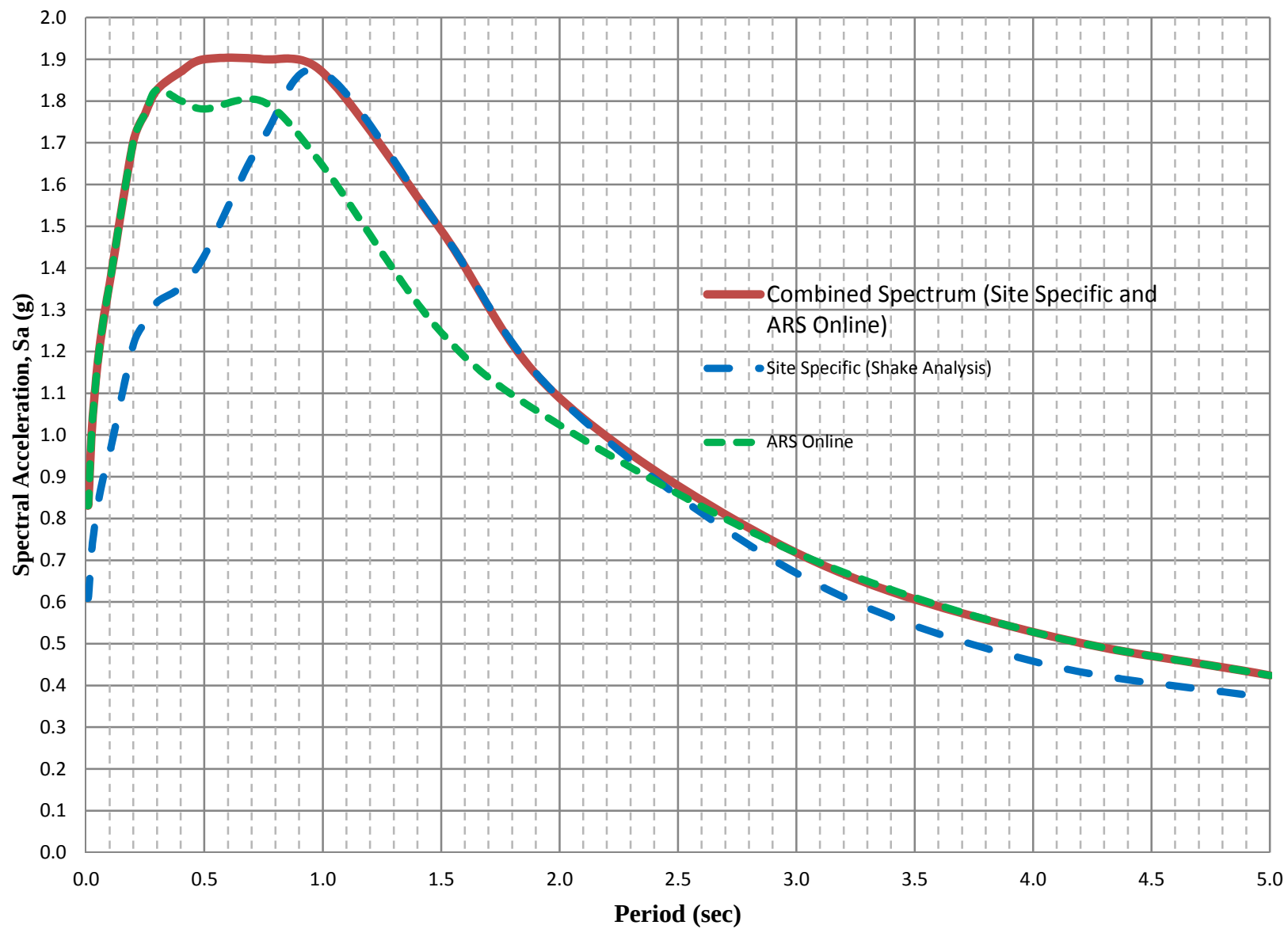


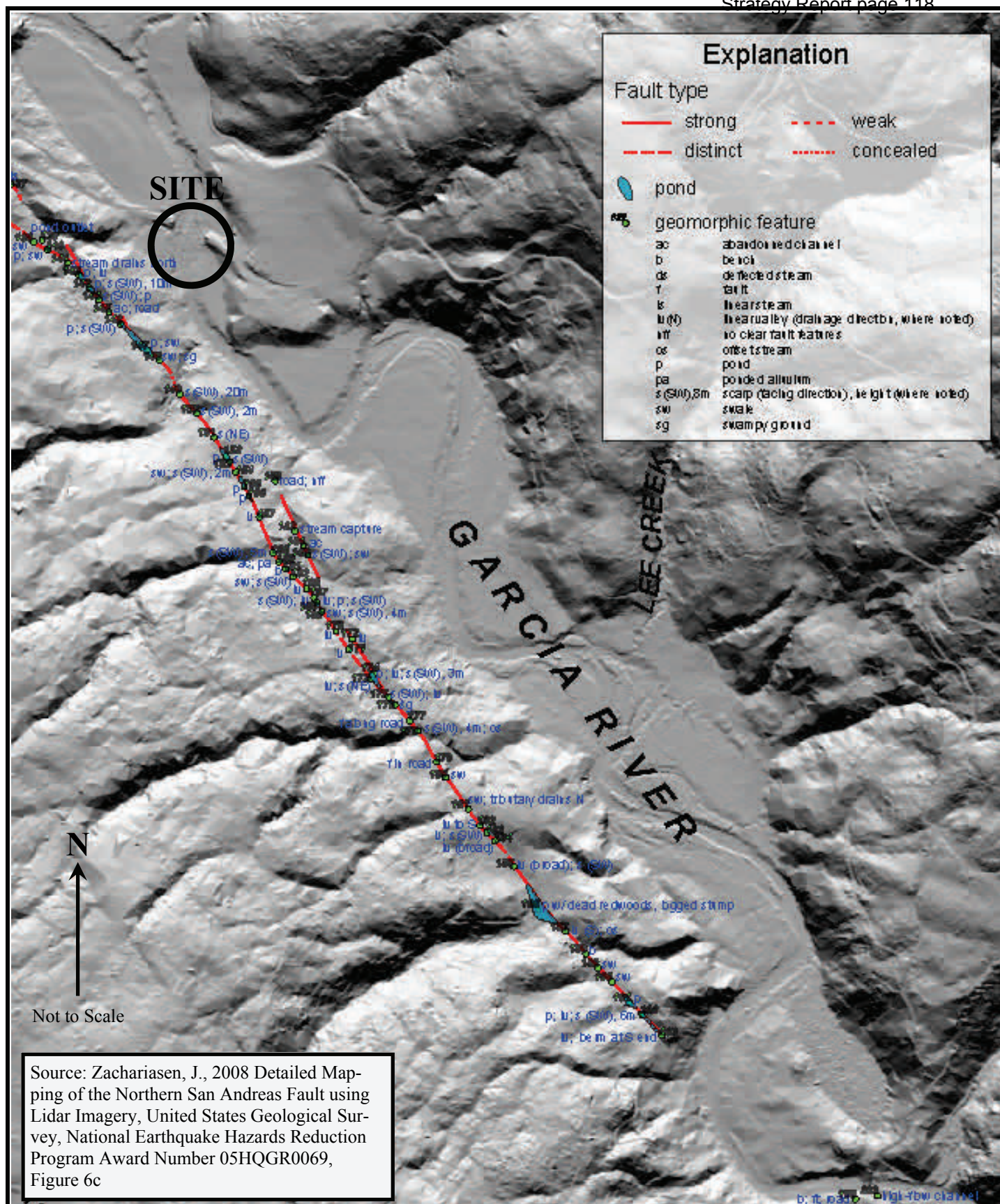
TABLE 1: Fault Traces used in Rupture Potential Analysis

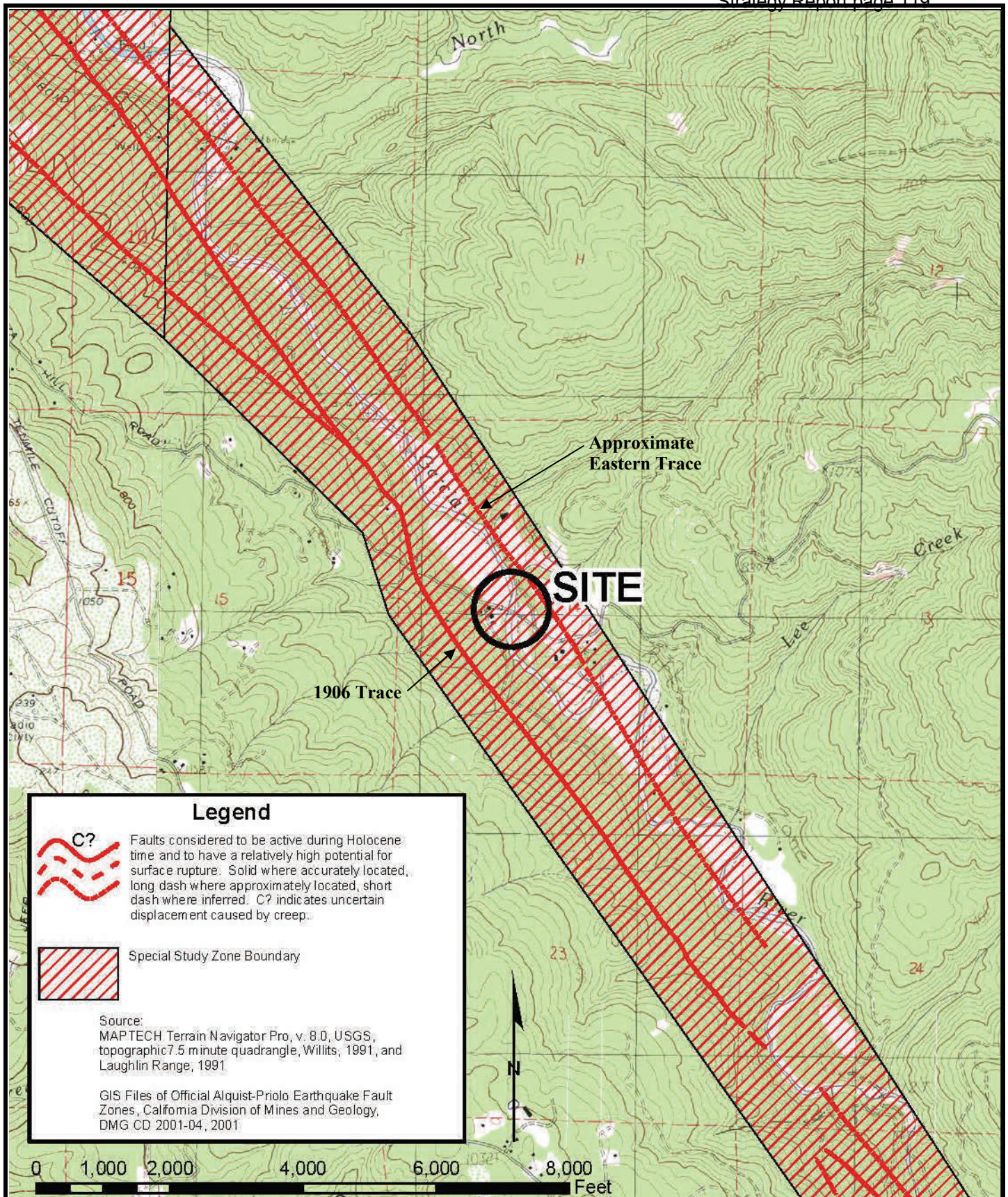
Trace Description	Reference	Quality of Mapping	Scale At Which Fault Was Mapped	Distance to Fault from Bridge (ft)	Observation and Measurement Description	Relative Amount of Slip and Qualitative Basis
1906 trace	J. Zachariasen, 2008	Accurate	1:10,000	830	Observable, not field measured. Measured in GIS from mapped 1906 trace	75%- most recent well defined rupture location
AP Trace/ LiDAR identified Trace	AP Map, California Geological Survey, 1974/LiDAR, USGS, 2003	Approximate	1:24,000	265	Observable, not field measured. Measured in GIS image from lineament feature (interpreted fault scarp) east of bridge near mapped AP fault strand	15%- interpreted as older fault rupture; based on mapping and geomorphology
LiDAR Inferred Trace at Bridge	LiDAR, USGS, 2003	Inferred	Resolution "few meters" variable	0	Not observed or measured. Fault inferred from lineaments and likely fault strands concealed in the active channel	10%- interpreted as older fault rupture based on geomorphology

Fault Data from Caltrans 2013 Fault Database:

- Fault = San Andreas (North Coast)
- Mmax = 8
- Slip Rate = 24 mm/yr

Aseismic Slip Factor = 0





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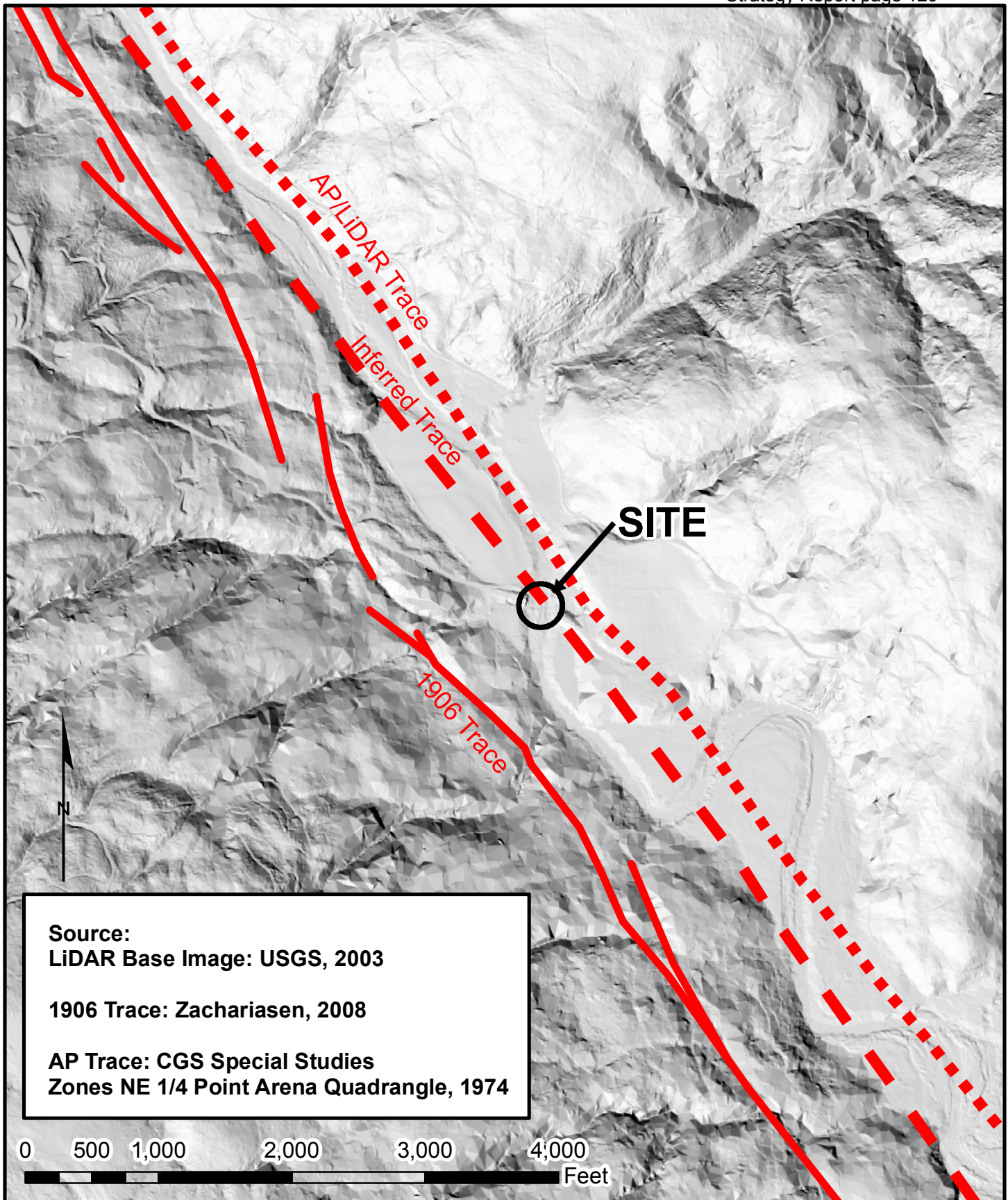
MAP OF ALQUIST PRIOLO SPECIAL STUDY ZONE MAP

