August 2002

US Army Corps
of Engineers
San Francisco
District

Structural Assessment of the Lagunitas Road Bridge

Corte Madera Creek Project

Post-100% Submittal

by Ben C. Gerwick, Inc.

Robert Filgas, P.E.
Neil J. Tuholski, S.E., P.E.
Wen Lin, Ph.D.

Prepared for San Francisco District, U.S. Army Corps of Engineers

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USACE

Prepared for U.S. Army Corps of Engineers, San Francisco District

Monitored by U.S. Army Corps of Engineers, San Francisco District

Facilitated by U.S. Army Corps of Engineers, San Francisco District

Preface

The work presented in this report was authorized by the San Francisco District U.S. Army Corps of Engineers, as part of the Corte Madera Creek Project. The study was conducted under Indefinite Delivery Contract (IDC), DACW07-00-D-0003.

This report summarizes the results of the as-built seismic analysis and two options for flood control measures for the Lagunitas Road Bridge over the Corte Madera Creek in the city of Ross. The responses of the as-built analysis are compared to the responses of the structure as modified to the configurations of the options to determine the affect of the proposed reconfigurations. This assessment was performed and the report written by Robert Filgas, Neil J. Tuholski, and Wen Lin, of Ben C. Gerwick, Inc.. The work was performed under the general direction of Mr. George Fong, Mr. James Miller, and Mr. Arnold Lee.

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1.0 General

1.1 Introduction

The Lagunitas Road Bridge assessment is in support of efforts underway for the Corte Madera Creek Project by the U.S. Army Corps of Engineers and was directed under Indefinite Delivery Contract (IDC) DACW07-00-D-0003. This phase of the project received a notice to proceed on March 5, 2002. The Lagunitas Road Bridge assessment evaluates the relative structural affects induced by various alternative bridge and creek bed configuration options being considered to alleviate local Corte Madera flooding problems.

Structural analysis techniques have developed sufficient accuracy and verification in recent years to be an effective analytical tool for relatively assessing the various structural configurations assumed during the design and construction of major projects similar to the currently under way Corte Madera Creek Project. For older bridges uncertainties do occur relative to the assumptions of structural details due to lack of or loss of construction documentation. Notwithstanding these assumptions, computerized structural assessment is an effective method to identify, estimate, quantify, and evaluate the relative effects of various creek to bridge configuration options.

1.2 Objective

The objectives of this assessment are to:

a. Analyze the bridge in its existing condition to determine key structural responses as a basis for comparison with optional stream bed to bridge configurations

b. Analyze the bridge for two proposed configuration conditions to determine key structural responses for comparison with the existing condition

c. Quantify relative effects of various configuration changes

The proposed configurations to be evaluated in this assessment are the dredged or lowered stream bed by four feet condition (to Elevation +6.5 feet NGVD) and/or the condition resulting from the construction of a large box culvert parallel to the stream flowline adjacent to the west headwall of the bridge. Achieving these objectives provides the basis for a preconstruction independent assessment of the measures needed to increase the stream flow while preventing detrimental affects on this historically significant bridge structure.

2.0 Lagunitas Road Bridge Description

2.1 General Description of Structure

The structure is generally described as follows:

Year Constructed:	1909 (1930 Estimated)
Location:	Lagunitas Road over Corte Madera Creek approximately 200 feet west of Sir Francis Drake Boulevard in the town of Ross, California.
Jurisdiction:	Town of Ross, California.
State Bridge Number:	27C0071
Length:	75’-1 ½”
Width:	37’– 5”
Elevation Datum:	Top of concrete deck elevation is 24.80 feet National Geodetic Vertical Datum (NGVD)
Traffic Lanes:	Two (2) auto lanes and one (1) pedestrian sidewalk
Number of Spans:	Three (3)
Skew:	Twenty Degrees
Superstructure:	Continuous reinforced concrete haunched T-beam girders (5) on solid wall piers and end diaphragm abutments supporting a reinforced concrete slab
Substructure:	Reinforced concrete pier walls on spread foundations
Abutments:	Reinforced concrete walls framed continuously into the deck slab and longitudinal girders with wing walls that are isolated from the headwalls.
Utilities:	Mounted on the north side of the bridge: 16” diameter cast/ductile iron pipe. 5” diameter steel pipe. 1” diameter steel pipes. Mounted on the south side of the bridge: 8 – 4” diameter steel pipes. 5” diameter steel pipe.

The 75-foot Lagunitas Road Bridge crosses over the Corte Madera Creek at a 20-degree skew. It is a two-lane bridge with a four-foot sidewalk on the south side and concrete railings on each side. Construction of the bridge was completed in 1909; however, Caltrans biennial bridge inspection documents indicated the bridge was constructed in 1930. Because no formal records or as-built drawings were available for this bridge, the structural configuration is based on drawings and sketches that were developed from field measurements for the Corps and presented in the initial Phase of the work. Additional field measurements were made during an earlier stage of this Phase to determine and clarify the cross-section and abutment wall dimensions of the bridge. The as-built drawings are presented in Appendix A. The bridge spans Corte Madera Creek running in an unimproved streambed section which is normally dredged on an annual basis.

2.2 Site Inspection

Site inspection was performed in the initial phase and results were documented in the report entitled "Background, Site, and Geotechnical Investigation of the Lagunitas Road Bridge" prepared under this contract and presented to the Corps. Additional field measurements were made to determine and clarify bridge cross-section and abutment wall dimensions, which led to the updated as-built drawings presented in Appendix A.

2.3 Structural Details

The bridge consists of five longitudinal continuous reinforced concrete haunched T-beams framed monolithically into the pier walls and the headwalls. Two diaphragms are framed into the girders in the larger midspan. The deck is a twelve-inch reinforced slab cantilevered along the south edge for the sidewalk.

Head and wing walls are located on both ends of the bridge structure. These walls do not appear to be founded on spread or strip footings. The head walls are integrally tied to the main deck and girder structure and will be considered as load carrying components of the foundation structure when determining the foundation stiffness and load capacity.

Foundations of the bridge are as described in the soils investigation report. The pier foundation was determined during the Geotechnical Investigation in Phase I to be a spread footing 32.5'x5.6'x2.25' approximately ten feet below the stream bed at the time of the investigation. The headwalls apparently are not supported on spread footings and the actual depth of embedment is unknown as indicated by the Geotechnical investigation. Both the pier strips and the headwalls are founded on stream alluvial material as described in the soils investigation report.

2.4 Utilities

The bridge supports a total of 12 utility pipes: one 16"-diameter cast iron pipe, one 5"- and one 1"-diameter steel pipes mounted along the North side of the bridge; and eight 4"-diameter and one 5"-diameter steel pipes mounted on the South side.

3.0 Basis of Assessment

3.1 Structural Criteria

The structural design and evaluation criteria used for these design calculations will be in accordance with the criteria and guidance in the Corps of Engineers manuals for engineering and design, industry standards, and other technical references as follows:

Engineering Manuals

a. EM 1110-2-2000, Standard Practice for Concrete, September 1985.
b. EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures, June 1992.

Technical Publications

a. *Building Code Requirements for Structural Concrete*, American Concrete Institute, ACI 318-99.
b. AASHTO Bridge Design Code
c. Caltrans Bridge Design Specifications
d. Caltrans Seismic Design Criteria

Emphasis has been placed on the Caltrans Bridge Design requirements for this assessment, in particular, the recently revised seismic requirements.

The definition of loads and applied forces are as given below:

a. Dead Loads (D): The actual dead weight of the structure including roadway, sidewalk, pipes and other utility services. Material densities will be assumed as follows:

Item	Unit Weight (lbs/ft3)
Water	62.5
Reinforced Concrete	150.0
Steel	490.0

b. Live Loads (L): Vehicle traffic lane load and truck load as specified by Caltrans Bridge Design Specification, Section 3.7.

c. Impact (I): Highway live loads are increased for those structural elements specified in the Bridge Design Specifications of Caltrans, Section 3.8, to allow for dynamic, vibratory and impact effects.

d. Earth Pressure Load (E)
Earth pressure forces on the structures due to backfilling.

e. Seismic Loads (MCE)
Appropriate dynamic analyses are performed using the Maximum Credible Earthquake (MCE) input ground motion acceleration response spectra at the appropriate elevation as provided by Caltrans Bridge Design Specification, Section 3.21 and Caltrans Seismic Design Criteria, Section 6.1.

Reinforced concrete structures including the main deck, girders, piers and head walls are evaluated using the load factor methodology as required by Caltrans Bridge Design Specification (2000), Section 3.22.

The factored load cases given by Table 3.22.1 in Section 3.22.1 of Caltrans Bridge Design Specification (2000) govern the structural evaluation of all the reinforced concrete structural components. For the seismic assessment of the Lagunitas Road Bridge, two load cases are considered as follows:

$$\mathbf{Group(N) = \gamma[\beta_D\, D + \beta_E\, E + \beta_L(L+I) + \beta_{EQ}\, EQ]}$$

Group	Load Combination	γ	β			
			D	L + I	E	EQ
II	LCDL	1.30	1.0	1.67	1.0	0
VII	LCMCET LCMECL	1.00	1.0	0	1.0	1.0

where
β_D = 0.75 when checking columns for maximum moment or maximum eccentricities and associated axial load; and when Dead Load effects are of opposite sign to the net effects of other loads in a Group.
β_D = 1.0 when checking columns for maximum axial load and associated moment.
β_D = 1.0 for flexural and positive moments in rigid frames.

The axial force (P) and bending moment (M) interaction diagram is used to evaluate the strength demands and capacity of critical sections identified from bridge structural analysis. Two critical sections were selected where a plastic hinge, if any, is most likely to form: (1) slab-girder (T-beam) cross-section at the mid-span of the bridge and (2) pier cross-section at the bottom of the haunched girder.

