Final Rehabilitation and Retrofit Strategy Report

Seventh Street Bridge Project
Bridge No. 38C-0023

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1.0 Executive Summary

The purpose of this report is to evaluate the conclusions established in the original feasibility study[1] and strategy report[2] performed by the URS Corporation under contract with the City of Modesto in 2001 for Seventh Street Bridge located in the City of Modesto.

Seventh Street Bridge was built in 1916 and carries an important two-lane urban road over the Tuolumne River and surrounding flood plain. The bridge is a historic and unique structure type called a "Canticrete" arch bridge, which is composed of steel trusses encased in concrete arches with mid-span joints, typically located in every other span. The structure is approximately 34'-8" wide and 1,165-feet long, and consists of 14 spans. The superstructure is attached to concrete abutments and piers supported on pile foundations.

Rehabilitation and retrofit or replacement of the structure is necessary for the following reasons. First, the bridge is classified as structurally deficient and has a sufficiency rating of 2 out of a possible 100 rating. There are a multitude of structural elements that exhibit areas of significant cracking and concrete spalling with some exposed reinforcement or structural steel, including the barrier railing and sidewalks, joint headers between cantilevers, deck soffit, floor beams, arches, pier curtain walls, and abutments. Second, the inventory and operating ratings for the bridge are 6.5 tons and 11 tons, while the structure is posted for four-tons. These load ratings are well below modern highway vehicular loadings and thus, no trucks are allowed on the bridge. Third, there are differential vertical defections at mid-span cantilever joints up to three-inches in magnitude, suggesting overstressing of the steel truss has occurred. Last, the structure does not satisfy the required freeboard criteria, scour issues have occurred in the past along the river piers, there remains the potential for structural instability after a flood event, and the bridge is classified as scour critical.

This report first presents an overview of the project and summarizes the existing roadway and bridge conditions. Next, the structural analyses performed to assess the vulnerabilities of the existing bridge are described in detail and potential vulnerabilities are identified and compared to the vulnerabilities recognized in the original feasibility study[1]. Rehabilitation and retrofit efforts required to repair the vulnerabilities are then discussed in detail and compared to the measures identified in the original feasibility study[1]. Last, an updated cost estimate for the recommended rehabilitation and retrofit measures is provided.

2.0 Project Description

The Seventh Street Bridge was constructed in 1916 and is located along Seventh Street in the City of Modesto, California. The structure is located about ½ mile east of Highway 99 at a longitude of 120° 59′ 38″ and a latitude of 37° 37′ 37″ (see Project Site Map in Figure 1). Seventh Street is an important two-lane urban road that carries traffic over the Tuolumne River and surrounding flood plain. The bridge is a unique structure type called a "Canticrete" arch bridge, which is composed of steel trusses encased in concrete arches with mid-span joints, typically located in every other span. The structure is approximately 1,165-feet long and consists of 14-spans that are attached to concrete abutments and piers supported on pile foundations. The bridge is eligible for inclusion in the National Register of Historic Places and is considered to be a significant structure.

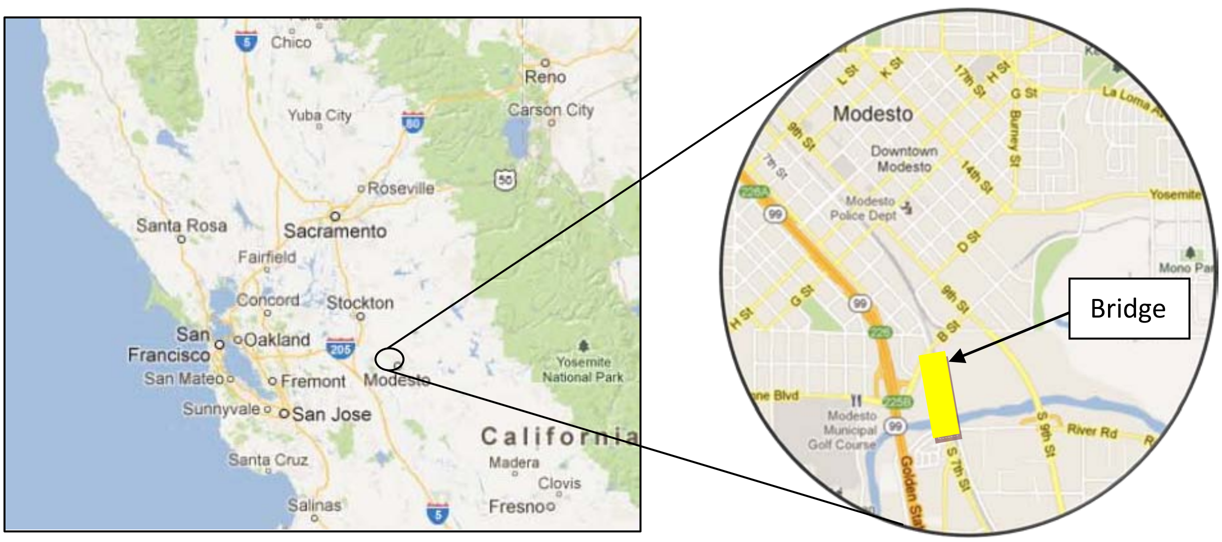

Figure 1- Project Site Map

The bridge is currently defined as structurally deficient with a load restriction of 4-tons and is in poor condition (see Section 3.2 of this report for further details). Consequently, rehabilitation and retrofit or replacement of the bridge is necessary and currently being investigated as part of this joint project between the County of Stanislaus and the City of Modesto. The purpose of the project is to enhance seismic safety and vehicular load capability, improve vehicular and pedestrian safety, and reduce congestion.

Previous and continuing study efforts supporting this project include preliminary environmental studies, preliminary hydraulics and scour analyses, preliminary foundations investigations, a historical property survey report, and structural rehabilitation/retrofit and replacement strategy reports.

3.0 Existing Conditions

3.1 Roadway

Seventh Street is classified as an on-system, urban, minor arterial and provides access over the Tuolumne River and surrounding flood plain between downtown Modesto and local farms, ranches, and residences located in the southern region of the city. Traffic counts performed by Fehr and Peers in 2012 indicated that the average daily traffic (ADT) along the roadway over the bridge is about 15,900 vehicles per day. According to the Caltrans inspection report dated October 13, 2011, closure of Seventh Street to traffic over the bridge necessitates an approximate one-mile long detour along Ninth Street. Based on the short roadway detour length, closure of Seventh Street in the vicinity of the bridge to facilitate rehabilitation and retrofit or replacement of the structure is feasible.

The roadway alignment in the vicinity of the bridge follows a reverse curve with the bridge located along a tangent between the two curves (see Figures 1 and 2). Along the tangent portion of the alignment, the cross section is crowned at the centerline of the roadway with cross slopes for drainage. The roadway profile consists of a crest type vertical curve with a maximum elevation change along the bridge profile of approximately two-feet. Seventh Street provides two 12-foot wide lanes, one lane for each direction of traffic, with no appreciable shoulders. The roadway asphalt is generally cracked and abraded with multiple potholes located along the bridge approaches and deck overlay. According to the Caltrans inspection report, the approach roadway alignment has an appraisal rating of 6 (satisfactory condition), though the approach railing is substandard.

Figure 2 - Seventh Street along Bridge

3.2 Bridge

3.2.1 Description of Bridge

Seventh Street Bridge was constructed in 1916 and is designated bridge number 38C-0023 (see Figure 3). The structure is approximately 34'-8" wide and 1,165-feet long, and consists of 14-spans, with two 54-foot spans, nine 84-foot spans, two 100-foot spans, and one 101-foot span. The 100-foot spans and 101-foot span are located over the low-flow Tuolumne River, while the remaining spans are above the surrounding flood plain.

Figure 3 – West Edge of Bridge Looking South

The bridge is a unique structure type called a "Canticrete" arch bridge in which the primary structural members that span between supports are composed of steel trusses embedded in concrete arches. This design relies solely on the steel truss to resist superstructure loads while the concrete serves to provide lateral support and corrosion protection to the steel truss members. The steel truss/concrete arch is typically continuous over one span and cantilevered over the adjacent spans with mid-span joints at the crown of the arch. This pattern is repeated along the length of the bridge such that the mid-span joints are located in every other span.

The superstructure consists of a concrete deck with an approximate three-inch thick asphalt overlay supported by transverse floor beams spaced at 8-feet 5-inches apart that connect to the steel truss/concrete arch members. A horizontal steel bracing system is embedded in the concrete deck, most likely to laterally support the steel trusses during construction prior to deck concrete placement. Floor beams or end diaphragms are used on each side of the mid-span joints to support the deck at the joint. At the north abutment, the superstructure is monolithically connected to the abutment by a 12-inch thick end diaphragm. At the south abutment, the superstructure steel truss/concrete arch bears on an abutment seat with a ¼" expansion joint. Both abutments are high cantilever types of abutments. At the piers, 12-inch thick diaphragm walls attach the superstructure to cellular concrete piers that vary in size and degree of architectural treatment. The abutments and piers are perpendicular to the axis of the superstructure and are supported on pile foundations. The piles consist of 14-inch square reinforced concrete piles approximately 20-feet long along the flood plain and potentially untreated timber piles of unknown length along the river spans. In general, concrete elements are lightly reinforced by current standards and the concrete arches are unreinforced, with the exception of the embedded steel truss.

The bridge provides two 12-foot wide lanes with four-foot wide sidewalks and no appreciable shoulders. Note that the current travel way across the bridge does not comply with the guidelines specified in the Caltrans Highway Design Manual[3] and the American Association of State Highway and Transportation Officials (AASHTO) Policy on Geometric Design of Highways and Streets[4]. Collectively, these documents recommend 12-foot wide lanes with eight-foot wide shoulders and six-foot wide sidewalks for this urban arterial street. In addition, the bridge barriers are obsolete and do not meet current AASHTO vehicular crash load safety guidelines for traffic barriers. It is also important to acknowledge that the barriers are formed around the upper chord of the steel truss/concrete arch members (see Sections 4.2.1.1 and 5.1.1 for implications).

3.2.2 Previous Investigations

Based on an engineering study performed on the Seventh Street Bridge by the Public Works Departments for the City of Modesto and Stanislaus County in March of 1976[5], the structure was originally designed for an H-12 truck loading but posted for 10 tons at the time of the study. Differential deflections between ends of cantilevers at some mid-span joints became apparent in the early 1960's and increased drastically at the northern-most joint in the early 1970's. At this location, the maximum differential deflection at the time of the study was approximately three-inches. The study concluded that this phenomenon was due to overstressing of the steel truss along the cantilevered portions of the superstructure. Due to these differential deflections, it can be inferred that the vertical shear keys initially installed between cantilevers, as shown in the original plans, are no longer functional and must have been damaged over time.

Remedial measures were later implemented at the northern-most joint by installing a steel frame consisting of W10 x 60 sections connected by gusset plates and three-inch diameter pins and bolted to concrete footings (see Figure 4); later, shims were required due to additional differential deflection between the cantilevers. In addition, an asphalt overlay was placed to eliminate the differential displacements at the mid-span joints along the roadway surface and the bridge was posted for 4 tons. The study also indicated that the timber piles in the river spans had been intermittently exposed based on observations by County staff.

Figure 4 - Span 13 Support at Joint

Based on field investigation notes produced by URS Corporation in January of 2001, the floor beams exhibit areas with deep spalls that expose the wire mesh and steel truss, which have corroded as a result. The report also notes that the bridge was designed prior to enactment of the first seismic design codes in 1918.

3.2.3 Caltrans Bridge Inspection Reports

The most recent Caltrans bridge inspection report dated October 13, 2011 for Seventh Street Bridge is provided for reference in Appendix G and indicates the following issues. Note that the inspection report numbers the spans and piers from south to north in ascending numerical order, regardless of the original numbering configuration in the as-built plans. The bridge is classified as structurally deficient with a sufficiency rating of 2, which is the lowest rating of any Caltrans District 10 bridges. Since the sufficiency rating is below 50, the structure is eligible for rehabilitation or replacement per federal guidelines. The inventory and operating ratings for the bridge are 6.5 tons and 11 tons, respectively. The condition ratings of structural elements are as follows: deck = 5 (fair condition), superstructure = 4 (poor condition), substructure = 5 (fair condition), and channel and channel protection = 6 (satisfactory condition). In addition, the bridge is eligible for historical significance.

Based on the 2011 inspection report, as well as earlier Caltrans inspection reports, the structure is generally in poor condition with significant signs of deterioration as follows.

1. A multitude of structural elements exhibit areas of significant cracking and concrete spalling with some exposed reinforcement, including the barrier railing and sidewalks, joint headers between cantilevers, floor beams, arches, pier curtain walls, abutments, and architectural features along the railing.
2. There is vehicular collision damage on the east arch near the south abutment.
3. The deck soffit exhibits concrete spalls up to three-feet wide, cracking, and efflorescence formation.
4. Large portions of the curtain walls at Piers 3 and 4 (river piers) have fractured resulting in large cavities up to ten-feet long and five-feet deep.
5. Large potholes are evident near both abutments and the north approach has settled by about three inches, causing an uneven driving surface.
6. Heavy graffiti is prevalent on most pier walls and abutments.
7. There is a two-inch vertical offset at Span 5 and an approximate one-inch vertical offset at Span 7 between the railings on opposite sides of the mid-span joints. It is also noted that, at Span 5, there is a one-inch vertical offset along the driving surface. In addition, there is a 3.5-inch vertical offset in Span 13 between the railings, as discussed above. The differential deflections at the joints have caused the joints to close and damaged the ends of the cantilevers along the deck, floor beams, and railing.
8. Based on previous Caltrans inspection reports, scour issues have occurred at Piers 3 and 4, with up to five-feet of undermining previously observed. Some scour remediation has been performed at these locations using sacked concrete. In addition, major cracking and differential settlement has been identified at Pier 3.
9. Prior to 1979, rehabilitation work was performed to repair cracking and spalling at the joints, refurbish cosmetic characteristics of the railing and sidewalks, and resurface the asphalt overlay.

3.2.4 Environmental Studies

Preliminary environmental studies have been performed and continue to be conducted to determine environmental issues associated with the project. According to the Preliminary Environmental Study (signed by Caltrans on October 11, 2012), a Complex Environmental Assessment (EA) will be required for National Environmental Policy Act (NEPA) review. Stanislaus County has indicated that a focused Environmental Impact Report (EIR) will be required for California Environmental Quality Act (CEQA) compliance. Additional environmental studies will be performed to confirm the following potential impacts:

- Traffic changes and related noise and air quality impacts;
- Aesthetic impacts from visual changes;
- Loss of the existing bridge as a historic resource;
- Potential disruption of hazardous materials;
- Increased water pollution during construction;
- Loss of riparian vegetation and channel margin wetlands;
- Barriers to anadromous fish migration;
- Loss of park habitat;
- Social and economic effects from property acquisition.

Agencies that have jurisdictional authority and must be involved in the project approval process include the U.S. Army Corps of Engineers, U.S. Fish and Wildlife Service, National Marine Fisheries Service, Central Valley Regional Water Quality Control Board, Central Valley Flood Protection Board, State Lands Commission, and California Department of Fish and Wildlife.

3.2.5 Hydraulics Report

Preliminary hydraulics and scour analyses were performed by WRECO in October of 2012. The estimated flow, in cubic feet per second (CFS), water surface elevation (WSE), and estimated scour depth at each pier for the 100-year, 200-year, and 500-year flood events are listed in Table 1 below.

For the 100-year event, the existing bridge has approximately 4.5-feet of freeboard at the north abutment and no freeboard at the south abutment. For the 200-year and 500-year events, the bridge is completely overtopped. The maximum scour listed in Table 1 accounts for local scour, long term scour, which is negligible, and contraction scour that varies from about two-feet to four-feet. See Appendices D and E with the hydraulics summary and scour analysis for further details.

Table 1 - Hydraulic Flow Information

Flood Event Reoccurrence	Flow (CFS)	WSE (Ft)	Scour Depth (Ft)
100 Year	70,000	75	6 – 18
200 Year	105,400	80	7 – 20
500 Year	154,000	86	NA

Flood Event Reoccurrence	Maximum Scour Below Bottom of Footing (Ft)	
	River Piers	Flood Plain Piers
100 Year	15	5
200 Year	19	11

3.2.6 Geotechnical Investigations

A preliminary foundation report (PFR) was produced by Taber Consultants in March of 2000 for the Seventh Street Bridge[6]. Based on this report, the native foundation material consists of an older alluvium layer expected to be stable under seismic loading and an upper more recent and weaker alluvium layer. The project site is near the Midway San Joaquin fault zone, the depth to bedrock material is about 100 feet, and the soil profile is Type D. The ground water table is approximately two to eight feet in elevation. Liquefaction potential is high and expected settlement due to lateral spreading ranges from two-inches to one-foot. The report speculates that the timber piles, but not the concrete piles, extend into the older alluvium layer. It is not known if the timber piles are treated, and thus, may deteriorate in wet-dry cycles.

As part of the effort associated with this report, the information presented in the original PFR[6] was reviewed and a foundation evaluation developed by CH2M HILL. The foundation evaluation provides estimates of the pile lateral stiffness values with group effect factors for each abutment and pier, soil passive resistance at abutment walls and pier caps, pile axial capacities, and the acceleration response spectral (ARS) curves. This information is presented in Appendix F. The estimated values from the foundation evaluation are used in the structural modeling for evaluation of the bridge.

3.2.7 Historical and Architectural Considerations

Architectural Resources Group produced a preliminary study and survey of Seventh Street Bridge in 1998[7] to evaluate rehabilitation or replacement of the bridge in accordance with the National Historic Preservation Act and to identify any archaeological or historic resources within the Area of Potential Effects (APE). In addition, a Historic Property Survey Report and Historic Archaeological Survey Report were performed by Foothill Resources in March of 1996[8]. The reports determined that the bridge is eligible for inclusion in the National Register of Historic Places and is considered to be a significant structure. However, no other historic cultural resources and no archaeological resources were identified in the APE. The reports did not address requirements by CEQA, NEPA, and Modesto's municipal preservation ordinance as these regulations relate to cultural resources.

Figure 5 – Lion Statue at Bridge Entrance

The Seventh Street Bridge is the only remaining major example in the San Joaquin Valley of the "City Beautiful" bridges, adorned in "Beaux Arts" classical details, and is the most impressive surviving example of John B. Leonard's "Canticrete" bridge design. The structure exhibits a wide variety of architectural details, as shown in Figures 2 and 3. Large recumbent lions on pedestals are located at the ends of the bridge railing that artistically signify the lions are protecting the bridge (see Figure 5). Due to these statues, the bridge is often referred to by the public as the "Lion Bridge". The barrier railing has an arched window design consisting of round arched openings alternating with panels, creating strong ornamental bands. At the approaches, the railing flares outward to accommodate concrete benches. The concrete sidewalks are scored in a checkerboard pattern with a thin layer of integrally-colored concrete. The piers are treated hierarchically, with the piers closest to the approaches interpreted as the largest and most significant. The architecturally more significant piers are treated with reveals along the base of the pier and ornamental pedestals with medallion-like lighting fixtures, or electroliers, at the top of the piers above the barrier railing. Due to the historic classification and detailed architectural treatment involved with the bridge, it is important that any rehabilitation and retrofit measures preserve the architectural character of the structure.

3.3 Utilities

There is an electrical line located along the east railing and sewer storm drains adjacent to both sides of the bridge. In addition, overhead electrical and telephone lines are located near both ends of the bridge. There are no other known utilities located in the vicinity of the bridge.

4.0 Structural Analysis

4.1 Modeling Approach and Assumptions

The design criteria used for the structural analysis consists of AASHTO LRFD[9] for dead and vehicular loads and Caltrans Seismic Design Criteria (SDC) version 1.6[10] for seismic loads. Although this is an evaluation of an existing structure, the goal of the rehabilitation is to provide a structure that will support current vehicular loads far into the future. Thus, AASHTO LRFD is used for vehicular loading in order to determine the rehabilitation efforts necessary to repair and upgrade the structure to meet vehicular loading requirements for new structures. The analyses investigate dead loads, vehicular loads, seismic loads, and settlement loads caused by liquefaction.

The existing structure is analyzed using the CSI Bridge program, which is an enhanced bridge modeling platform that utilizes the SAP2000 finite element modeling software produced by Computers and Structures, Inc. The steel truss elements are modeled using frame elements with the section properties generated in a section builder module of the CSI Bridge program. All diagonal and vertical truss elements are assigned moment releases at both ends to capture the truss pinned connection behavior. Since the top and bottom chord members are continuous at panel points, no moment releases are assigned between adjacent chord members. Two-dimensional shell elements are used to model the bridge deck, concrete arches surrounding the steel truss members, floor beams that connect the two arches, and diaphragm walls. The concrete arch shell elements connect to the steel truss members at the truss nodes. The steel truss members embedded in the concrete deck and floor beams do not significantly contribute to the stiffness of the structure and thus, are not explicitly modeled in the analyses. The concrete arch surrounding the steel truss is assumed to provide continuous lateral bracing to the steel members.

Each pier is modeled using a single frame element with rotation fixity and lateral springs at the footing elevations to simulate the foundation stiffness. Soil springs are similarly used at the abutments. The soil spring stiffness values used in the model include the summation of the stiffness associated with the piles and the passive soil pressure, except at the piers where liquefaction is expected at the elevation of the pile cap, where the passive soil resistance is assumed to be zero.

Screenshots from the CSI Bridge model that shows a single frame of the bridge between mid-span joints is provided in Figures 6 and 7 below. In Figure 6, the blue lines represent the steel truss members and the red shell elements represent the concrete arches and deck. A small gap can be seen at each end of the figure that represents the mid-span joints at the end of the cantilevers. In Figure 7, only the frame elements that represent the steel truss members and piers are shown.

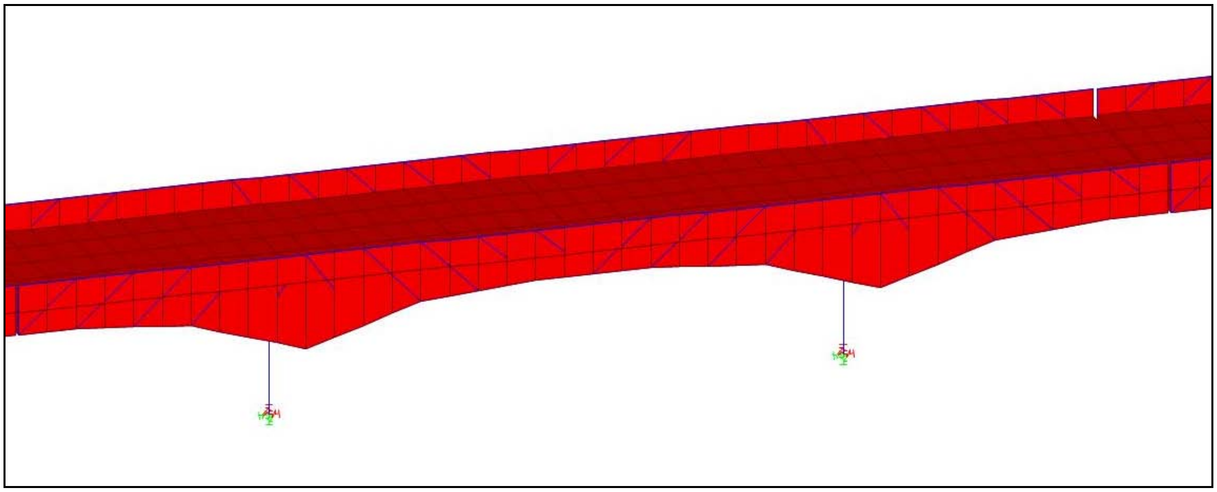

Figure 6 – Single Frame from CSI Bridge Model

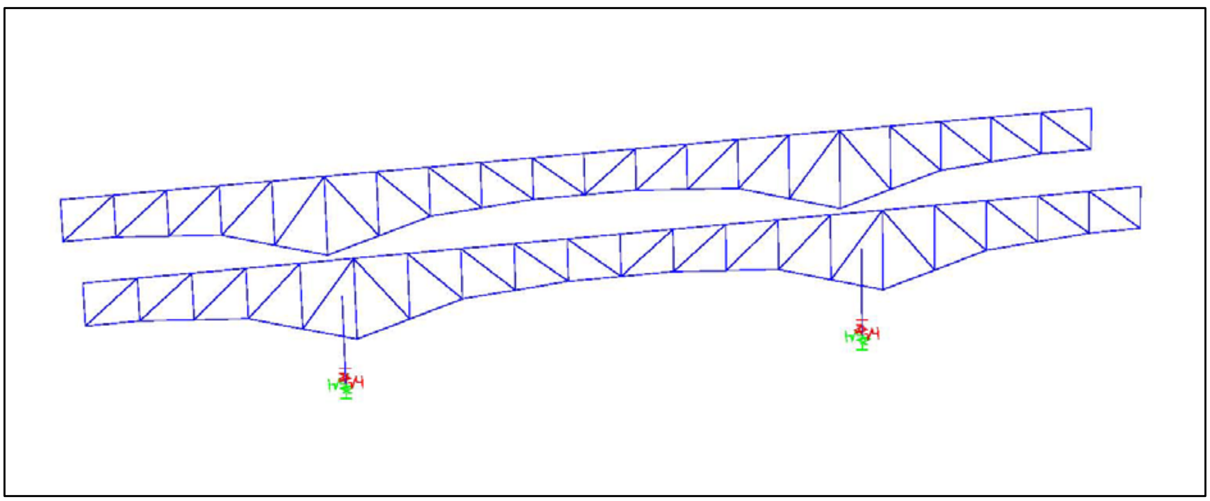

Figure 7 - Single Frame from CSI Bridge Model with only Frame Elements Shown

As described above, the superstructure is composed of steel trusses embedded in un-reinforced concrete arches. The concrete contributes both mass and stiffness to the behavior of the structure, although the stiffness component of the concrete applies only to uncracked concrete due to the un-reinforced or lightly-reinforced nature of the structure. Capturing the influence of the concrete on the structure in a refined manner requires non-linear material and geometric behavior associated with cracking of the concrete arches and necessitates use of a non-linear model with a time history analysis for the seismic evaluation. However, development of a non-linear model is not warranted at the vulnerability assessment level and unnecessary to sufficiently capture the basic performance of the structure and identify the fundamental vulnerabilities of the bridge. Therefore, a linear-elastic model that approximates the concrete behavior as described below, with a multi-modal spectral analysis for the seismic evaluation, is employed for evaluation of the structure.