Garcia River Bridge (Retrofit)
Bridge No. 10C-0034
Mendocino County, California

File No. 1615.1

November 2014

Figure



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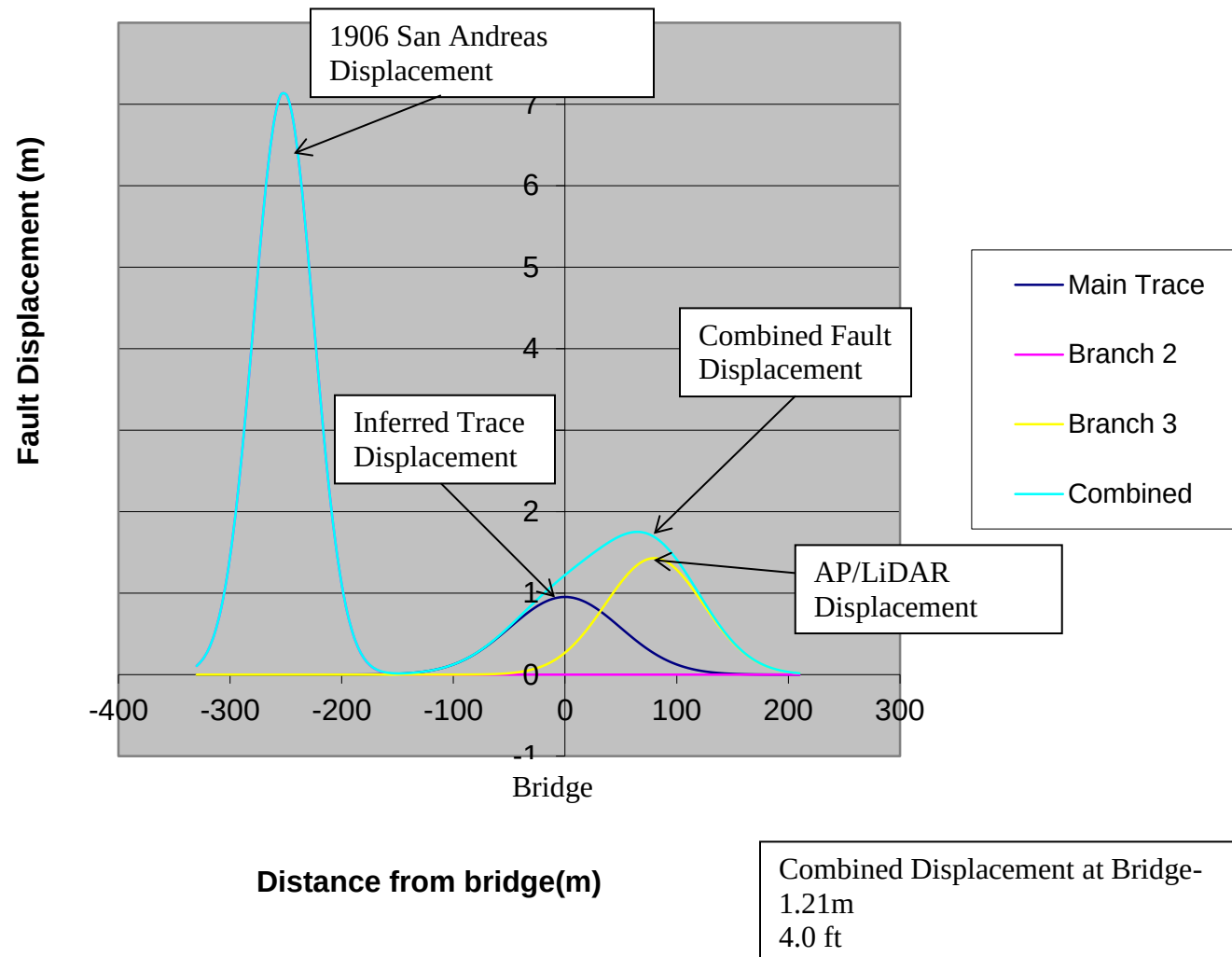
USGS LIDAR IMAGE WITH FAULT TRACES

Garcia River Bridge (Retrofit)
Bridge No. 10C-0034
Mendocino County, California

File No. 1615.1

November 2014

Figure



APPENDIX 7

FAULT RUPTURE REVIEW

West Sacramento Office:
2491 Boatman Ave. ▪ West Sacramento, CA 95691
(916) 375-8706 ▪ Fax (916) 375-8709



Main Auburn Office: (530) 887-1494
Fresno Office: (559) 438-8411
Modesto Office: (209) 522-6273

Geotechnical ▪ Geo-Environmental ▪ Construction Services ▪ Forensics

File No. 1615.1
February 17, 2015

Mr. Eric Fredrickson
Mark Thomas & Company, Inc.
7300 Folsom Boulevard, Suite 203
Sacramento, CA 95826

Subject: Fault Rupture Review
Addendum to the Preliminary Foundation Report
Garcia River Bridge (Retrofit), Bridge No. 10C-0034
Mendocino County, California

Dear Mr. Fredrickson,

Blackburn Consulting (BCI) prepared a Preliminary Foundation Report (PFR) dated June 27, 2014 for use by Mark Thomas & Company, Inc. (MTCO) to develop an updated seismic retrofit strategy for the Garcia River Bridge at Eureka Hill Road in Mendocino County, California.

BCI received Caltrans' draft PFR review comments and previously responded to those comments (see our Response to Comments dated 11/7/2014). In follow-up, we further discussed the fault rupture component with Caltrans; based on our discussions and Caltrans' current approach when using their displacement hazard spreadsheet, we provide the revised fault rupture criteria included below.

The primary modification to our analysis is removal of the inferred fault at the bridge site. We previously included an inferred trace based primarily on: 1) our interpretation of geomorphic expression of possible faulting through the bridge site and, 2) past practice by Caltrans to infer a somewhat random strand adjacent to highly active faults (such as the San Andreas) in other Caltrans Fault Rupture Potential evaluations.

Based on our discussions with Caltrans and Judith Zachariasen who performed detailed mapping of the northern San Andreas¹, the presence of an "active" fault trace at the bridge site is not supported by existing data. Rupture would most likely occur along the mapped 1906 trace

¹ Zachariasen, J., 2008, Detailed Mapping of the Northern San Andreas Fault Using LIDAR Imagery

Fault Rupture Review, Addendum to Preliminary Foundation Report

Garcia River Bridge (Retrofit), Bridge No. 10C-0034,

Mendocino County, California

BCI File No. 1615.1

February 17, 2015

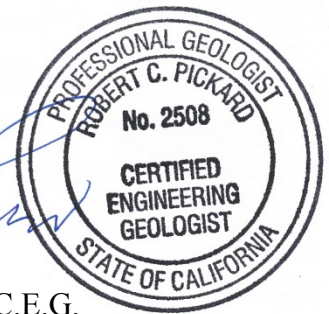
located west of the bridge site. Caltrans also indicated that the methodology of Peterson² (used in the Caltrans spreadsheet for evaluation of fault displacement hazard) incorporates off-fault displacement and that it is not necessary to place an inferred strand at the bridge site when the active trace is well-located. There may be past shearing at the bridge site upon which sympathetic rupture could occur; inclusion of the eastern trace (from the Alquist Priolo Special Study Zone (AP) mapping) in our analysis covers this and results in potential displacement at the bridge site.

Maps that show the fault traces used in our analysis are attached. Table 1 (attached) presents fault information included in our analysis. Based on the available site information, we weight the contribution from the 1906 trace at 85% and 15% for the AP trace. Our analysis results in potential fault displacement at the bridge site of approximately 0.25 meters (0.82 feet) as shown on the attached graph.

We expect the information presented in our Response to Comments (11/7/2014) and in this addendum to be included in the Foundation Design Report when that task is authorized.

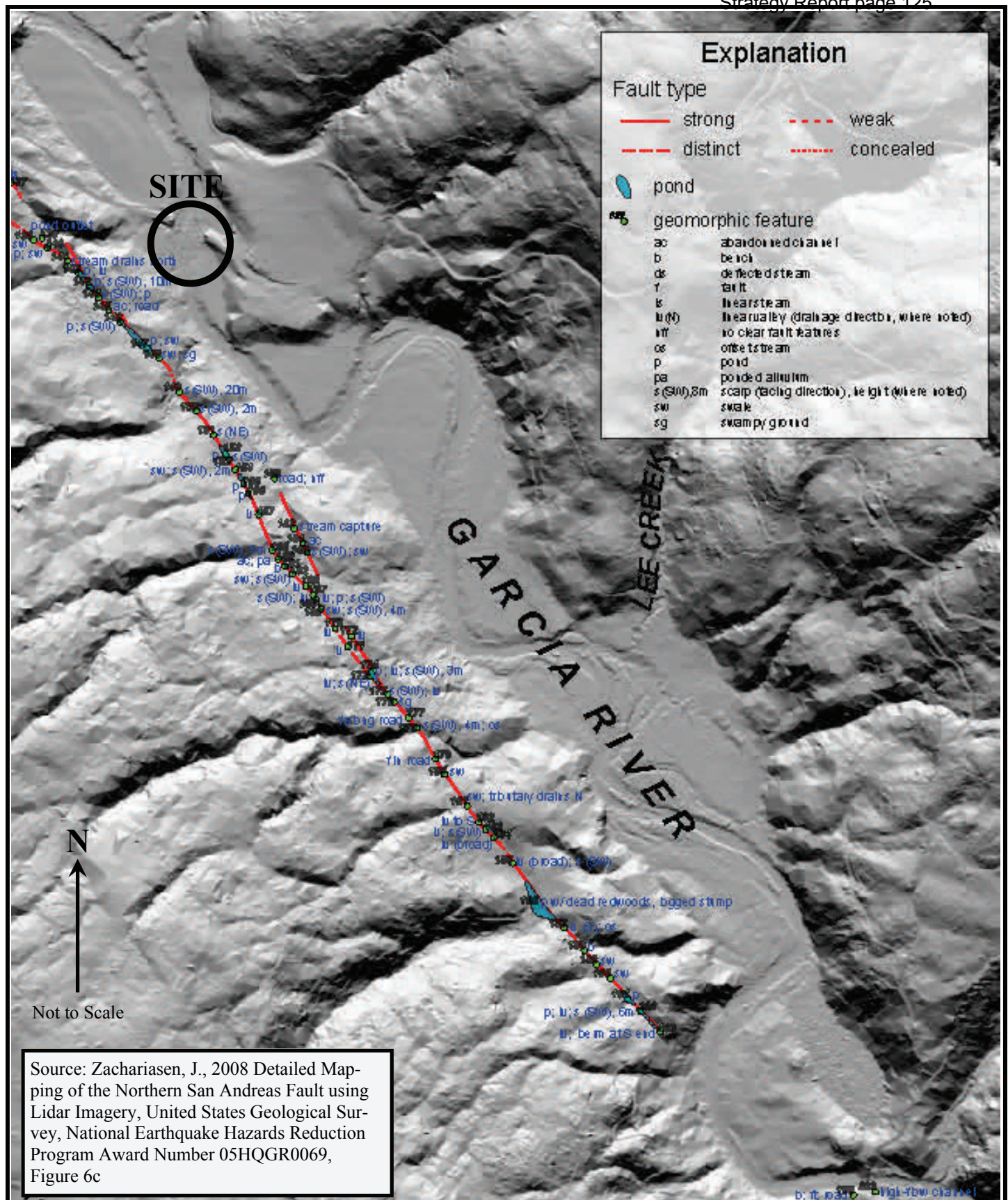
Please call if you have questions or require additional information.

Sincerely,

BLACKBURN CONSULTING
Patrick Fischer, C.E.G., C.E.
Engineering Geologist, Principal
Rob Pickard, P.G., C.E.G.
Senior Engineering Geologist

Attached: Map of 1906 Fault Trace (from Figure 6c, Zachariasen, 2008)
 Alquist Priolo Special Study Zone Map
 USGS LIDAR Image with Fault Traces
 Table 1 - Fault Traces Used in Rupture Potential Analysis
 Fault Rupture Displacement Estimation (Probabilistic Offset Graph)

² Petersen, M.D., et al, 2011, Fault Displacement Hazard for Strike-Slip Faults; Bulletin of the Seismological Society of America, Vol. 101, No. 2, pp. 805-825.