The following material and geotechnical assumptions have been made

Concrete (1909):	f'_c =3,000 psi	ε_{cu} = 0.004
Reinforcement (1909):	f_y = 30,000 psi	ε_{sh} = 0.0125
	f_u = 60,000 psi	ε_u = 0.12

Depth to bedrock: approx. 25-30 feet
Expected peak rock acceleration: 0.4g

3.2 Methodology

The response spectrum analysis was performed in both longitudinal and transverse directions using Caltrans design ARS spectrum (Caltrans Seismic Design Criteria, 2001). The peak rock acceleration of 0.4g for the Maximum Credible Earthquake was identified from 1996 Caltrans California Seismic Map (Appendix C) at the location of the Lagunitas Road Bridge. The results were combined using a 100%-30% combination of the analytical results.

The soil-foundation properties were investigated and equivalent linear foundation stiffness and damping were determined by the URS Corporation for as-built conditions and various scenarios of placement of a bypass concrete culvert behind the existing abutment, as well as lowering of the stream channel. These results are documented in the Geotechnical report. The site seismological conditions are characterized by Caltrans ARS or design elastic response spectrum for the Maximum Credible Earthquake (MCE) event, where the peak ground acceleration (PGA) of 0.4g was determined at the location of the bridge from Caltrans 1996 Seismic Hazard Map.

4.0 Existing Structure Analysis

4.1 Structural Loads

The dead load was applied to the structure as a one-g acceleration in the global model as defined by a gravity static load specification. The one-g inertial gravity dead load was determined by the material density input for each type of element. A weight summary was calculated by the SAP 2000 input echo and was checked initially after running each analysis. The weight of rigid elements that were used to model eccentric continuity was excluded by inputting their weight and mass density as zero.

The live load was defined by CALTRANS Bridge Design Specifications Section 3.4 through 3.12. The AASHTO lane and HS20-44 load definition is essentially identical to the CALTRANS applied live load. A lane load of 0.640 kips per linear foot was utilized for this evaluation. These loads were applied as a static pressure to the global structural model.

The definition of loads and applied forces was given in Section 3. The live load and earth pressure loads were applied in a distributed manner to the analytical model in order to adequately account for the bridge skew and width-to-length ratio.

The seismic load design/analysis methodology is specified by CALTRANS Bridge Design Specifications Section 3.21. Seismic inertia forces were applied to the global structural model for an appropriate dynamic analyses (modal superposition) using the Maximum Credible Earthquake (MCE) input ground motion acceleration response spectra at the appropriate elevation as provided by Caltrans Bridge Design Specification, Section 3.21. Seismic inertial internal forces are determined by the material density input for each type of element and the acceleration present at the mass established for each dynamic degree of freedom. Inertial forces for rigid structural modeling elements were excluded by inputting their mass density as zero. Bridge deck, girder, and pier component assessment forces were obtained from the global force summary output for each individual seismic load direction analysis case and for the various combination load cases at locations that provide structural resultants associated with the comparisons of the existing structure and the proposed modified or reconfigured structure.

4.2 Foundation Stiffness

The underlayment geological stratigraphy and determination of soil properties were presented in Geotechnical Data Report, Lagunitas Road Bridge Project, January 30, 2002 by URS. The soil-foundation properties were investigated and equivalent linear foundation stiffness were determined for existing conditions behind the abutment headwall and pier side faces. The reaction force stiffness for the bridge components supported vertically by soil bearing for the various load cases were also presented in this report.

The reaction force soil stiffness for each of the directly supported bridge pier footers and headwalls were reviewed and an appropriate method of representation in the global structural model was determined. The following soil springs were used:

Pier Footing Base - Vertical Bearing	Vertical Linear Springs
Pier Footing Base – Horizontal Shear	Horizontal Linear Springs
Pier Vertical Faces - Horizontal Bearing	Horizontal Linear Springs at Nodes
Headwall Base - Vertical Bearing	Vertical Linear Springs
Headwall Base – Horizontal Shear	Horizontal Linear Springs
Headwall Vertical Faces – Horiz. Bearing	Horizontal Linear Springs at Nodes

The local area soil stiffness were included as individual element springs attached to the nodal points of the global structural model and provide resultant static and dynamic soil force and pressure resistances for the analyses. Global force summaries provide resultant soil force/stress conditions for each load condition or combination.

4.3 Structural Model

The bridge was modeled with plate (shell) elements for the main deck, beam (frame) elements for the main girders, and shell elements for the piers, pier footings and headwalls. The main girders were rigidly attached to the main deck with rigid links to assure monolithic behavior. This level of discretization was deemed necessary in order to obtain sufficiently accurate individual girder axial and bending design forces, main deck warping shears, and pier and headwall torsional responses from the skewed and relatively short to wide span ratios. It was anticipated that the dynamic response of the structure would be strongly coupled in the horizontal directions and possibly coupled horizontal to vertical.

The elemental material and structural properties were input for each type of element to with a consideration to assure minimal duplication at elemental intersections. Output requirements in terms of internal and external force and displacement extraction was considered in the topological configuration of the various elements. A weight summary by element type was calculated by the SAP 2000 input echo function. Individual element type weight was checked after the global model was completed prior to utilizing the results of each analysis. The weight of rigid elements were excluded by inputting their weight and mass density as zero. Soil springs modeling the soil to structure interface were attached to the piers and headwalls as discussed above.

4.4 SAP 2000 Analysis

The analysis was performed by the general purpose structural analysis program SAP 2000 Non-Linear Version 7.40. Pertinent parts of output computer files were printed and placed in the project calculation files in support of this assessment. Results for individual loads and load combinations were specified.

4.5 General Response Results

The general response of the bridge was evaluated by assessing the dynamic properties, static deformations and forces and the structural responses to critical MCE seismic load combinations.

The dynamic mode shapes indicated substantial coupling in the horizontal direction due to the skew configuration. The dynamic motion is characterized by the first three modes as shown by the participation factors. The periods and participation factors are presented in Table 4-1. The primary mode in the direction longitudinal to the bridge centerline has a period of 0.246 seconds (4.06 hertz). This period is slightly to the lower period side of the peak seismic input of 1.43 g's at 0.300 seconds. Thus the seismic acceleration this mode of the structure will be subjected to is essentially the maximum peak input acceleration. For the additional modes, approximately 28 % of the bridge mass will be subjected to lower accelerations, but these accelerations could increase if local structural failures increase the modal periods during the course of the seismic event.

The location of the centerline girder, girder three, at midspan of the main span for which assessment forces were developed is shown as Section A on Figure 4-1. The resultant axial force and bending moment for midspan of girder three are presented in Table 4-3. These structural forces and deformations are dependent on the assumed material properties and reinforcement and are provided for comparative purposes only.

The resultant forces in midspan girder three are governed primarily by the dead and live vertical loads and their associated load factors. The affect of horizontal seismic loads on these midspan girder forces are minimal even with a input ground motion magnitude of approximately 1.4 g. Comparison of the resultant forces with respect to their capacities based on axial force vs. bending moment interaction diagrams, P-M diagrams, indicates the capacity of girder three considerably exceeds the factored analysis demands.

The resultant factored load displacements at midspan girder three are given in Table 4-4. The vertical deflection under normal operating conditions with the design live load is less than an eighth of an inch. The largest horizontal component of movement at misspend is predicted to be in the longitudinal direction with the full seismic inertia force along the longitudinal axis of the bridge. A predicted movement of slightly over three-quarters of an inch is in the direction inducing weak axis bending in the piers.

Resultant axial forces and bending moments of Pier 1 are determined at Section B on Figure 4-1 for the comparative assessment with proposed modified bridge configurations. These forces are shown in Table 4-5 for the relevant normal operating and seismic load combinations. The axial load and the bending moment of the entire pier section about its weaker lateral axis indicate that plastic hinges will form at the tops of the piers for the seismic load combination with the primary seismic motion along the longitudinal axis of the bridge. The magnitude of the bending moment as calculated by this linear analysis, without pushover analysis data, would indicate that the structural will incur sufficient rotation to extensively spall and thus be unusable following the seismic event.

Soil bearing pressures and shear stresses under the Pier 1 footing are provided in Table 4-6 for the relevant normal operating and seismic load combinations to be used for comparative assessment. The bearing pressures are unacceptablely high for the seismic load combination with the primary seismic motion along the longitudinal axis of the bridge. These soil stresses indicate that the structure will incur soil failure and subsequent horizontal offset and vertical settlement during the postulated seismic event.

Table 4-1 Existing Structure Significant Modal Periods and Participation

Mode	Period T (secs)	Participation Factor - X	Participation Factor Y
1	0.2461	71.61	14.19
2	0.1408	6.51	31.79
3	0.1391	9.10	44.50

Table 4-2 Existing Structure Total Seismic Inertial Force

Seismic Input Ground Motion Load Direction	Total Inertial Force X - Direction (kips)	Total Inertial Force Y - Direction (kips)
(MCEHL) MCE Horizontal Longitudinal X Direction	1396.7 (1.06 g)	822.1 (0.63 g)
(MCEHT) MCE Horizontal Transverse Y Direction	822.1 (0.63 g)	1305.8 (1.00 g)

Notes: 1. Equivalent static inertia load g's based on 1311.5 kips of total weight

Table 4-3 Existing Structure Midspan Center Girder Force Summary

Load Combination and Load Factors	Axial Force (kips)	Bending Moment (k-ft)
(LCNORMAL) 1.3D + 2.17L + 1.3EHS	-135.22	167.91
(MCEHL) (1) D + 0.1L + EHD + MCE		
Max	52.19	68.06
Min	-71.46	-16.19
(MCEHT) (2) D + 0.1L + EHD + MCE		
Max	49.20	67.26
Min	-71.47	-0.64

Notes:
1. Seismic direction combination 1.00 MCE(x) + 0.30 MCE(y)
2. Seismic direction combination 0.30 MCE(x) + 1.00 MCE(y)
3. Force Sign Convention (-) compression (+) tension