Implementation of a linear elastic modeling approach requires a unique method to sufficiently capture the behavior of the structure without utilizing non-linear material and geometric properties. Note that modeling only the weight and mass, and not the stiffness, of the concrete arches would lead to extremely conservative results in the steel truss members. As an approximate method for capturing the stiffness, as well as the cracking behavior, of the concrete arches, the concrete is considered to contribute to the stiffness of the structure at locations where the concrete is uncracked for dead and vehicular loads. A separate elastic analysis was performed with dead and vehicular loads only to determine where the tensile stress exceeds the rupture stress of the concrete. At these locations, the concrete arch shell elements are assigned an elastic modulus of zero for all the full seismic and vehicular analyses to approximately capture the presence of cracking in the concrete. In general, the concrete arch shell elements with zero elastic modulus are located near the mid-spans of the continuous spans below the deck and at the piers above the deck. Since the stress in the concrete deck, floor beams, and diaphragm walls generally does not exceed the concrete rupture stress, none of the shell elements used to represent these members in the model are assigned an elastic modulus of zero.

Assigning the arch concrete material an elastic modulus equal to zero in the aforementioned regions above the deck causes the arch out-of-plane demands to be conservatively large. This is due to the way that the concrete mass is lumped at the model nodes and because the concrete is not being counted on for any resistance, which is conservative for the out-of-plane response. Ultimately, this level of conservatism does not impact the overall conclusion that these elements are not adequate for the Extreme-I loading case, as discussed in Section 4.2.2.1, since vertical and diagonal members located in regions with uncracked concrete (where concrete is allowed to contribute to the resistance) exhibit inadequacy issues as well. Note that the same behavior is not exhibited below the deck where the arch concrete is assigned an elastic modulus of zero since the floor beams provide lateral out-of-plane support to the truss at the panel points.

In the seismic analysis, the mid-span joints between cantilevers are connected using body constraints that allow independent translation and rotation in all directions for the tension model and fixed translation along the bridge longitudinal axis with the other degrees of freedom released for the compression model. The results from these two models are enveloped for the seismic response of the structure and the tension model is used for the vehicular analysis.

The following material properties are assumed in the analyses based on the AASHTO Manual for Bridge Evaluation (second edition, Sections 6A.5.2 and 6A.6.2) [11] and Caltrans SDC[10].

- Structural steel yield stress (Fy) = 30-ksi (built between 1905 and 1936);

- Structural steel ultimate stress (Fu) = 60-ksi (built between 1905 and 1936);
- Concrete compressive strength (f'c) = 2.5-ksi (built prior to 1959);
- Expected concrete compressive strength (f'ce) = 3.25-ksi (1.3 f'c per Caltrans SDC[10], the minimum f'ce = 5-ksi was not applied given the age of the structure and unknown quality of the concrete, which was not designed as a structural material, but rather to provide corrosion protection to the steel truss);
- Reinforcing yield stress (fy) = 33-ksi (unknown grade built prior to 1954).

In addition, the following assumptions are used in the analyses to evaluate the structure:

- The steel truss member connections, which consist of gusset plates with rivets, are not checked since connection details are not included in the as-built plans and the details cannot be measured in the field as the truss members are embedded in concrete. It is important to acknowledge that, due to the design practices at the time the bridge was constructed, it is unlikely that the connections were designed for the full ultimate capacities of the connecting truss members, as required by current code.
- The Seventh Street Bridge Modification Plans from 1978 indicate that a hanger plate retrofit strategy was employed to connect the cantilevers at the mid-span joints. Based on field inspection pictures of the structure, the hanger plate retrofit strategy was never constructed and thus, is not included in the model.
- The remaining fatigue life has not been investigated but could be a concern based on the age of the structure, the ADT, and potential overstressing of the steel truss, as identified in the aforementioned engineering study performed in March of 1976[5].
- The differential settlement at adjacent piers due to liquefaction is assumed to be about six-inches based on the original PFR[6] and geotechnical engineering judgment.

4.2 Vulnerabilities

The following sections present the vulnerabilities identified by the structural analyses described above. The vulnerabilities are categorized first by loading conditions and then by structural element. Detailed analysis results are summarized in Appendix A of this report and include support reactions, as well as maximum demand/capacity ratios (DCR's) in graphical format for the steel truss and floor beam members, for the various loading conditions investigated in this report. The results for the Extreme-I limit state are presented without the liquefaction induced settlement load included. Including the settlement load further increases the member demands and worsens the vulnerabilities identified in this report.

Table 2 provides a brief summary of the maximum DCR values for most of the structural elements identified in the vulnerabilities. Note that DCR values greater than 1.0 mean that the demand is greater than the capacity and thus, the member is inadequate for the given limit state. The values listed for the vertical and diagonal members are conservative for the Extreme-I Limit State due to the inertial forces associated with the concrete mass above the deck, as discussed above.

Table 2 - Maximum Element DCR Values

Element	Strength-I Limit State	Extreme-I Limit State
Truss Top Chord	1.2	4.1
Truss Bottom Chord	1.6	1.7
Truss Vertical	2.5	13.1
Truss Diagonal	2.6	2.7
Concrete Deck	1.6	-
Deck Floor Beam	1.5	-
Diaphragm Wall to Pier Connection	-	5.0
Concrete Piles	> 1	> 1
Timber Piles	Unknown	> 1

4.2.1 Vehicular Load Evaluation

The following vulnerabilities to vehicular loads are identified by the structural analysis and evaluation of the bridge.

4.2.1.1 Deck and Barriers

- The barrier railing contains the top chord of the steel truss, which is a primary structural member, and thus, truck collision to the barrier railing presents a structural vulnerability.
- The longitudinal flexure capacity of the concrete deck between floor beams is not adequate for the Strength-I limit state, with a maximum DCR approximately equal to 1.6. Note that this calculation does not utilize two-way shear action in the slab due to the width of the deck relative to the distance between floor beams, as well as the fact that the deck is very lightly reinforced in the transverse direction (i.e. #3 bars spaced at about two-feet).

4.2.1.2 Floor Beams

- The top chord of the steel trusses embedded in the concrete floor beams is not adequate for the Strength-I limit state, with a maximum DCR equal to about 1.5 for compressive loads.

4.2.1.3 Arch Truss

- Some top and bottom chord members near the mid-span of the continuous truss spans are not adequate in compression and tension, respectively, for the Strength-I limit state, with a maximum DCR equal to about 1.2.
- Most bottom chord members near the piers are not adequate in compression for the Strength-I limit state, with a maximum DCR equal to about 1.6.
- Most vertical and diagonal members near mid-span of the continuous truss spans are not adequate for the Strength-I limit state, with maximum DCR's equal to about 2.5 and 2.6, respectively.

4.2.1.4 Substructure

- Based on an ultimate pile axial compressive capacity of 100 kips, as determined in the geotechnical evaluation provided in Appendix F, and a resistance factor of about 0.5, the concrete piles at most flood plain piers and the south abutment are not adequate for the Strength-I limit state. Note that the resistance factor prescribed by AASHTO LRFD can be as large as 0.95 and is highly dependent of the method used for establishing the pile ultimate resistance. A resistance factor of 0.5 is conservatively used due to lack of information about the original geotechnical investigation and pile construction.
- The current condition of the timber piles in the river spans is unknown and the piles may be vulnerable to future deterioration since the piles are potentially untreated.

4.2.2 Seismic Load Evaluation

The following vulnerabilities due to seismic loads are identified by the structural analysis.

4.2.2.1 Arch Truss

- Most top and bottom chord members near mid-span of the continuous truss spans are not adequate in compression and tension, respectively, to resist the Extreme-I limit state. The maximum DCR values for the Extreme-I limit state, are equal to 4.1 for the top chord (Frame J-K, compression model) and 1.5 for the bottom chord (Frame L-M, tension model).

- Most bottom chord members near the piers are not adequate in compression to resist the Extreme-I limit state, with a maximum DCR equal to 1.7 (Frame L-M. compression model).
- Most vertical and diagonal members near regions of the arch with tension demands are not adequate to resist the Extreme-I limit state, with maximum DCR's equal to 13.1 and 2.7, respectively. Note that the maximum DCR for the vertical and diagonal members is conservative due to the inertial forces associated with the concrete mass above the deck, as discussed above.

4.2.2.2 Substructure

- At the south abutment, where there is an expansion joint, the seat width is about 14-inches, which is less than 30-inches and thus, not sufficient per Section 7.8.3 of Caltrans SDC[10].
- The connection between the diaphragm wall and the pier is not adequate to resist lateral seismic demands at each pier. This conclusion assumes that the shear capacity of the connection is equal to the shear friction capacity from the vertical reinforcement component only (i.e. #3 bars at two-feet). Neglecting the shear friction capacity component attributed concrete surface cohesion is warranted since the concrete will likely crack during a seismic event and uplift, as well as settlement, may occur. Based on this approach, the maximum DCR for this connection is approximately 5.0.
- The concrete and timber pile embedment into the footings provides very little tension capacity. As a result, the foundations are vulnerable to pile pull-out and the ultimate geotechnical axial resistance of the piles is not adequate to resist compressive loads caused by overturning of the piers in the longitudinal direction; thus, pile plunging will likely occur.
- Note that the moment and shear capacity of the piers is adequate to resist seismic and settlement loads, although pile plunging effects could lead to excessive flexural cracking of the piers.

4.2.3 Hydraulic Scour Evaluation

Based on the preliminary hydraulics and scour analyses performed by WRECO, the maximum potential scour depths are significant and well below the bottom of footing elevations (see Section 3.2.5 of this report and Appendix E for details). Due to the magnitude of the scour depths relative to the pile lengths, the axial and lateral capacities of the piles for all limit states could be severely compromised due to both the 100-year and 200-year storm events. Thus, retrofit or replacement of the pile foundations is necessary to ensure stability of the bridge during these scour events.

5.0 Rehabilitation and Retrofit Strategy

The following sections present the rehabilitation measures necessary to address the vulnerabilities described in Section 4.2 of this report. Element level rehabilitation and retrofit are discussed for the superstructure and then the substructure. Next, a brief summary of other necessary repairs is presented and construction staging issues are discussed. Last, an approximate cost estimate for the rehabilitation and retrofit efforts is provided.

The vulnerabilities identified in this report are consistent with the vulnerabilities that were identified in the original draft Seventh Street Bride Preliminary Strategy Report[2] and draft Seventh Street Bridge Feasibility Study[1]. Since the original rehabilitation and retrofit strategies appear to provide a practical and cost-effective solution, alternative retrofit methods have not been developed. The rehabilitation and repair methods described below are therefore in general agreement with that presented in the original reports. Note the purpose of the rehabilitation and retrofit strategy is to repair the structure to meet current seismic criteria and support modern vehicular loads, including HS-20 and Caltrans permit trucks, except as noted below in Section 5.1.1. It should be acknowledged, however, that the feasibility and adequacy of these rehabilitation and retrofit strategies are based on engineering judgment and have not been evaluated or designed in detail. Furthermore, note that the rehabilitation and retrofit measures are designed to maximize preservation of the architectural integrity of the structure.

In addition to the rehabilitation and retrofit strategies described below, there are a multitude of bridge elements with concrete spalling, exposed reinforcement, and other miscellaneous forms of deterioration or damage that need maintenance and repair (see Section 3.2.3 of this report for details).

5.1 Superstructure

The following rehabilitation and retrofit measures are necessary for the bridge superstructure (see Appendix B for concept drawings of measures).

5.1.1 Deck and Barrier Rail

- A collision protective system along the existing barriers is needed to prevent vehicular collision impact on the barrier railing since the railing contains the top chord of the steel truss, which is a primary structural element. Note that severe damage to the top chord of the steel truss due to a substantial vehicular collision could potentially cause localized partial collapse of the structure. This measure cannot be implemented without further reducing the substandard roadway and sidewalk widths. As a result, a collision prevention system is considered not feasible and this vulnerability will therefore remain in the structure.

- Since the longitudinal flexure capacity of the concrete deck between floor beams is not adequate for the Strength-I limit state, remedial measures for the deck are necessary. Deck replacement is recommended rather than rehabilitation due to the age and condition of the deck, the under-reinforced nature of the deck, and feasibility of other repair strategies. During deck replacement, the joint seals at the expansion joints should be replaced as well.

5.1.2 Floor Beams

- The deck floor beams are not adequate for the Strength-I limit state. The deck floor beams should therefore be replaced or/and an interior longitudinal beam, such as an arched girder, should be constructed along the centerline of the structure to reduce the maximum demands in the floor beams. This work could be accomplished during replacement of the deck, as described above. Assuming the deck is replaced and an interior longitudinal beam is constructed, replacement rather than retrofit of the floor beams is recommended.

5.1.3 Arch Truss

- There are numerous vulnerabilities associated with the steel truss members for both vehicular and seismic loading, as discussed in Sections 4.2.1.3 and 4.2.2.1 of this report. Retrofit of the steel truss is cost-prohibitive due to the concrete arch that surrounds the steel truss members. However, one promising retrofit scheme that does not significantly impact the aesthetics of the bridge involves constructing an interior longitudinal beam, such as an arched girder, along the centerline of the structure to reduce the vehicular and seismic demands on the existing steel trusses. Note this work could be accomplished during replacement of the deck and floor beams, as described above.
- The differential displacements between adjacent cantilever tips at the mid-span joints should be removed by jacking the cantilevers into their original position and connecting the cantilevers together for vertical support, perhaps using a detail similar to the hanger plate retrofit presented in the aforementioned 1978 modification plans.

5.2 Substructure

The following rehabilitation and retrofit measures are necessary for the bridge substructure.

5.2.1 Abutments

- At the south abutment, a seat extension should be constructed to comply with the required 30-inch seat length specified in Section 7.8.3 of Caltrans SDC[10].

5.2.2 Piers

- The connection between the diaphragm walls and the piers is not adequate to resist lateral seismic demands. Strengthening the connection is therefore necessary and could be accomplished during the aforementioned deck replacement effort by removing and replacing the diaphragm walls. The new diaphragm walls would likely be thicker than the existing walls to prevent shear failure of the walls during a seismic event.

5.2.3 Pile Foundations

- The ultimate geotechnical axial resistance of both the concrete and timber piles is not adequate to resist compressive loads and pile connection details are not capable of resisting tension demands caused by seismic overturning of the piers in the longitudinal direction. In addition, the ultimate geotechnical axial resistance of the concrete piles is not adequate to resist the Strength-I limit state. The timber piles are potentially untreated and have likely been exposed to wet-dry cycles over the lifetime of the bridge, and thus, deterioration of the timber piles cannot be precluded. Furthermore, the pile lengths are not sufficient to resist the scour depth associated with the design flood events without plunging or settlement of the foundations and potential collapse of the structure.

  Consequently, retrofit of the pile foundations is necessary. The proposed retrofit strategy consists of installing a combination of large-diameter cast-in-drilled-hole (CIDH) piles and cast-in-steel-shell (CISS) piles through the existing pier cap, varying between three-feet and seven-feet in diameter, with a new pile cap and infill wall inside each existing pier. The pile cap would attach directly to the new diaphragm wall. Installing the new foundations inside the existing piers could be accomplished during replacement of the deck and diaphragm walls and does not impact the architectural integrity of the bridge since it is not visible to the public. Note that the piles would be designed for hydraulic and scour demands. Noise mitigation measures would likely not be necessary for building this type of foundation provided the piles are constructed during the daytime.

5.3 Other Repairs

- Architectural repairs that are necessary include patching the lion statues and benches, removing biological growth and applying a waterproof coating, cleaning and painting the bronze plaques at the bridge approaches, removing and replacing mismatched patches previously placed, and possible replacing the existing lighting fixtures with the original light fixtures detailed on the as-built plans. Note that the concrete used to patch the existing spalls should be designed to match the color and texture of the existing concrete.
- Modifications to the sidewalk approaches are necessary to provide wheel chair access to the sidewalks and satisfy Americans with Disabilities Act (ADA) requirements.
- Graffiti should be removed and counter-measures should be employed to deter or mitigate future graffiti and vandalism. Such measures that should be considered include protective coatings on concrete surfaces, landscaping to cover areas targeted by graffiti vandals, and fencing to limit access to the bridge.

5.4 Construction Staging

Rehabilitation and retrofit of the bridge will require a long-duration closure of Seventh Street over the bridge to facilitate implementation of the remedial measures. The most likely detour utilizes Ninth Street, located about ¼ mile east of Seventh Street, to carry traffic over the Tuolumne River. Closure of Seventh Street over the bridge will have some impact on traffic in the area and will necessitate re-routing the public bus transportation system that currently uses Seventh Street.

5.5 Estimate

The vulnerabilities and corresponding rehabilitation and retrofit strategies discussed herein are very similar to that identified in the original draft Seventh Street Bridge Feasibility Study[1]. Consequently, the cost estimate for the rehabilitation and retrofit efforts presented below is based on the cost estimates from that study and provided in Appendix C. Note that there were two separate original cost estimates for the structure, one estimate to address rehabilitation of the superstructure for vehicular loads (i.e. $2,741,000) and another estimate to address retrofit of the substructure for seismic and liquefaction settlement loads (i.e. $5,690,000). The total rehabilitation and retrofit cost estimate presented in this report is based on the summation of the original cost estimates with updated pricing. It is acknowledged that there is some overlap between the two estimates (e.g. partial deck replacement for seismic retrofit and complete deck replacement for vehicular rehabilitation), though such overlap is very minor in terms of the overall cost estimate and therefore may be ignored. The original estimates include ten percent for mobilization and 25 percent for contingencies, but do not include contractor time related overhead (TRO). Note that the unit quantities in the original estimates are assumed to be accurate and thus, not rigorously confirmed. The original cost estimates are escalated to an assumed mid-point of construction of 2017 to account for inflation and the cost associated with the temporary structure listed in the seismic retrofit estimate (i.e. $1,000,000) is removed since closure of the bridge is feasible with detours. The total cost estimate listed below does not include a collision prevention system for the barrier since it is not feasible to provide such as system without further reducing the lane width or removing the sidewalks.

Table 3: *Cost Estimate Summary for Bridge Rehabilitation and Retrofit Strategies*

Description	Cost Estimate	Cost per SQFT Bridge Deck
Rehabilitation/Retrofit	$13,590,000	$335

Note: Estimates does not include roadway approach costs.

5.6 Remaining Deficiencies

The rehabilitation and retrofit measures presented in this report address repair of vulnerabilities related to seismic and modern vehicular loadings. These measures, however, do not mitigate deficiencies that pertain to the following performance criteria and conditions:

- Functional obsolescence of the existing structure: The existing structure has no shoulders, creating an unsafe condition for drivers and cyclists. The existing sidewalks could be removed to provide shoulders, but they would be substandard in width and result in a loss of pedestrian access. There is no feasible way to widen the structure to provide room for shoulders as the truss embedded in the concrete arch extends above the roadway surface.
- Freeboard inadequacy for the 100, 200, and 500 year flood events: The structure has zero freeboard for the 100 year event and partly impounds the 200 year event. There is no practical way to raise the bridge to provide the minimum required freeboard of 3' for the 100 year event.
- Remaining life of the existing steel truss and questionable durability of the concrete arch and abutments: The concrete that encases the embedded steel truss prevents inspection and monitoring of the condition of the steel members. The presence and propagation of fatigue cracks and corrosion in the members cannot be observed or repaired. Because of the inability to closely inspect and monitor the aged steel members and the fact that the bridge is non-redundant, structural

deterioration cannot be assessed and failure of any one of the embedded steel members will result in likely collapse. Additionally, regions of the concrete exhibit significant cracking and spalling that appears to be due to alkali-silica reaction (ASR). There are no practical mitigations for ASR. The ASR will continue to cause cracking in the concrete and will be an on-going inspection and maintenance need. An extensive and expensive test program would be required to determine the exact condition of the existing concrete and embedded steel.

- Collision performance of the existing barriers: The existing barriers are not capable of resisting design crash loads and since the barriers are a component of the bridge's primary structural system, damage to them can lead to bridge collapse. The only way to protect the bridge from this vulnerability would be the installation of supplemental barriers in front of the existing barriers. This would require removal of the existing sidewalks and loss of pedestrian access on the bridge. It would also reduce the potential shoulder width improvement provided by removing the sidewalks.
- Continuing deterioration of bridge architectural features, such as the barrier railing and recumbent lion statues: Maintenance of the architectural features will require an on-going inspection and repair program to minimize their continued deterioration.
- ADA requirements for the existing sidewalks (if sidewalks remain on bridge): If the existing sidewalks remain they will require significant improvements to provide adequate disabled access across the bridge.

It is important to acknowledge that mitigation of the above deficiencies either requires additional maintenance efforts that will likely increase over time or generally cannot be accomplished by implementing rehabilitation strategies suggested in this report. In addition, since there would probably be unforeseen damage and deterioration that becomes exposed only after the retrofit is started, a larger than standard contingency would be carried through final design when estimating retrofit costs. Replacement of the entire structure may therefore be more cost-effective.

6.0 References

1. Seventh Street Bridge Feasibility Study, URS Corporation, January 2001
2. Seventh Street Bridge Preliminary Strategy Report, URS Corporation, January 2001
3. Caltrans Highway Design Manual, May 2012
4. AASHTO Policy on Geometric Design of Highways and Streets, 2011
5. Summary of Engineering Studies - Seventh Street Bridge, City of Modesto Public Works Department and Stanislaus County Public Works Department, March 1976
6. Preliminary Foundation Investigation, Seventh Street Bridge over Tuolumne River, Taber, March 2000
7. Preliminary Study and Survey of Seventh Street Bridge, Architectural Resources Group, 1998
8. Historic Property Survey Report and Historic Archaeological Survey Report, Foothill Resources, March 1996
9. American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, 4th edition, with Caltrans 2011 Amendments
10. Caltrans Seismic Design Criteria, Version 1.6
11. AASHTO Manual for Bridge Evaluation (MBE), Second Edition

6.0 References

1. Seventh Street Bridge Feasibility Study, URS Corporation, January 2001
2. Seventh Street Bridge Preliminary Strategy Report, URS Corporation, January 2001
3. Caltrans Highway Design Manual, May 2012
4. AASHTO Policy on Geometric Design of Highways and Streets, 2011
5. Summary of Engineering Studies - Seventh Street Bridge, City of Modesto Public Works Department and Stanislaus County Public Works Department, March 1976
6. Preliminary Foundation Investigation, Seventh Street Bridge over Tuolumne River, Taber, March 2000
7. Preliminary Study and Survey of Seventh Street Bridge, Architectural Resources Group, 1998
8. Historic Property Survey Report and Historic Archaeological Survey Report, Foothill Resources, March 1996
9. American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, 4th edition, with Caltrans 2011 Amendments
10. Caltrans Seismic Design Criteria, Version 1.6
11. AASHTO Manual for Bridge Evaluation (MBE), Second Edition

Appendix A – Structural Analysis Results

CSiBridge

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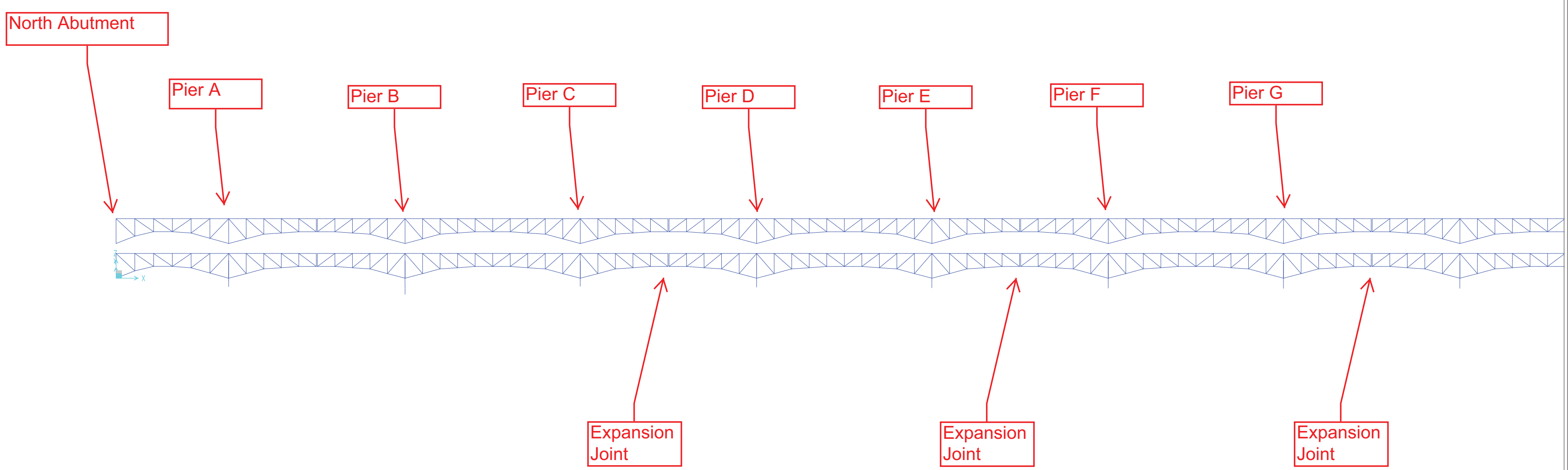


FIGURE 1 - Global Analysis Model, Piers A-G

CSiBridge v15.2.0 - File:7th St_Global Model_Tension_v8 - 3-D View - Kip, ft, F Units