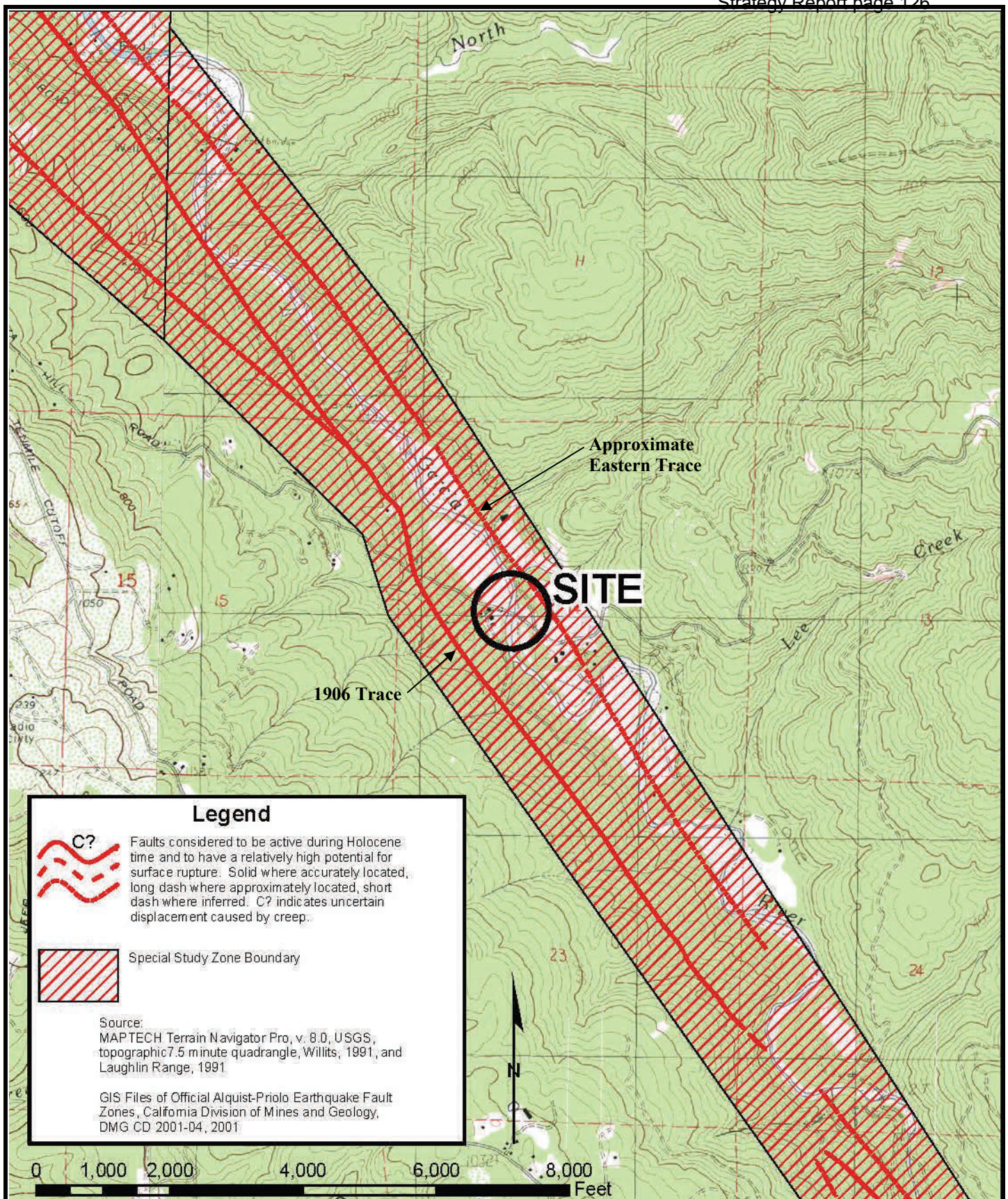
MAP OF 1906 FAULT TRACE

Garcia River Bridge (Retrofit)
Bridge No. 10C-0034
Mendocino County, California

File No. 1615.1

February 2015

Figure



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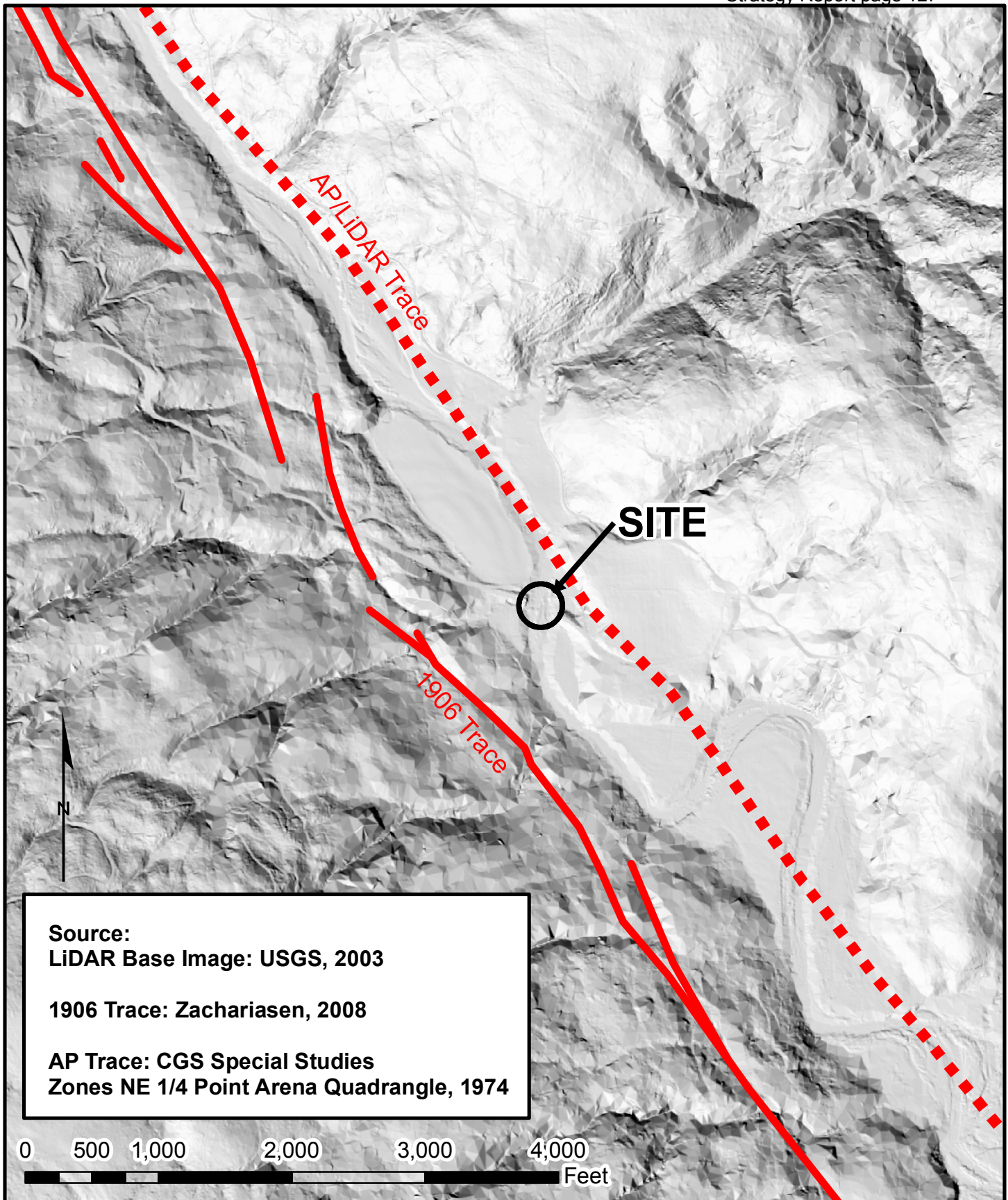
MAP OF ALQUIST PRIOLO SPECIAL STUDY ZONE MAP

Garcia River Bridge (Retrofit)
Bridge No. 10C-0034
Mendocino County, California

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February 2015

Figure



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USGS LIDAR IMAGE WITH FAULT TRACES

Garcia River Bridge (Retrofit)
Bridge No. 10C-0034
Mendocino County, California

File No. 1615.1

February 2015

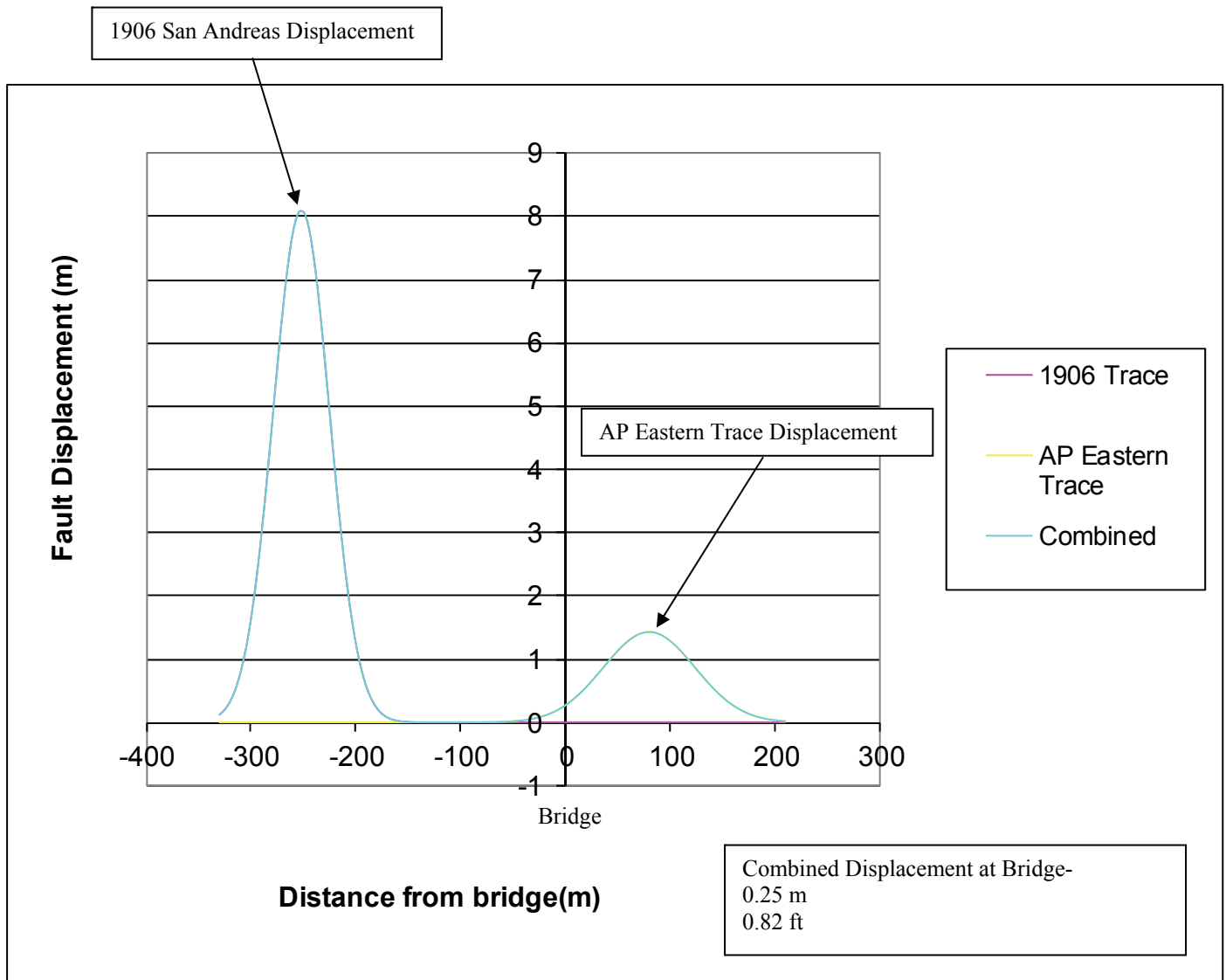
Figure

TABLE 1: Fault Traces used in Rupture Potential Analysis						
Trace Description	Reference	Quality of Mapping	Scale At Which Fault Was Mapped	Distance to Fault from Bridge (ft)	Observation and Measurement Description	Relative Amount of Slip and Qualitative Basis
1906 trace	J. Zachariasen, 2008	Accurate	1:10,000	830	Observable, not field measured. Measured in GIS from mapped 1906 trace	85% - most recent well defined rupture location
AP Trace/ LiDAR identified Trace	AP Map, California Geological Survey, 1974; LiDAR imagery USGS, 2003	Approximate	1:24,000	265	Observable, not field measured. Measured in GIS image from lineament feature (interpreted fault scarp) east of bridge near mapped AP fault trace	15% - interpreted as older fault rupture; based on mapping and geomorphology

Fault Data from Caltrans 2013 Fault Database:

- Fault = San Andreas (North Coast)
- Mmax = 8
- Slip Rate = 24 mm/yr

Aseismic Slip Factor = 0



APPENDIX 8

DRAFT LOCATION HYDRAULIC STUDY REPORT

Eureka Hill Road - Garcia River Bridge Retrofit
Mendocino County, California
Federal-Aid Project No. BRLRT-5910(041)
Existing Bridge No. 10C-0034

Draft Location Hydraulic Study Report



Prepared for:



Prepared by:



July 2013

**Eureka Hill Road - Garcia River Bridge Retrofit
Mendocino County, California
Federal-Aid Project No. BRLRT-5910(041)
Existing Bridge No. 10C-0034**

Draft Location Hydraulic Study Report

Submitted to:
Mendocino County

This report has been prepared by or under the supervision of the following Registered Engineer. The Registered Civil Engineer attests to the technical information contained herein and has judged the qualifications of any technical specialists providing engineering data upon which recommendations, conclusions, and decisions are based.

Han-Bin Liang, Ph.D., P.E.
Registered Civil Engineer

Date

July 2013

Table of Contents

Executive Summary	iii
Acronyms	v
Location Hydraulic Study Form	vi
Summary Floodplain Encroachment Report.....	ix
1 General Description	1
1.1 Project Need.....	1
1.2 Present Condition.....	1
1.3 Proposed Improvements.....	3
1.3.1 Retrofit Alternative 1.....	4
1.3.2 Retrofit Alternative 2.....	4
1.3.3 Selected Alternative.....	4
1.4 Regulatory Setting	10
1.4.1 Executive Order 11988.....	10
1.4.2 California's National Flood Insurance Program.....	10
1.5 Design Standards	11
1.5.1 FEMA Standards	11
1.5.2 Bridge Design Standards	11
1.6 Vertical Datum.....	11
2 Affected Environment.....	12
2.1 Geographic Location.....	12
2.2 Watershed Description.....	12
2.3 FEMA Floodplains.....	12
2.4 Traffic	12
2.5 Traffic Interruptions for Base Flood (Q_{100})	12
3 Environmental Consequences and Project Adverse Effects	15
3.1 Hydrologic Assessment	15
3.2 Hydraulic Assessment.....	16
3.3 Summary of Potential Encroachments.....	20
3.3.1 Potential Traffic Interruptions Due to the Proposed Action.....	20
3.3.2 Potential Adverse Effects on Natural and Beneficial Floodplain Values.....	20
3.3.3 Support of Probable Incompatible Floodplain Development	20
3.3.4 Longitudinal Encroachments	21
3.4 Risk Associated with the Proposed Action	21
4 Avoidance, Minimization, and/or Mitigation Measures	22
4.1 Minimize Adverse Effects to Floodplains	22
4.2 Restore and Preserve Natural and Beneficial Floodplain Values	22
4.3 Alternatives to Significant Encroachments.....	22
4.4 Alternatives to Longitudinal Encroachments.....	23
4.5 Coordination with Local, State, and Federal Water Resources and Floodplain Management Agencies.....	23
5 References.....	24

Figures

Figure 1. Project Location Map	1
Figure 2. Project Vicinity Map	2
Figure 3. Existing Bridge Elevation View.....	3
Figure 4. Retrofit Alternative 1 General Plan.....	6
Figure 5. Retrofit Alternative 2 General Plan.....	7
Figure 6. Abutment Retrofit Details	8
Figure 7. Elliptical Casing Detail for Bent Retrofit.....	9
Figure 8. Project Watershed Map	13
Figure 9. Floodplain Map of Project Vicinity.....	14
Figure 10. Location of Stream Gage.....	15
Figure 11. 100-Year Water Surface Comparison	17
Figure 12. Upstream Face of Existing Bridge, Looking Downstream (Northwest)	18
Figure 13. Upstream Face of Alternative 1 Bridge, Looking Downstream (Northwest) .	18
Figure 14. Upstream Face of Alternative 2 Bridge, Looking Downstream (Northwest) .	19

Tables

Table 1. Hydraulic Summary for the 100-Year Storm Event	16
---	----

Photos

Photo 1. Upstream Face of Bridge (Facing North)	3
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Appendices

Appendix A Federal Emergency Management Agency Flood Insurance Rate Map	
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Executive Summary

The Garcia River Bridge (Bridge No. 10C-0034) at Eureka Hill Road Retrofit Project (Project) is located in Mendocino County, California along County Road 505, at Mile Marker 4.92. The City of Point Arena is approximately 5 mi west of the Project location. The purpose of this study is to summarize the hydrology, compare the hydraulics for the existing and proposed bridge alternatives, and discuss the floodplain risk assessment for the Project.