Table 4-4 Existing Structure Midspan Center Girder Displacement

Load Combination and Load Factors	Global Displ. X – Long. (inches)	Global Displ. Y – Trans. (inches)	Global Displ. Z – Vert. (inches)
(LCNORMAL) 1.3D + 2.17L + 1.3EHS	-0.0761	-0.0274	-0.1026
(MCEHL) (1) D + 0.1L + EHD + MCE			
Max	0.7767	0.3692	0.0059
Min	-0.7767	-0.3692	-0.0745
(MCEHT) (2) D + 0.1L + EHD + MCE			
Max	0.3546	0.2789	0.0068
Min	-0.3546	-0.2789	-0.0745

Notes:
1. Seismic direction combination 1.00 MCE(x) + 0.30 MCE(y)
2. Seismic direction combination 0.30 MCE(x) + 1.00 MCE(y)
3. See Figure 4-1 for Global Coordinate System

Table 4-5 Existing Structure Top of Pier 1 Force Summary

Load Combination and Load Factors	Axial Force (kips)	Bending Moment (k-ft)
(LCNORMAL) 1.3D + 2.17L + 1.3EHS	-370.53	-1078.99
(MCEHL) (1) D + 0.1L + EHD + MCE		
Max	855.48	7245.31
Min	-892.24	-7252.23
(MCEHT) (2) D + 0.1L + EHD + MCE		
Max	511.98	3378.69
Min	-587.22	-3371.71

Notes:
1. Seismic direction combination 1.00 MCE(x) + 0.30 MCE(y)
2. Seismic direction combination 0.30 MCE(x) + 1.00 MCE(y)
3. Force Sign Convention (-) compression (+) tension

Table 4-6 Existing Structure Pier 1 Footing Base Stress Summary

Load Combination and Load Factors	Bearing Pressure (ksf)	Shear Stress (ksf)
(LCNORMAL) 1.3D + 2.17L + 1.3EHS	-3.47 avg -6.11 max	1.04 avg 1.25 max
(MCEHL) (1) D + 0.1L + EHD + MCE	-9.12 avg -19.05 max	3.76 avg 3.97 max
(MCEHT) (2) D + 0.1L + EHD + MCE	-6.90 avg -11.94 max	3.53 avg 3.64 max

Notes:
1. Seismic direction combination 1.00 MCE(x) + 0.30 MCE(y)
2. Seismic direction combination 0.30 MCE(x) + 1.00 MCE(y)
3. Force Sign Convention (-) compression (+) tension

5.0 Reconfiguration Option 1 Structure Analysis

5.1 Description

The bridge to stream bed reconfiguration option 1 is defined as lowering the stream bed by four (4) feet uniformly to Elevation +6.5 feet NGVD.

5.2 Structural Loads

The definition of loads and applied forces were as given in Section 3. The live load and earth pressure loads were applied in a distributed manner due to the bridge skew and width to length ratio as modified for a lower stream bed elevation.

5.3 Foundation Stiffness

The soil-foundation properties were investigated and equivalent linear foundation stiffness and damping were determined for the scenario of lowering of the stream channel. These results were documented in the Geotechnical report.

5.4 Revised Global Structural Model

The bridge with the proposed modifications was modeled with plate (shell) elements for the main deck, beam (frame) elements for the main girders, and shell elements for the piers, pier footings and headwalls. Soil springs modeling the soil-to-structure interface which were attached to the piers and headwalls, as described in Section 4, were revised to reflect the excavation of the stream bed. The dynamic response of the structure was thus revised to the proposed bridge configuration considered as an option. The analyses were rerun and forces were extracted for comparison with the existing condition.

5.5 SAP 2000 Analysis

The analysis of the bridge in the option 1 configuration was performed by the general purpose structural analysis program SAP 2000 Non-Linear Version 7.40. As was the case with the existing structure analysis, results for individual loads and load combinations were specified and relevant parts of output computer files were printed and placed in the project calculation files in support of this assessment.

5.6 General Response Results

An evaluation of the Option 1 modified bridges' general response utilized structural parameters consistent with the existing bridge structure assessment in order to facilitate a direct comparison and quantify the effects due to stream bed excavation.

The dynamic structural properties of the modified structure are summarized in Table 5-1. The horizontal motion of the structure is controlled by three primary modes.. The presence of coupling of the horizontal seismic input is indicated by the modal participation in the longitudinal direction for lateral input motion an vice versa. The total inertia forces applied to the bridge for each seismic direction is provided in Table 5-2. This force indicates response by a relatively stiff structure with a period near the peak of the input ground motion accelerations and a strong coupling of horizontal motions as measured by equivalent static accelerations.

Girder three central midspan factored axial and bending moment forces are shown in Table 5-3. The static vertical loads govern for this structural section with horizontal seismic factored loads having minor affect.. The excess capacity over demand based on the PM curves is predominant.

The factored load displacements at midspan girder three are provided in Table 5-4. The vertical deflection under normal operating conditions with the design live load is less than an eighth of an inch. The largest predicted horizontal component of movement at this midspan section is slightly in excess of three quarters of an inch in the longitudinal direction with the full longitudinal seismic inertia applied.

Factored axial forces and bending moments are provided at Section B on Figure 4-1. These forces at the top of pier 1 are shown in Table 5-5 for the relevant normal operating and seismic load combinations of the modified structure. The pier section lateral weak axis axial load and the bending moment demands exceed the calculated capacity as indicated on P-M curves and thus will form plastic hinges at the tops of the piers for the seismic load combination with the primary seismic motion along the longitudinal axis of the bridge. The structural section will incur sufficient rotation to extensively spall and thus be unusable following the seismic event.

Soil bearing pressures and shear stresses under the Pier 1 footing are provided in Table 5-6. The bearing pressures exceed soil strength for seismic inertia loads applied along the longitudinal axis of the bridge. These soil stresses indicate soil failure and subsequent permanent deformation during the postulated seismic event.

Table 5-1 Option 1 Modified Structure Significant Modal Periods and Participation

Mode	Period T (secs)	Participation Factor - X	Participation Factor Y
1	0.2492	71.30	14.37
2	0.1429	9.32	44.50
3	0.1410	6.42	30.90

Table 5-2 Option 1 Modified Structure Total Seismic Inertial Force

Seismic Input Ground Motion Load Direction	Total Inertial Force X - Direction (kips)	Total Inertial Force Y - Direction (kips)
(MCEHL) MCE Horizontal Longitudinal X Direction	1394.3 (1.06 g)	825.8 (0.63 g)
(MCEHT) MCE Horizontal Transverse Y Direction	825.8 (0.63 g)	1299.4 (0.99 g)

Notes: 1. Equivalent static inertia load g's based on 1311.5 kips of total weight

Table 5-3 Option 1 Configuration Structure Midspan Center Girder Force Summary

Load Combination and Load Factors	Axial Force (kips)	Bending Moment (k-ft)
(LCNORMAL) 1.3D + 2.17L + 1.3EHS	-174.18	159.94
(MCEHL) (1) D + 0.1L + EHD + MCE		
Max	53.02	68.32
Min	-133.66	-51.95
(MCEHT) (2) D + 0.1L + EHD + MCE		
Max	49.88	67.41
Min	-133.65	-35.97

Notes: 1. Seismic direction combination 1.00 MCE(x) + 0.30 MCE(y)
2. Seismic direction combination 0.30 MCE(x) + 1.00 MCE(y)
3. Force Sign Convention (-) compression (+) tension

Table 5-4 Option 1 Configuration Structure Midspan Center Girder Displacement

Load Combination and Load Factors	Global Displ. X – Long. (inches)	Global Displ. Y – Trans. (inches)	Global Displ. Z – Vert. (inches)
(LCNORMAL) 1.3D + 2.17L + 1.3EHS	-0.1046	-0.0372	-0.09298
(MCEHL) (1) D + 0.1L + EHD + MCE			
Max	0.7971	0.3818	0.0269
Min	-0.7971	-0.3818	-0.0769
(MCEHT) (2) D + 0.1L + EHD + MCE			
Max	0.3660	0.2895	0.0269
Min	-0.3660	-0.2895	-0.0769

Notes:
1. Seismic direction combination 1.00 MCE(x) + 0.30 MCE(y)
2. Seismic direction combination 0.30 MCE(x) + 1.00 MCE(y)
3. See Figure 4-1 for Global Coordinate System

Table 5-5 Option 1 Modified Structure Top of Pier 1 Force Summary

Load Combination and Load Factors	Axial Force (kips)	Bending Moment (k-ft)
(LCNORMAL) 1.3D + 2.17L + 1.3EHS	-356.31	-1199.33
(MCEHL) (1) D + 0.1L + EHD + MCE		
Max	867.75	7321.28
Min	-895.93	-7313.99
(MCEHT) (2) D + 0.1L + EHD + MCE		
Max	555.54	3438.89
Min	-591.94	-3437.64

Notes:
1. Seismic direction combination 1.00 MCE(x) + 0.30 MCE(y)
2. Seismic direction combination 0.30 MCE(x) + 1.00 MCE(y)
3. Force Sign Convention (-) compression (+) tension

Table 5-6 Option 1 Modified Structure Pier 1 Footing Base Stress Summary

Load Combination and Load Factors	Bearing Pressure (ksf)	Shear Stress (ksf)
(LCNORMAL) 1.3D + 2.17L + 1.3EHS	-2.87 avg -6.27 max	1.32 avg 1.65 max
(MCEHL) (1) D + 0.1L + EHD + MCE	-8.93 avg -18.73 max	3.76 avg 3.96 max
(MCEHT) (2) D + 0.1L + EHD + MCE	-6.88 avg -11.94 max	3.53 avg 3.65 max

Notes:
1. Seismic direction combination 1.00 MCE(x) + 0.30 MCE(y)
2. Seismic direction combination 0.30 MCE(x) + 1.00 MCE(y)
3. Force Sign Convention (-) compression (+) tension

6.0 Reconfiguration Option 2 Structure Analysis

6.1 Description

The bridge to stream bed configuration option 2 is defined as lowering the stream bed by four (4) feet uniformly to Elevation +6.5 feet NGVD, as well as constructing a box culvert, approximately 10 feet high and 19 feet wide, adjacent and parallel to the west headwall.