CSiBridge 1/2/13 15:47:24

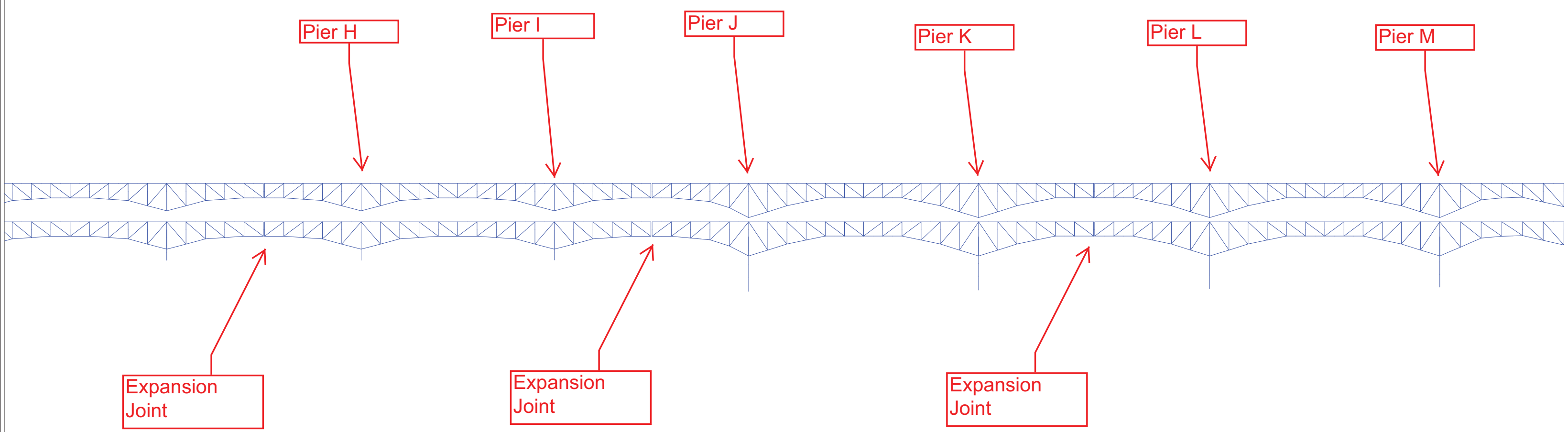


FIGURE 2 - Global Analysis Model, Piers H-M

CSiBridge v15.2.0 - File:7th St_Global Model_Tension_v8 - 3-D View - Kip, ft, F Units

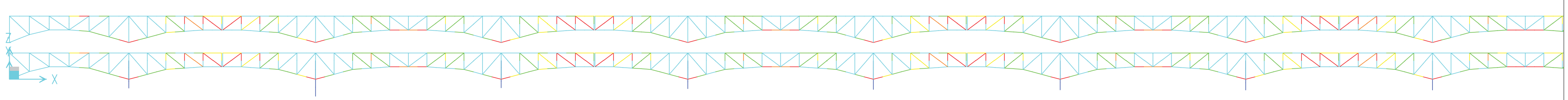
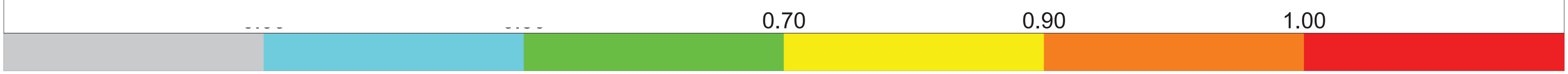


FIGURE 3 - Strength-I Limit State, Piers A-G



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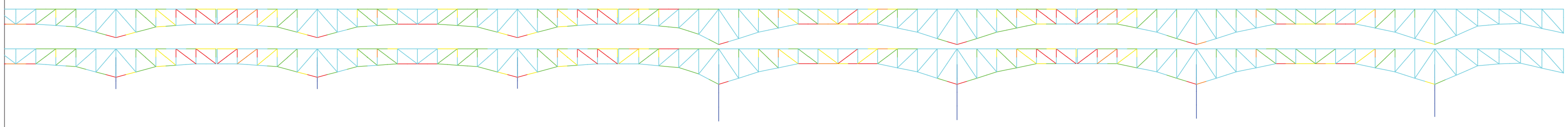
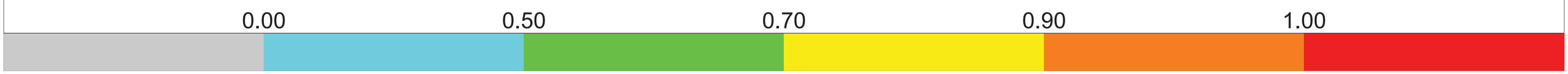


FIGURE 4 - Strength-I Limit State, Piers H-M



CSiBridge v15.2.0 - File:7th St_Global Model_Strength_E=0 - Steel P-M Interaction Ratios (AISC360-05/IBC2006) - Kip, ft, F Units

CSiBridge

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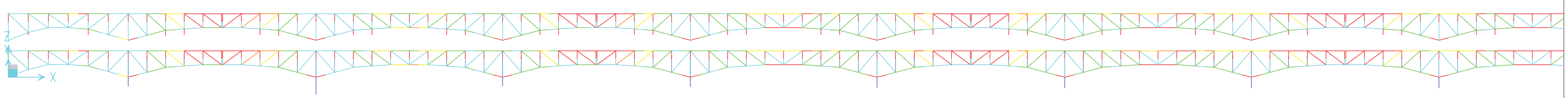


FIGURE 5 - Compression Model, Extreme-I Limit State (DL+ARS), Piers A-G

0.00 | 0.50 | 0.70 | 0.90 | 1.00

CSiBridge v15.2.0 - File:7th St_Global Model_Compression_v8 - Steel P-M Interaction Ratios (AISC360-05/IBC2006) - Kip, ft, F Units

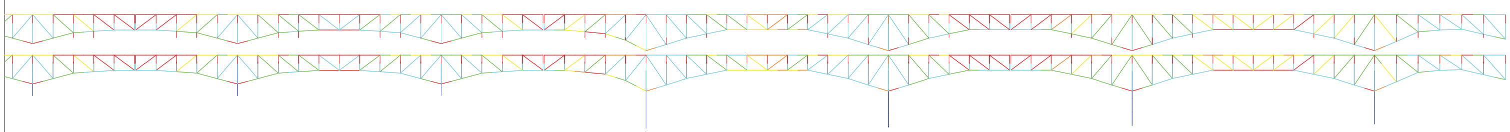
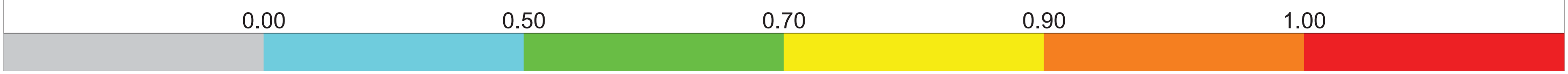


FIGURE 6 - Compression Model, Extreme-I Limit State (DL+ARS), Piers H-M



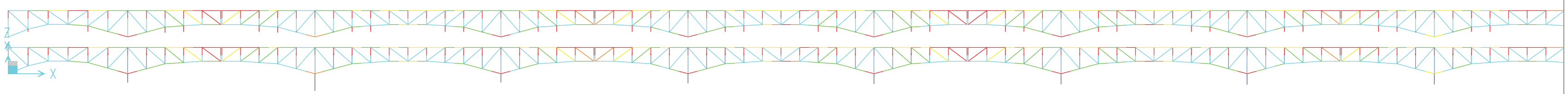


FIGURE 7 - Tension Model, Extreme-I Limit State (DL+ARS), Piers A-G

0.00	0.50	0.70	0.90	1.00

CSiBridge

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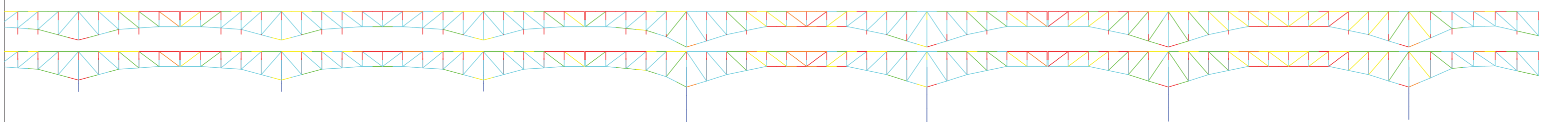
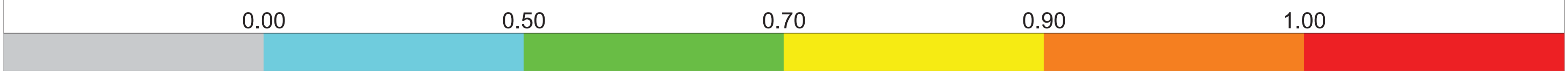


FIGURE 8 - Tension Model, Extreme-I Limit State (DL+ARS), Piers H-M



CSiBridge v15.2.0 - File:7th St_Global Model_Tension_v8 - Steel P-M Interaction Ratios (AISC360-05/IBC2006) - Kip, ft, F Units

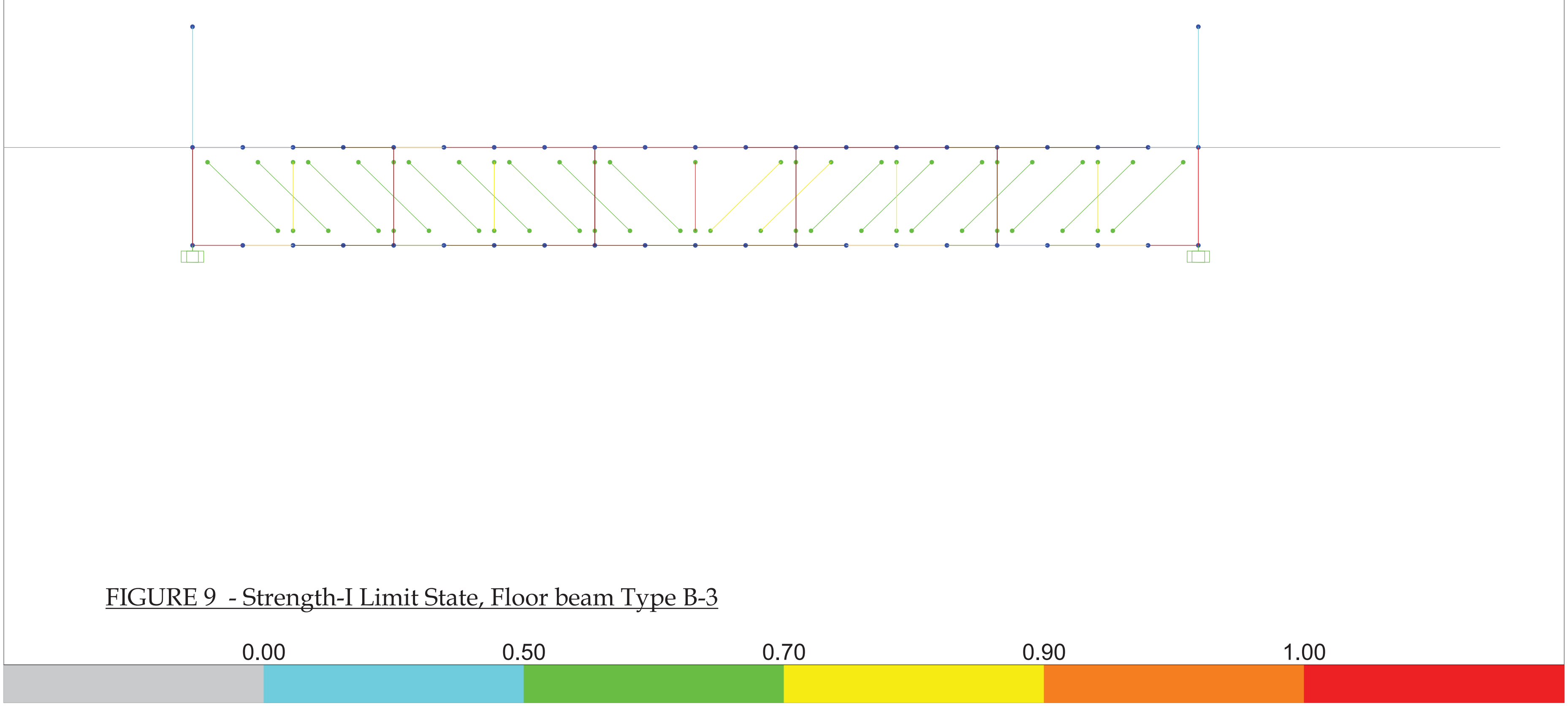


FIGURE 9 - Strength-I Limit State, Floor beam Type B-3

CSiBridge v15.2.0 - File:7th St_Trans LL_B-3_v5 - Steel P-M Interaction Ratios (AISC360-05/IBC2006) - Kip, ft, F Units

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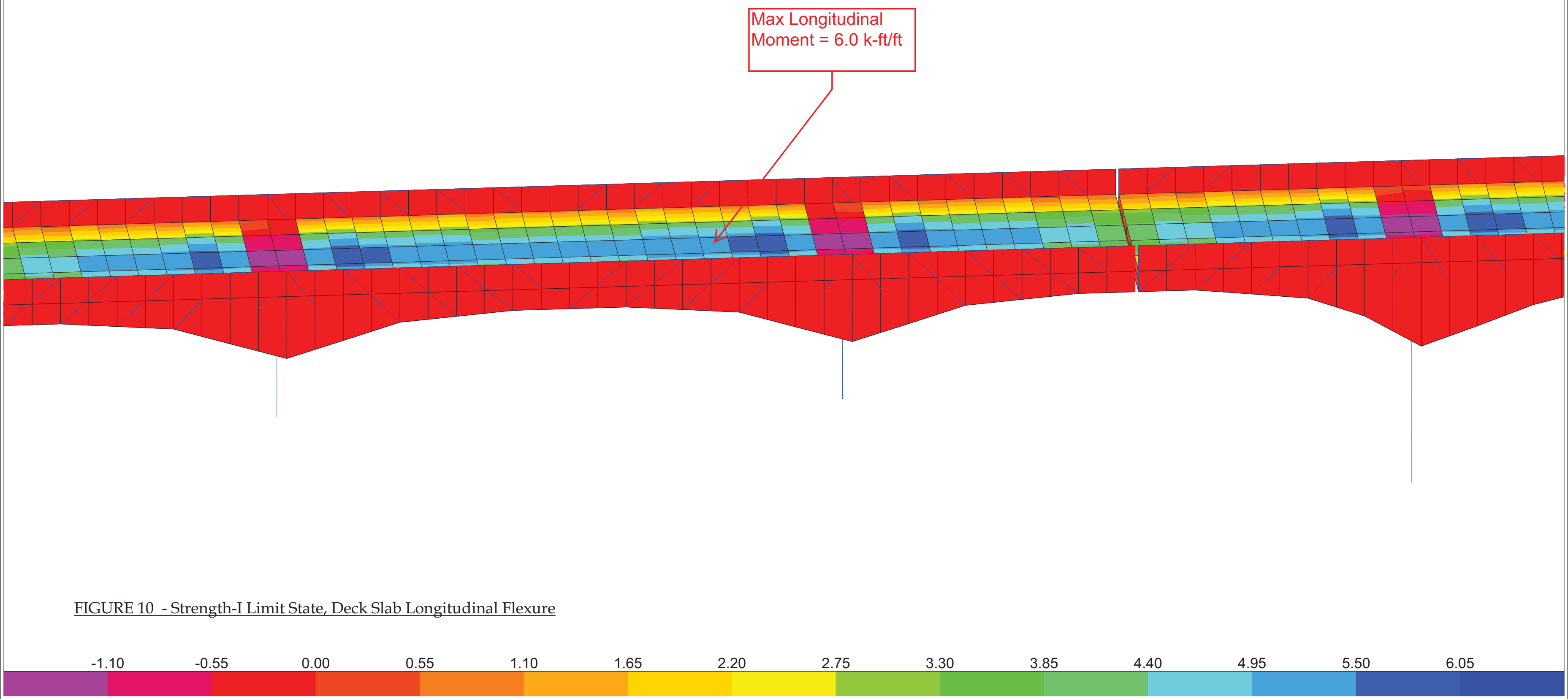


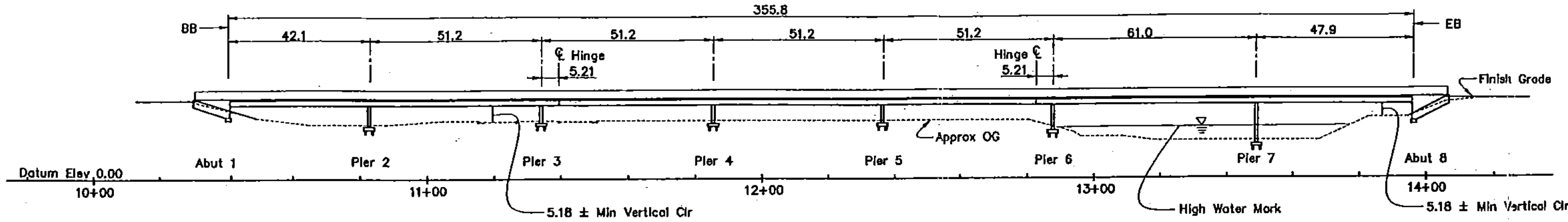
FIGURE 10 - Strength-I Limit State, Deck Slab Longitudinal Flexure

CSiBridge v15.2.0 - File:7th St_Global Model_Strength - Resultant M11 Diagram (STR-I Combo - Max) - Kip, ft, F Units

Support	DL (kip)	HL93 (kip)
N. Abut	128.9	185.4
Pier A	1731.0	376.3
Pier B	1214.2	366.1
Pier C	1122.5	388.1
Pier D	1200.5	366.1
Pier E	1139.4	388.0
Pier F	1144.0	366.1
Pier G	1213.9	388.1
Pier H	1145.5	366.1
Pier I	1142.4	388.0
Pier J	1905.9	378.8
Pier K	2020.8	435.3
Pier L	2037.5	412.0
Pier M	2357.5	353.3
S. Abut	128.9	185.4

FIGURE 11 – Dead and Live Load Vertical Support Reactions

Appendix B – Original General Plans for Rehabilitation and Retrofit Strategy



ELEVATION
1:800

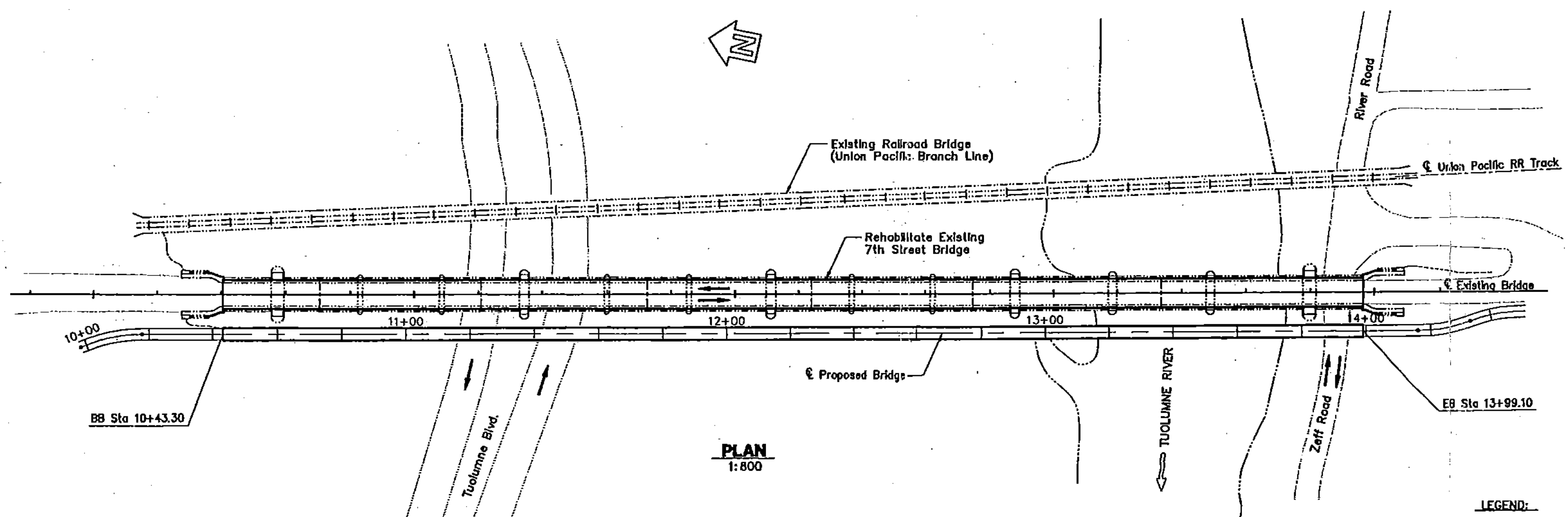
VEHICULAR TRAFFIC

1. ✓ New alignment. No traffic at the site.
2. ✓ Traffic will be detoured away from the site.
3. ___ Traffic will be carried on the structure. Stage construction will be required.
4. ✓ Traffic will pass under the structure on Tuolumne Boulevard and Zeff Road
   - A. ___ No falsework allowed over traffic.
   - B. ✓ Falsework opening(s) required.

	Temporary Vertical Clearance	Width of Traffic Opening
Tuolumne Blvd. North End	4.572	10.973
Tuolumne Blvd. South End	4.572	10.973
Zeff Road	4.572	10.973

COST ESTIMATE

Date of Estimate	= 01-14-2001
Structure Depth (New)	= 2.44
Length (New)	= 355.8
Width (New)	= 4.20
Area (New)	= 1494
Total Cost Includes Rehab Work On Existing Structure and New Strucuture 10% Mobilization & 25% Contingency	
Total Cost	= $5,775,000



PLAN
1:800

LEGEND:
- → Indicates direction of traffic
- ------ Denotes existing structure
- ——— Indicates new construction

General plan for Live Load Improvements from Original Rehabilitation and Retrofit Study

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

ALL DIMENSIONS IN METERS UNLESS OTHERWISE NOTED

Metric

PLOT DATE : 11/13/00

NO.	REVISIONS	BY	DATE
△			
△			
△			
△			
△			

BENCH MARK
ELEVATION DESCRIPTION ___ DATUM ___

DESIGN BY : JE
DRAWN BY : MB
CHECKED BY : WDL
SCALE : AS SHOWN
DATE : 01/16/01
PROJ NO. : H300007140.10

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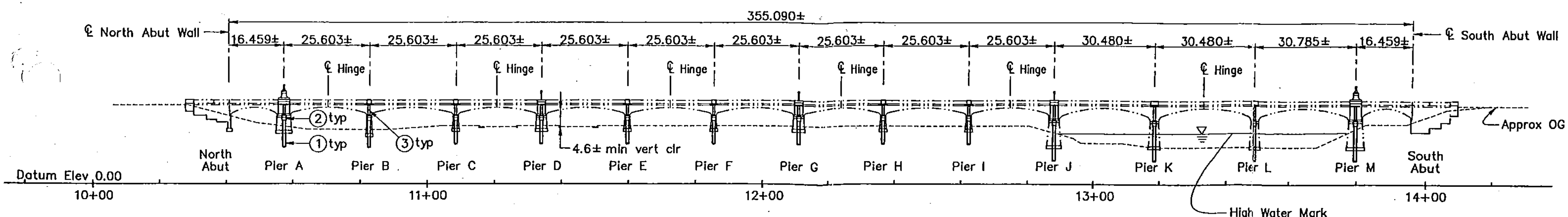
7TH STREET BRIDGE FEASIBILITY STUDY
ALTERNATIVE 1-B (1)
CALIFORNIA

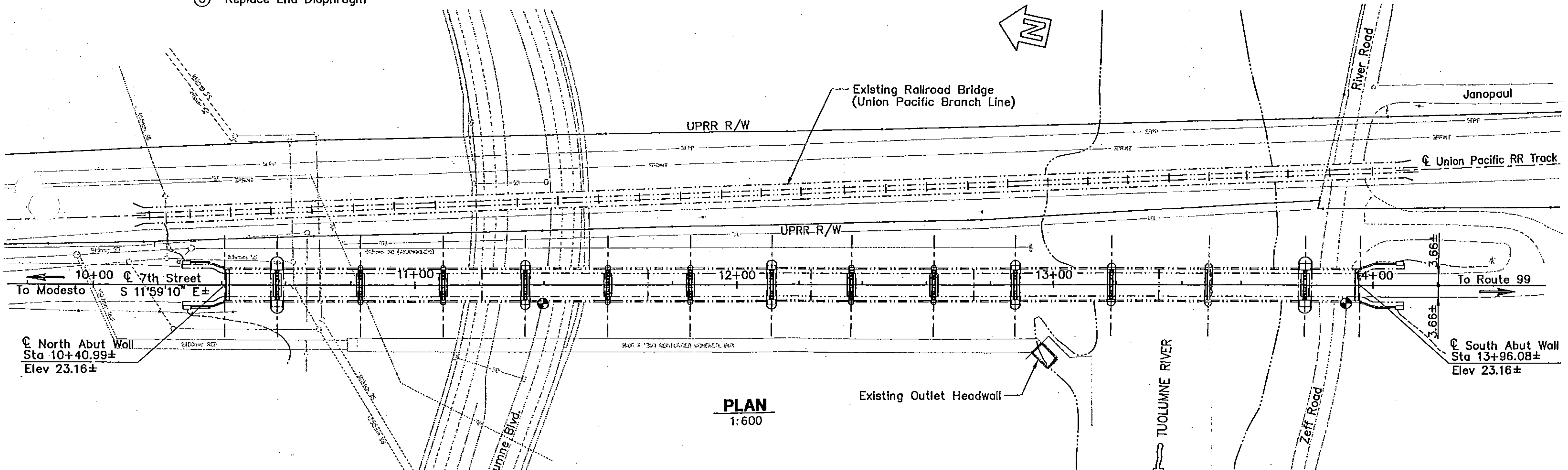
1 of 2



ELEVATION
1:600

Retrofit Notes:

1. 915 mm dia. CISS or 1525 mm dia. CIDH Piling
2. Pile Extension and Bent Cap
3. Replace End Diaphragm



PLAN
1:600

Legend:

- Denotes existing construction
- Denotes new construction
- Denotes direction of traffic
- Denotes existing Minimum Vertical Clearance

Note:
For "TYPICAL SECTION" see "GENERAL PLAN NO. 2" sheet.