The existing Garcia River bridge is a four-span, 320-ft long by 22 ft, 8 in.-wide structure that was built by the U.S. Army Corps of Engineers in 1969. The bridge consists of a reinforced concrete box girder superstructure supported by cast in steel shell piles, and the abutments are seat type abutments.

The Project includes seismic retrofits of the Eureka Hill Road bridge. The improvements will strengthen the structural components to resist seismic loads on the structure and loads imposed by soil movement. Support is provided at the ends of the superstructure near the abutments. Two alternatives were considered for the Project. Both alternatives include adding steel column casings to all bent columns and providing bumpers to the superstructure soffit. Alternative 1 also includes adding four 72-in. cast-in-drilled-hole (CIDH) piles at each end of the bridge, and Alternative 2 also includes remediation of in-situ soils by adding stone columns and compaction soil grouting at each end of the bridge. The Project has selected Alternative 1 as the preferred retrofit alternative, which is listed in the Seismic Retrofit Strategy Report prepared in 1998.

The 100-year peak discharge for Garcia River was estimated using a weighted flow estimate based on equations and gaging station data published in the U.S. Geological Survey's study for determining the magnitude and frequency of floods in California, *Methods for Determining Magnitude and Frequency of Floods in California, Based on Data through Water Year 2006* (Gotvald et. al. 2012). The 100-year flow at the Project site was estimated to be 27,990 cfs.

The existing and proposed conditions hydraulic models show that the 100-year water surface elevation does not overtop the bridge or the surrounding approach roadways, and no traffic interruptions are anticipated from the base flood. The existing bridge is located within a 100-year base floodplain, but the nature of the Project would not facilitate incompatible base floodplain development. Because the Project is proposing retrofit improvements, the proposed bridge will maintain the existing bridge alignment and profile. It will maintain local access and not create new access to developed land; therefore the Project will not support incompatible floodplain development. Because the proposed alignment is roughly perpendicular to the 100-year base floodplain, it would not be considered a longitudinal encroachment to the base floodplain.

Both alternatives will involve minimal encroachments on the base floodplain due to the widening of the bents associated with the steel casings and the addition of the bumpers to

the superstructure. Alternative 1 will include an additional encroachment from the CIDH piles. Alternative 2 will include an additional temporary encroachment due to the soil remediation work. However, both alternatives will have little effect on the water surface elevation for the base flood: Alternative 1 will result in no change and Alternative 2 will result in an increase of 0.01 ft. The overall risk associated with the Project is considered to be low.

Potential adverse effects to the natural and beneficial floodplain values include temporary loss of vegetation and potential effects to wildlife and fish species and habitats. Environmental impacts that could result from the construction activities could be minimized with measures such as best management practices, revegetation, boundary establishment for work around sensitive habitat, erosion control measures, or other requirements that are part of the Project's permit conditions.

Because the proposed condition would not significantly change the base flood elevation within the Project vicinity, a floodplain map revision and a Letter of Map Revision are not anticipated. Regulatory permits and approvals may be required from the U.S. Army Corps of Engineers, the California Department of Fish and Wildlife, the Regional Water Quality Control Board, and the Federal Emergency Management Agency as the Project enters into the final design phase.

Acronyms

ADT	Average Daily Traffic
BFE	Base Flood Elevation
BIR	Bridge Inspection Report
BMP	best management practice
Caltrans	California Department of Transportation
CFR	Code of Federal Regulations
CIDH	Cast-in-Drilled-Hole
CR	County Road
DWR	Department of Water Resources
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
HEC-RAS	Hydrologic Engineering Centers River Analysis System
NFIP	National Flood Insurance Program
NGVD 29	National Geodetic Vertical Datum of 1929
NOAA	National Oceanic and Atmospheric Administration
SFHA	Special Flood Hazard Area
USGS	United States Geological Survey

LOCATION HYDRAULIC STUDY FORM

Dist. 1 Co. Mendocino Rte. Eureka Hill Road P.M. EA: 01-280364
Federal-Aid Project Number: BRLRT-5910(041)

Floodplain Description:

The Project is located within FIRM Panel 060183 0975 B, effective June 1, 1983. The FIRM indicates that the Project site is within an area classified by FEMA as a SFHA Zone A, an area with a 1% annual chance of flooding with no depths or base flood elevations shown.

1. Description of Proposal *(include any physical barriers i.e. concrete barriers, soundwalls, etc. and design elements to minimize floodplain impacts)*
The Project proposes to retrofit the existing bridge, which is located along County Road 505 at Mile Marker 4.92 approximately 5 mi east of the City of Point Arena. Two alternatives were considered for the Project. Both alternatives include adding steel column casings to all bent columns and providing bumpers to the superstructure soffit. Alternative 1 also includes adding four 72-in. Cast-in-Drilled-Hole (CIDH) piles at each end of the bridge, and Alternative 2 also includes remediation of in-situ soils by adding stone columns and compaction soil grouting at each end of the bridge. The Project has selected Alternative 1 as the preferred retrofit alternative, which is listed in the Seismic Retrofit Strategy Report prepared in 1998.

2. ADT: Current (2011) 90 Projected (2027) 625

3. Hydraulic Data: Base Flood Q100= 27,990 CFS
WSE100= see table below *The flood of record, if greater than Q100:*
Q= N/A CFS WSE= N/A
Overtopping flood Q= 55,800 CFS WSE= 112 ft NGVD 29

Alternative	Water Surface Elevation (ft NGVD 29)
Existing	104.4
Alternative 1	104.4
Alternative 2	104.5

Are NFIP maps and studies available? NO YES ☒

4. Is the highway location alternative within a regulatory floodway ?
NO ☒ YES YES

5. Attach map with flood limits outlined showing all buildings or other improvements within the base floodplain.

Potential Q100 backwater damages:

A. Residences? NO ☒ YES YES
B. Other Bldgs? NO ☒ YES YES
C. Crops? NO ☒ YES YES
D. Natural and beneficial Floodplain values? NO ☒ YES YES

"Natural and beneficial flood-plain values" shall include but are not limited to fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, aquaculture, forestry, natural moderation of floods, water quality maintenance, and groundwater recharge.

6. Type of Traffic:

A. Emergency supply or evacuation route? NO ☒ YES YES
B. Emergency vehicle access? NO ☒ YES YES
C. Practicable detour available? NO ☒ YES YES
D. School bus or mail route? NO YES ☒

LOCATION HYDRAULIC STUDY FORM cont.

Dist. 1 Co. Mendocino Rte. Eureka Hill Road K.P. _____
Federal-Aid Project Number: BRLRT-5910(041)
EA 01-280364 Bridge No. 10C-0034

7. Estimated duration of traffic interruption for 100-year event hours: N/A

8. Estimated value of Q100 flood damages (if any) – moderate risk level.

A.	Roadway	\$ <u>N/A</u>
B	Property	\$ <u>N/A</u>
	Total	\$ <u>N/A</u>

9. Assessment of Level of Risk Low ✓
Moderate _____
High _____

For High Risk projects, during design phase, additional Design Study Risk Analysis may be necessary to determine design alternative.

LOCATION HYDRAULIC STUDY FORM cont.

Dist. 1 Co. Mendocino Rte. Eureka Hill Road K.P. _____
Federal-Aid Project Number: BRLRT-5910(041)
EA 01-280364 Bridge No. 10C-0034

PREPARED BY:

Signature:

I certify that I have conducted a Location Hydraulic Study consistent with 23 CFR 650 and that the information summarized in items numbers 3, 4, 5, 8, and 9 of this form is accurate.

District Hydraulic Engineer (capital and 'on' system projects) Date _____

Local Agency/Consulting Hydraulic Engineer (local assistance projects) Date _____

Is there any longitudinal encroachment, significant encroachment, or any support of incompatible Floodplain development? NO ✓ YES _____

If yes, provide evaluation and discussion of practicability of alternatives in accordance with 23 CFR 650.113

Information developed to comply with the Federal requirement for the Location Hydraulic Study shall be retained in the project files.

I certify that item numbers 1, 2, 6 and 7 of this Location Hydraulic Study Form are accurate and will ensure that Final PS&E reflects the information and recommendations of said report:

District Project Engineer (capital and 'on' system projects) Date _____

Local Agency Project Engineer (local assistance projects) Date _____

CONCURRED BY:

I have reviewed the quality and adequacy of the floodplain submittal consistent with the attached checklist, and concur that the submittal is adequate to meet the mandates of 23 CFR 650.

District Project Manager (capital and 'on' system projects) Date _____

Local Agency Project Manager (Local Assistance projects) Date _____

District Local Assistance Engineer (or District Hydraulic Branch for very complex projects or when required expertise is unavailable. Note: District Hydraulic Branch review of local assistance projects shall be based on reasonableness and concurrence with the information provided). Date _____

I concur that the natural and beneficial floodplain values are consistent with the results of other studies prepared pursuant to 23 CFR 771, and that the NEPA document or determination includes environmental mitigation consistent with the Floodplain analysis.

District Senior Environmental Planner (or Designee) Date _____

Note: If a significant floodplain encroachment is identified as a result of floodplains studies, FHWA will need to approve the encroachment and concur in the Only Practicable Alternative Finding.

SUMMARY FLOODPLAIN ENCROACHMENT REPORT

Dist. 1 Co. Mendocino Rte. Eureka Hill Road K.P. _____
Federal-Aid Project Number (Local Assistance) BRLRT-5910(041)
Project No.: _____ Bridge No. 10C-0034
Limits: The Project proposes to retrofit the existing bridge, which is located along County Road 505 at Mile Marker 4.92 approximately 5 mi east of the City of Point Arena.

Floodplain Description: The Project is located within FIRM Panel 060183 0975 B, effective June 1, 1983. The FIRM indicates that the Project site is within an area classified by FEMA as a SFHA Zone A, an area with a 1% annual chance of flooding with no depths or base flood elevations shown.

- | | No | Yes |
|---|----------|----------|
| 1. Is the proposed action a longitudinal encroachment of the base floodplain? | <u>✓</u> | _____ |
| 2. Are the risks associated with the implementation of the proposed action significant? | <u>✓</u> | _____ |
| 3. Will the proposed action support probable incompatible floodplain development? | <u>✓</u> | _____ |
| 4. Are there any significant impacts on natural and beneficial floodplain values? | <u>✓</u> | _____ |
| 5. Routine construction procedures are required to minimize impacts on the floodplain. Are there any special mitigation measures necessary to minimize impacts or restore and preserve natural and beneficial floodplain values? If yes, explain. | <u>✓</u> | _____ |
| 6. Does the proposed action constitute a significant floodplain encroachment as defined in 23 CFR, Section 650.105(q). | <u>✓</u> | _____ |
| 7. Are Location Hydraulic Studies that document the above answers on file? If not explain. | _____ | <u>✓</u> |

PREPARED BY:

_____ Date _____
District Project Engineer (capital and 'on' system projects)

_____ Date _____
Local Agency/Consulting Hydraulic Engineer (local assistance projects)

CONCURRED BY:

_____ Date _____
District Project Manager (capital and 'on' system projects)

_____ Date _____
District Local Assistance Engineer (Local Assistance projects)

I concur that impacts to natural and beneficial floodplain values are consistent with the results of other studies prepared pursuant to 23 CFR 771, and that the NEPA document or determination includes environmental mitigation consistent with the Floodplain analysis.

_____ Date _____
District Senior Environmental Planner (or Designee)

Note: If a significant floodplain encroachment is identified as a result of floodplains studies, FHWA will need to approve the encroachment and concur in the Only Practicable Alternative Finding.

1 GENERAL DESCRIPTION

The Garcia River Bridge at Eureka Hill Road Retrofit Project (Project) is located in Mendocino County, California along County Road (CR) 505, at Mile Marker 4.92. The City of Point Arena is approximately 5 mi west of the Project location. See Figure 1 for the Project Location Map and Figure 2 for the Project Vicinity Map.

The purpose of this study is to summarize the hydrology, compare the hydraulics for the existing and proposed bridge alternatives, and discuss the floodplain risk assessment for the Project.

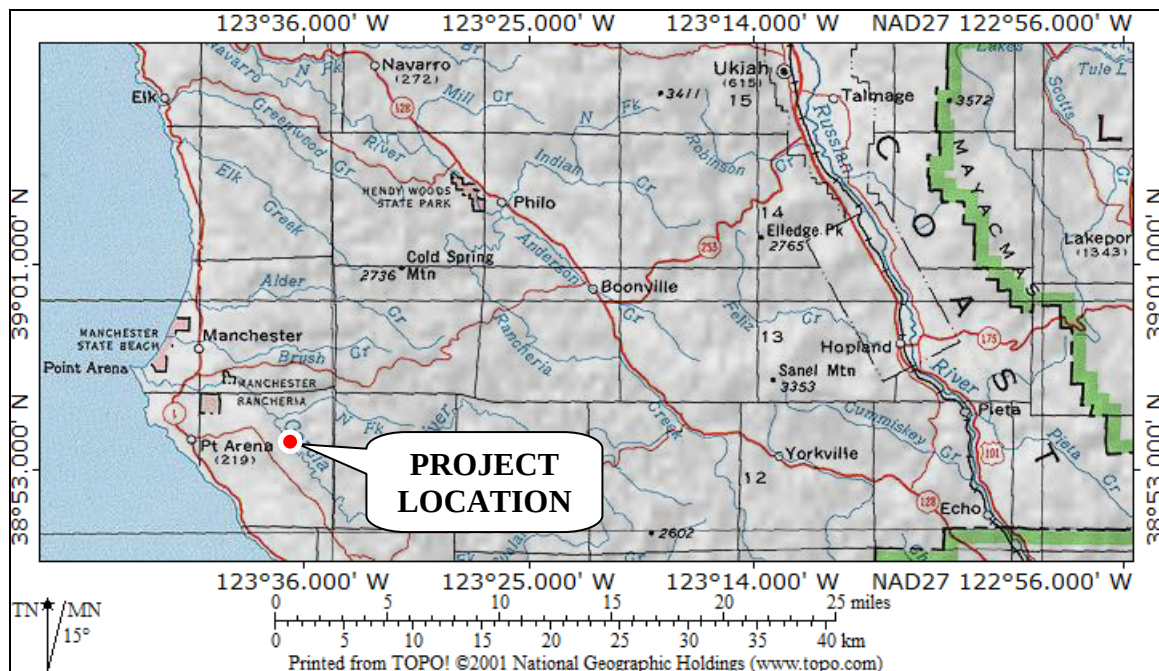


Figure 1. Project Location Map

Source: United States Geological Survey (USGS)

1.1 Project Need

The Eureka Hill Road bridge has been reviewed and the bridge foundations are subject to severe damage resulting from potential ground rupture, soil liquefaction, or slope failure during the design earthquake, and the bridge is eligible for funding for seismic rehabilitation.