6.2 Structural Loads

The definition of loads and applied forces due to lowering of the stream bed were as given in Section 3. There is no resultant change in the bridge structure loading or responses from the box culvert behind the west headwall, provided: (1) the location of the culvert is a minimum of 10 feet to the west of the bridge headwall, (2) the base elevation of the culvert is +12.5 feet NGVD, and (3) the weight of the box culvert filled with water is less than or equal to the weight of the excavated soil. This is based on the Final Geotechnical Analysis Report by URS, Section 9.0, dated April 30, 2002. This assumption was made after consultation with Sverdrup Civil, who performed a constructability study of the box culvert and determined that the culvert could not be constructed any closer to the headwall due to alignment requirements, as well as continued use of the bridge during construction.

6.3 Conclusion

As a result of the aforementioned conditions resulting in no appreciable structural effects from the box culvert, no further analysis is required for this option at this time.

7.0 Comparison of Structural Responses

7.1 Seismic Input and Dynamic Properties

The comparison of the dynamic structural properties of the existing and modified structures indicated very minor changes as a result of the proposed modifications. The dynamic characteristics of the bridge were essentially unchanged as shown by the significant modal periods of the bridge. The largest affect of the Option 1 modification was to increase one period by 1.4% and its associated participation factor by 2.4% as shown in Table 7-1. Slight changes in modal periods reversed the sequential order for modes 2 and 3. This will have an inconsequential effect of the modal deformation and their amplitudes. The presence of coupling due to transverse seismic input remained as determined by the modal participation. The additional flexibility of the pier base did not measurably affect the dynamic properties.

A seismic total inertial force comparison between the existing structure and the Option 1 reconfigured structure is shown in Table 7-2. The differences indicating that the total inertial forces can be considered identical is a measure of the dynamic properties as being essentially identical.

7.2 Displacements

The predicted movements at the main span centerline as shown in Table 7-3, indicated minor differences between the existing structure and the option 1 reconfiguration. What appears as a large change in the response to the normal static dead and live load combination is a small change in the small secondary horizontal deflection. The proposed stream bed excavation of the option 1 modifications provided a slightly more flexible horizontal structural stiffness.

No major shifts in the load combinations for horizontal seismic structural response were noted. The predicted displacements indicated the seismic horizontal affect load paths were changed by less than 4%. The option 1 primary affect was to increase these horizontal deformations.

7.3 Section Forces

A comparison for girder 3 axial load and bending moment forces at the central midspan showed the effect of the pier horizontal stiffness reduction on the static load condition with a modest increase for axial load and a decrease for bending moment . This increase can be considered inconsequential when compared to the available capacities. The forces at this section are nearly uncoupled from seismic affects.

Resultant structural section force changes at Section B in Figure 4-1 are provided in Table 7-5. The affects of the modifications are insignificant when compared to the high demand relative the capacity and the damage predicted. The small change percentages should have insignificant affect on the onset of structural damage and the extent or sequence of damage. Consideration of the assumed reinforcement is greater than the modified structure affects at the top of the piers.

7.4 Foundation Reactions

The largest percentage change that the option 1 modifications have on the structure are the changes to the foundation reactions at the base of the pier footings for static dead and live loads.. These changes are shown in Table 7-6. Excavating the foundation soils adjacent to the pier footings increases the shear stress under the pier footings from existing shear stress magnitudes but should not exceed acceptable design or assessment values. Although the seismic load combination bearing stresses are slightly reduced, the peak bearing pressures exceed design or assessment limits. The improvement effected by the option modifications to the bearing pressures will not preclude soil failures.

Table 7-1 Comparison of Significant Modal Periods and Participation Factors For Option 1 Modified Structure

Mode	Period T (secs)	Participation Factor - X	Participation Factor Y
1	+1.26%	-0.43%	+1.27%
2	+1.49%	+2.42%	0 %
3	+1.37%	-0.99%	-2.80%

Notes:
1. Existing Structure used as baseline for comparison.
2. Modes 2 and 3 interchanged.

Table 7-2 Comparison of Seismic Inertial Force For Option 1 Modified Structure

Seismic Input Ground Motion Load Direction	Total Inertial Force X - Direction (kips)	Total Inertial Force Y - Direction (kips)
(MCEHL) MCE Horizontal Longitudinal X Direction	-0.17%	+0.45%
(MCEHT) MCE Horizontal Transverse Y Direction	+0.45%	-0.49%

Notes:
1. Existing Structure used as baseline for comparison.

Table 7-3 Comparison of Significant Midspan Center Girder Displacement Changes

Load Combination and Load Factors	Global Displ. X – Long. (inches) (1)	Global Displ. Y – Trans. (inches) (1)	Global Displ. Z – Vert. (inches) (1)
(LCNORMAL) 1.3D + 2.17L + 1.3EHS	+34.8%	(2)	-9.4%
(MCEHL) D + 0.1L + EHD + MCE			
Max	+2.6	+3.4	(2)
Min	+2.6	+3.4	+3.2
(MCEHT) D + 0.1L + EHD + MCE			
Max	+3.2%	+3.8	(2)
Min	+3.2%	+3.8	+3.2

Notes:
1. Percentage change based on existing structure
2. Change not significant due to extremely small displacement

Table 7-4 Comparison of Center Girder Midspan Section Force Changes

Load Combination and Load Factors	Axial Force (percent) (1)	Bending Moment (percent) (1)
(LCNORMAL) 1.3D + 2.17L + 1.3EHS	+28.81%	-4.75%
(MCEHL) D + 0.1L + EHD + MCE	+1.59% +1.41%(3)	+0.38% +1.86%(2)
(MCEHT) D + 0.1L + EHD + MCE	+1.38% +1.41%(3)	+0.22% +1.84%(2)

Notes:
1. Percentage change based on existing structure force demand
2. Percentage change based on existing structure force P-M curve Capacity P =1203 k (comp) and M = 1918 k-ft
3. Percentage change based on existing structure force P-M curve Capacity P =4416 k (comp) and M = 0.0 k-ft

Table 7-5 Comparison of Top of Pier 1 Section Force Changes

Load Combination and Load Factors	Axial Force (percent) (1)	Bending Moment (percent) (1)
(LCNORMAL) 1.3D + 2.17L + 1.3EHS	-3.84%	-11.15%
(MCEHL) D + 0.1L + EHD + MCE	+1.43% +0.41%	+1.05% +0.85%
(MCEHT) D + 0.1L + EHD + MCE	+8.51% +0.80%	+1.78% +1.96%

Notes:
1. Percentage change based on existing structure

Table 7-6 Comparison of Pier 1 Base Stress Changes

Load Combination and Load Factors	Bearing Pressure (percent) (1)	Shear Stress (percent) (1)
(LCNORMAL) 1.3D + 2.17L + 1.3EHS	-17.29% avg +2.62% max	+26.92% avg +32.00% max
(MCEHL) D + 0.1L + EHD + MCE	-2.08% avg -1.68% max	(2) avg -0.25% max
(MCEHT) D + 0.1L + EHD + MCE	-0.29% avg (2) max	(2) avg +0.28% max

Notes:
1. Percentage change based on existing structure
2. Change not significant due to extremely small soil stress

8.0 Conclusions and Recommendations

8.1 Conclusions

The structural assessments performed for this work demonstrate the important parameters to consider in such a bridge to stream configuration study. When scrutinizing a carefully planned construction project like the Corte Madera Creek project , it is found that many of the specific affects of the optional configuration changes provided have been identified and contingency mitigation measures introduced as appropriate. This structural analysis is based upon the assumption that liquefaction during the MCE does not occur. For further discussion regarding liquefaction, refer to the Final Geotechnical Analysis Report prepared by URS, dated April 30, 2002, Section 5.0, Liquefaction Analyses.

Specific conclusions include:

1. The general dynamic response to a maximum credible earthquake event in terms of its displacement mode shapes and total induced inertial inertia loads will not be significantly affected by the streambed excavation for option one.

2. The central midspan girder normal operating axial forces increase and the negative seismic bending moments increase, but the excess of capacity over demand for these girder sections render the increases inconsequential.

3. Assessment results indicate axial force and bending moment demands at the top of the pier walls from seismic load conditions substantially exceed the pier capacities. This condition will cause plastic hinges to form in the pier walls at the base of the girder haunches. The piers are predicted to incur spalled concrete damage and be immediately unusable during the postulated seismic event. The affect of the option 1 modifications do not have a significant affect on this condition.

4 Soil bearing pressures under the pier footings from seismic load conditions exceed the soil capacities. This condition will cause soil failures and permanent deformations under the pier footings.

These specific conclusions are summarized from the structural parameters that were selected for assessment as indicative of the major affects caused by the option 1 modifications. Additional data can be made available for assessment.

8.2 Recommendations

Based on the results and conclusions of this relative configuration assessment, the following recommendations are made:

1. The affects of the Option 1 stream bed excavation will reduce structural integrity of the bridge girders and piers by slight increases in seismic inertia forces and displacements. The impact of these changes could be negligible to a seismically retrofitted bridge.
2. As built drawings of the bridge be developed to confirm the assumed reinforcement and the calculated capacities. This effort will require some destructive examination with subsequent concrete repair.
3. Perform a seismic assessment with the confirmed reinforcement.

During the course of this relative configuration assessment, potential seismic response deficiencies with respect to the current AASHTO design requirements have been identified. As proposed by the Federal Highway Administration and Caltrans, an effort to identify all seismic deficiencies, evaluate the consequences of seismic damage, and initiate a program for reducing this seismic risk should be undertaken. The following recommendations should be considered for evaluating and upgrading the seismic resistance of the existing bridge structure and the supporting soil conditions:

1. Complete the comprehensive seismic assessment as noted above to identify all seismic deficiencies.
2. Evaluate and implement one of the strategies to mitigate the liquefaction potential of the supporting soils as described in the Final Geotechnical Analysis Report prepared by URS, dated April 30, 2002, Section 11.0, Recommendations.
3. An additional alternative should be to consider a pile underpinning retrofit to meet the AASHTO requirements for elastic foundation response.
4. A retrofit to provide confinement and assure the required ductility of the plastic hinges at the top of the piers in the longitudinal motion direction may be required considering the foundation retrofit strategy and the as-built reinforcement conditions.