General plan for Seismic Improvements from Original Rehabilitation and Retrofit Study

NOTE:
CONTRACTOR SHALL VERIFY ALL C... ROLLING FIELD DIMENSIONS BEFORE OR...NG OR FABRICATING ANY MATERIAL.

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50

Metric

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ROSEVILLE, CA 95661 (916) 784-3800

SR. PROJECT ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
Walt D. LaFranchi
No. 34946
Exp. 9/30/00
CIVIL
STATE OF CALIFORNIA

REVISIONS					
NO.	BY	DATE	DESCRIPTION	APPROVED BY	DATE

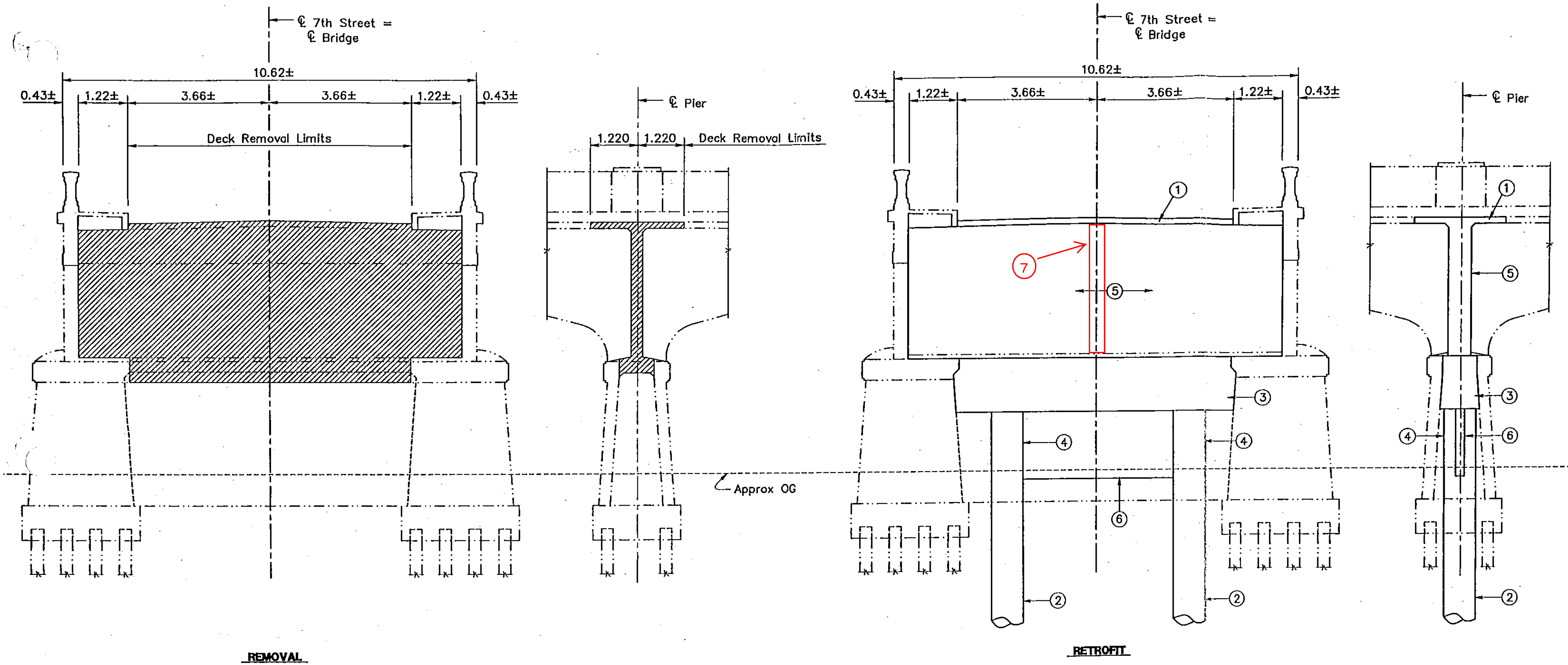
ACCEPTED BY THE CITY ENGINEER

???? RCE ?????

CITY OF MODESTO
7 TH STREET BRIDGE
SEISMIC RETROFIT
GENERAL PLAN NO. 1

BRIDGE NO. 38C-0023

DESIGNED BY	DRAWN BY	CHECKED BY	JOB NUMBER	SCALE	SHEET
J. England	L. Davis		H399000166	As Shown	1 OF 2



TYPICAL SECTION
1:50

Details of Seismic Improvements from Original Rehabilitation and Retrofit Study. Live Load Improvements added.

Legend:

- Denotes existing construction
- Denotes new construction
- Denotes limits of Bridge Removal (Portion)

Retrofit Notes:

1. Replace Portion of Deck
2. 915 mm dia. CISS or 1525 mm dia. CIDH Piling
3. Bent Cap
4. 915 mm or 1525 mm Column
5. 600 mm End Diaphragm
6. Infill Wall
7. New Center Girder (live load retrofit)

...: CONTRACTOR SHALL VERIFY ALL C... ...LLING FIELD DIMENSIONS BEFORE ORD... ...G OR FABRICATING ANY MATERIAL.

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50

Metric

URS
ENGINEERING • PLANNING • SURVEYING
1360 LEAD HILL BLVD, SUITE 100
ROSEVILLE, CA 95661 (916) 784-3800

SR. PROJECT ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
Walt D. LaFranchi
No. 34946
Exp. 9/30/00
CIVIL
STATE OF CALIFORNIA

REVISIONS

NO.	BY	DATE	DESCRIPTION	APPROVED BY	DATE
△					
△					
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ACCEPTED BY THE CITY ENGINEER

???? RCE ?????

CITY OF MODESTO
7 TH STREET BRIDGE
SEISMIC RETROFIT
GENERAL PLAN NO. 2

BRIDGE NO. 38C-0023

DESIGNED BY	DRAWN BY	CHECKED BY	JOB NUMBER	SCALE	SHEET 2
[illegible]	[illegible]		[illegible]	[illegible]	[illegible]

Appendix C – Cost Estimate for Rehabilitation and Retrofit Strategy

Seventh Street Bridge Project
Cost Estimate Summary

By: C. Serroels Date: 1/4/13

Cost Estimates from Original Rehabilitation and Retrofit Studies:			
Vehicular Improvements:		$	2,741,000.00
Seismic Improvements:		$	5,690,000.00
	Subtotal:	$	8,431,000.00
Adjustments to Previous Estimates:			
Temporary Bridge (See note 1):		$	(1,388,889.00)
	Subtotal:	$	7,042,111.00
Escalation to 2017 (See note 2):			
93% Escalation:		$	6,549,163.23
	Tot:	$	13,591,274.23
	Use:	**$**	**13,590,000.00**

Bridge Deck Area:

Length:	1165	ft
Width:	34.83	ft
Area:	40577	sq ft

Cost/SF:	**$**	**335**

Notes:

1) Previous estimate for Seismic Improvements includes $1,000,000 for a temporary bridge. It is assumed in this estimate that detours will be provided and that a temporary bridge will not be required. The Seismic Improvements total includes 10% mobilization and 25% contingency for a total of $1,388,889.
2) Previous estimate was prepared in 2000. It is assumed that the mid-point of construction will be in 2017. Years 2000-2008 are escalated at 5% per year, years 2009-2017 are escalated at 3% per year. Total escalation = 93%.

01/23/01

Alternative 1-B (1)

Deck Rehabilitation

URS CORPORATION — JOB NO. H300007140.10

X PLANNING ESTIMATE ____ BRIDGE GENERAL PLAN ESTIMATE ____ 60% ESTIMATE

Bridge: 7th Street (Alternative 1-B (1)) — Br. No.: 38C-0023
Type: Canticrete Truss-Arch (Existing) — District: 10 County: Sta Route: City PM:
No. Spans: (14) Fourteen

Quantities and Price by JRE 11/24/00

Width (M)	Length (M)	Area (M2)
10.2	355.1	3622

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT
1	Bridge Removal Location A (Deck)	LS	1	$700,000.00	$700,000.00
2	Bridge Removal Location B (Diaphragm Hinge)	LS	1	$55,000.00	$55,000.00
3	Bridge Removal Location C (Partial Diaphragm)	LS	1	$53,000.00	$53,000.00
4	Bridge Removal Location D (Unsound Concrete)	LS	1	$15,000.00	$15,000.00
5	Structural Concrete, Bridge	M3	1330	$500.00	$665,000.00
6	Drill & Bond Dowels	M	790	$50.00	$39,500.00
7	Joint Seal (MR=)	M	60	$90.00	$5,400.00
8	Bar Reinforcing Steel (Bridge)	KG	142000	$1.20	$170,400.00
9	Clean Structural Steel (Existing Bridge)	LS	1	$5,000.00	$5,000.00
10	Miscellaneous Metal (Restrainer-Pipe Type)	KG	3550	$10.00	$35,500.00
11	Bridge Deck Drainage System	KG	1670	$10.00	$16,700.00
12	Concrete Barrier (Type 27)	M	710	$300.00	$213,000.00
13					
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27					

COMMENTS:

Cost Per Square Meter
$757

Cost Per Square Foot
$70

SUBTOTAL		$1,973,500
MOBILIZATION	10%	$219,278
SUBTOTAL BRIDGE ITEMS		$2,192,778
CONTINGENCIES (25%)		$548,194
BRIDGE TOTAL		$2,740,972
WORK BY RAILROAD/UTILITY FORCES		
BRIDGE REMOVAL (CONTINGENCIES INCL.		
GRAND TOTAL		$2,740,972
FOR BUDGET PURPOSES - SAY		$2,741,000

URS CORPORATION				JOB NO. H300000209.10
___ PLANNING ESTIMATE	_X_ BRIDGE GENERAL PLAN ESTIMATE		___ 60% ESTIMATE	

Bridge:	7th Street Earthquake Retrofit	Br. No.: 38C - 0023			
Type:	Canticrete truss-arch	District: 10	County: Sta	Route: City	PM:
No. Spans:	(14) fourteen	Width (M)	Length (M)	Area (M2)	
Quantities and Price by JRE 11/13/00		10.62	355.09	3771	

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT
1	Bridge Removal (Portion), Location A	LS	1	$240,000.00	$240,000.00
2	Furnish 915mm Cast-in-Steel Shell Concrete Pile	M	440	$250.00	$110,000.00
3	Drive 915mm Cast-in-Steel Shell Concrete Pile	EA	12	$5,000.00	$60,000.00
4	1525mm Cast-in-Drill-Hole Concrete Piling	M	430	$1,500.00	$645,000.00
5	Structural Concrete, Bridge	M3	660	$700.00	$462,000.00
6	Drill and Bond Dowel	M	300	$65.00	$19,500.00
7	Bar Reinforcing Steel	KG	250000	$1.20	$300,000.00
8	Miscellaneous Metal, (Bridge)	KG	210000	$6.00	$1,260,000.00
9	Temporary Bridge	LS	1	$1,000,000.00	$1,000,000.00
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COMMENTS:			
	SUBTOTAL		$4,096,500
	MOBILIZATIOI	10%	$455,167
Cost Per Square Meter	SUBTOTAL BRIDGE ITEMS		$4,551,667
$1,509	CONTINGENCIES (25%)		$1,137,917
	BRIDGE TOTAL		$5,689,583
Cost Per Square Foot	WORK BY RAILROAD/UTILITY FORCE:		
$140	BRIDGE REMOVAL (CONTINGENCIES INCL.		
	GRAND TOTAL		$5,689,583
	FOR BUDGET PURPOSES - SAY		$5,690,000

Appendix D – Preliminary Hydraulics Report

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www.wreco.com

Draft Memorandum

Date: October 23, 2012

To: Hans Strandgaard – CH2M HILL

From: Kazuya Tsurushita/Han-Bin Liang/Chris Sewell – WRECO

Project 7th Street Bridge Replacement Project, Stanislaus County

Subject: Summary of Preliminary Hydrology and Hydraulics

The purpose of the proposed 7th Street Bridge Replacement Project (Project) is to improve movement and safety for motor vehicles, pedestrians, and bicyclists across the Tuolumne River on the 7th Street Bridge. The Project is proposing to replace the existing structurally deficient bridge with the following intent:

- Provide full truck carrying capacity;
- Expand vehicular capacity of the 7th Street corridor; and
- Improve safety for motor vehicles, pedestrians, and bicyclists.

The purpose of this memorandum is to summarize the hydrologic studies of Tuolumne River to determine the design flows at the Project location, present the preliminary hydraulic analyses for the existing bridge, and discuss the various hydraulic criteria for which the bridge design is subject to.

Hydrology

Tuolumne River is a tributary of San Joaquin River, and its confluence is approximately 16 mi downstream (west) of the Project location. The Tuolumne River watershed includes drainage areas in Tuolumne and Stanislaus counties (see Figure 1). The watershed area of Tuolumne River at the confluence with San Joaquin River is approximately 2,000 square miles (mi^2).

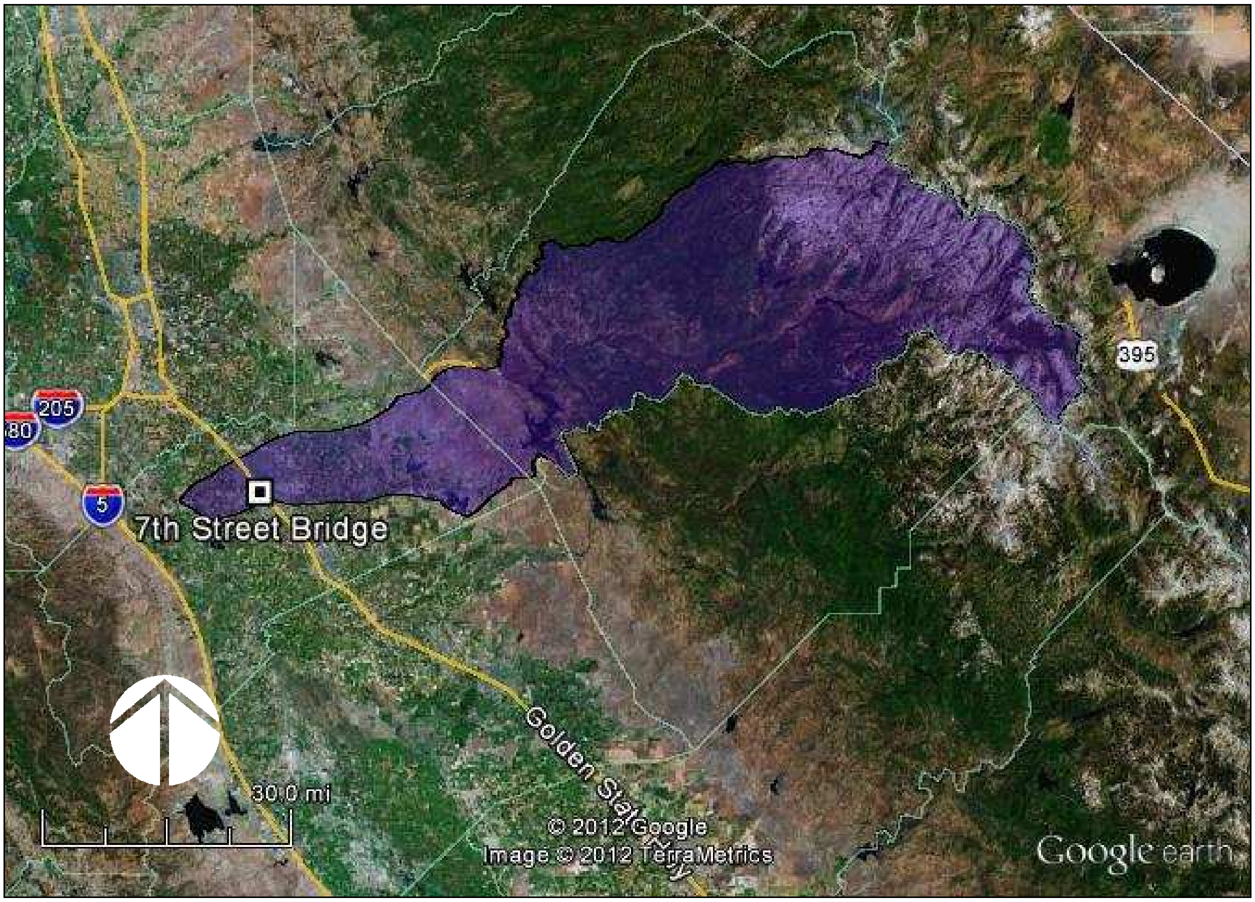

Figure 1. Watershed Area of Tuolumne River at the Confluence with San Joaquin River

Source: Google Earth

Previous Hydrologic Studies

Available information from Federal Emergency Management Agency (FEMA), Central Valley Flood Protection Board (CVFPB), United States Geological Survey (USGS), and the hydraulic study of Tuolumne River at the 9th Street bridge was investigated to identify the design discharges of Tuolumne River at the Project location. The information from these sources is described in the following sections.

A. FEMA

The FEMA Flood Insurance Study (FIS) for Stanislaus County, California and Incorporated Areas provided the 1% annual exceedance probability flood (100-year flood or Q_{100}) and the 0.2% annual exceedance probability flood (500-year flood or Q_{500}) of Tuolumne River in the Project vicinity. The 0.5% annual exceedance probability flood (200-year flood or Q_{200}) was calculated by interpolating between the available flows. The FEMA peak flows of Tuolumne River in the Project vicinity are presented in Table 1, and the locations are shown in Figure 2.

The Project location is within the County of Stanislaus, and crosses into the City of Modesto to the North. Table 1. FEMA FIS Hydrologic Data, Tuolumne River

Location	Peak Discharge (cfs)				
	Q_{10}	Q_{50}	Q_{100}	$Q_{200}^{(1)}$	Q_{500}
At Modesto	10,500	32,000	70,000	105,400	154,000
At Waterford	9,000	10,000	42,000	119,000	225,000

Source: FEMA, 2008

Note: (1) The 200-year flows are interpolated using the available flows from the FEMA FIS.

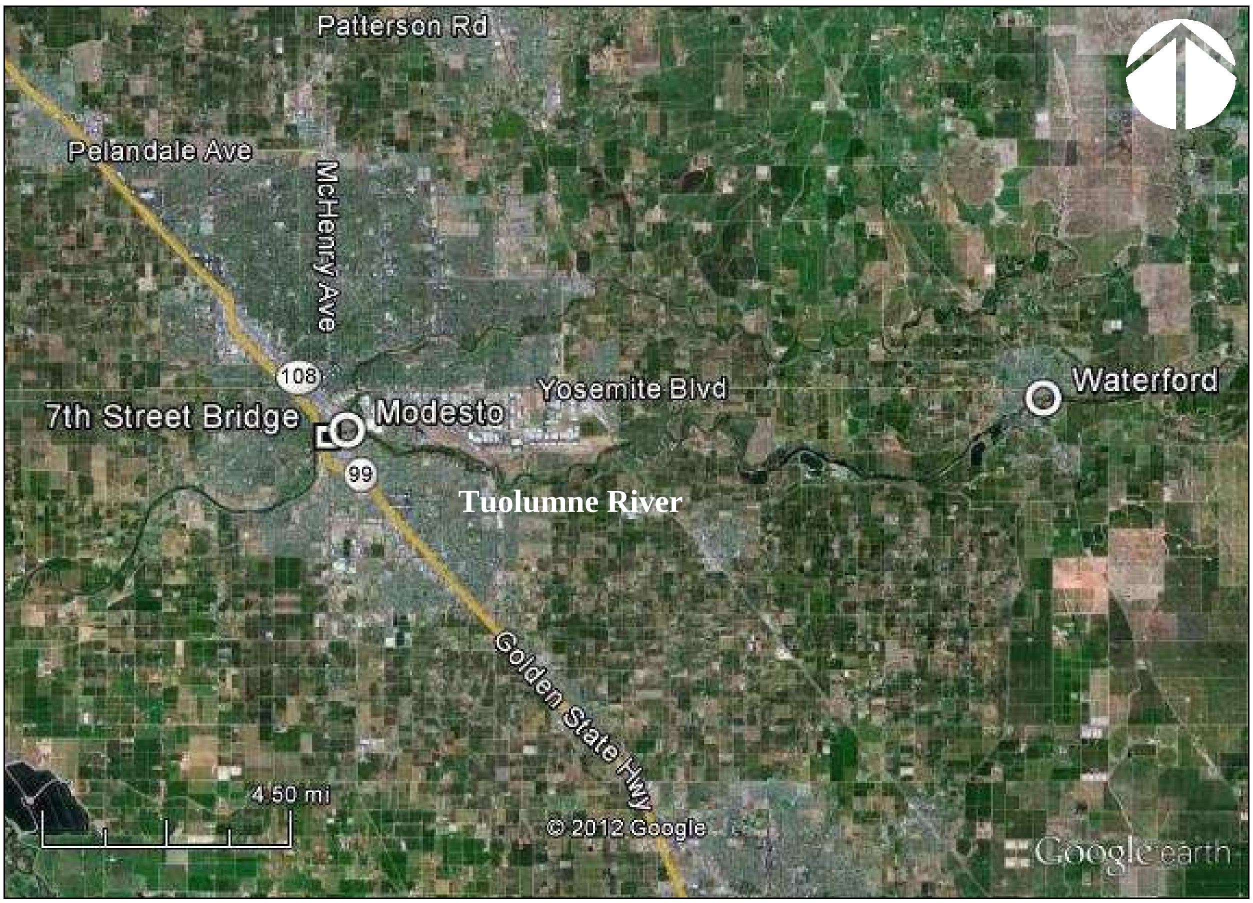

Figure 2. FEMA FIS Peak Flow Locations

Source: FEMA, 2008 and Google Earth

B. Central Valley Flood Protection Board

According to California Code of Regulations, Table 8.1, Tuolumne River, from La Grange Dam to the San Joaquin River confluence, is within the jurisdiction of the CVFPB. The *Designated Floodway Program* published in 1990, provided by the CVFPB, included the CVFPB design flows of Tuolumne River at the Project location. The design flows and locations are summarized in Table 2 and Figure 3. The Project location is located between Whitmore Avenue and Mitchell Road.

Table 2. CVFPB Design Flows

Location	Design Flow (Q_{100}) (cfs)
San Joaquin River to Extension of Whitmore Avenue	44,000
Extension of Whitmore Avenue to Mitchell Road	44,000
Mitchell Road to La Grange Dam	44,000

Source: CVFPB, 1990

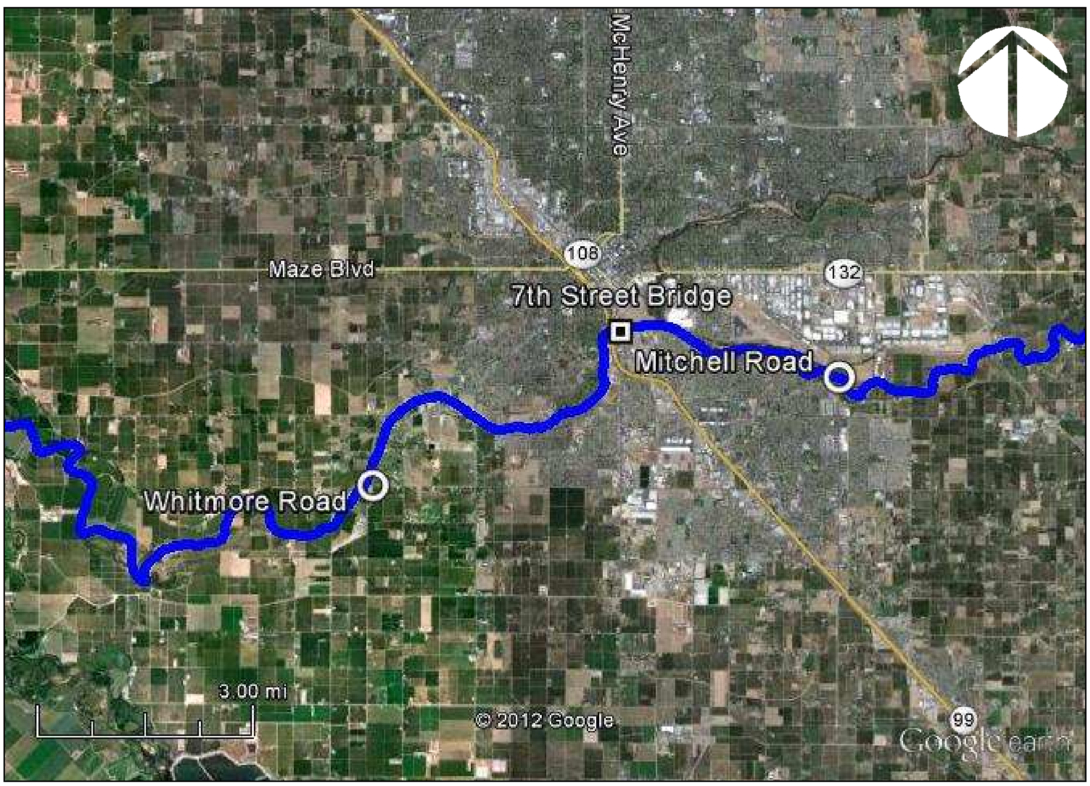

Figure 3. CVFPB Design Flow Locations

Source: Google Earth and CVFPB, 1990

Notes:

- Whitmore Road is the extension of Whitmore Avenue.
- La Grange Dam located approximately 30 mi east of the Project location, and is not shown due to the scale of the figure.

C. USGS Gaging Station

USGS stream gaging station 11290000 is located approximately 0.5 mi upstream (east) of the proposed 7th Street bridge over Tuolumne River (see Figure 4). This station has annual peak flows of Tuolumne River recorded in 1895, 1940, and from 1943 to 2010. The historical high flow recorded by this stream gaging station is 57,000 cfs in December 9, 1950 (see Table 3).