1.2 Present Condition

The existing Garcia River bridge is a four-span, 320-ft long by 22 ft, 8 in.-wide structure that was built by the U.S. Army Corps of Engineers in 1969. Figure 3 shows the elevation view of the existing bridge, and Photo 1 shows the upstream face of the bridge facing north. The bridge consists of a reinforced concrete box girder superstructure supported by cast in steel shell piles, and the abutments are seat type abutments. Sacked concrete slope protection is in place in front of both abutments. The California

Department of Transportation (Caltrans) Bridge Inspection Report (BIR) issued in 2011 does not indicate any history of scour.

A Foundation Investigation prepared in 1997 summarized findings:

The Project is located in the San Andres Fault Zone. Active traces of the fault are mapped approximately 400 ft east and 1,000 ft west of the bridge. Preliminary evaluation of liquefaction potential concluded that:

- 1 to 2 ft of ground movement could occur at the west bank, affecting Abutment 1 and Pier 2;
- Movement in excess of 2 ft could occur at the east bank, affecting Pier 4 and Abutment 5;
- Channel soils are susceptible to erosion, scour, and liquefaction, affecting the lateral stability of piles at Pier 3.

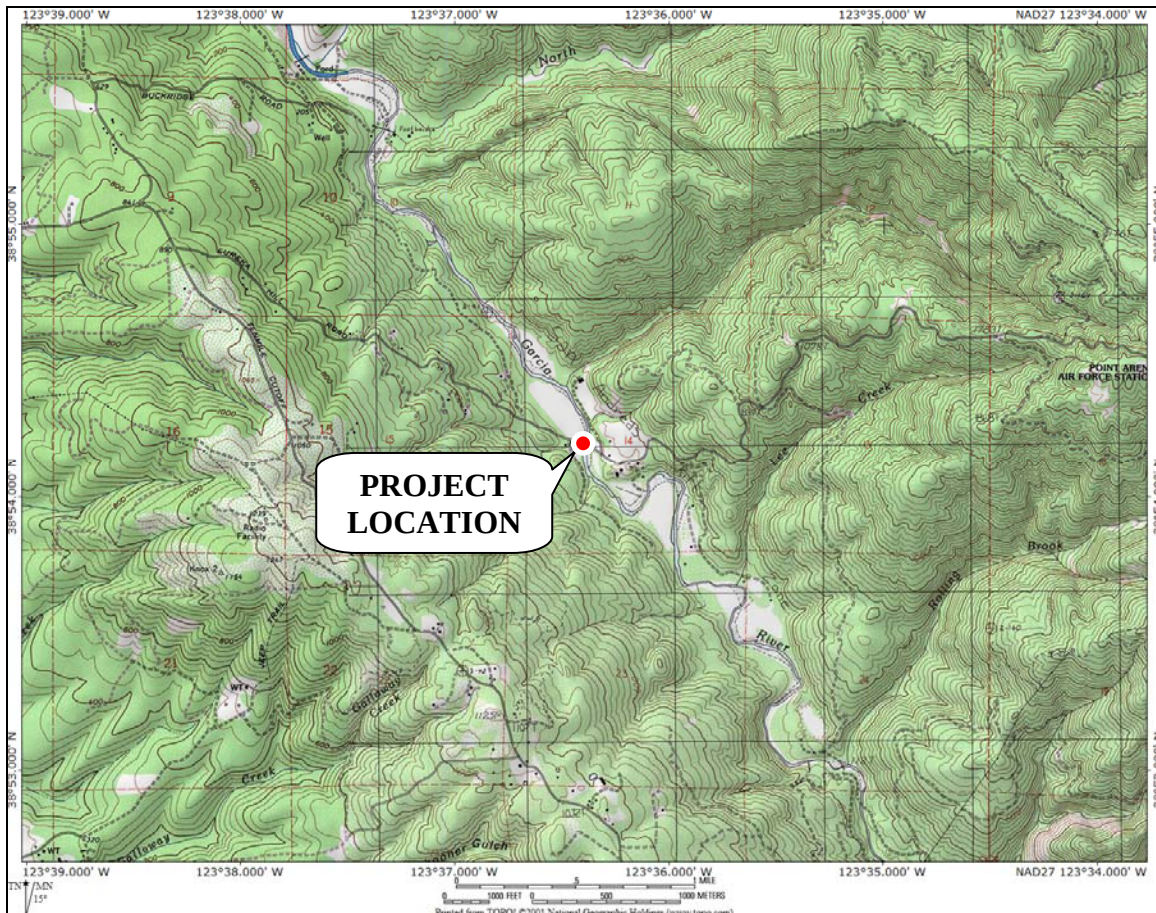


Figure 2. Project Vicinity Map

Source: USGS

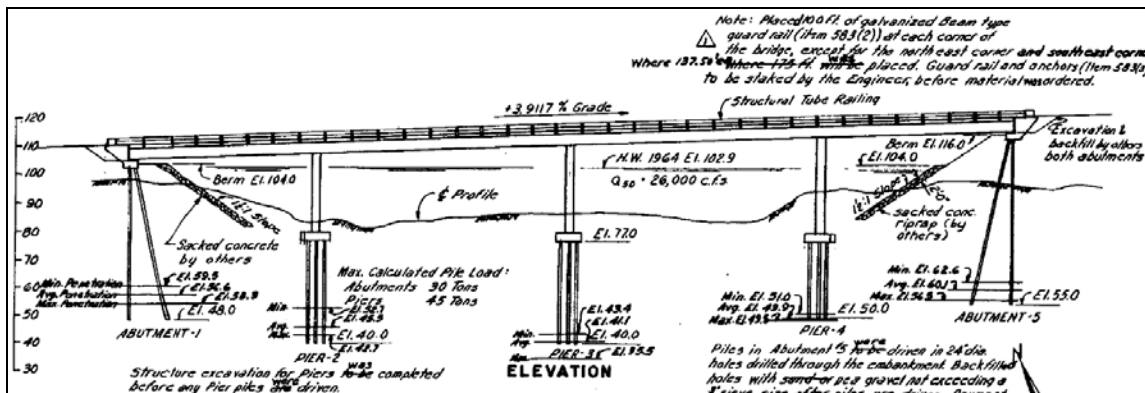


Figure 3. Existing Bridge Elevation View

Source: Caltrans



Photo 1. Upstream Face of Bridge (Facing North)

Source: Caltrans

1.3 Proposed Improvements

The Project includes seismic retrofits of the Eureka Hill Road bridge (Bridge No. 10C-0034) over the Garcia River in Mendocino County. The improvements will strengthen the structural components to resist seismic loads on the structure and loads imposed by soil movement. Support is provided at the ends of the superstructure near the abutments. Two alternatives are being considered. Both alternatives are described in the following sub-sections.

1.3.1 Retrofit Alternative 1

The following work will be performed for this alternative:

- Add steel column casings to all bent columns, and add piles and top mats of reinforcing to all bent foundations.
- Add four 72-in. cast-in-drilled-hole (CIDH) piles at each end of the bridge, two on each side, drilled to an elevation below the level of competent soil for the support of the superstructure in the event the soil supporting the abutments migrates into the channel during an earthquake.
- Provide concrete girders between the CIDH pile groups for vertical support of the superstructure.
- Add bumpers to the superstructure soffit.

See Figure 4 for the General Plan, Figure 6 for the abutment details, and Figure 7 for the bent/pier details.

1.3.2 Retrofit Alternative 2

The following work will be performed for this alternative:

- Add steel column casings to all bent columns, and add piles and top mats of reinforcing to all bent foundations.
- Add bumpers to the superstructure soffit.
- Remediate the in-situ soils by adding stone columns and compaction soil grouting at each end of the bridge. The areas under the superstructure will require compaction soil grouting.

See Figure 5 for the General Plan, Figure 6 for the abutment details, and Figure 7 for the bent/pier details.

1.3.3 Selected Alternative

The Project has selected Alternative 1 as the preferred retrofit alternative, which is listed in the Seismic Retrofit Strategy Report prepared in 1998.

These improvements will require temporary access to the streambed in order to construct column casings to the bent columns and top mats to the bent foundations. However, all temporary access will be removed once the seismic improvements have been completed, and the disturbed area will be restored to the pre-project conditions. The resulting spoils of CIDH piles will be disposed of outside of the public right-of-way, in accordance with the contract specifications.

As is standard with all roadway projects, the contractor will be required to install temporary best management practices (BMPs) to control any runoff or erosion from the Project site into the surrounding waterways. These temporary BMPs will be installed prior to any construction operations and will be in place for the duration of the contract.

The removal of these BMPs will be the final operation along with the Project site cleanup.

There is a telephone/fiber optic conduit attached to the bridge on the north side and a telephone line attached to the south side of the bridge. These utilities may be in conflict with the retrofit work near the abutments.

1.3.3.1 Construction Staging

In order to complete the needed seismic retrofit work, it is likely that the Eureka Hill Road bridge will need to be limited to one lane traffic control through the Project site for some construction activities, including drilling CIDH piles. There are limited east-west connections between State Route 1 and 128, and closing Eureka Hill Road and detouring traffic would equate to a 125-mile detour around the Project site.





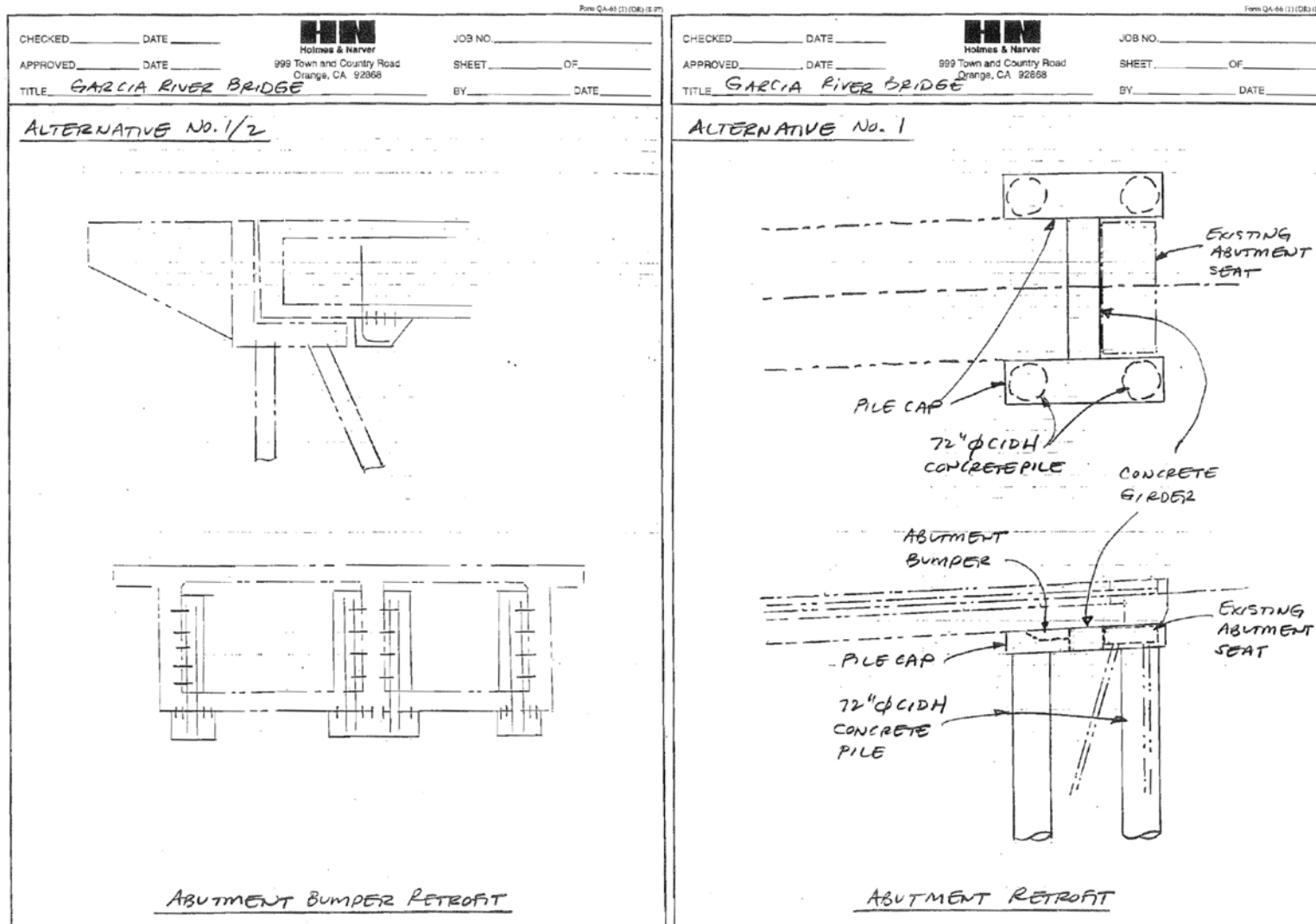


Figure 6. Abutment Retrofit Details

Source: Holmes and Narver, Inc. for Quincy Engineering, Inc.