These recommendations are based on conservative preliminary estimates of the bridge importance and acceptable damage. Final retrofit recommendations should be based on the results of a complete and comprehensive seismic evaluation program.

9.0 References

URS Corporation, April 30, 2002, *Final Geotechnical Analysis Report, Lagunitas Road Bridge Project, Ross, California*

FHWA-RD-94-052, May 1995, *Seismic Retrofitting Manual for Highway Bridges*, U.S. Department of Transportation, Federal Highway Administration

ATC, 1996, ATC-32, *Improved Seismic Design Criteria for California Bridges: Provisional Recommendations*, Applied Technology Council, Redwood City, California.

Caltrans, 1994, *Bridge Design Specification Manual*, California Department of Transportation, Sacramento, California

Caltrans, 1999*, Caltrans Seismic Design Criteria Version 1.1*, California Department of Transportation, Sacramento, California

Priestley M.J.N., F. Seible, and G.M. Calvi, 1996, *Seismic Design and Retrofit of Bridges*, John Wiley & Sons, New York

Priestley M.J.N. and F. Seible, 1991, *Seismic Assessment and Retrofit of Bridges*, SSRP – 91/03, University of California, San Diego

Appendices

1 Calculations – General
2 Reference Drawings
3 Calculations – Structural Loads
4 Calculations – Foundation Stiffness
5 Calculations – Existing Bridge Global Analysis
6 Calculations – Proposed Configuration Analyses

2.0 Reference Drawings

As-built structural drawings were prepared from field measurements and are presented in Appendix A. Because no drawings or formal records of the bridge were available, bridge dimensions were determined from field measurements made during Phase I of this project. Additional measurements were made at the beginning of this stage to clarify and verify the dimensions of the bridge cross-section, headwall abutments, and sidewalk.

The pier foundation was determined from the Geotechnical Investigation in Phase I, which indicated that the pier is supported on a spread footing of 32.5' x 5.6' x 2.25', approximately 10 ft. below the streambed at the time of investigation. The headwall apparently is not supported on a spread footing and the actual depth of embedment is unknown as indicated by the Geotechnical Investigation. Design or construction drawing information is not available for the reinforcement in the bridge deck, girders, piers, and headwalls.

Assumptions

- The East and West headwalls extend to the same elevation as the piers.
- 4-1" – square steel reinforcing bars were placed in the bottom of the girder
- 1-1/4" – square steel reinforcing bars for the pier
- Concrete compressive strength of 3,000 psi
- ASTM A16 for steel reinforcement with a yield strength of 30,000 psi and an ultimate tensile strength of 60,000 psi

DRAWING INDEX

---	COVER SHEET
G-01	INDEX OF DRAWINGS, GENERAL NOTES, ABBREVIATIONS AND SYMBOLS
G-02	DESIGN CRITERIA
S-01	PLAN AND ELEVATION
S-02	PIER AND ABUTMENT ELEVATIONS
S-03	SECTIONS AND DETAILS
S-04	REINFORCEMENT DETAILS
01-349T-1	TOPOGRAPHICAL SURVEY LAGUNITAS BRIDGE

GENERAL NOTES:

REINFORCED CONCRETE

1. REINFORCING STEEL IS ASSUMED TO BE STEEL BARS CONFORMING TO ASTM A15 UNLESS NOTED OTHERWISE.
2. CAST-IN PLACE CONCRETE STRENGTH IS ASSUMED AS FOLLOWS:

	MINIMUM COMPRESSIVE STRENGTH AT 28 DAYS
GIRDERS, PIERS AND DECK	3000 PSI
ALL OTHER CONCRETE	3000 PSI

3. CONSTRUCTION JOINTS HAVE NOT BEEN DOCUMENTED
4. CONCRETE COVER SHALL BE ASSUMED AS 1½" UNLESS NOTED OTHERWISE.

MISCELLANEOUS STEEL

1. MISCELLANEOUS STEEL SHAPES AND PLATES, SHALL BE ASSUMED TO CONFORM TO ASTM A7.
2. RIVETS SHALL BE ASSUMED TO CONFORM TO ASTM A141.

ABBREVIATIONS AND SYMBOLS FOR STRUCTURAL DRAWINGS

AB	ANCHOR BOLT
ALT	ALTERNATE
BOF	BOTTOM OF FOOTING
BOT	BOTTOM
BRG	BEARING
CIP	CAST-IN-PLACE
CJ	CONSTRUCTION JOINT
CLR	CLEAR
CONC	CONCRETE
CONN	CONNECTION/CONNECTING
CONST	CONSTRUCTION
CONT	CONTINUOUS
d	REBAR DIAMETER
DIA	DIAMETER
DIM	DIMENSIONS
DN	DOWN
DWG	DRAWING
E	EPOXY COATED
EF	EACH FACE
EW	EACH WAY
EA	EACH
EHW	EXTREMELY HIGH WATER
EL	ELEVATION
ELECT	ELECTRICAL
ELW	EXTREMELY LOW WATER
EQ	EQUAL
EXIST	EXISTING
EXP JT	EXPANSION JOINT
FF	FAR FACE
FND	FOUNDATION
FIN	FINISH
GA	GAUGE
GALV	GALVANIZED
HORIZ	HORIZONTAL
ID	INSIDE DIAMETER
INTERM	INTERMEDIATE
JT	JOINT
LLH	LONG LEG HORIZONTAL
LONGIT	LONGITUDINAL
MAX	MAXIMUM
MIN	MINIMUM
NF	NEAR FACE
OC	ON CENTER
OD	OUTSIDE DIAMETER
OPP	OPPOSITE
OPNG	OPENING
pcf	POUNDS PER CUBIC FOOT
PL	PLATE
PSF	POUNDS PER SQUARE FOOT
PSI	POUNDS PER SQUARE INCH
RAD	RADIUS
REINF	REINFORCEMENT
REF	REFERENCE
SHT	SHEET
SIM	SIMILAR
SPA	SPACING/SPACE
SPEC	SPECIFICATION
SQ	SQUARE
STA	STATION
STD	STANDARD
STL	STEEL
SYM	SYMMETRICAL
T	THICKNESS
THRU	THROUGH
TOT	TOTAL
TYP	TYPICAL
UNO	UNLESS NOTED OTHERWISE
VERT	VERTICAL
W/	WITH
XS	EXTRA STRONG
Ø	DIAMETER
±	APPROXIMATELY
℄	CENTER LINE

Revisions			
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Checked by: N. TOHOLSKI

US Army Corps of Engineers

CORTE MADERA CREEK PROJECT
LAGUNITAS ROAD BRIDGE
STRUCTURAL EVALUATION

INDEX OF DRAWINGS, GENERAL NOTES ABBREVIATIONS AND SYMBOL

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Plot Date:

DESIGN CRITERIA

REFERENCES

AASHTO-LRFD	: STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 2nd EDITION. THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, INC. (1998)
CALTRANS BDS	: BRIDGE DESIGN SPECIFICATIONS, CALTRANS, AUGUST 1986 WITH REVISIONS THROUGH 2001
ACI 318-99	: AMERICAN CONCRETE INSTITUTE BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE AND COMMENTARY (1999)
AISC-LRFD	: AMERICAN INSTITUTE OF STEEL CONSTRUCTION, MANUAL OF STEEL CONSTRUCTION - LOAD AND RESISTANCE FACTOR, 2nd EDITION (1994)

GENERAL

DESIGN LOADS

1.0 DEAD LOAD (D):

WEIGHT OF STRUCTURE AND ALL PERMANENTLY ATTACHED COMPONENTS AND SYSTEMS.

UNIT WEIGHTS: REINFORCED CONCRETE 150 pcf
STEEL 490 pcf

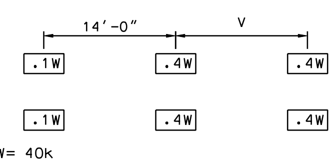
2.0 LIVE LOADS (L):

UNIFORM LOAD AND VEHICLE LOADS SHALL BE APPLIED TO THE SAME AREA SIMULTANEOUSLY.

BRIDGE DECK (EXCEPT AS NOTED BELOW)

UNIFORM LOAD (LU) 640 PSF

TRUCK LOAD (LH) AASHTO HS20-44



A 30% IMPACT FACTOR LOAD (I) SHALL BE APPLIED TO THE LIVE LOAD, AT THE ELEVATION OF DECK SLAB.

3.0 WIND LOADS: (W): NOT APPLICABLE FOR THIS EVALUATION.

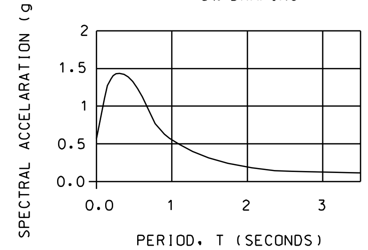
4.0 TEMPERATURE LOADS (T): NOT APPLICABLE FOR THIS EVALUATION.

5.0 EARTH PRESSURE LOADS (E): SEE GEOTECHNICAL REPORT.

6.0 SEISMIC LOADS (EQ):

DESIGN EARTHQUAKE LEVEL	MCE
PROBABILITY OF EXCEEDANCE IN 50 YRS	10%
DAMPING	5%
PEAK GROUND ACCELERATION, PGA	0.40g

ACCELERATION RESPONSE SPECTRUM
- 5% DAMPING



DESIGN CRITERIA

THE PIERS, GIRDERS, AND DECK STRUCTURE SHALL BE EVALUATED FOR THE FOLLOWING LOAD COMBINATIONS AND FACTORS:

GROUP	γ	D	L + I	E	EQ
I	1.30	1.0*	1.67	1.0	0
VII	1.0	1.0	0**	1.0	1.0***

* USE 0.90 FOR CHECKING MEMBERS FOR MINIMUM AXIAL LOAD AND MAXIMUM MOMEMT.