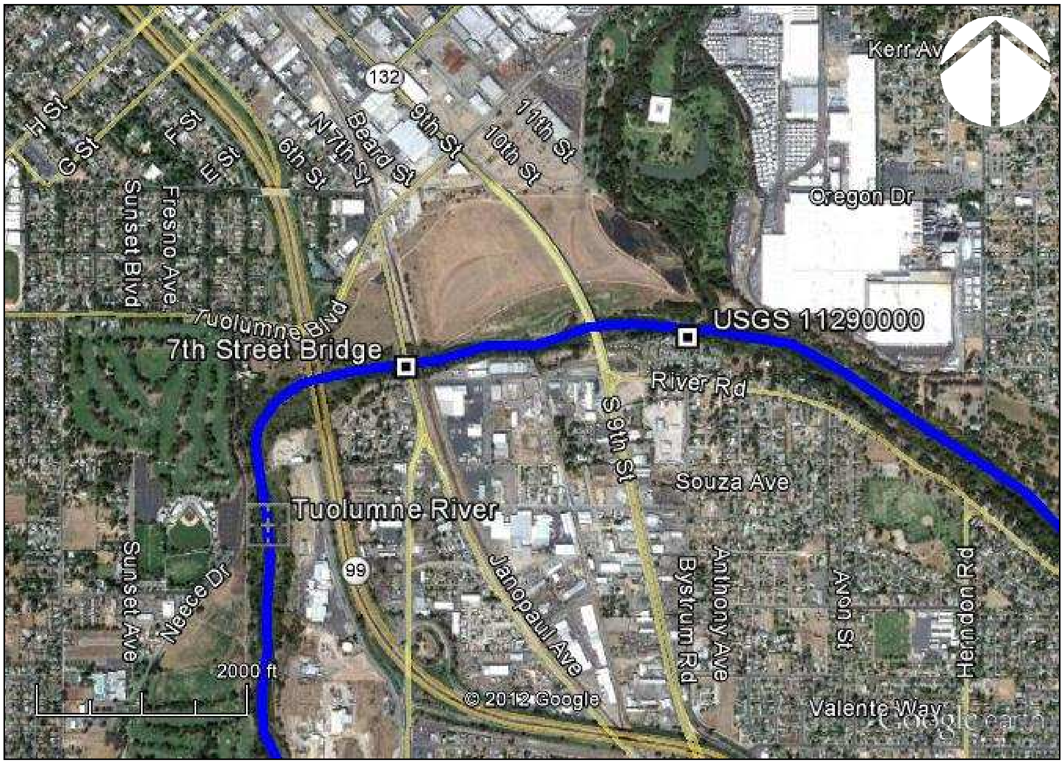

Figure 4. Location of USGS Stream Gaging Station

Source: Google Earth and USGS

Table 3. Recorded Annual Peak Flows in USGS Station 1290000

	Full Record (1895, 1940, 1943-2010)
Number of Records (yr)	70
Highest Annual Peak Flow (cfs)	57,000 (December 9, 1950)
Lowest Annual Peak Flow (cfs)	445 (March 16, 1977)
Mean Annual Peak Flow[1] (cfs)	8,220

Note: (1) The mean annual peak flow is rounded up to nearest 10 cfs.

Source: USGS

The Log-Pearson Type III (LPIII) distribution is a statistical distribution method used to estimate the annual exceedance probability of peak flows. The LPIII distribution has been used for several decades for the flood frequency analysis in the United States. The LPIII distribution analysis with and without the generalized skew coefficient was completed using annual peak flow discharge data from USGS stream gaging station 11290000 (see Table 4). The generalized skew coefficients are used in the LPIII distribution to stabilize flood frequency estimation. A map of the generalized skew coefficient is provided in USGS' *Bulletin 17B, Guidelines for Determining Flood Flow Frequency*.

Table 4. USGS Hydrologic Data, Log-Pearson Type III Distribution

Recurrence Interval (yr)	LP III Distribution (cfs)	LP III Distribution with Generalized Skew Coefficient (cfs)
50	37,500	36,800
100	49,200	47,900
200	63,100	61,000
500	85,600	81,800

Note: Flows in the table are rounded up to nearest 100 cfs.

D. 9th Street Bridge Report

Norman S. Braithwaite Incorporated prepared the draft hydraulic study of 9th River Bridge over Tuolumne River, located approximately 0.35 mi upstream (east) of the Project location (see Figure 5), in March 2000. The flood frequency relationship estimated by the FEMA Flood Hazard Mitigation Study was selected as the design 50- and 100-year flows of Tuolumne River for the draft hydraulic study. The FEMA design 50- and 100-year flows from the study were 906 cms (32,000 cfs) and 1,982 cms (70,000 cfs), respectively, which are identical to the design 50- and 100-year flows from the FEMA FIS (see Table 1).

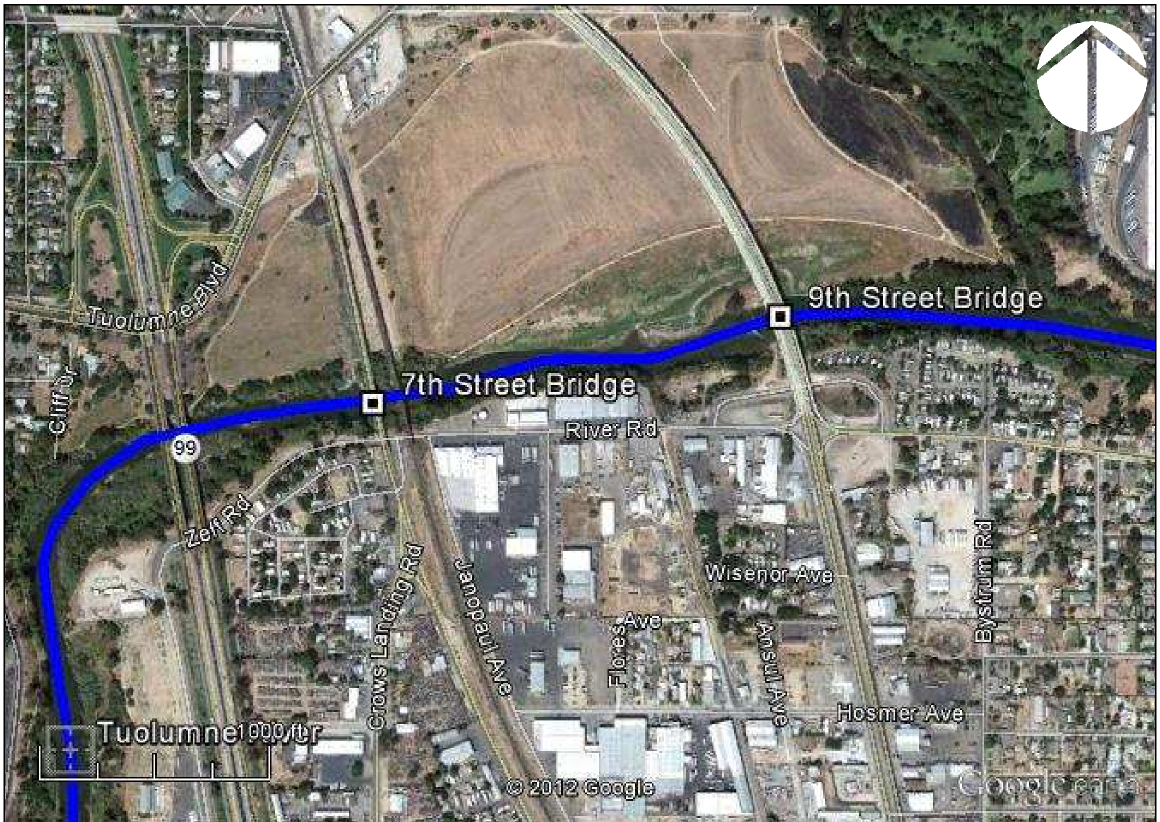

Figure 5. Location of 9th Street Bridge

Source: Google Earth

E. Hydrology Summary and Recommendation

The design 100-year flow of Tuolumne River at 7th Street bridge varies from 44,000 cfs from CVFPB to 70,000 cfs from FEMA FIS. According to the California State Reclamation Board, United States Army Corps of Engineers (USACE) reevaluated the hydrology of the Calfornia's Central Valley, including Tuolumne River, after the 1997 storm event. According to USACE's report published in 1999, the 100-year flow of Tuolumne River downstream of Don Pedro Dam was revised from 44,000 cfs to 70,000 cfs.

The design 200-year flows of Tuolumne River at 7th Street bridge were not available from FEMA FIS and CVFPB. Based on the annual peak flows recorded in the USGS gaging station and by interpolation using FEMA design 100- and 500-year flows, the design 200-year flow varied from 61,000 cfs to 105,400 cfs.

Based on the FEMA FIRM and the corresponding hydraulic model of Tuolumne River, FEMA's 100-year flood flows (70,000 cfs) with an elevation of approximately 75 ft, NAVD would overtop the southern banks and flood the adjacent properties (see Figure 6 and Figure 7). The toe of the bank

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elevation along the southern side of the studied reach is approximately 70 ft, NAVD; this area would be completely inundated.

The design 100- and 200-year flows of 70,000 cfs and 105,400 cfs were the most conservative design flows from the available studies. The proposed 7th Street bridge over Tuolumne River in the City of Modesto should be designed to have sufficient freeboard and structure foundation with the design 100-year flow of 70,000 cfs or design 200-year flow of 105,400 cfs.

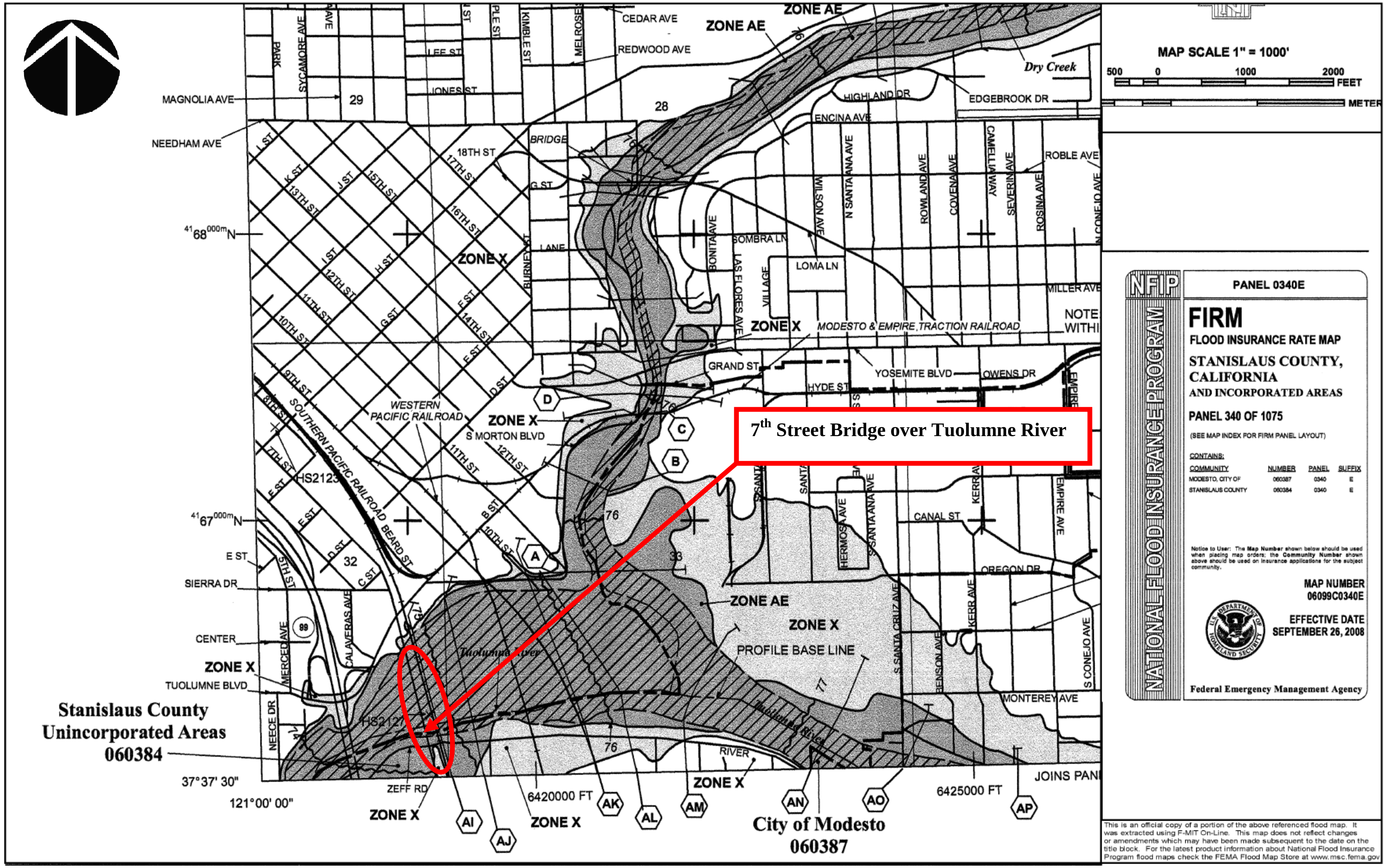

Figure 6. FEMA FIRM at the Project Location, Northern Panel

Source: FEMA, 2008

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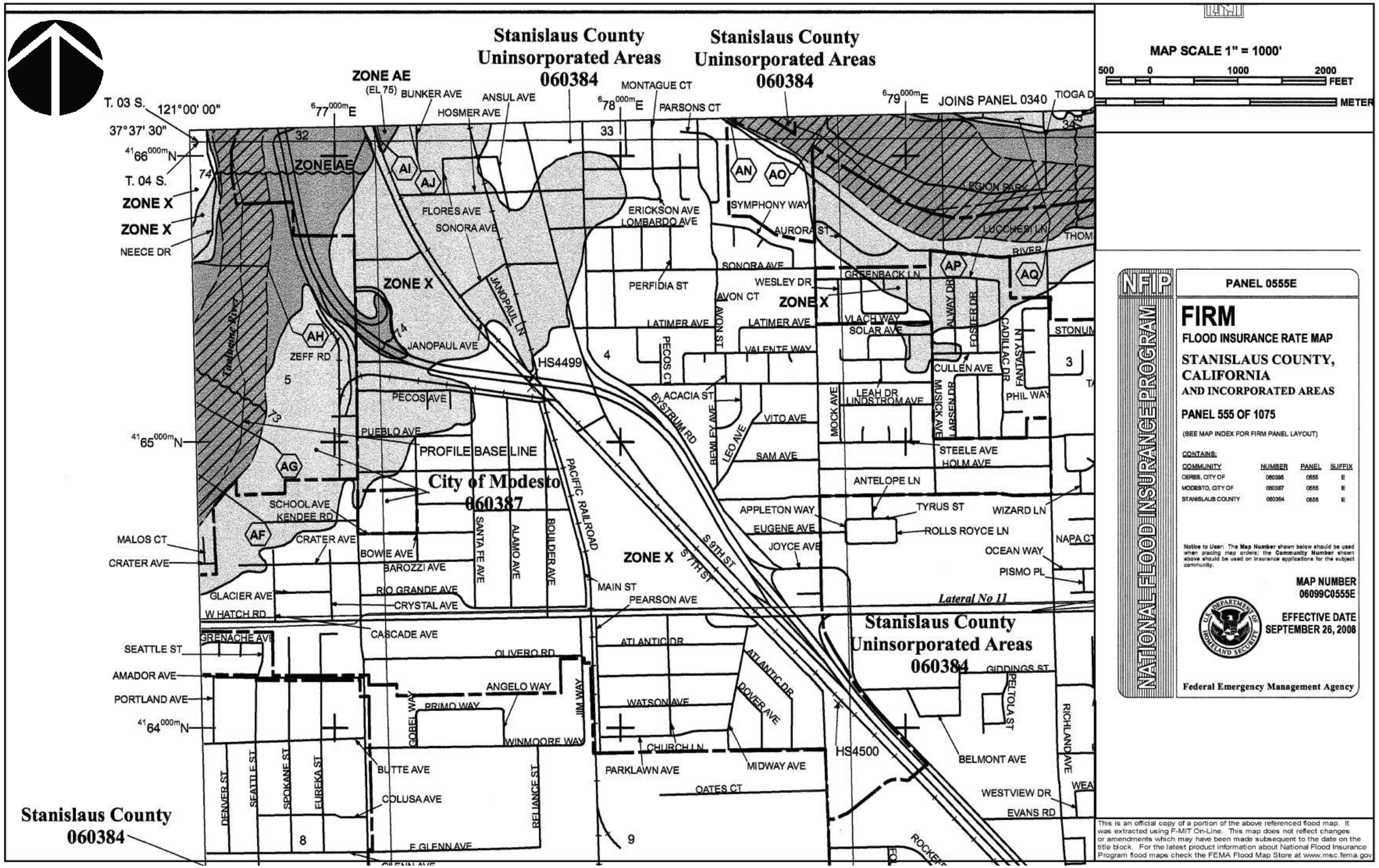

Figure 7. FEMA FIRM at the Project Location, Southern Panel

Source: FEMA, 2008

Preliminary Hydraulic Analysis

The FEMA hydraulic model of Tuolumne River in the Project vicinity provided by Michael Baker Corporation was used as the base hydraulic model for the preliminary hydraulic analysis. The channel cross sections of Tuolumne River at the Project location, based on the survey in 2012 provided by CH2M HILL were used to replace the existing cross sections upstream and downstream of 7th Street bridge, 7th Street Railroad bridge, and 9th Street bridge. The other cross sections in the model remained unchanged.

The design 100- , 200-, and 500-year flows from FEMA FIS were used in the hydraulic analysis. As discussed in Section A, the design 200-year flow at the Project location was interpolated using the design 100- and 500-flows. The water surface elevations (WSEs) of Tuolumne River at the 7th Street bridge are summarized in Table 5 and Figure 8.

Table 5. Summary of FEMA Hydraulic Model at 7th Street Bridge

Location		100-year Storm Event (70,000 cfs) (ft, NAVD)	200-year Storm Event (105,400 cfs) (ft, NAVD)	500-year Storm Event (154,000 cfs) (ft, NAVD)
7th Street Bridge	Immediately Upstream	75.2	80.6	86.2
	Bridge Upstream Face	75.1	79.7	85.9
	Bridge Downstream Face	75.0	79.7	85.8
	Immediately Downstream	75.0	80.5	85.9

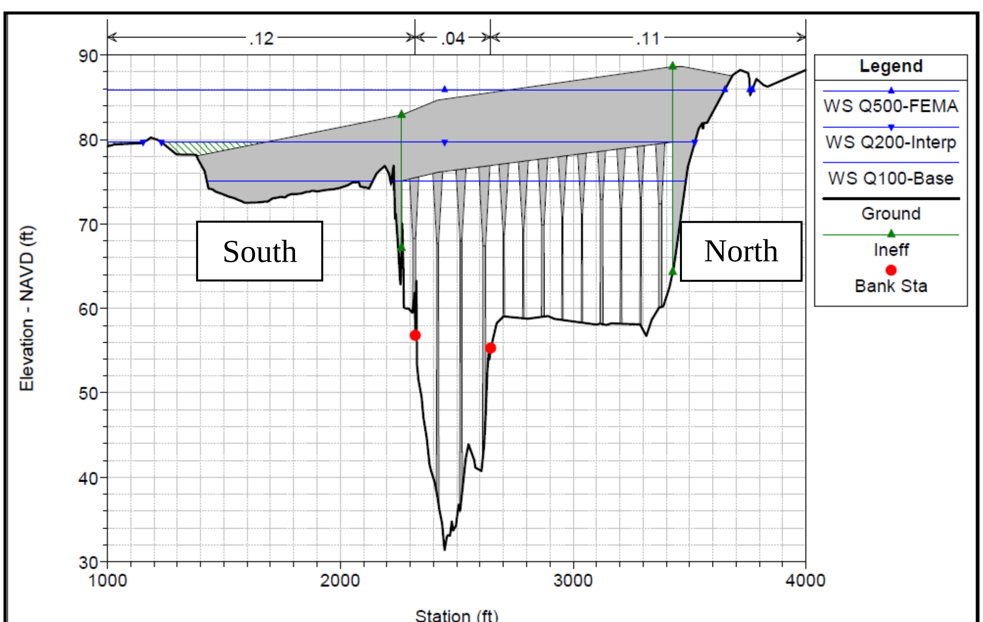

Figure 8. Cross-Sectional View of the Existing 7th Street Bridge, Looking Downstream

The north abutment of the existing bridge has approximately 4.5 ft of freeboard during the design 100-year storm event, but the south abutment would not have freeboard (see Table 6). The existing 7th Street bridge would not be overtopped during the design 200-year storm event, but the approach area south of the bridge would be overtopped (see Figure 8). During the design 500-year storm event, the southern portion of the existing 7th Street bridge would be overtopped (see Figure 8).

Table 6. Summary of Available Freeboards and Floodplain Widths

Design Storm Event (yr)	WSE (Immediately Upstream of the Bridge) (ft, NAVD)	WSE with 3 ft Freeboard (ft, NAVD)	Freeboard of Existing Bridge		Floodplain Width (ft)
			South Abutment (ft)	North Abutment (ft)	
100	75.2	78.2	-	4.5	1,690
200	80.6	83.6	-	-	2,230
500	86.2	89.2	-	-	4,380

The soffit elevation of the proposed 7th Street bridge would have to be 83.6 ft, NAVD or higher to provide the minimum 3 ft of freeboard over the 200-year WSE. The soffit elevation of the proposed 7th Street bridge would have to be 78.2 ft, NAVD or higher to provide the minimum 3 ft of freeboard over the 100-year WSE. The approach area of the bridge would not overtop during 100-year storm event.

Bridge Design Hydraulic Criteria

The hydraulic design of the bridge should conform to the CVFPB, FHWA, Caltrans, and Stanislaus County's freeboard criteria. Applicable sections from these agencies' design standards are summarized below. The most stringent criteria is set by the CVFPB, which requires that the bridge soffit be 3 ft above the 200-year storm event.

Central Valley Flood Protection Board:

According to the Barclays Official California Code of Regulations – Title 23. Waters - Division 1. Central Valley Flood Protection Board, Volume 32, Article 8 Standards, § 128 Bridges:

> (10)(A) The bottom members (soffit) of a proposed bridge must be at least three (3) feet above the design flood plane. The required clearance may be reduced to two (2) feet on minor streams at sites where significant amounts of stream debris are unlikely.

Federal Highways Administration, Highway Bridge Program:

According to the Code of Federal Regulation Title 23, SUBCHAPTER G--ENGINEERING AND TRAFFIC OPERATIONS, Part 650-Bridges, Structures, and Hydraulics, Subpart A-Location and Hydraulic Design of Encroachments on Flood Plains, § 650.115 Design standards:

> (2) The design flood for encroachments by through lanes of Interstate highways shall not be less than the flood with a 2-percent chance of being exceeded in any given year. No minimum design flood is specified for Interstate highway ramps and frontage roads or for other highways.
>
> (3) Freeboard shall be provided, where practicable, to protect bridge structures from debris- and scour-related failure.

California Department of Transportation:

The Caltrans Highway Design Manual and bridge design manuals set the hydraulic criteria for highway bridges to the Q50, or the flood-of-record, the greater of which shall be designated as the "design flood", with adequate freeboard provided above the design flood to pass anticipated drift, AND shall convey the Q100 with no freeboard.

County of Stanislaus:

The Stanislaus County Department of Public Works Standards and Specifications 2007 edition indicates that the design of Bridges shall be to Caltrans standards, and that fill below 2 feet above the 100-year flood (Q100) elevation shall be protected from erosion by slope protection as approved by the Engineer.

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Recommendations

Based on the preliminary investigations into the hydraulic characteristics of the existing 7th Street bridge over Tuolumne River, WRECO recommends designing the proposed structure to pass the 100-year storm event with 3 ft of freeboard.

Under existing conditions, the bridge can pass the 100-year storm event with no freeboard, and is overtopped during the 200-year storm event. There would be tremendous impacts to adjacent infrastructure if the bridge were designed to pass the 200-year storm event with 3 ft of freeboard. The bridge soffit would need to be raised by at least 9.1 ft, and the roadway approaches would accordingly also need to be considerably raised to meet roadway geometric standards. Doing so would result in significant impacts to adjacent properties as well as nearby roadways. In addition, the limit of the existing 200-year floodplain extends approximately 1,600 ft south of the existing bridge's southern abutment. Even if the bridge itself and the roadway approach are raised to pass the 200-year storm event with 3 ft of freeboard, there would still be portions of the roadway that would remain within the 200-year floodplain. Because the southern roadway would be overtopped during the 200-year event, the bridge would be inaccessible from the south, and vehicles approaching from the north would be unable to proceed beyond.

Because of the reasons stated above, designing the bridge to pass the 100-year storm event with 3 ft of freeboard is considered to be more feasible than designing the bridge to pass the 200-year storm event with 3 ft of freeboard.

Appendix E – Preliminary Scour Analysis

WRECO
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Draft Memorandum

Date: October 23, 2012

To: Hans Strandgaard – CH2M HILL

From: Kazuya Tsurushita/Han-Bin Liang/Chris Sewell – WRECO

Project 7th Street Bridge over Tuolumne River Replacement Project, Stanislaus County

Subject: Summary of Scour Analysis of the Existing 7th Street Bridge

---

The purpose of the proposed 7th Street Bridge Replacement Project (Project) in the City of Modesto, Stanislaus County, California, is to improve movement and safety for motor vehicles, pedestrians, and bicyclists across Tuolumne River on 7th Street. The Project is proposing to replace the existing structurally deficient bridge with a new structure including the following improvements:

- Provide full truck carrying capacity;
- Expand vehicular capacity of the 7th Street corridor; and
- Improve safety for motor vehicles, pedestrians, and bicyclists.

The purpose of this memorandum is to summarize the scour analysis of the existing 7th Street bridge over Tuolumne River.