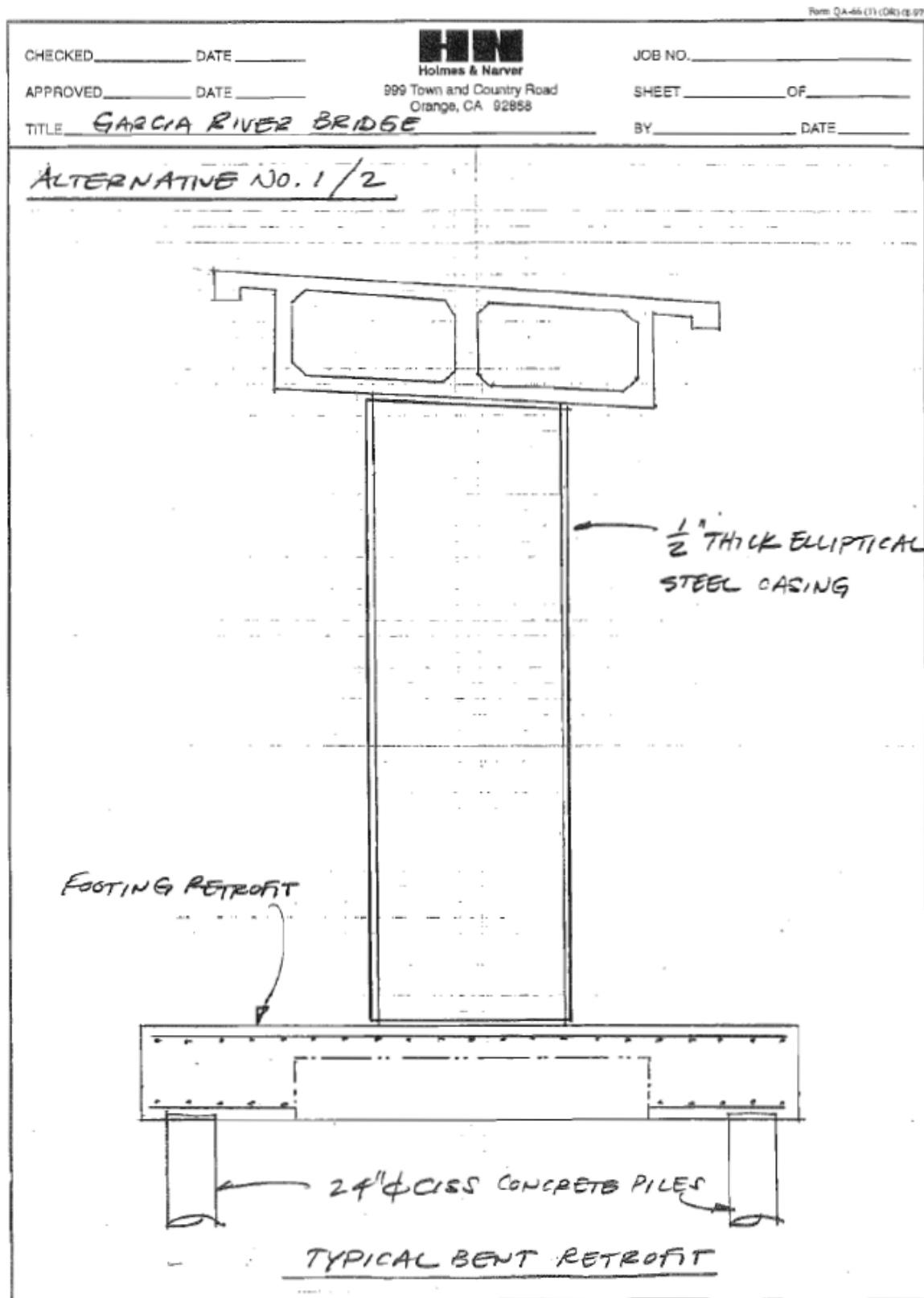


Figure 7. Elliptical Casing Detail for Bent Retrofit

Source: Holmes and Narver, Inc. for Quincy Engineering, Inc.

1.4 Regulatory Setting

1.4.1 Executive Order 11988

Executive Order 11988 (Floodplain Management) directs all federal agencies to avoid, to the extent possible, long- and short-term adverse impacts associated with the occupancy and modification of floodplains, and to avoid direct and indirect support of floodplain development wherever there is a practicable alternative. Requirements for compliance are outlined in Title 23, Code of Federal Regulations, Part 650, Subpart A (23 CFR 650A) titled “Location and Hydraulic Design of Encroachment on Floodplains.”

If the preferred alternative involves significant encroachment onto the floodplain, the final environmental document (final environmental impact statement or finding of no significant impact) must include:

- The reasons why the proposed action must be located in the floodplain.
- The alternatives considered and why they were not practicable, and
- A statement indicating whether the action conforms to applicable State or local floodplain protection standards.

1.4.2 California’s National Flood Insurance Program

The Federal Emergency Management Agency (FEMA) is the nationwide administrator of the National Flood Insurance Program (NFIP), which is a program that was established by the National Flood Insurance Act of 1968 to protect lives and property, and to reduce the financial burden of providing disaster assistance. Under the NFIP, FEMA has the lead responsibility for flood hazard assessment and mitigation, and it offers federally backed flood insurance to homeowners, renters, and business owners in communities that choose to participate in the program. FEMA has adopted the 100-year floodplain as the base flood standard for the NFIP. FEMA is also concerned with construction that would be within a 500-year floodplain for proposed projects that are considered “critical actions”, which are defined as any activities where even a slight chance of flooding is too great. FEMA issues the Flood Insurance Rate Maps (FIRMs) for communities that participate in the NFIP. These FIRMs present delineations of flood hazard zones.

In California, nearly all of the State’s flood-prone communities participate in the NFIP, which is locally administered by the California Department of Water Resources’ (DWR) Division of Flood Management. Under California’s NFIP, communities have a mutual agreement with the State and Federal government to regulate floodplain development according to certain criteria and standards, which is further detailed in the NFIP. Typically, each county (or community) has a Flood Insurance Study (FIS), which is used to locally develop FIRMs and Base Flood Elevations (BFE).

The FIS for Mendocino County, California Unincorporated Areas was researched for floodplain information. The summary of discharges table from the FIS (effective date June 16, 1992) for Mendocino County, California and Incorporated Areas does not list Garcia River.

1.5 Design Standards

1.5.1 FEMA Standards

FEMA standards are employed for design, construction, and regulation to reduce flood loss and to protect resources. Two types of standards are often employed: design criteria and performance standards.

A design criteria or specified standard dictates that a provision, practice, requirement, or limit be met; e.g. using the 1% flood and establishing floodway boundaries so as not to cause more than a 1-ft increase in flood stages.

A performance standard dictates that a goal is to be achieved, leaving it to the individual application as to how to achieve the goal; e.g. providing protection to the regulatory flood, keeping post-development stormwater runoff the same as pre-development, or maintaining the present quantity and quality of water in a wetland.

The 1% annual chance flood and floodplain have been adopted as a common design and regulatory standard in the United States. The NFIP adopted it in the early 1970s, and it was adopted as a standard for use by all federal agencies with the issuance of Executive Order 11988. States or local agencies are free to impose a more stringent standard within their jurisdiction.

1.5.2 Bridge Design Standards

The hydraulic design of the bridge should follow the Federal Highway Administration (FHWA) and Caltrans' criteria.

The FHWA criterion for the hydraulic design of bridges is that they be designed to pass the 2% probability of annual exceedance flow (50-year recurrence interval design discharge) with adequate freeboard, where practicable, to account for debris and bedload.

The Caltrans criteria for the hydraulic design of bridges is that they be designed to pass the 2% probability of annual exceedance flow (50-year design discharge) or the flood of record, whichever is greater, with adequate freeboard to pass anticipated drift. Two feet (2 ft) of freeboard is commonly used in preliminary bridge designs. The bridge should also be designed to pass the 1% probability of annual exceedance flow (100-year design discharge, or base flood). No freeboard is added to the base flood.

1.6 Vertical Datum

The Project references the National Geodetic Vertical Datum of 1929 (NGVD 29).

2 AFFECTED ENVIRONMENT

2.1 Geographic Location

The Project is in southeast Mendocino County, California on Eureka Hill Road at the crossing of the Garcia River, along County Road (CR) 505 at Mile Marker 4.92. The Project is located approximately 5 mi east of the City of Point Arena at latitude 38°54' North and longitude 123°36' West.

2.2 Watershed Description

The Project is located in the Mendocino Coast Hydrologic Unit, in the Garcia River Hydrologic Area of the North Coast Hydrologic Basin (North Coast Water Network 2011). The Project falls within the jurisdiction of the North Coast Regional Water Quality Control Board. The Garcia River watershed covers approximately 114 square mi in Mendocino County (National Oceanic and Atmospheric Administration [NOAA] 2012). The drainage for the Project site location is 83.1 square mi (see Figure 8 for the watershed delineation).

2.3 FEMA Floodplains

The Project site is located within FIRM panel number 0601830975B, effective June 1, 1983. The FIRM indicates that the Project site is within an area classified by FEMA as SFHA Zone A, which is an area with a 1% annual chance of flooding with no depths or BFEs shown.

2.4 Traffic

The average daily traffic (ADT) based on the Caltrans BIR (2011) for the Project was 90 in 2011, and it is projected to be 625 in 2027. Eureka Hill Road bridge is not used for emergency supply or evacuation and emergency vehicle access. There is no practicable detour available, although the construction will be staged to allow one lane traffic control. Eureka Hill Road bridge is used as a mail route. The proposed retrofit bridge will maintain these traffic uses.

2.5 Traffic Interruptions for Base Flood (Q_{100})

According to the existing conditions hydraulic model generated in support of this study, the 100-year water surface elevation does not overtop the Eureka Hill Road bridge over Garcia River. Therefore, traffic interruptions for the base flood are not anticipated. The potential traffic interruption for the proposed action is discussed in Section 3.3.1.

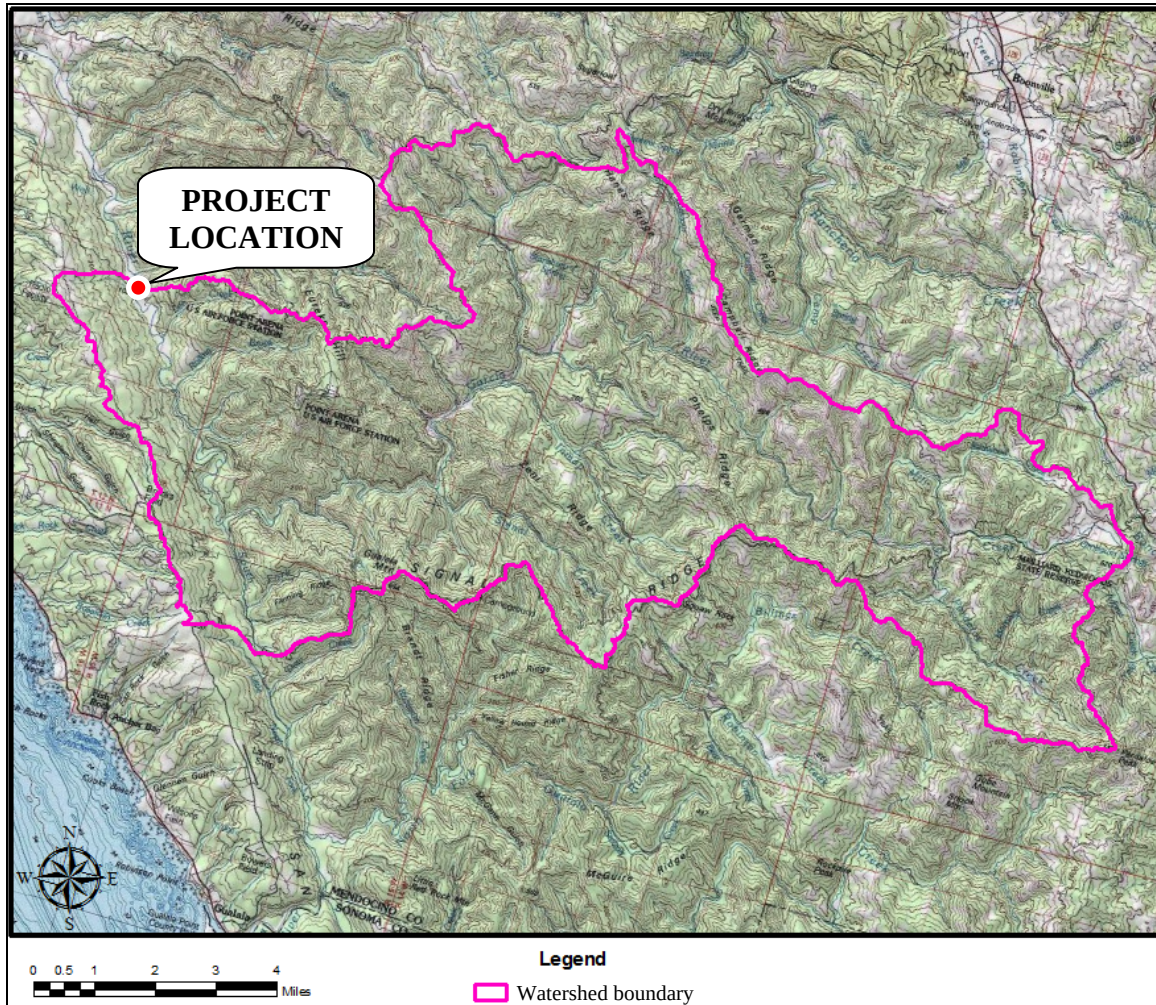


Figure 8. Project Watershed Map

Source: USGS

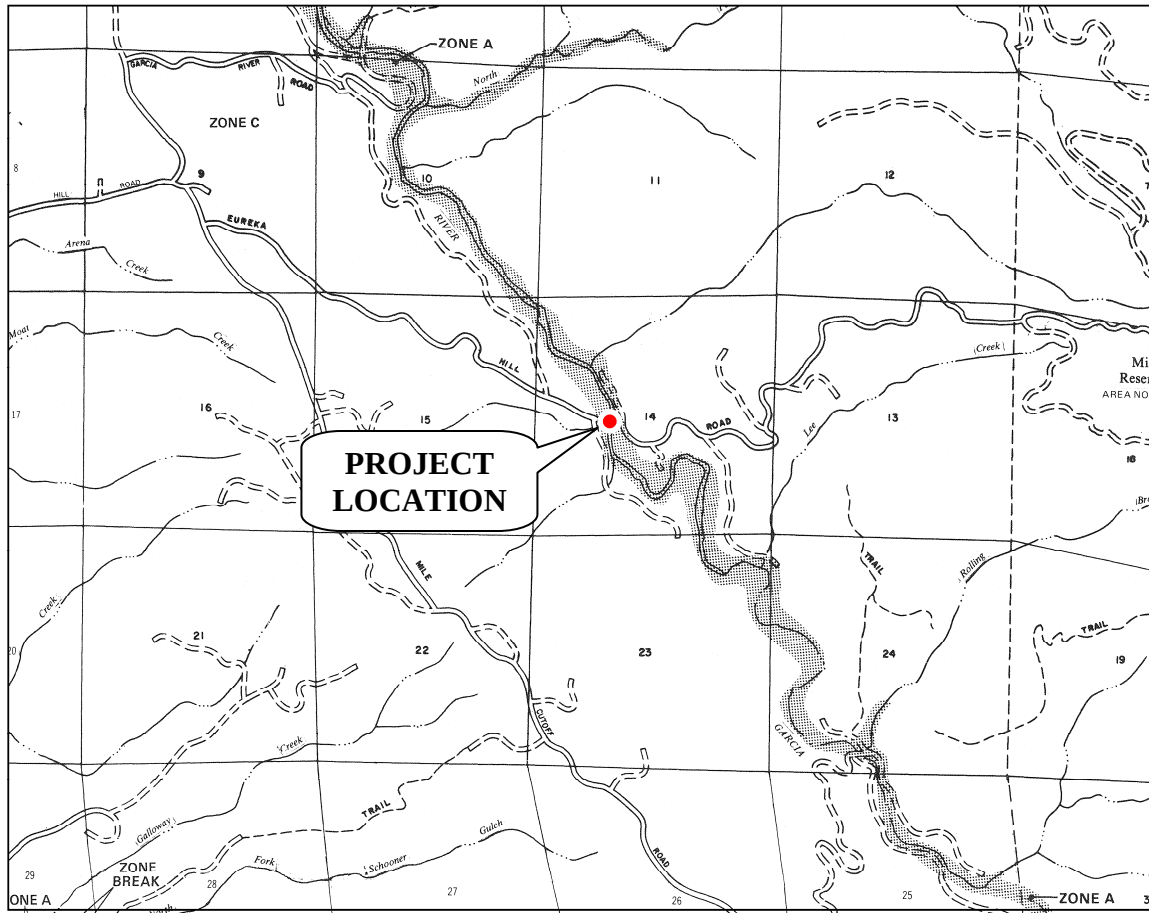


Figure 9. Floodplain Map of Project Vicinity

Source: FEMA

3 ENVIRONMENTAL CONSEQUENCES AND PROJECT ADVERSE EFFECTS

3.1 Hydrologic Assessment

The 100-year peak discharge for Garcia River was estimated using a weighted flow estimate based on equations and gaging station data published in the USGS's study for determining the magnitude and frequency of floods in California, *Methods for Determining Magnitude and Frequency of Floods in California, Based on Data through Water Year 2006* (Gotvald et. al. 2012). The peak discharge for USGS gage no. 11467600, which is located approximately 2.5 mi downstream (northwest) of the Project site (see Figure 10), was adjusted following the method to estimate the flow at an ungaged site near a stream gage. The 100-year flow at the Project site was estimated to be 27,990 cfs. The effective FIS for Mendocino County, California (Unincorporated Areas) had no published flow rates for Garcia River.