** 10% OF THE UNIFORM DECK LIVE LOAD IS ASSUMED FOR HORIZONTAL EARTHQUAKE MASS CALCULATIONS.

*** EARTHQUAKE SHALL BE APPLIED IN THE FOLLOWING LOAD DIRECTION COMBINATION:
1.0 LONG. + 0.3 TRANS.
0.3 LONG. + 1.0 TRANS.

PIER ASSESSMENT CRITERIA:

1. THE PIERS SHALL BE ASSESSED IN ACCORDANCE WITH THE REQUIREMENTS OF CALTRANS BDS.

SUPERSTRUCTURE ASSESSMENT CRITERIA:

1. THE DECK STRUCTURE SHALL BE ASSESSED IN ACCORDANCE WITH THE REQUIREMENTS OF CALTRANS BDS.
2. SEISMIC FUNCTIONAL REQUIREMENTS INCLUDE:

EARTH QUAKE LEVEL	PERFORMANCE OBJECTIVE
MCE	THE BRIDGE STRUCTURE SHALL REMAIN IN SERVICE. REPAIRS SHALL BE MINOR AND REQUIRE NO SIGNIFICANT INTERRUPTION OF BRIDGE OPERATIONS.

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US Army Corps of Engineers

CORTE MADERA CREEK PROJECT
LAGUNITAS ROAD BRIDGE
STRUCTURAL EVALUATION

DESIGN CRITERIA

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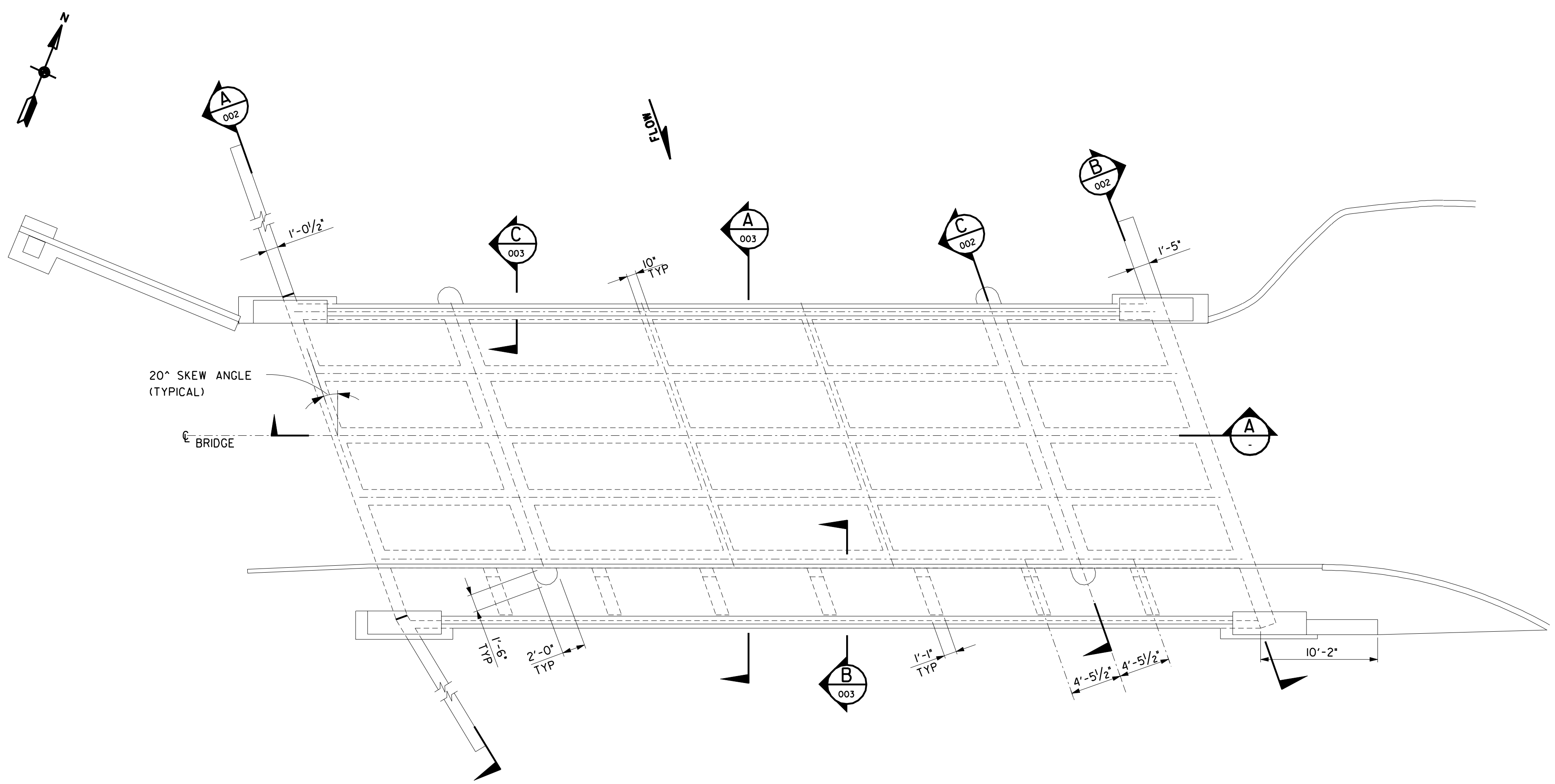

PLAN
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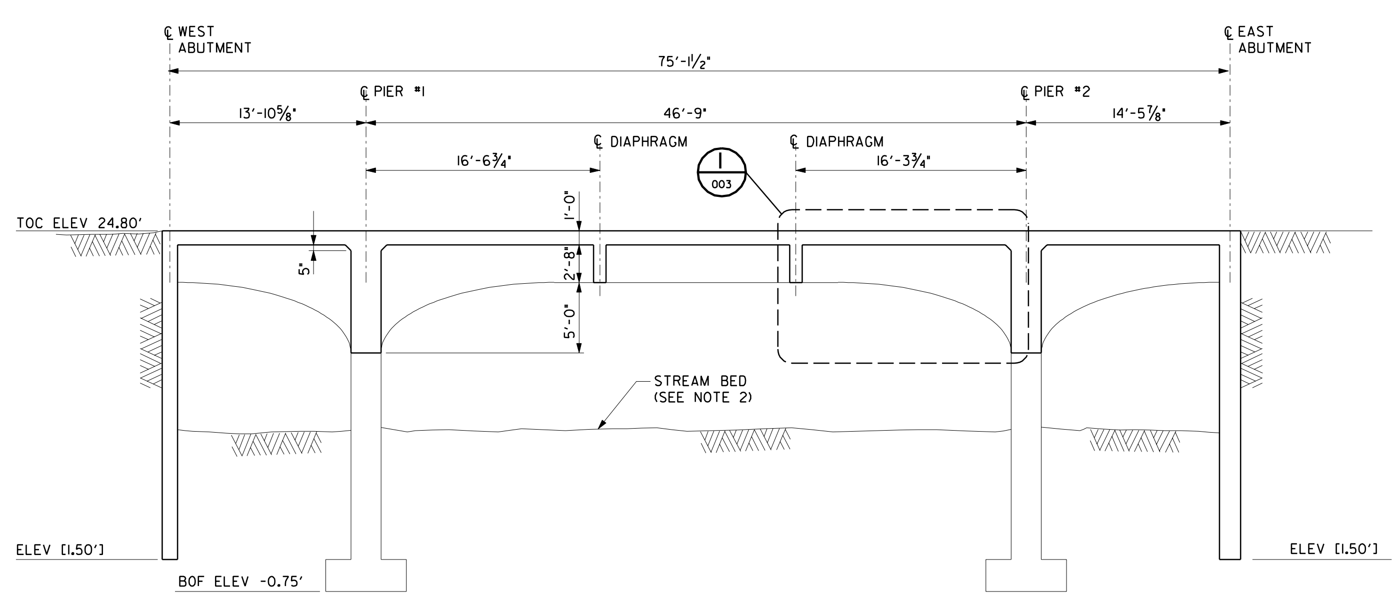
NOTES:

1. DATA MARKED WITH [ ] ARE ASSUMED.
2. CREEK BED ELEVATION AS DETERMINED DURING GEOTECH INVESTIGATION 10-15-01 TO 10-19-01
3. ELEVATIOINS ARE BASED ON NATIONAL GEODETIC VERTICAL DATUM (NGVD).