Scour Design Criteria

The bridge scour was evaluated per the criteria described in the Federal Highway Administration's (FHWA's) *Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges, Fifth Edition* (HEC-18) (April 2012). Hydraulic data was obtained from the HEC-RAS model for the existing 7th Street bridge over Tuolumne River. The 1% annual exceedance probability flood (100-year flood or Q_{100}) was selected for the scour design flood frequency for the existing 7th Street bridge. The 0.5% annual exceedance probability flood (200-year flood or Q_{200}) was selected for the scour design check flood frequency. Scour analysis with Q_{200} as the design flood was performed to calculate a more conservative scour hole elevation, which would provide guidelines for the bridge design with a minimized risk of failure.

The total scour of the bridge is the sum of local scour (pier or abutment), channel contraction scour, and long-term scour. Channel contraction scour and local scour at the bridge structures were calculated following the criteria described in the FHWA's HEC-18. The long-term bed elevation change was based on the historical channel surveys available in the California Department of Transportation's (Caltrans') Bridge Inspection Reports (BIRs) for the 7th Street bridge.

The design 100- and 200-year flows of Tuolumne River used in the hydraulic analysis of the existing 7th Street bridge were based on the peak 100- and 500-year flows from the Federal Emergency Management Agency's (FEMA) Flood Insurance Study (FIS) for Stanislaus County and Incorporated

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Areas. Because the 200-year flow was not available from the FEMA FIS, it was interpolated by using the peak 100- and 500-year flows of Tuolumne River at the City of Modesto. The design 100- and 200-year flows were 70,000 cfs and 105,400 cfs, respectively.

The length, width, and size of piers of the existing 7th Street bridge in the hydraulic model were based on the bridge as-builts provided by CH2M HILL. Based on the field observations, the material subject to scour at the Project location will be clay with a median particle diameter size of approximately 0.003 mm.

The following sections explain WRECO's analysis

- A. Long-Term Bed Elevation Change
- B. Contraction Scour
- C. Pier Scour
- D. Abutment Scour
- E. Total Scour Depths and Evaluation

A. Long-Term Bed Elevation Change

The channel bed elevation may fluctuate over time as a result of changes in local sediment transport capacity and availability. Channel aggradation occurs when more sediment is supplied by watershed erosion and upstream channel flow than can be transported locally. Only channel degradation is considered for the purposes of analyzing scour.

The long-term bed elevation change of Tuolumne River at the Project location over the anticipated lifetime of the 7th Street bridge was estimated based on the comparison of the channel bed elevations from the Caltrans supplemental bridge inspections dated November 2, 1995 and November 17, 1972 (see Figure 1). The comparison of the surveyed channel cross sections showed that there were no signs of channel degradation between 1972 and 1995. From this information, WRECO determined that the historic long-term bed elevation changes of Tuolumne River at the existing 7th Street Bridge has been were insignificant.

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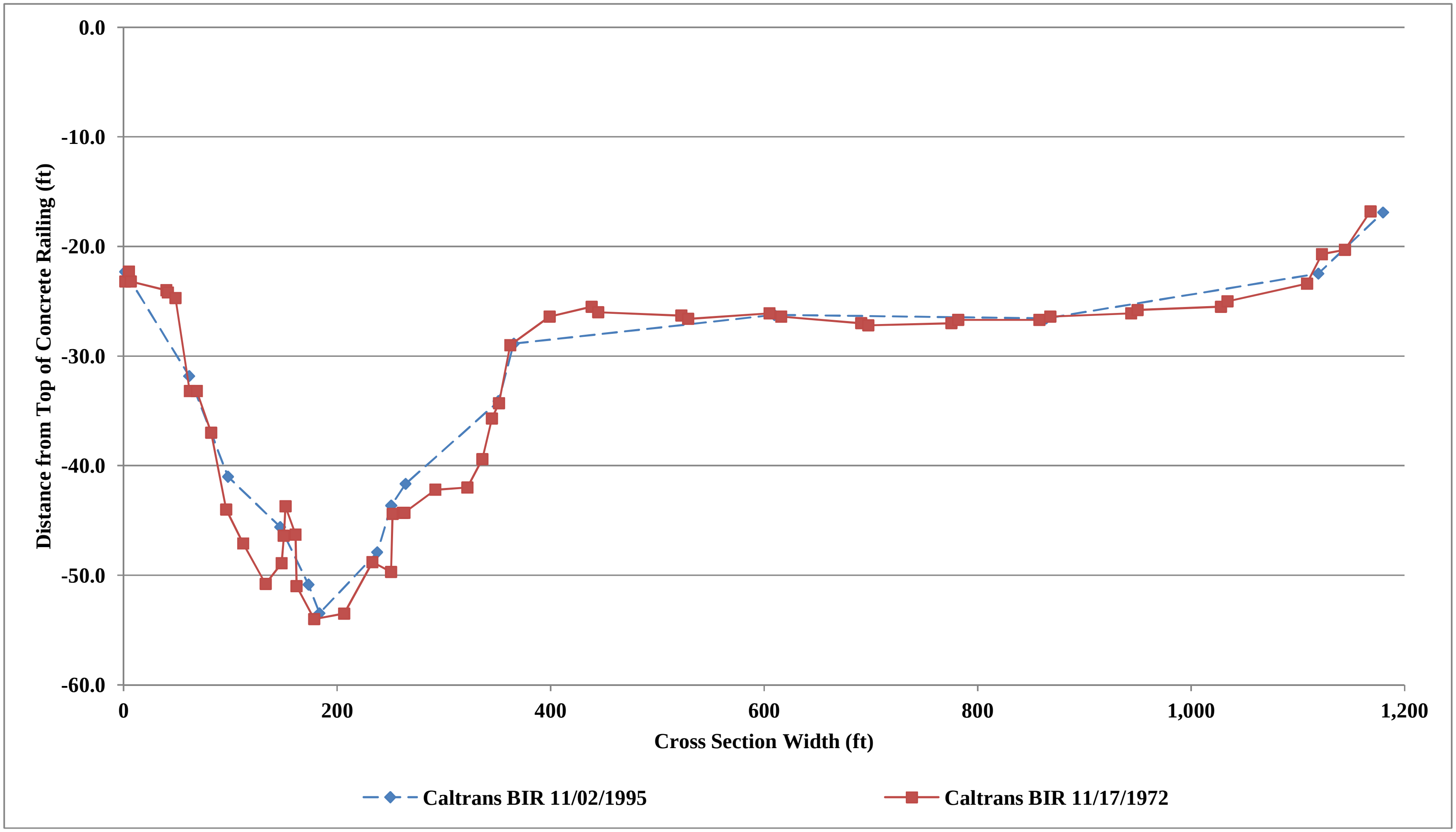

Figure 1. Comparison of the Historical Channel Cross Sections

Source: Caltrans

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B. Contraction Scour

Contraction scour occurs when the flow area of a stream is reduced by either: 1) the natural contraction of the stream channel; 2) a bridge structure; or 3) the overbank flow forced back into the channel by roadway embankments at a bridge approach.

From the continuity equation, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contracted section, and more bed material is removed from the contracted reach than is transported into the reach. This increase in bed material transport from the reach lowers the natural bed elevation, resulting in an increased flow area. Thus, the velocity and shear stress decrease until relative equilibrium is reached; i.e, the quantity of bed material that is transported into the reach is equal to that removed from the reach, or the bed shear stress is decreased to a value such that no sediment is transported out of the reach. Contraction scour, in a natural channel or at a bridge crossing, involves removal of material from the bed across all or most of the channel width (FHWA 2012).

Live-bed contraction scour occurs at a bridge when sediment or bed materials from upstream are transported into the bridge cross section. If the critical velocity (V_c) is less than the mean channel velocity, live-bed contraction scour is assumed (HEC-18, equation 6.2). Clear-water contraction scour occurs when there is no sediment or bed material from upstream being transported into the bridge cross section. If V_c is greater than the mean channel velocity, clear-water scour is assumed (HEC-18, equation 6.4). The V_c was calculated using equation 6.1 in HEC-18 and outputs from the hydraulic analysis. The critical velocity with the assumed particle size of 0.003 mm is summarized in Table 1.

Table 1. Critical Velocity Summary

Recurrence Interval (yr)	Location	Average Flow Depth (ft)	Critical Velocity (ft/sec)	Average Flow Velocity (ft/sec)	Contraction Scour Type
100	North Overbank	15.7	0.4	1.1	Live Bed
	Main Channel	31.6	0.4	4.7	Live Bed
	South Overbank	12.9	0.4	0.8	Live Bed
200	North Overbank	20.0	0.4	1.4	Live Bed
	Main Channel	37.0	0.4	5.8	Live Bed
	South Overbank	18.3	0.4	1.1	Live Bed

The average flow velocities at the main channel, north overbank, and south overbank during the design 100- and 200-year storm events were faster than the critical velocity; thus, the live-bed contraction scour equation was used to calculate the contraction scour. The contraction scour depths of Tuolumne River at the existing 7th Street bridge during the design 100- and 200-year storm events are shown in Table 2.

Table 2. Contraction Scour Summary

Recurrence Interval (yr)	Location	Scour Depth (ft)
100	North Overbank	4.0
	Main Channel	0.0
	South Overbank	2.9
200	North Overbank	4.2
	Main Channel	2.2
	South Overbank	2.2

C. Pier Scour

Pier scour is caused by vortices forming at the base of the pier. The scour depth at the pier is determined by pier design, flow characteristics (flow rate, local flow velocity at the pier, and local flow depth at the pier), and sediment particle size distribution. The pier widths for the existing bridge were based on the widths of the pile caps of the piers from the bridge as-builts, which provided the most conservative design widths. The outputs from the hydraulic analyses were used for the flow velocities and flow depths at the piers. The Colorado State University (CSU) equation, referenced in HEC-18, was used to determine local scour at the piers. The local scour depths of the existing bridge piers during the design 100- and 200-year storm events are shown in Table 3 and Table 4, respectively.

Table 3. Pier Scour Summary, Design 100-year Storm Event

Proposed Bridge Structure	Pier Width (ft)	Local Flow Depth (ft)	Local Flow Velocity (ft)	Scour Depth (ft)
Pier A	15.0	15.0	1.1	9.1
Pier B	8.0	17.0	1.2	6.4
Pier C	8.0	16.9	1.2	6.4
Pier D	12.5	17.0	1.2	8.6
Pier E	8.0	16.8	1.2	6.4
Pier F	8.0	16.5	1.2	6.3
Pier G	12.5	16.1	1.2	8.4
Pier H	8.0	16.3	1.2	6.3
Pier I	8.0	16.0	1.2	6.3
Pier J	12.5	31.3	4.6	16.6
Pier K	12.5	39.0	5.1	17.8
Pier L	12.5	37.3	5.3	18.1
Pier M	16.5	16.3	0.9	8.9

Table 4. Pier Scour Summary, Design 200-year Storm Event

Proposed Bridge Structure	Pier Width (ft)	Local Flow Depth (ft)	Local Flow Velocity (ft)	Scour Depth (ft)
Pier A	15.0	20.5	1.5	10.9
Pier B	8.0	22.5	1.6	7.6
Pier C	8.0	22.4	1.6	7.6
Pier D	12.5	22.5	1.6	10.1
Pier E	8.0	22.3	1.6	7.5
Pier F	8.0	22.0	1.6	7.5
Pier G	12.5	21.6	1.6	9.9
Pier H	8.0	21.8	1.6	7.4
Pier I	8.0	21.5	1.6	7.5
Pier J	12.5	36.8	5.7	18.6
Pier K	12.5	44.6	6.2	19.8
Pier L	12.5	42.8	6.5	20.1
Pier M	16.5	16.3	1.2	10.2

D. Abutment Scour

High flow events would cause local scour at the abutments. A vortex is formed on the upstream end and along the toe of the abutment due to the flow obstruction caused by the abutments. The highly turbulent flow caused by the abutments generates erosive shear action, which subsequently causes scour.

Froehlich's equation would be used for cases where the abutment length (L) is small in comparison to the flow depth (y_1) ($L/y_1 < 25$). The HIRE equation would be applicable when the ratio of the projected abutment length to the flow depth is greater than 25 ($L/y_1 > 25$). For both the north and south abutments for the existing bridge during the design 100- and 200-year storm events, Froehlich's equation was used to calculate the local scour, which is summarized in Table 5.

Table 5. Abutment Scour Summary

Recurrence Interval (yr)	Bridge Component	Scour Depth (ft)	Equation
100	North Abutment	12.6	Froehlich
	South Abutment	11.3	Froehlich
200	North Abutment	21.0	Froehlich
	South Abutment	15.9	Froehlich

E. Total Scour Depths and Evaluation

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Total scour is the sum of local scour, contraction scour, and long-term bed elevation change. The itemized total scour depths for the abutments and piers of the existing 7th Street bridge during the design 100- and 200-year storm events are shown in Table 6 and Table 7, respectively.

There were no significant changes in the depths and location of the thalweg of Tuolumne River at the Project location between 1972 and 1995. The scour hole elevations at the existing bridge piers and abutments were calculated by subtracting the total scour depth at each location from the existing ground elevation from the 2012 survey (see Table 6 and Table 7).

The bridge as-builts from 1916 did not provide the vertical datum of the elevations. In this analysis, the elevations from the as-builts were assumed to be similar to the elevations referring to North American Vertical Datum. Based on this assumption, concrete piles below the bridge abutments and pile caps of all of the bridge piers would be exposed during the design 100- and 200-year storm events.

Table 6. Total Scour Summary, Design 100-year Storm Event

Location	Bridge Component (from As-Builts)	Local Scour (ft)	Contraction Scour (ft)	Long-term Scour (ft)	Total Scour (ft)	Ground Elevation (ft, NAVD)	Scour Hole Elevation(1) (ft, NAVD)	Bottom of Pile Cap Elevation(2) (ft)
Left (North) Overbank	North Abutment	12.6	4.0	0.0	16.6	64.3	47.7	50.0
	Pier A	9.1	4.0	0.0	13.1	60.1	47.0	48.0
	Pier B	6.4	4.0	0.0	10.4	58.1	47.7	45.0
	Pier C	6.4	4.0	0.0	10.4	58.2	47.8	50.0
	Pier D	8.6	4.0	0.0	12.6	58.1	45.5	50.0
	Pier E	6.4	4.0	0.0	10.4	58.3	47.9	50.0
	Pier F	6.3	4.0	0.0	10.3	58.6	48.3	50.0
	Pier G	8.4	4.0	0.0	12.4	59.0	46.6	50.0
	Pier H	6.3	4.0	0.0	10.3	58.8	48.5	50.0
	Pier I	6.3	4.0	0.0	10.3	59.1	48.7	50.0
Channel	Pier J	16.6	0.0	0.0	16.6	43.8	27.2	33.0
	Pier K	17.8	0.0	0.0	17.8	36.0	18.2	33.0
	Pier L	18.1	0.0	0.0	18.1	37.8	19.7	33.0
Right (South) Overbank	Pier M	8.9	2.9	0.0	11.8	59.5	47.7	33.0
	South Abutment	11.3	2.9	0.0	14.1	63.0	48.8	50.0

Notes:
(1): Scour hole elevation is the ground elevation minus the total scour depth.
(2): The datum was not specified in the bridge as-built dated 1916. The elevations from the as-built are shown in the table.

Table 7. Total Scour Summary, Design 200-year Storm Event

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Location	Bridge Component (from As-Builts)	Local Scour (ft)	Contraction Scour (ft)	Long-term Scour (ft)	Total Scour (ft)	Ground Elevation (ft, NAVD)	Scour Hole Elevation(1) (ft, NAVD)	Bottom of Pile Cap Elevation(2) (ft)
Left (North) Overbank	**North Abutment**	21.0	4.2	0.0	**25.2**	**64.3**	**39.1**	**50.0**
	Pier A	10.9	4.2	0.0	**15.1**	**60.1**	**45.0**	**48.0**
	Pier B	7.6	4.2	0.0	**11.8**	**58.1**	**46.3**	**45.0**
	Pier C	7.6	4.2	0.0	**11.8**	**58.2**	**46.4**	**50.0**
	Pier D	10.1	4.2	0.0	**14.3**	**58.1**	**43.8**	**50.0**
	Pier E	7.5	4.2	0.0	**11.7**	**58.3**	**46.6**	**50.0**
	Pier F	7.5	4.2	0.0	**11.7**	**58.6**	**46.9**	**50.0**
	Pier G	9.9	4.2	0.0	**14.1**	**59.0**	**44.9**	**50.0**
	Pier H	7.4	4.2	0.0	**11.6**	**58.8**	**47.2**	**50.0**
	Pier I	7.5	4.2	0.0	**11.7**	**59.1**	**47.3**	**50.0**
Channel	**Pier J**	18.6	2.2	0.0	**20.8**	**43.8**	**23.0**	**33.0**
	Pier K	19.8	2.2	0.0	**22.0**	**36.0**	**14.0**	**33.0**
	Pier L	20.1	2.2	0.0	**22.3**	**37.8**	**15.5**	**33.0**
Right (South) Overbank	**Pier M**	10.2	2.2	0.0	**12.4**	**59.5**	**47.1**	**33.0**
	South Abutment	15.9	2.2	0.0	**18.0**	**63.0**	**45.0**	**50.0**

Notes:
(1): Scour hole elevation is the ground elevation minus the total scour depth.
(2): The datum was not specified in the bridge as-built dated 1916. The elevations from the as-built are shown in the table.

Appendix F – Evaluation of Foundations and ARS Curves for Existing Structure

7th Street Bridge Replacement, Stanislaus County

Evaluation of Foundations for Existing Structure

PREPARED FOR: Chris Serroels/SAC
Hans Strandgaard/SAC

PREPARED BY: D.E. Harris/SAC

DATE: January 7, 2013

PROJECT NUMBER: 437598

This memorandum was prepared to summarize estimates of foundation stiffness for use in evaluating existing 7th Street Bridge crossing the Tuolumne River, in Modesto California. These estimates were developed based on geotechnical information presented by Taber Consultants in their geotechnical report for the 7th Street Bridge Replacement Project, prepared in 1996. The evaluation was also based on pile arrangements as shown on the As-Built drawings provided by Stanislaus County.

The evaluation of foundation stiffness included the following:

- Review of pile lateral stiffness and estimates of average group factors for each of the abutment and pier arrangements
- Estimates of passive resistance of walls or pier caps
- Estimates of axial capacity values for pile foundations

Each of these is discussed in greater detail as follows.

Pile Groups – Lateral Resistance

Taber consultants provided estimates of the lateral stiffness of single piles in the attachments to their report, and a copy of their summary is attached. Separate calculations were not performed to confirm these values. It is recommended that these should be used a preliminary analysis gauge the acceptability of the foundation conditions. If the analysis indicates that the conditions may be acceptable, then more detailed analysis will likely be necessary to evaluate the single pile stiffness, including more detailed consideration of the pile structural condition, soil profile, etc.

TABLE 1
Pile Group Factors for Lateral Resistance

Location	Transverse	Longitudinal
North and South Abutments	0.9	0.8
Pier A	0.7	0.5
Piers B, C, E, F, H, and I	0.5	0.4
Piers D, G	0.4	0.5
Piers J, L, K, and M	0.5	0.4

An order of magnitude estimate of the “average” pile group factor for each pier was made, based on the group factor values presented in the AASHTO LRRD Design code. Pile group factors vary depending on the pile spacing in the direction of loading, the pile spacing in the direction perpendicular to the load, and the number of rows. For the existing structure the pile spacing differs significantly, but frequently the center to center spacing is as small as 2.1B, where B is the width of the pile.

Estimated average values for group factors to be used in conjunction with the single pile stiffness values provided by Taber Consultants, are presented in Table 1. The values were estimated by estimating the p-y multiplier values (as recommended/required in the AASHTO code) for each pile in the group, and then calculating the weighted average value.

Passive Resistance of Walls or Pile Caps

The lateral stiffness of retaining walls or pile caps may be estimated based on the height of the wall, as follows:

$K_{EFF} \approx 33 \cdot H\ L$ (klf)

K_{EFF} is the lateral stiffness of the wall, per unit length of the wall

H is the height of the wall and L is the length of the wall, both in feet

Axial Capacity Estimates

The estimated pile capacity values provided by Taber appear to overestimate the capacity of the piles significantly. In two cases the tension values are larger than the compression values. Therefore the Taber results for axial capacity do not appear to be reliable. Estimated axial capacity values are provided below. Note that there is very little information available for the timber pile foundations. Because their total length is not known, they were estimated to be about 30 ft long.

TABLE 2
Axial Capacity of Piles for Existing Bridge

Location	Side Resistance (kip)	Toe Resistance (kip)	Ultimate Resistance (kip)
Abutments and Piers A through I	90	10	100
Pier J, K, L, and M	200	25	225

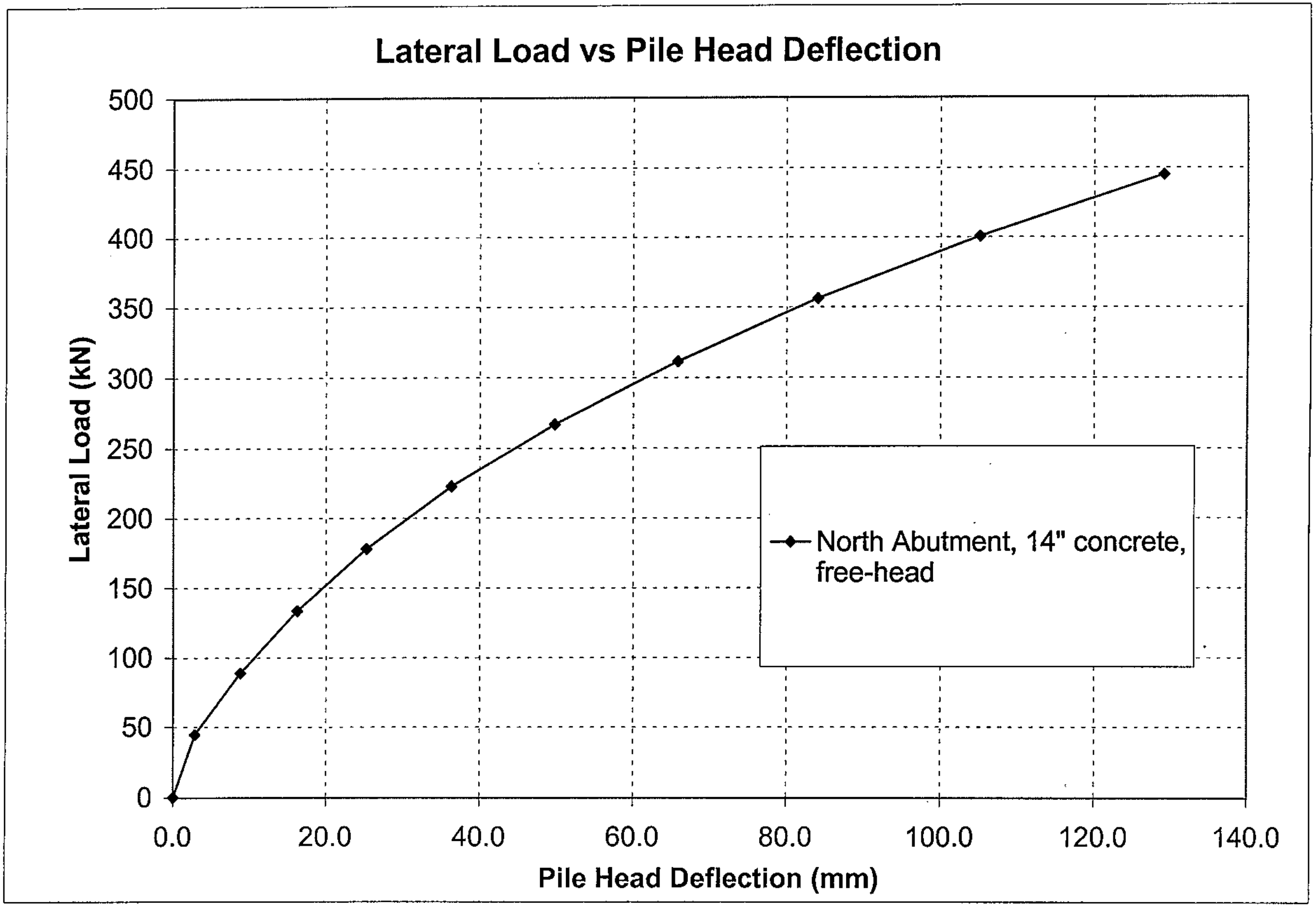
It should also be noted that the piles are relatively short, and there appears to be liquefiable material near the toe or below the toe of the piles at Pier E through H. Liquefaction of these materials could result in significant post-earthquake settlement of the structure.