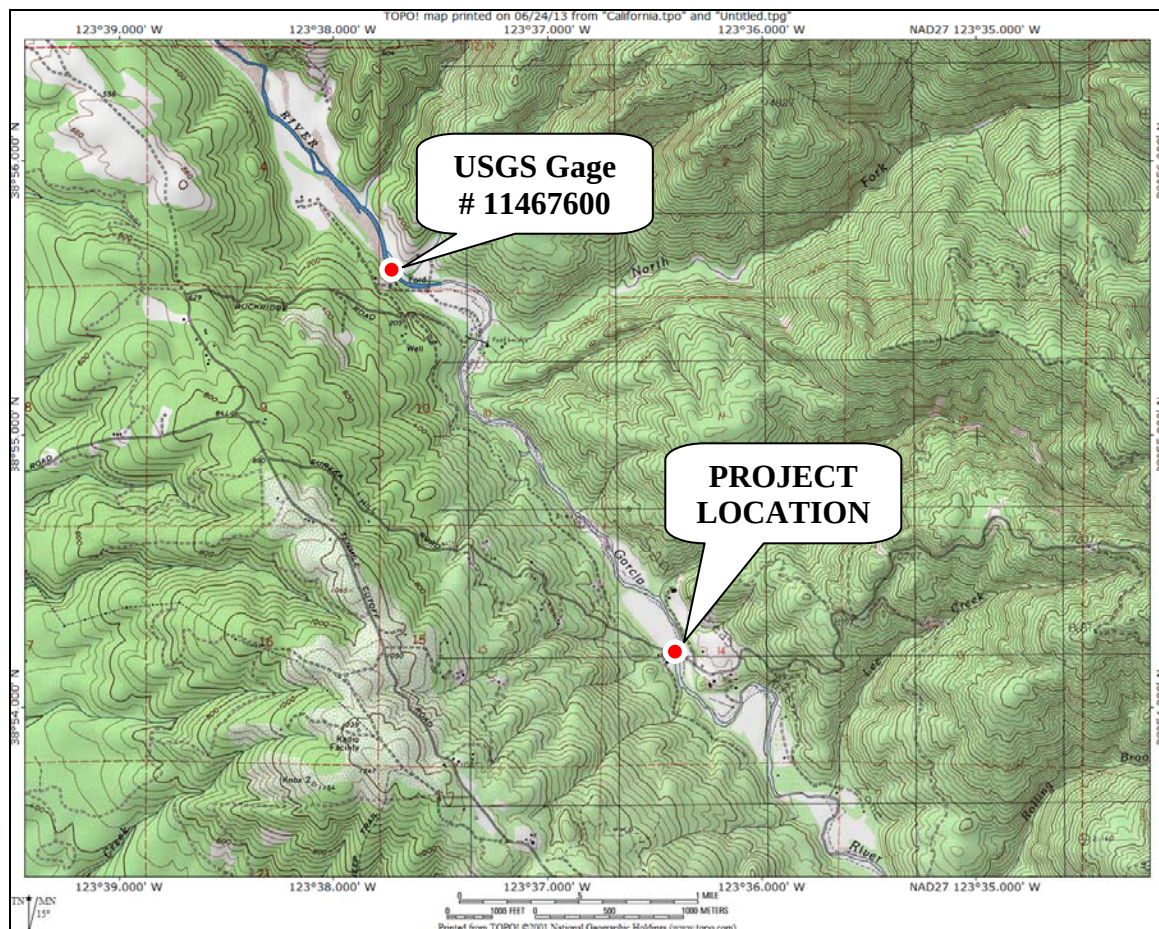


Figure 10. Location of Stream Gage

Source: USGS

3.2 Hydraulic Assessment

The hydraulic analyses were performed for the existing and proposed conditions. The hydraulic analyses were performed using the U.S. Army Corps of Engineers' Hydrologic Engineering Centers River Analysis System (HEC-RAS) modeling software Version 4.1.0.

The soffit elevations, water surface elevations, and average channel velocities at the upstream side of the bridges for the evaluated conditions are summarized in Table 1. The water surface elevation profiles along the studied stream reach are presented in Figure 11 for the existing and proposed alternatives. The cross sections at the upstream sides of the existing and proposed bridges are shown in Figure 12 and Figure 13.

Table 1. Hydraulic Summary for the 100-Year Storm Event

Alternative	Bridge Soffit Elevation (ft NGVD 29)	Water Surface Elevation (ft NGVD 29)	Average Channel Velocity (ft/s)
Existing	107.0	104.4	6.9
Alternative 1	107.6	104.4	6.9
Alternative 2	107.5	104.5	6.9

Alternative 1 would result in no change in water surface at the bridge, and Alternative 2 would result in an increase in water surface elevation of 0.01 ft at the bridge. The changes in water surface elevation for Alternative 2 primarily result from the slight changes to the bridge geometry due to the increase in pier width from the steel column casings and the added bumpers. Alternative 1 also includes the same geometry differences like Alternative 2, but with additional encroachment from the CIDH piles. This added encroachment results in a decrease in flow area, which increases the velocity slightly (0.07 ft/s), but is not enough to change the water surface elevation. For both alternatives, the steel casings only increase the width of each of the three bents by 1 in.

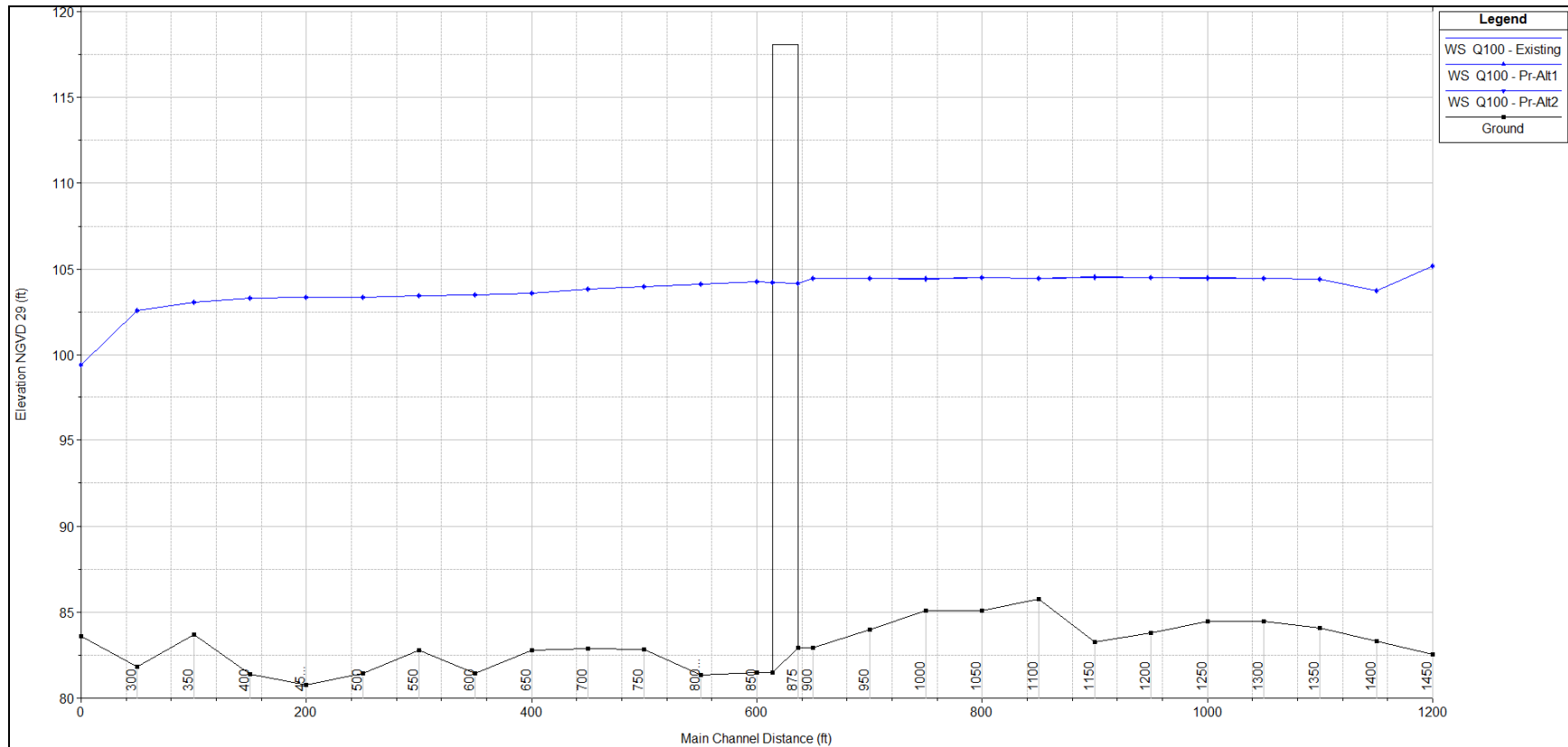


Figure 11. 100-Year Water Surface Comparison

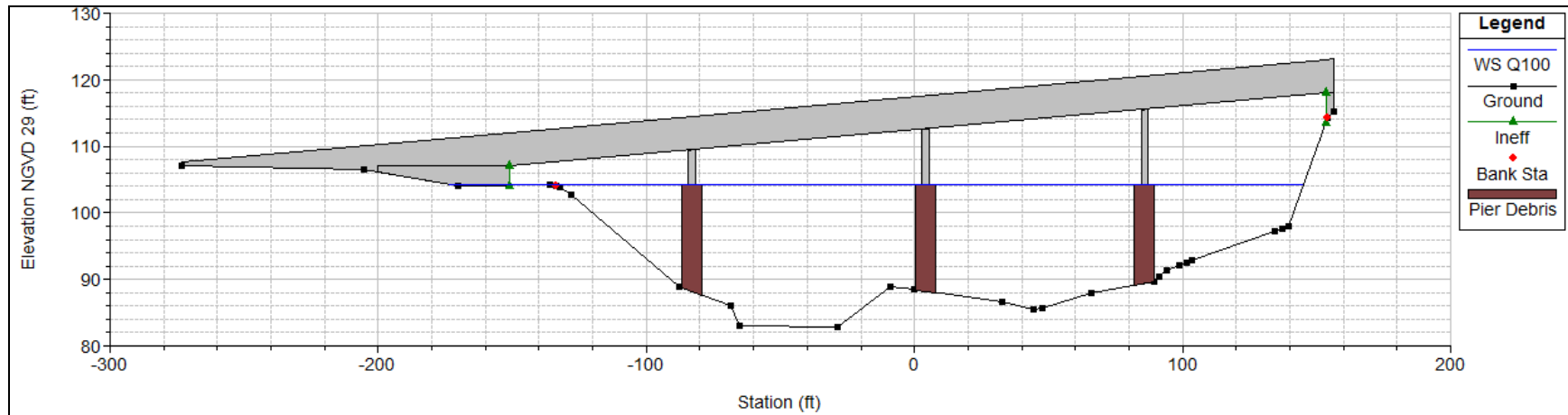


Figure 12. Upstream Face of Existing Bridge, Looking Downstream (Northwest)

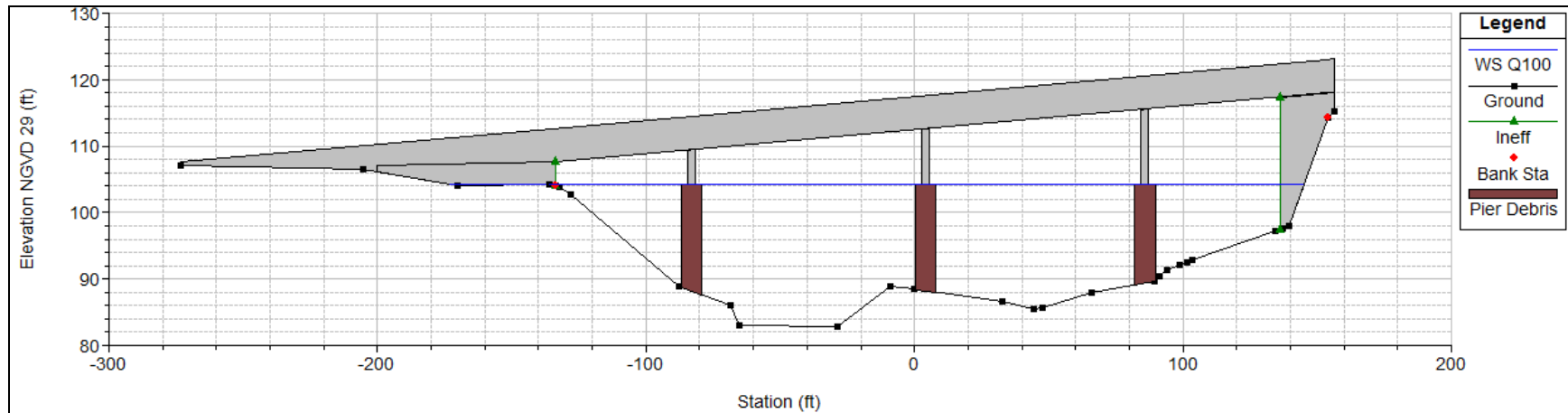


Figure 13. Upstream Face of Alternative 1 Bridge, Looking Downstream (Northwest)