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PLAN AND ELEVATION

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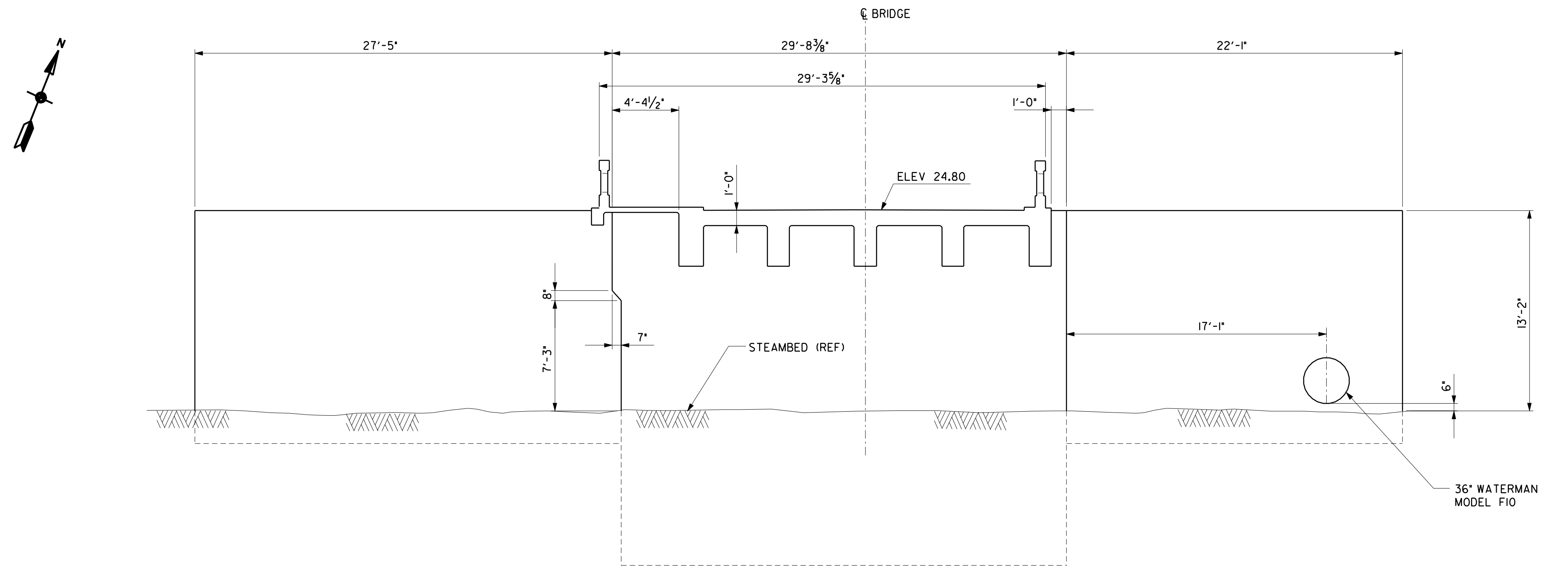

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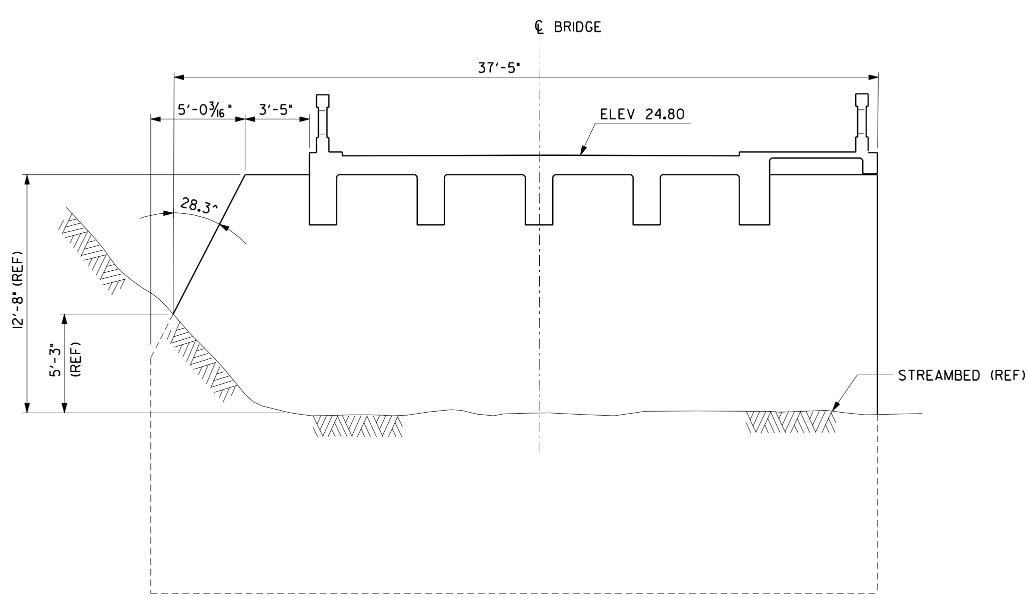

HEADWALL

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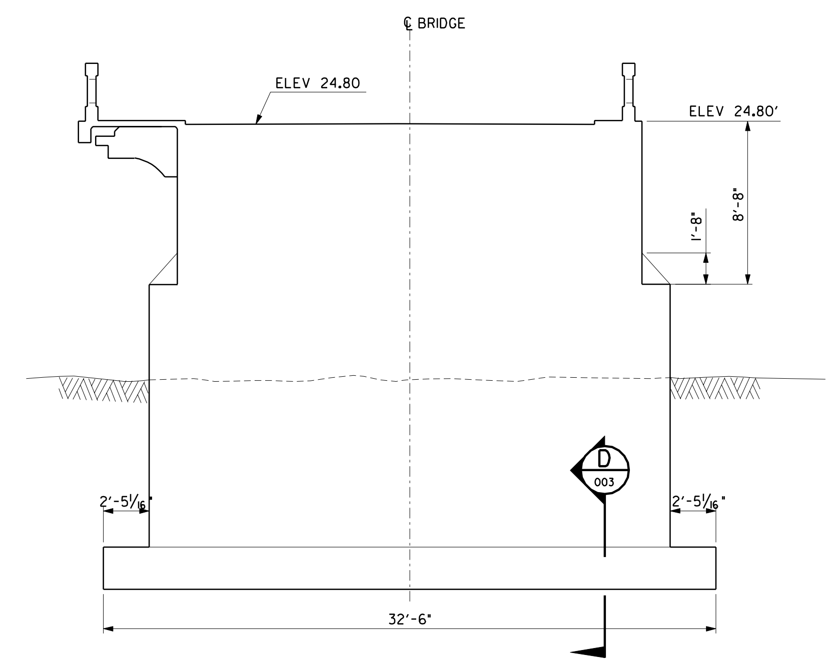

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LAGUNITAS ROAD BRIDGE
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PIER AND ABUTMENT ELEVATIONS

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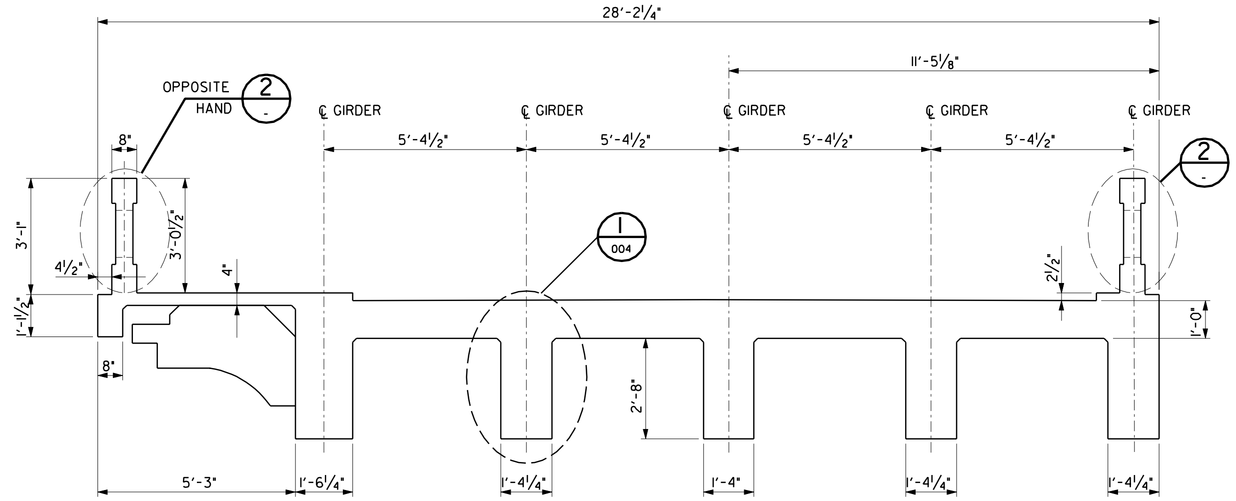

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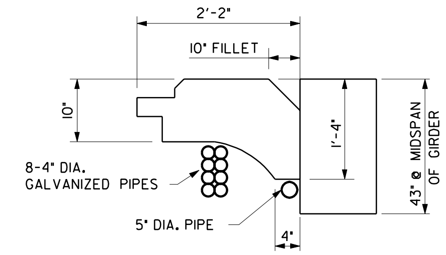

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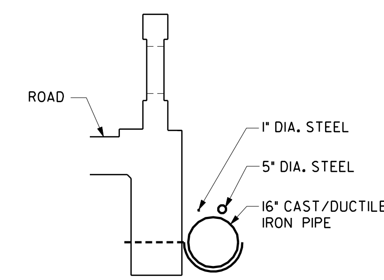

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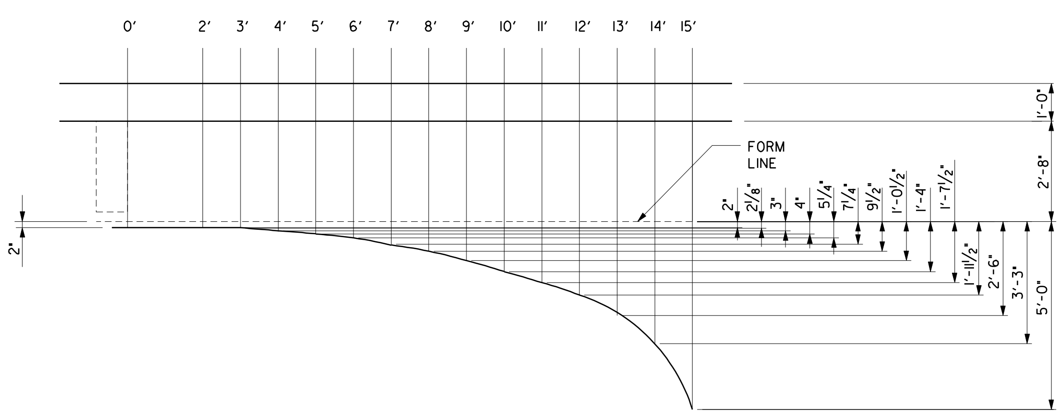

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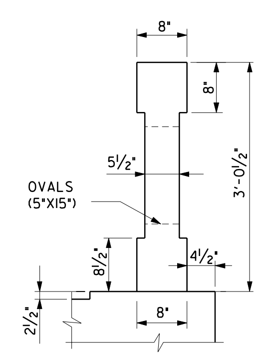