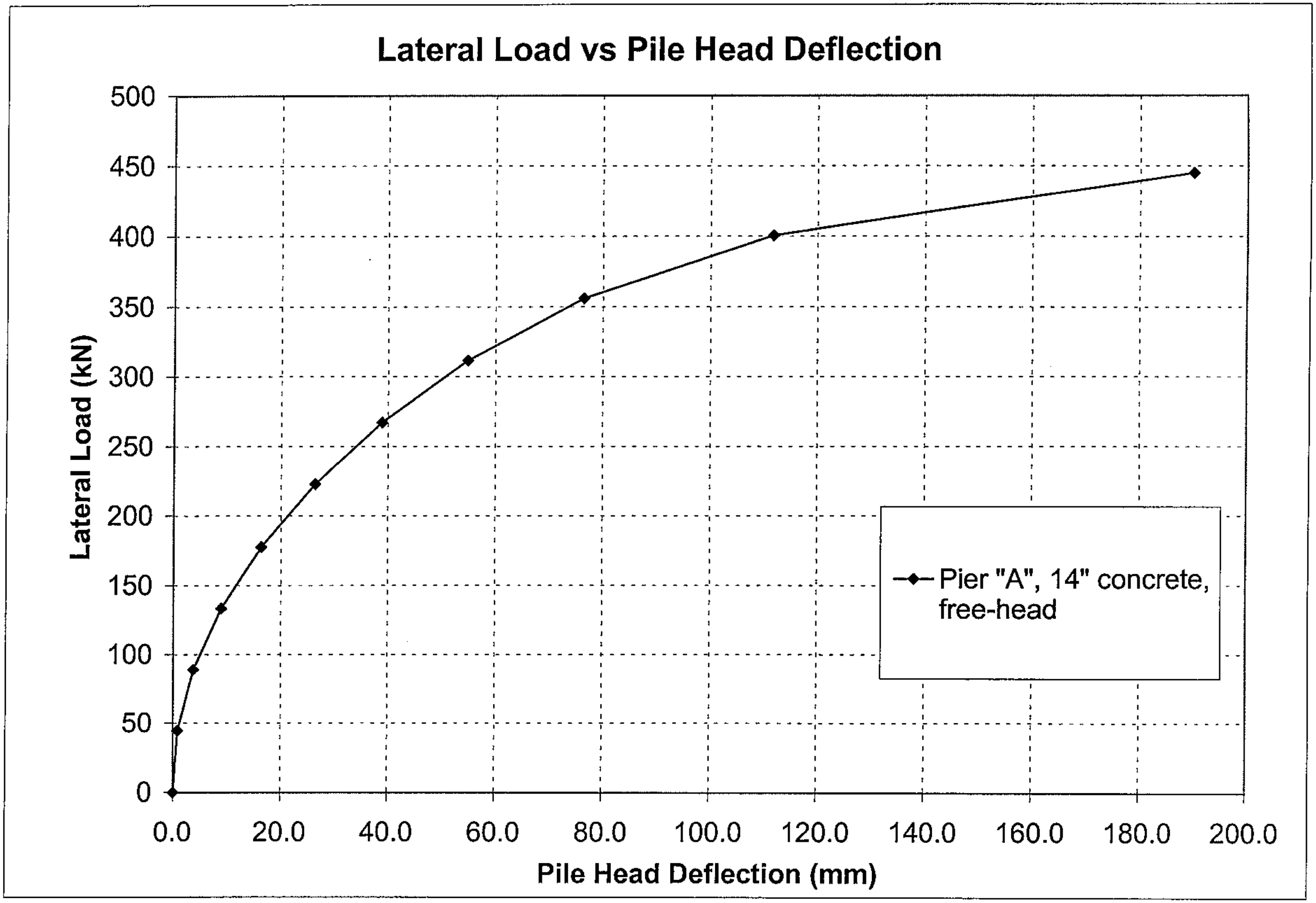
Estimated Capacities of Existing Piles

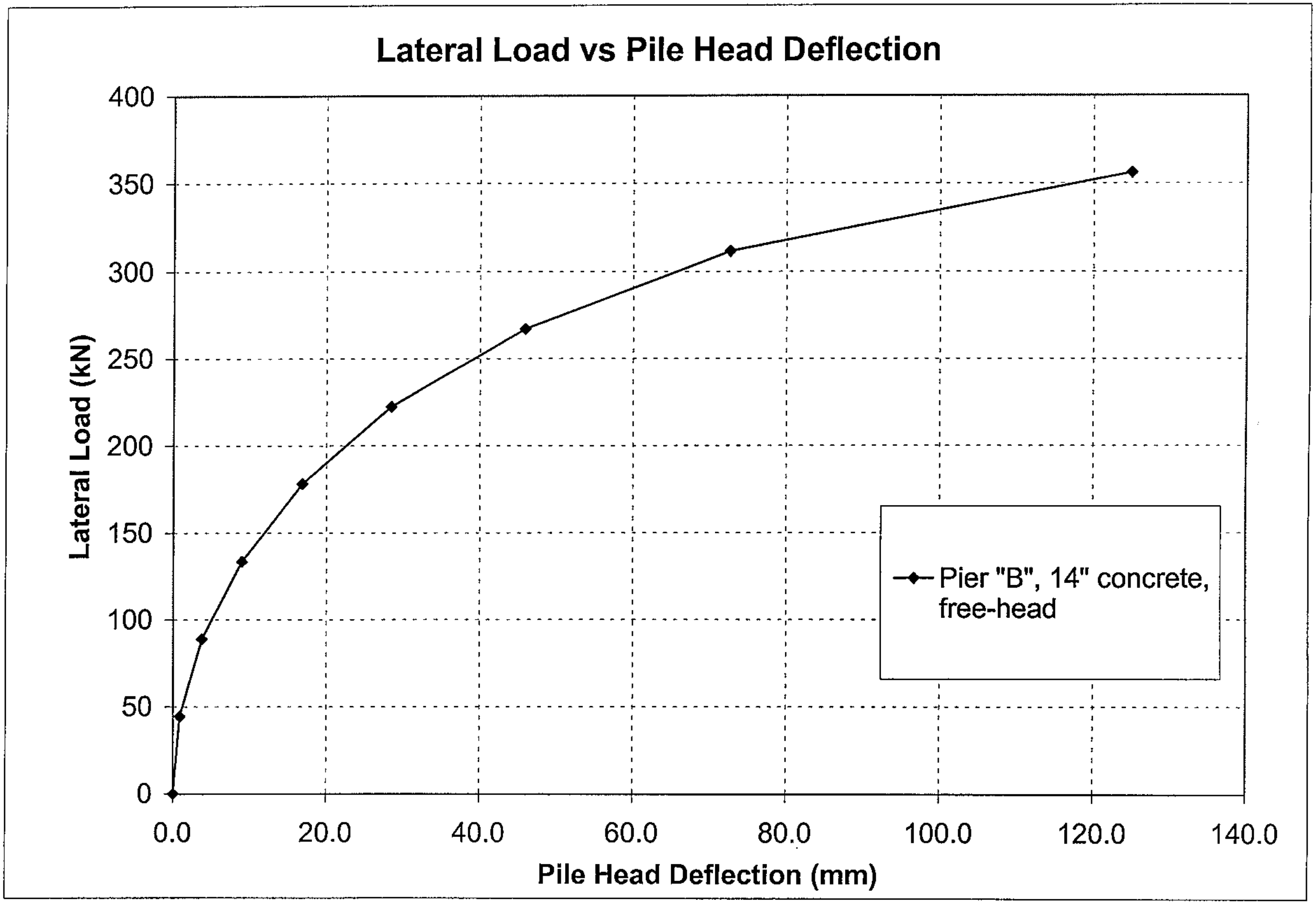
Support	Plan Footing Elev. (m)	Pile Type	Estimated Pile Tip Elev. (m)	Estimated Ultimate Pile Capacity (kN)		Est. Lateral Stiffness (kN/mm @ 75 mm)	Liquefaction Induced	
				Compression	Tension		Negative Skin Friction (kN/pile)	Settlement (mm)
N. Abut.	16.24	concrete	10.45	900	450	4.5	xx	xx
Pier-A	15.63	concrete	9.84	460	500	4.7	xx	85
Pier-B	14.72	concrete	8.93	390	440	4.2	xx	85
Pier-C	16.24	concrete	10.45	800	400	2.9	xx	85
Pier-D	16.24	concrete	10.45	800	400	2.9	xx	xx
Pier-E	16.24	concrete	10.45	400	400	2.9	xx	50
Pier-F	16.24	concrete	10.45	290	290	2.7	xx	50
Pier-G	16.24	concrete	10.45	290	290	2.7	xx	50
Pier-H	16.24	concrete	10.45	110	65	2.1	xx	300
Pier-I	16.24	concrete	10.45	110	65	2.1	xx	300
Pier-J	11.06	timber	6?	350	50	0.4	50	15
Pier-K	11.06	timber	6?	400	65	see note 7	50	xx
Pier-L	11.06	timber	6?	400	65	see note 7	50	xx
Pier-M	11.06	timber	6?	450	90	2.3	xx	xx
S. Abut.	16.24	concrete	10.45	900	450	4.4	xx	xx

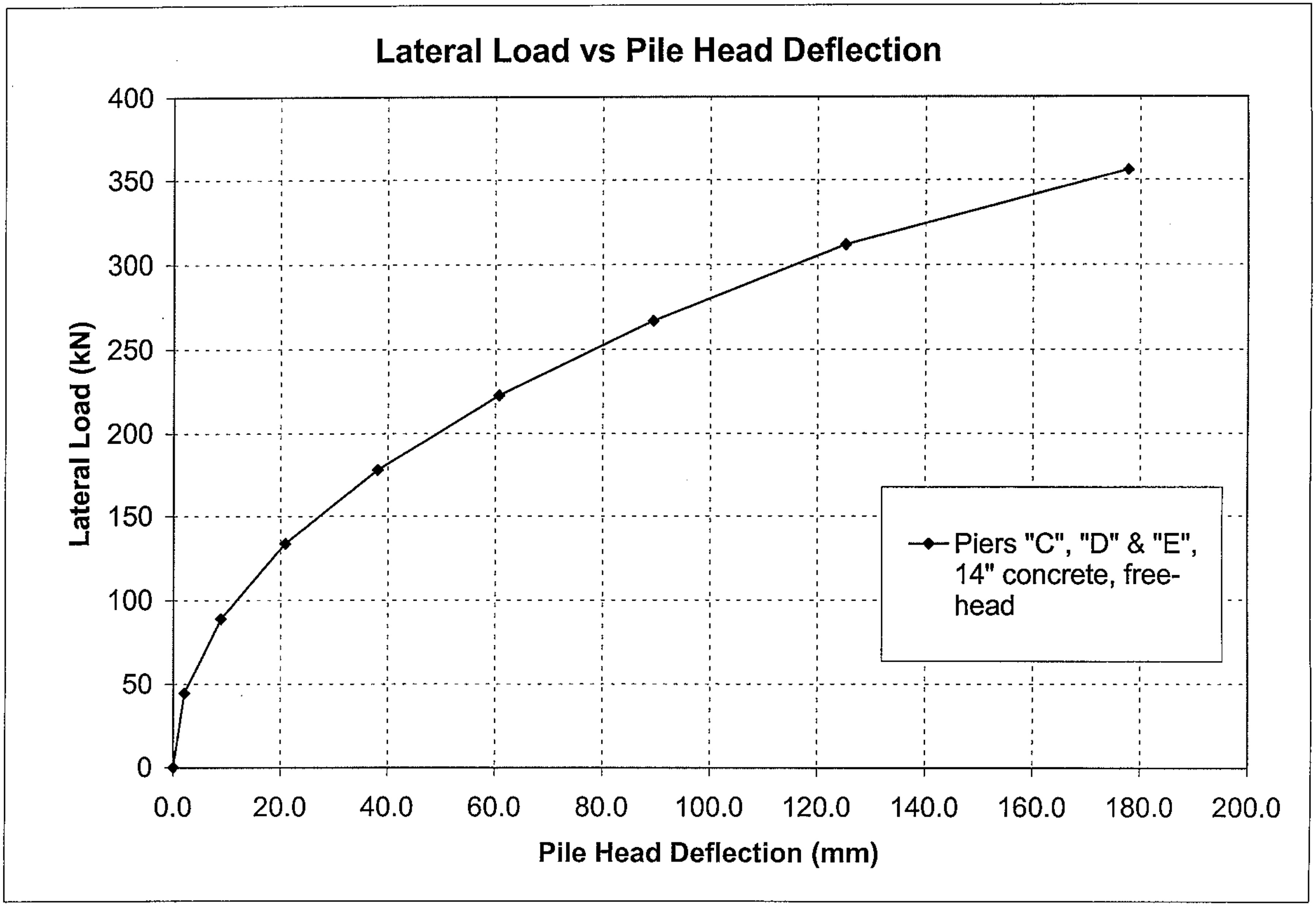
Notes:

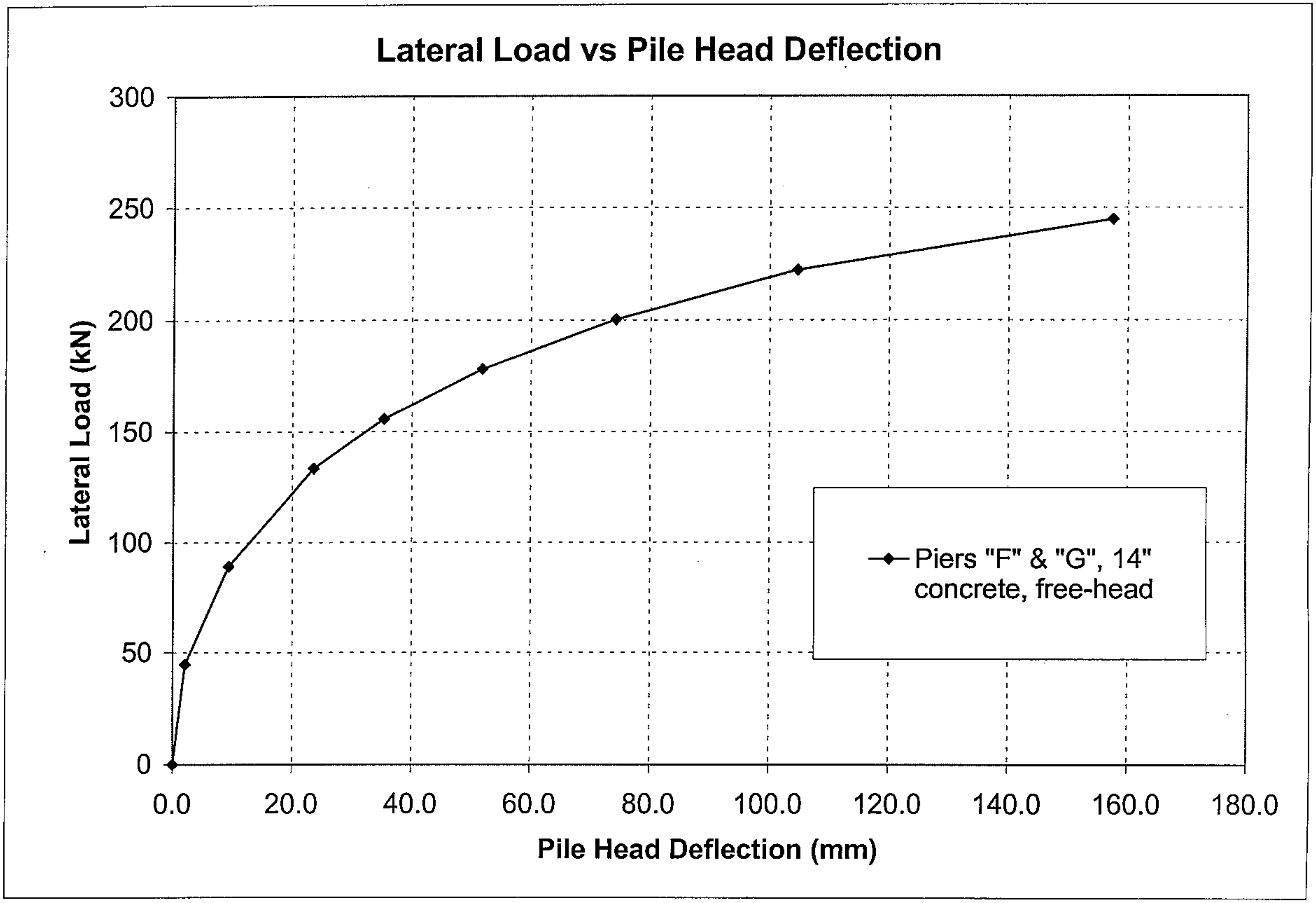
1) Concrete = 356 mm square pre-cast, reinforced concrete; Timber = assume 305 mm minimum butt diameter and 203 mm min. tip diameter Pacific Coast Douglas Fir.
2) Pile tip elevations based on 6.1 m pile casting length for concrete with 0.3 m embedment in footing; assumed 5 m length for timber with 0.91 m embedment in footing.
3) Typical pile spacing is in the range of 760 mm to 840 mm center-center.Pier footings are shown as plain concrete, i.e. without steel reinforcing.
4) Ultimate compressive capacities may be higher at some locations.
5) Pile structural details and connections may control available pile capacities and lateral stiffness.
6) Assume free-head for concrete piles and fixed-head for timber piles.
7) At Piers K & L, the piles are not sufficiently embedded to develop "fixity" in the ground; suggest considering them as "fixed" in footing and "pinned" about 3.6 m below base of footing.
8) Pile capacity and lateral stiffness estimates are for "liquefied" soil conditions; they may be significantly greater where liquefaction has not occurred.
9) Liquefaction may impose settlements or incremental compressive loads through negative skin friction.

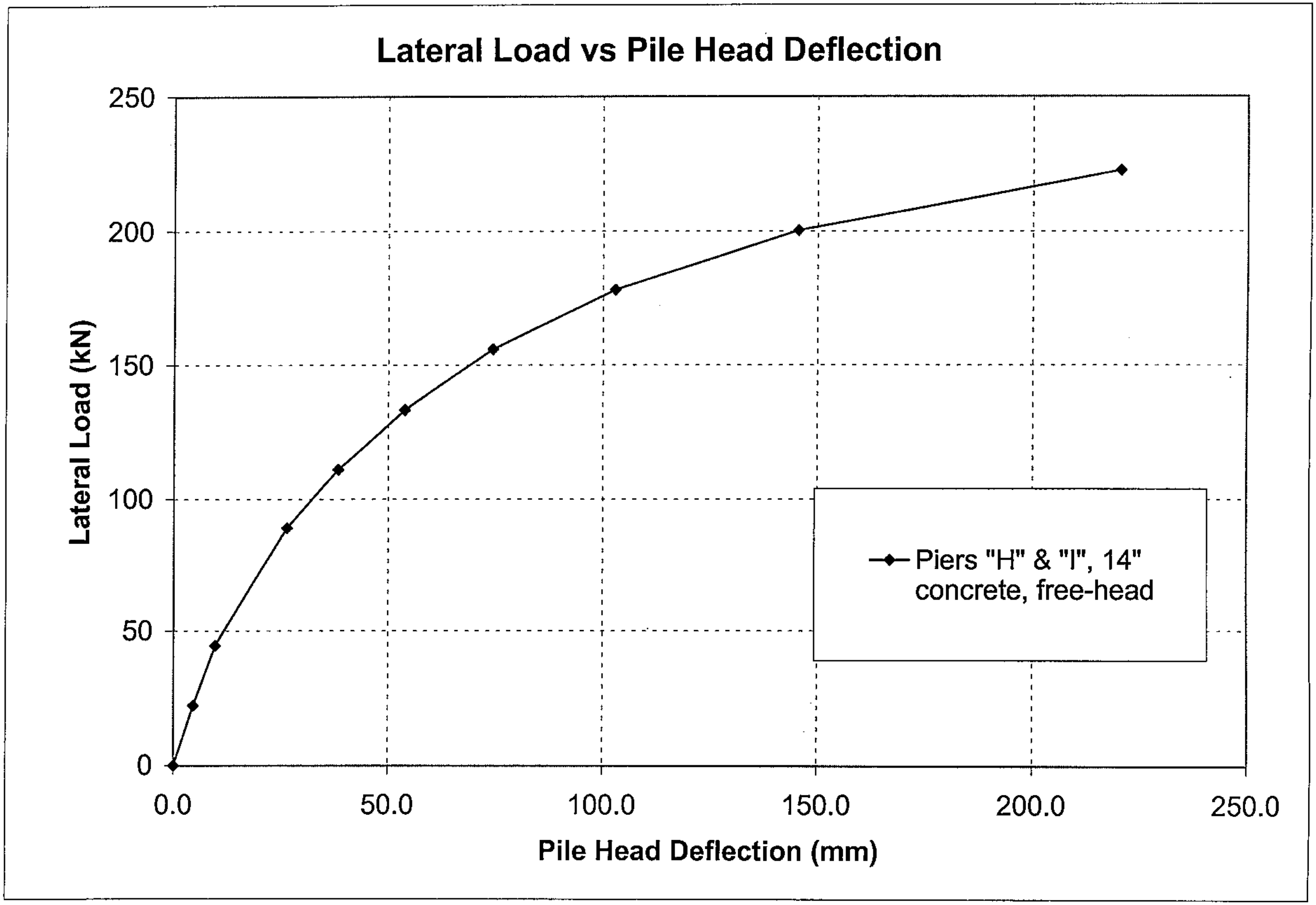

Lateral Load vs Pile Head Deflection
Lateral Load (kN)
500
450
400
350
300
250
200
150
100
50
0
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
Pile Head Deflection (mm)
North Abutment, 14" concrete, free-head

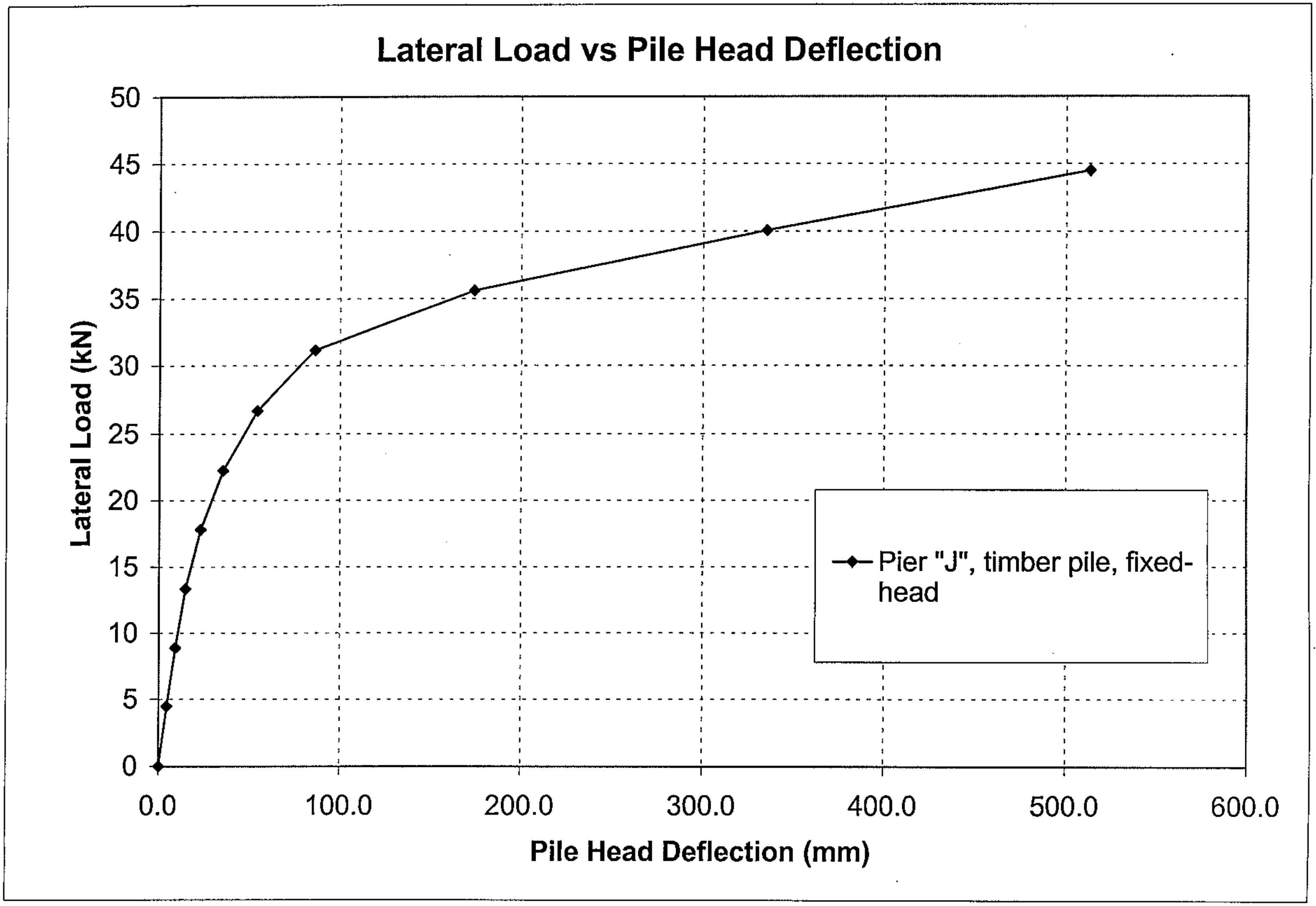

Lateral Load vs Pile Head Deflection
Lateral Load (kN)
0
50
100
150
200
250
300
350
400
450
500
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
200.0
Pile Head Deflection (mm)
Pier "A", 14" concrete, free-head


Lateral Load vs Pile Head Deflection
Lateral Load (kN)
400
350
300
250
200
150
100
50
0
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
Pile Head Deflection (mm)
Pier "B", 14" concrete, free-head