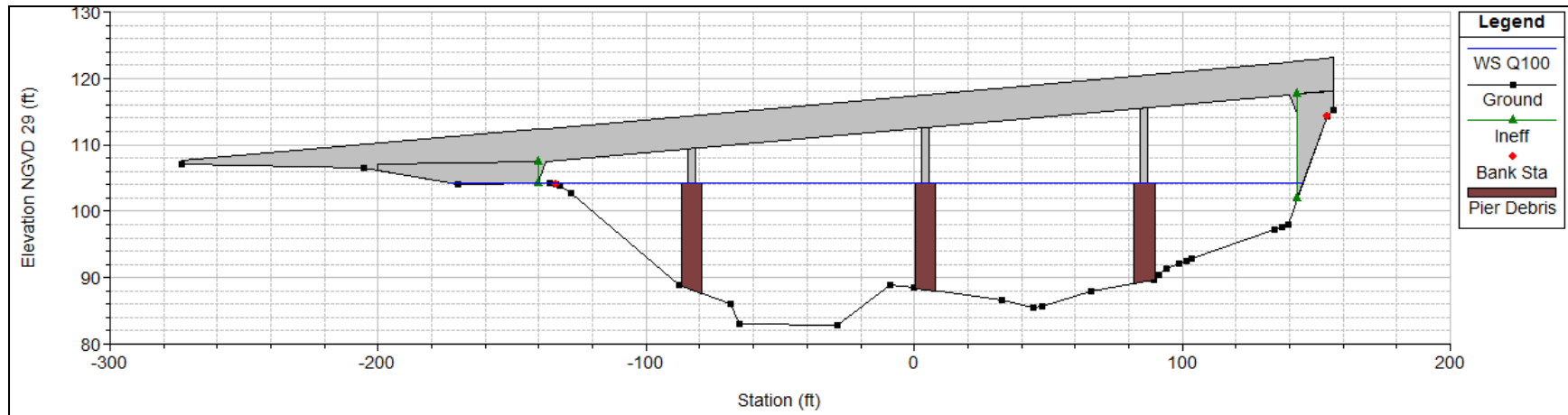


Figure 14. Upstream Face of Alternative 2 Bridge, Looking Downstream (Northwest)

3.3 Summary of Potential Encroachments

The FHWA defines a significant encroachment as a highway encroachment, and any direct support of likely base floodplain development, that would involve one or more of the following construction or flood-related impacts: 1) significant potential for interruption or termination of a transportation facility that is needed for emergency vehicles or provides a community's only evacuation route; 2) a significant risk; or 3) a significant adverse impact on the natural and beneficial floodplain values (1994). The following sections discuss the potential impacts to the floodplain that result from the proposed action. The risk associated with implementation of the action is discussed in Section 3.4.

3.3.1 Potential Traffic Interruptions Due to the Proposed Action

For both proposed retrofit alternatives, Alternatives 1 and 2, the 100-year flood is contained within the main channel and overbank areas, and therefore, no traffic interruptions are anticipated from the base flood. The cross sections of the proposed alternatives at the upstream face of the bridges are shown in Figure 13 for Alternative 1 and Figure 14 for Alternative 2.

3.3.2 Potential Adverse Effects on Natural and Beneficial Floodplain Values

Natural and beneficial floodplain values include, but are not limited to: fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, aquaculture, forestry, natural moderation of floods, water quality maintenance, and groundwater recharge.

Natural and beneficial floodplain values associated with the base floodplain at the Project site include habitat for fish, habitat for wildlife, plants, open space, natural beauty, outdoor recreation, and forestry. According to NOAA, Garcia River provides habitat for Coho salmon. The existing and proposed bridge piers and abutments are within the 100-year base floodplain.

3.3.3 Support of Probable Incompatible Floodplain Development

As defined by the FHWA, the support of incompatible base floodplain development will encourage, allow, serve, or otherwise facilitate incompatible base floodplain development, such as commercial development or urban growth.

The existing bridge is currently located within a 100-year base floodplain, and the Project would not support incompatible floodplain development. The Project will maintain local access and not create new access to developed land. The nature of the retrofit measures proposed for the Project will not facilitate incompatible base floodplain development.

3.3.4 Longitudinal Encroachments

As defined by the FHWA, a longitudinal encroachment is an action within the limits of the base floodplain that is longitudinal to the normal direction of the floodplain.

A longitudinal encroachment is “[a]n encroachment that is parallel to the direction of flow. Example: A highway that runs along the edge of a river is, usually considered a longitudinal encroachment.” The requirement for consideration of avoidance alternatives must be included in a Location Hydraulic Study by including an evaluation and a discussion of the practicability of alternatives to any significant encroachment or any support of incompatible floodplain development.

The proposed Project is not considered a longitudinal encroachment to the base floodplain.

3.4 Risk Associated with the Proposed Action

As defined by the FHWA, risk shall mean the consequences associated with the probability of flooding attributable to an encroachment. It shall include the potential for property loss and hazard to life during the service life of the bridge and roadway.

The potential risk associated with the implementation of the proposed action includes but is not limited to: 1) change in land use, 2) change in impervious surface area, 3) fill inside the floodplain, or 4) change in the 100-year water surface elevation. The measures to minimize the potential floodplain impacts associated with the action are summarized in Section 4.

The potential adverse effects to the floodplain for the proposed condition would be minimal due to the nature of the retrofit work.

- The Project does not propose to change the land uses within the Project limits.
- The Project does not propose to change the impervious surface areas within the Project limits.
- There will be some minor fill inside the floodplain attributed to the increased pier widths from the steel column casings and the added bumpers for both alternatives. Alternative 1 also includes an additional encroachment from the CIDH piles at both ends of the bridge. Alternative 2 would have more temporary construction impacts resulting from the remediation of the in-situ soils within the 100-year base floodplain.
- Alternative 1 results in no change to the water surface elevation and Alternative 2 only results in a 0.01 ft increase in water surface elevation.

The overall potential risk associated with the proposed bridge is considered low.

4 AVOIDANCE, MINIMIZATION, AND/OR MITIGATION MEASURES

4.1 Minimize Adverse Effects to Floodplains

Changes in land use and impervious surface areas were avoided. There will be some minor fill inside the floodplain attributed to the widened piers from the steel column casings and the added bumpers for both alternatives and the additional CIDH piles at both ends of the bridge for Alternative 1. However, based on the hydraulic modeling, the proposed retrofit bridge would result in minimal adverse effects to the floodplain (Alternative 1 results in no change in water surface elevation and Alternative 2 only increases the water surface elevation by 0.01 ft). The Project will have minimal adverse effects to the floodplains within the Project limits.

4.2 Restore and Preserve Natural and Beneficial Floodplain Values

The retrofit improvements will require temporary access to the streambed in order to construct column casings to the bent columns and top mats to the bent foundations. However, all temporary access will be removed once the seismic improvements have been completed, and the disturbed area will be restored to the pre-project conditions. The resulting spoils of CIDH piles will be disposed of outside of the public right-of-way, in accordance with the contract specifications.

In order to avoid impacting habitat for the Coho salmon, the construction window will need to avoid the migration and spawning season.

Environmental impacts that could result from the construction activities could be minimized with measures such as temporary BMPs, revegetation, a boundary establishment for work around sensitive habitat, erosion control measures, or other requirements that are part of the Project's permit conditions. The temporary BMPs will be installed prior to any construction operations, and will be in place for the duration of the contract. The removal of these BMPs will be the final operation with the Project site cleanup. Permits or approvals may be required from the U.S. Army Corps of Engineers, the California Department of Fish and Wildlife, and the Regional Water Quality Control Board.

4.3 Alternatives to Significant Encroachments

The proposed retrofit alternatives would maintain the existing roadway alignment and roadway profile. Although there will be some minor encroachment to the floodplain, the impacts to the floodplain would be minimal. Therefore, the floodplain encroachments will not be significant and other alternatives were not evaluated.

4.4 Alternatives to Longitudinal Encroachments

Because the proposed retrofit bridge would not be considered a longitudinal encroachment, no special alternatives were considered to address the minimal adverse effects to the floodplains.

4.5 Coordination with Local, State, and Federal Water Resources and Floodplain Management Agencies

The proposed condition would not significantly change the water surface elevations within the Project vicinity (there would be no change for Alternative 1 and an increase of 0.01 ft for Alternative 2). Therefore, a floodplain map revision and a Letter of Map Revision are not anticipated. Regulatory permits and approvals, as mentioned in Section 4.2, would be required as the Project enters into the final design phase.

5 REFERENCES

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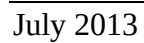
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Appendix A Federal Emergency Management Agency Flood Insurance Rate Map



APPENDIX 9

AS-BUILT PLANS

UNITED STATES DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

AS CONSTRUCTED PLANS FOR
PROJECT R-AD 39(2)
PT. ARENA AIR FORCE STATION ACCESS ROAD

POINT ARENA AIR FORCE STATION
MENDOCINO COUNTY
CALIFORNIA

10C.34

CALIF. R-AD 39(2)
SHEET 1
As-Constructed

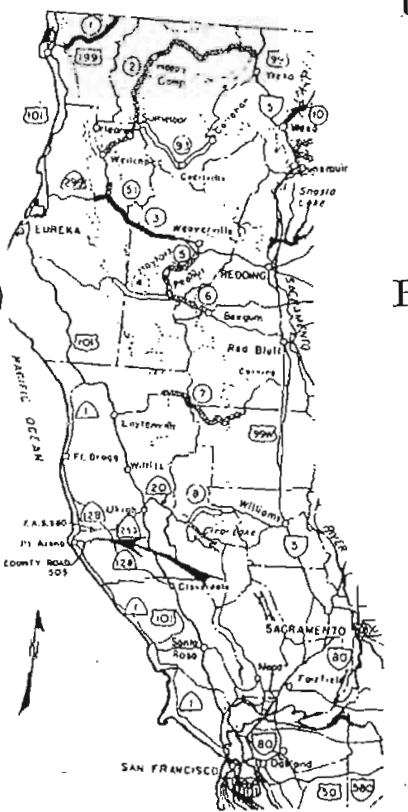
INDEX TO SHEETS

Sheet	Description
1	Title Sheet
2-3	Garcia River Bridge (P.G. 1847)
Standards	
10	C-302 Date Details
11	C-302 Standard Guardrail
12	C-302 Standard Guardrail (Flared)
13	C-302 Standard Guardrail (Tied Anchors)
14	C-302 Social Project Sign (C-302)
15	C-302 Social Identification Sign (Balance Access)

DESCRIPTION OF PROJECT

Project: Calif. R-AD 39(2), Pt. Arena Air Force Sta. Access Road
Improvement: Concrete Box Girder Bridge
Location: Project is located approximately 5.5 Miles east of Pt. Arena, which is 47 Miles Southeast of Fort Bragg, California consisting of a Bridge over the Garcia River.

Bridge: Garcia River Bridge (P.G. 1847)
Station: 7+07.13 to 7+10.87
Width: 22 Feet
Length: 0.061 Mile (321.75 feet)
As-Constructed Mileage



KEY MAP - NORTHERN CALIFORNIA

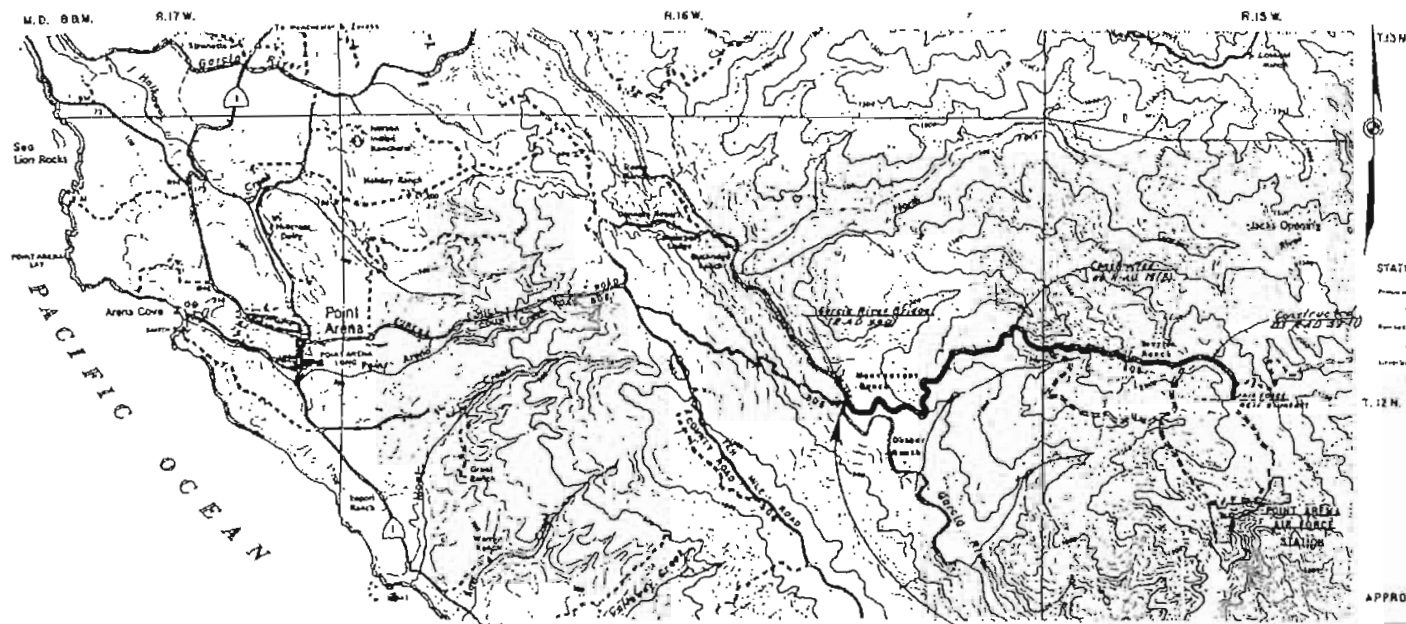
LEGEND:
- INTERSTATE NO.
- STATE NO. 1640 IN VICINITY OF ACTIVE PROJECT ONLY
- FOREST HWY. NO. CLASS 1 (CLASS 2 & 3, CLASS 3)
- FEDERAL AND CONNECTED ROADS
- NATIONAL FOREST, PARK & BUREAU OF LAND MANAGEMENT
SCALE 0 10 20 MILES

ROUTE MAP

SCALE OF MILES 0 10 20

Sheet	Improvement	Grade	Subgrade	Base	Surface	Drainage	Other	Comments
1	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete	Concrete

Drawn by: Oct 22, 1968



STATUS OF PLANS

Prepared: _____
Reviewed: _____
Approved: _____
Date: _____

APPROVED

DATE

PROJECT CALIFORNIA R-AD 39(2)
G-15
FIELD DRAWING NO. 144
SEQUENCE NO. 144
E&R

"As-Constructed" Revisions Nov. 1970