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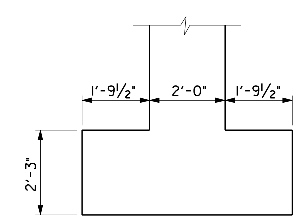

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U.S.ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

US Army Corps of Engineers

CORTE MADERA CREEK PROJECT
LAGUNITAS ROAD BRIDGE
STRUCTURAL EVALUATION

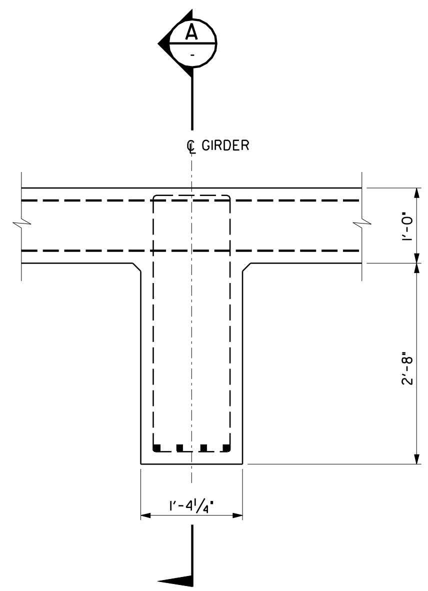
Designed by: R. FILGAS
Drawn by: J. GARCIA
Checked by: N. TUHOLSKI

SECTIONS AND DETAILS

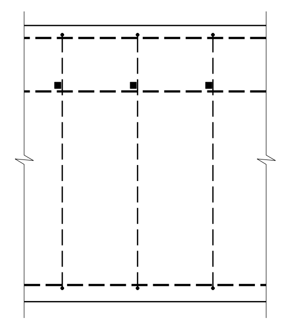
Scale: AS NOTED
Date:
Drawing Code:
Sheet reference number: S-03
Sheet ____ of ____

FILE: $FILE$
DATE: $DATE$ @ $TIME$ By: $USER$

Plot Date:



DETAIL 1/003
SCALE: ½"=1'-0"



SECTION A/-
SCALE: ½"=1'-0"

NOTES:

1. DESIGN OR AS-BUILT DRAWINGS HAVE NOT BEEN LOCATED.
2. REINFORCEMENT AS INDICATED BY SOLID LINES IS DETERMINED BY VISUAL INSPECTION IN AREAS OF MINIMAL OR SPALLED CONCRETE COVER.
3. REINFORCEMENT AS INDICATED BY DASHED LINE (----) IS ASSUMED AS CONTEMPORARY FOR DESIGN AND CONSTRUCTION.
4. MAIN GIRDER BOTTOM REINFORCEMENT IS SQUARE REBAR, ASSUMED TO BE IN ACCORDANCE WITH SPECIFICATION ASTM A-16.

Revisions			
Symbol	Descriptions	Date	Approved

BEN C. GERWICK, INC.
20 CALIFORNIA STREET, SUITE 400
SAN FRANCISCO 94111
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STRUCTURAL EVALUATION

Designed by: R. FILGAS
Drawn by: J. GARCIA
Checked by: N. TUHOLSKI

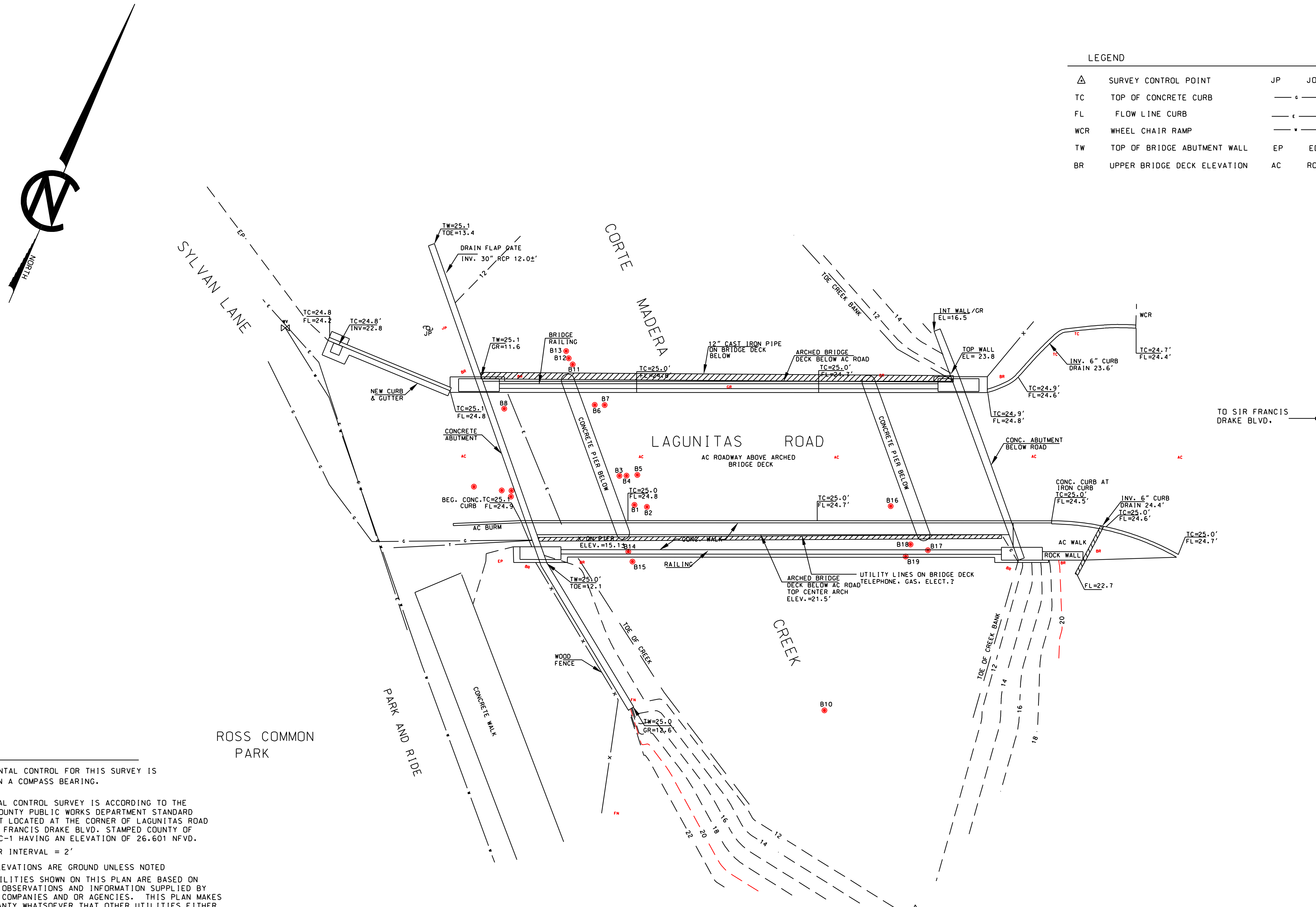
REINFORCEMENT DETAILS

Scale: AS NOTED
Date:
Drawing Code:
Sheet reference number: S-04
Sheet ____ of ______

FILE: $FILE$
DATE: $DATE$ @ $TIME$ By: $USER$

Plot Date:

REVISIONS		
DATE	BY	NO.



LEGEND

△	SURVEY CONTROL POINT	JP	JOINT POLE
TC	TOP OF CONCRETE CURB	— G —	GAS LINE
FL	FLOW LINE CURB	— E —	ELECTRIC
WCR	WHEEL CHAIR RAMP	— W —	WATER
TW	TOP OF BRIDGE ABUTMENT WALL	EP	EDGE OF PAVEMENT
BR	UPPER BRIDGE DECK ELEVATION	AC	ROAD BED ELEVATION

NOTES:

1. HORIZONTAL CONTROL FOR THIS SURVEY IS BASED ON A COMPASS BEARING.
2. VERTICAL CONTROL SURVEY IS ACCORDING TO THE MARIN COUNTY PUBLIC WORKS DEPARTMENT STANDARD MONUMENT LOCATED AT THE CORNER OF LAGUNITAS ROAD AND SIR FRANCIS DRAKE BLVD. STAMPED COUNTY OF MARIN RC-1 HAVING AN ELEVATION OF 26.601 NFVD.
3. CONTOUR INTERVAL = 2'
4. SPOT ELEVATIONS ARE GROUND UNLESS NOTED
5. THE UTILITIES SHOWN ON THIS PLAN ARE BASED ON SURFACE OBSERVATIONS AND INFORMATION SUPPLIED BY UTILITY COMPANIES AND OR AGENCIES. THIS PLAN MAKES NO WARRANTY WHATSOEVER THAT OTHER UTILITIES EITHER SURFACE OF SUBSURFACE DO OR DO NOT EXIST. PRIOR TO SITE PLANNING OR CONSTRUCTION ACTIVITIES THE CONTRACTOR SHALL ASCERTAIN THE TRUE LOCATION OF ANY UNDERGROUND UTILITIES AND SHALL BE RESPONSIBLE FOR DAMAGES TO ANY PUBLIC OR PRIVATE UTILITIES WHETHER SHOWN OR NOT SHOWN HEREON.
6. ANY DISCREPANCY BETWEEN THE ELECTRONIC FILE PROVIDED TO CLIENT AND THE SIGN AND SEALED HARD COPY, THE HARD COPY WILL PREVAIL.
7. FOR A COMPLETE LIST OF POINTS AND ELEVATIONS SEE ELECTRONIC FILE.

GRAPHIC SCALE

8 0 4 8 16 32

(IN FEET)
1 inch = 8 ft.

PREPARED BY:

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PLS 7053

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TOPOGRAPHIC SURVEY
LAGUNITAS BRIDGE
ROSS, CALIFORNIA

DATE: NOV. 2001
SCALE: 1"=8'
DRAWN: LC
CHECKED:
JOB NO: 01-349T

1

SHEET 1 OF 1