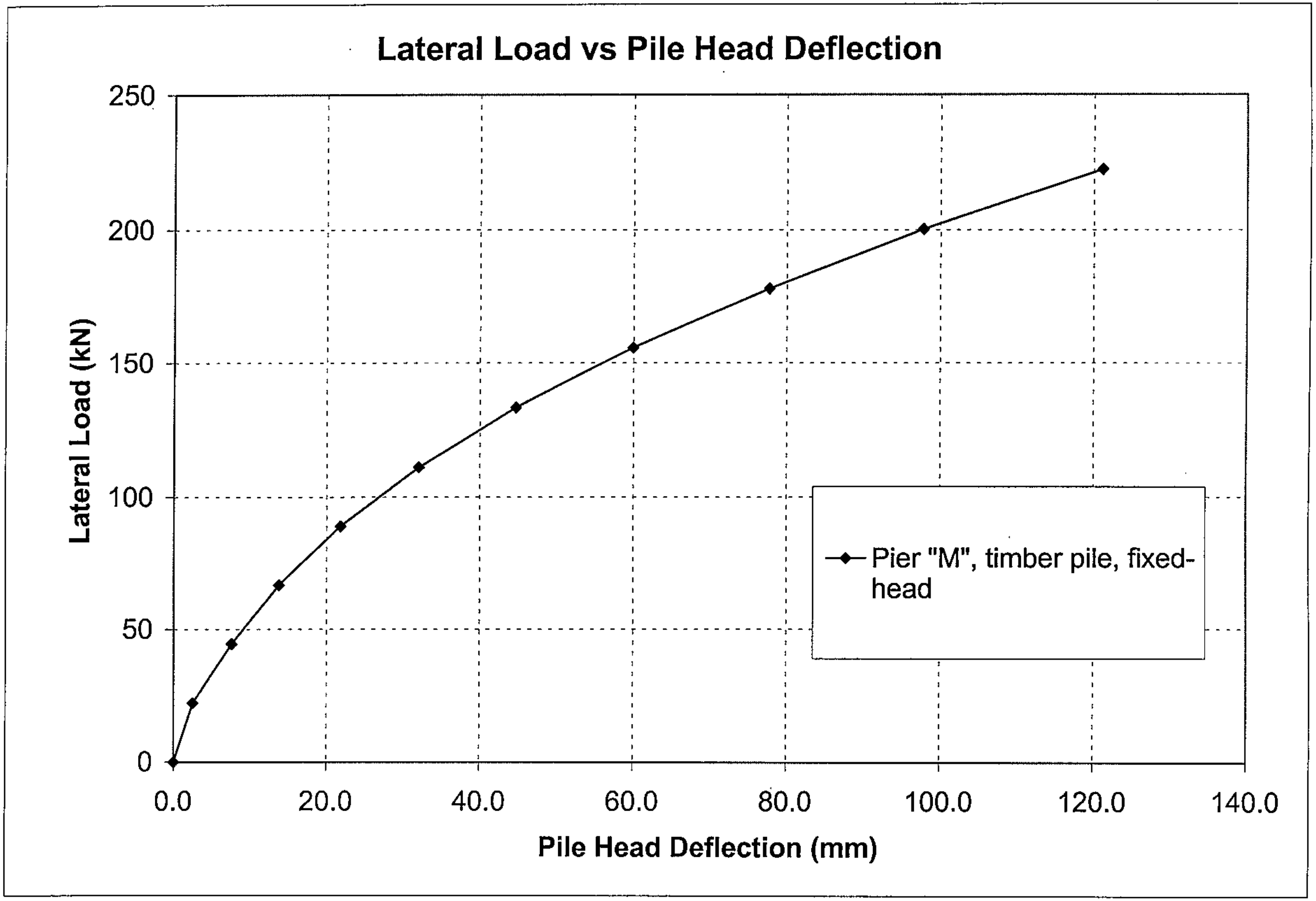

Lateral Load vs Pile Head Deflection
Lateral Load (kN)
400
350
300
250
200
150
100
50
0
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
200.0
Pile Head Deflection (mm)
Piers "C", "D" & "E", 14" concrete, free-head

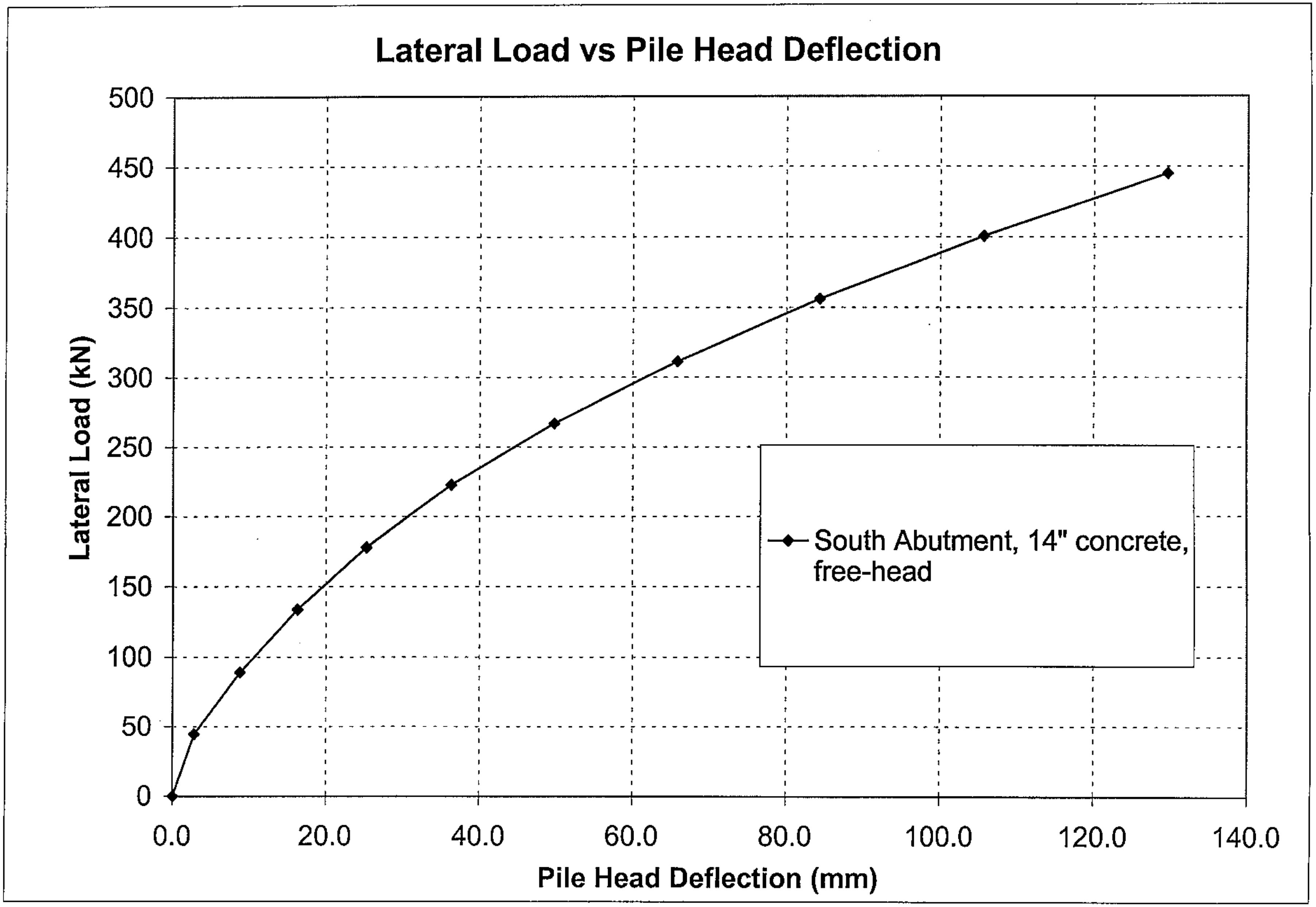

Lateral Load vs Pile Head Deflection
300
250
200
150
100
50
0
Lateral Load (kN)
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
Pile Head Deflection (mm)
Piers "F" & "G", 14" concrete, free-head


Lateral Load vs Pile Head Deflection
250
200
150
100
50
0
Lateral Load (kN)
0.0
50.0
100.0
150.0
200.0
250.0
Pile Head Deflection (mm)
Piers "H" & "I", 14" concrete, free-head


Lateral Load vs Pile Head Deflection
Lateral Load (kN)
50
45
40
35
30
25
20
15
10
5
0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
Pile Head Deflection (mm)
Pier "J", timber pile, fixed-head

Br. No. 38C-023
3/24/2000




Lateral Load vs Pile Head Deflection
Lateral Load (kN)
500
450
400
350
300
250
200
150
100
50
0
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
Pile Head Deflection (mm)
South Abutment, 14" concrete, free-head

7th Street Bridge Caltrans ARS Curves

PREPARED FOR: Dean Harris/SAC

PREPARED BY: Diana Worthen/SAC

DATE: January 7, 2013

PROJECT NUMBER: 437598

The Caltrans seismic spectra for the 7th Street Bridge were determined using the ARS Online Tool (Caltrans, 2009a). The latitude and longitude at the center of the bridge are 37.626519°N and 120.9936°W.

According to the Caltrans Seismic Design Criteria Figure B.12 (Caltrans, 2009b), the project site is classified as Site Class D. Site Class D is defined as having the top 100 feet of soil having an average SPT N-value between 15 and 50 blows per foot (bpf) and/or undrained shear strength between 1,000 and 2,000 psf.

SPT N-values in the top 100 feet of the subsurface profile ranged from 2 to 141 bpf, with an average of 29 bpf, as determined using the ASCE 7-05 $\bar{N}$ method (ASCE, 2005; Section 20.4.2).

The shear wave velocity was estimated based on the seismic site class. The average SPT N-value was compared to the range of SPT N-values and shear wave velocities for Site Class D in the Caltrans Seismic Design Criteria (Caltrans, 2009b) to find an approximate value. The shear wave velocity used in this analysis was 256 meters per second (m/s).

The Caltrans ARS Online Tool was used to develop the following response curves:

minimum deterministic spectrum,

deterministic spectrum for the nearest faults,

probabilistic spectrum for a 5% probability of exceedance in 50 years, and

an envelope curve for all these spectra.

Caltrans provides a QC/QA checklist to verify the results of the ARS Online Tool and requires completion of the checklist for Caltrans projects. As a quality measure this tool was used to check the findings for the 7th Street evaluation.

Deterministic Spectra

The ARS Online Tool determines the most significant faults based on the site latitude and longitude and the Caltrans fault database (Caltrans, 2007; Caltrans, 2009c). For this site, deterministic spectra were calculated for the two faults: Great Valley fault 7 and San Andreas fault zone (Santa Cruz Mountains section). These spectra were calculated as the arithmetic average of median response spectra calculated using the Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) ground motion prediction equations (GMPE's). Only faults active in the last 700,000 years (late Quaternary age) and capable of producing an earthquake of M_w=6.0 or greater were considered.

The ARS Online Tool also calculates a minimum deterministic spectrum to account for the potential for earthquakes occurring on previously unknown faults. This minimum spectrum is defined as the average of the median predictions of the Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) GMPE's for a scenario M_w = 6.5, vertical, strike-slip event occurring at a distance of 12 km. This spectrum is intended to represent the possibility of a wide range of magnitude-distance scenarios. Although a rupture distance of 12 km strictly meets the criteria for application of a directivity adjustment factor, the near-fault factor is not applied for this spectrum. (Caltrans, 2009b). At this site, the minimum deterministic spectrum is the envelope deterministic curve.

The QC/QA checklist was completed using the spreadsheet provided by Caltrans with velocity profile parameters and fault data determined from the ARS Online Tool and the site shear wave velocity. The spreadsheet compares the envelope spectrum of the two fault-based ARS Online spectra to a spectrum calculated by the spreadsheet using input parameters and the Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) GMPE's. Basin factors and near-fault factors were not applied. According to the figures provided (Caltrans, 2009b), the site is not located within a basin. A near-fault factor is required for rupture distances less than 25 km. Based on the ARS Online Tool, the rupture distance for the Great Valley fault 7 was 25.3 km, and the rupture distance for the San Andreas fault zone (Santa Cruz Mountains section) was 96.1 km. The Eastern California Shear Zone minimum spectrum was also not considered as, according to the figure provided (Caltrans, 2009b), the site is not located within that zone. The Caltrans comparison spreadsheet showed that the ARS Online Tool deterministic spectrum was 10% higher than the spreadsheet-based spectrum at periods higher than 2.2 seconds, with a maximum difference of 47%. Despite this, the minimum deterministic spectrum still controls for deterministic spectra.

Probabilistic Curve

The ARS Online Tool calculates the probabilistic spectrum from the 2008 USGS Seismic Hazard Map for the 5% probability of exceedance in 50 years. The raw values from the hazard map are adjusted for soil amplification based on the input shear wave velocity, using site amplification factor based on an average of those derived from the Boore-Atkinson (2008), Campbell-Bozorgnia (2008), and Chiou-Youngs (2008) GMPE's. These are the same models used to develop the hazard map.

The QC/QA checklist was completed using the spreadsheet provided by Caltrans with velocity profile parameters determined from the ARS Online Tool, the fault distance determined by the 2008 USGS Deaggregation Tool (USGS, 2012), and the site latitude, longitude, and shear wave velocity. The spreadsheet compares the probabilistic spectrum determined by the ARS Online Tool to two curves. One 2008 USGS deaggragated hazard curve is approximated using 4 points calculated by the spreadsheet based on site latitude, longitude, and shear wave velocity; rupture distance from the 2008 USGS Deaggregation Tool (USGS, 2012); and velocity profile parameters determined by the ARS Online Tool. A second deaggregated hazard curve is plotted by direct values from the 2008 USGS Deaggregation Tool (USGS, 2012). The spreadsheet-calculated deaggregated curve differs from the ARS Online Tool probabilistic spectrum by no more than 3.3%. The spectrum of values directly from the 2008 USGS deaggregation is up to 15.9% higher than the ARS Online spectrum at the short-period peak (T = 0.2 seconds), and differs by no more than 2.7% at long periods (T = 2 seconds). In the comparison, the spectrum directly from the 2008 USGS deaggregation controls.

Envelope Spectrum

The envelope spectrum based on ARS Online Tool curves is equal to the probabilistic spectrum for this site.

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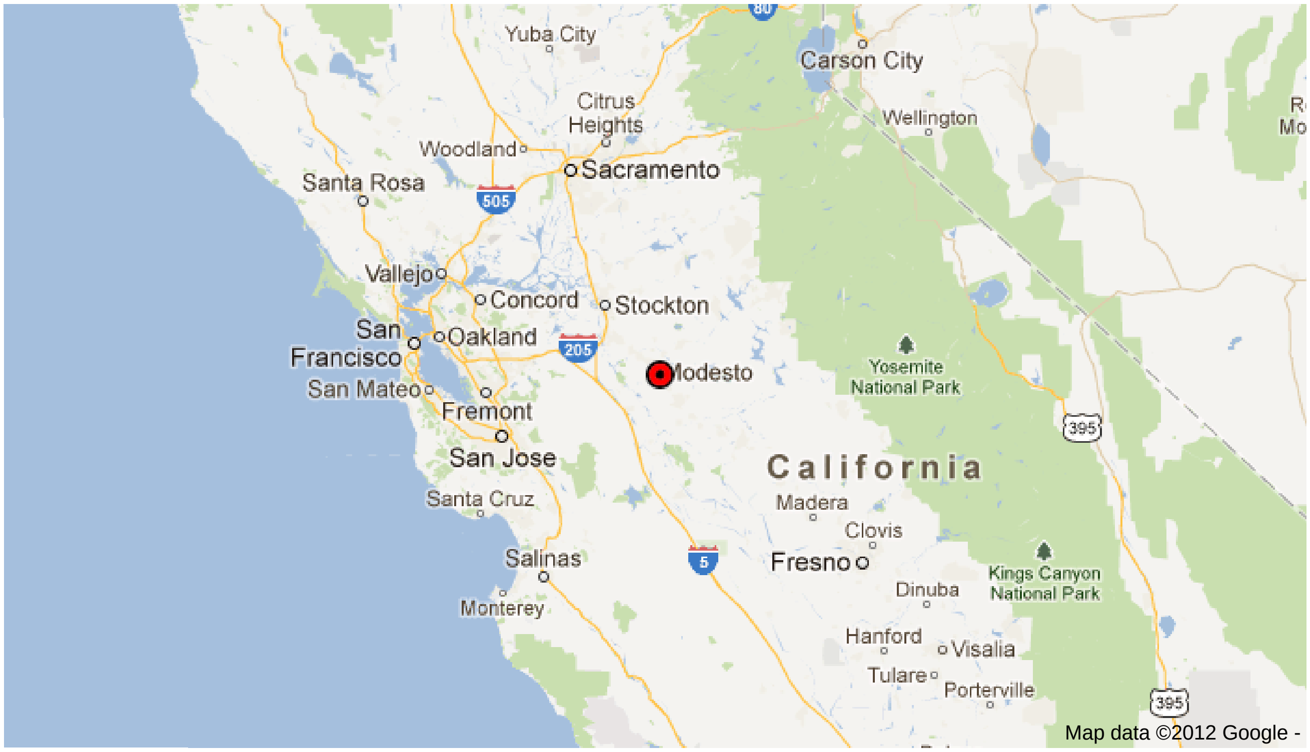
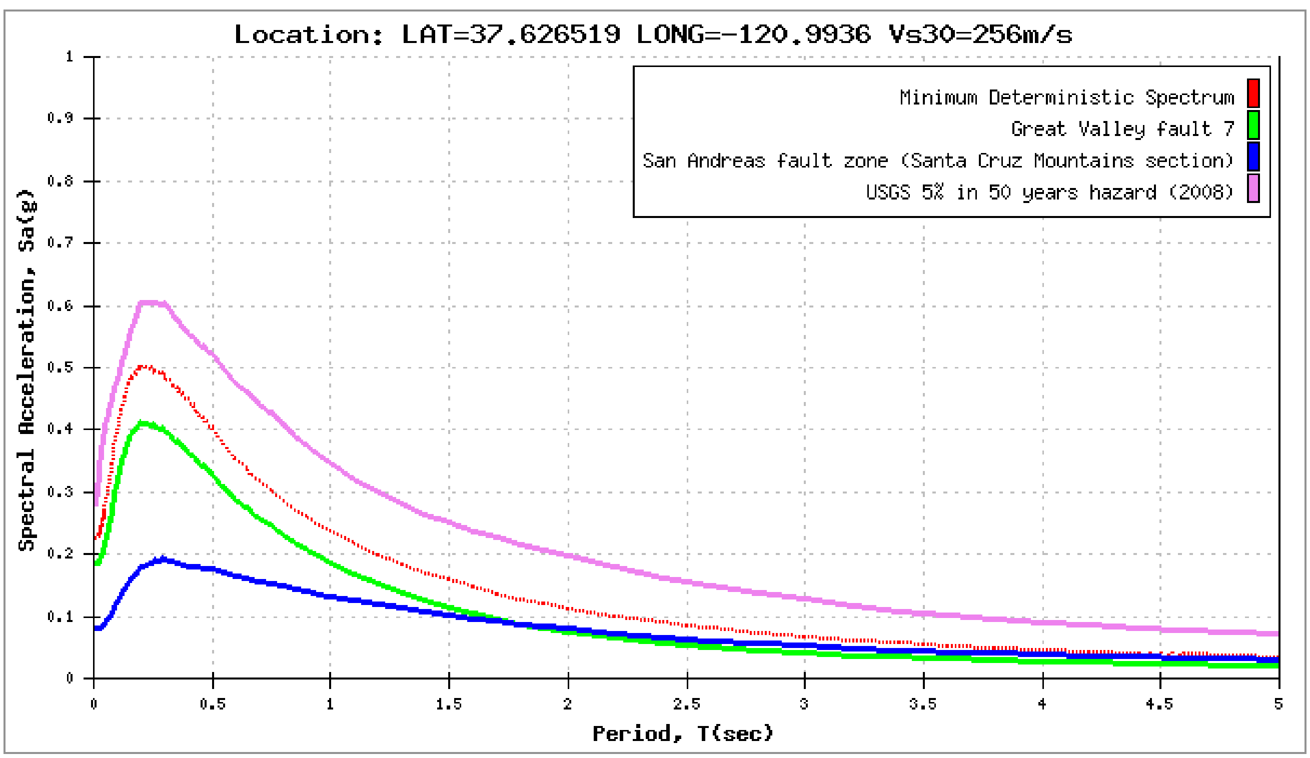
SELECT SITE LOCATION



CALCULATED SPECTRA



SITE DATA

Shear Wave Velocity, Vs_{30}:	256 m/s
Latitude:	37.626519
Longitude:	-120.993600
Depth to Vs = 1.0 km/s:	331 m
Depth to Vs = 2.5 km/s:	2.00 km

DETERMINISTIC

Great Valley fault 7

Fault ID:	25
Maximum Magnitude (MMax):	6.7
Fault Type:	R
Fault Dip:	15 Deg
Dip Direction:	W
Bottom of Rupture Plane:	10.00 km
Top of Rupture Plane(Ztor):	7.00 km
Rrup	25.25 km
Rjb:	24.26 km
Rx:	24.26 km
Fnorm:	0
Frev:	1

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Not Applied)	SA(Final Spectrum)
0.01	0.185	1.000	1.000	0.185
0.02	0.187	1.000	1.000	0.187
0.022	0.189	1.000	1.000	0.189
0.025	0.191	1.000	1.000	0.191
0.029	0.194	1.000	1.000	0.194
0.03	0.195	1.000	1.000	0.195
0.032	0.197	1.000	1.000	0.197
0.035	0.201	1.000	1.000	0.201
0.036	0.202	1.000	1.000	0.202
0.04	0.206	1.000	1.000	0.206
0.042	0.209	1.000	1.000	0.209
0.044	0.212	1.000	1.000	0.212
0.045	0.213	1.000	1.000	0.213
0.046	0.214	1.000	1.000	0.214
0.048	0.217	1.000	1.000	0.217
0.05	0.220	1.000	1.000	0.220
0.055	0.229	1.000	1.000	0.229
0.06	0.239	1.000	1.000	0.239
0.065	0.248	1.000	1.000	0.248
0.067	0.252	1.000	1.000	0.252

0.07	0.258	1.000	1.000	0.258
0.075	0.267	1.000	1.000	0.267
0.08	0.277	1.000	1.000	0.277
0.085	0.288	1.000	1.000	0.288
0.09	0.298	1.000	1.000	0.298
0.095	0.308	1.000	1.000	0.308
0.1	0.317	1.000	1.000	0.317
0.11	0.334	1.000	1.000	0.334
0.12	0.349	1.000	1.000	0.349
0.13	0.363	1.000	1.000	0.363
0.133	0.367	1.000	1.000	0.367
0.14	0.376	1.000	1.000	0.376
0.15	0.387	1.000	1.000	0.387
0.16	0.393	1.000	1.000	0.393
0.17	0.398	1.000	1.000	0.398
0.18	0.402	1.000	1.000	0.402
0.19	0.406	1.000	1.000	0.406
0.2	0.409	1.000	1.000	0.409
0.22	0.409	1.000	1.000	0.409
0.24	0.408	1.000	1.000	0.408
0.25	0.407	1.000	1.000	0.407
0.26	0.405	1.000	1.000	0.405
0.28	0.402	1.000	1.000	0.402
0.29	0.400	1.000	1.000	0.400
0.3	0.399	1.000	1.000	0.399
0.32	0.392	1.000	1.000	0.392
0.34	0.385	1.000	1.000	0.385
0.35	0.381	1.000	1.000	0.381
0.36	0.378	1.000	1.000	0.378
0.38	0.370	1.000	1.000	0.370
0.4	0.363	1.000	1.000	0.363
0.42	0.356	1.000	1.000	0.356
0.44	0.348	1.000	1.000	0.348
0.45	0.344	1.000	1.000	0.344
0.46	0.341	1.000	1.000	0.341
0.48	0.334	1.000	1.000	0.334
0.5	0.327	1.000	1.000	0.327
0.55	0.306	1.000	1.000	0.306
0.6	0.288	1.000	1.000	0.288
0.65	0.272	1.000	1.000	0.272
0.667	0.267	1.000	1.000	0.267
0.7	0.257	1.000	1.000	0.257
0.75	0.244	1.000	1.000	0.244
0.8	0.231	1.000	1.000	0.231
0.85	0.218	1.000	1.000	0.218
0.9	0.207	1.000	1.000	0.207
0.95	0.197	1.000	1.000	0.197
1	0.187	1.000	1.000	0.187
1.1	0.168	1.000	1.000	0.168
1.2	0.152	1.000	1.000	0.152

1.3	0.137	1.000	1.000	0.137
1.4	0.125	1.000	1.000	0.125
1.5	0.114	1.000	1.000	0.114
1.6	0.104	1.000	1.000	0.104
1.7	0.095	1.000	1.000	0.095
1.8	0.088	1.000	1.000	0.088
1.9	0.081	1.000	1.000	0.081
2	0.075	1.000	1.000	0.075
2.2	0.065	1.000	1.000	0.065
2.4	0.057	1.000	1.000	0.057
2.5	0.054	1.000	1.000	0.054
2.6	0.051	1.000	1.000	0.051
2.8	0.045	1.000	1.000	0.045
3	0.041	1.000	1.000	0.041
3.2	0.037	1.000	1.000	0.037
3.4	0.034	1.000	1.000	0.034
3.5	0.033	1.000	1.000	0.033
3.6	0.032	1.000	1.000	0.032
3.8	0.030	1.000	1.000	0.030
4	0.028	1.000	1.000	0.028
4.2	0.026	1.000	1.000	0.026
4.4	0.024	1.000	1.000	0.024
4.6	0.023	1.000	1.000	0.023
4.8	0.022	1.000	1.000	0.022
5	0.021	1.000	1.000	0.021

San Andreas fault zone (Santa Cruz Mountains section)

Fault ID:	310
Maximum Magnitude (MMax):	7.9
Fault Type:	RLSS
Fault Dip:	90 Deg
Dip Direction:	V
Bottom of Rupture Plane:	15.00 km
Top of Rupture Plane(Ztor):	0.00 km
Rrup	96.09 km
Rjb:	96.09 km
Rx:	96.09 km
Fnorm:	0
Frev:	0

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Not Applied)	SA(Final Spectrum)
0.01	0.079	1.000	1.000	0.079
0.02	0.080	1.000	1.000	0.080
0.022	0.080	1.000	1.000	0.080
0.025	0.081	1.000	1.000	0.081
0.029	0.082	1.000	1.000	0.082
0.03	0.082	1.000	1.000	0.082
0.032	0.082	1.000	1.000	0.082

0.035	0.084	1.000	1.000	0.084
0.036	0.084	1.000	1.000	0.084
0.04	0.085	1.000	1.000	0.085
0.042	0.086	1.000	1.000	0.086
0.044	0.087	1.000	1.000	0.087
0.045	0.088	1.000	1.000	0.088
0.046	0.088	1.000	1.000	0.088
0.048	0.089	1.000	1.000	0.089
0.05	0.090	1.000	1.000	0.090
0.055	0.093	1.000	1.000	0.093
0.06	0.096	1.000	1.000	0.096
0.065	0.099	1.000	1.000	0.099
0.067	0.100	1.000	1.000	0.100
0.07	0.102	1.000	1.000	0.102
0.075	0.105	1.000	1.000	0.105
0.08	0.108	1.000	1.000	0.108
0.085	0.112	1.000	1.000	0.112
0.09	0.115	1.000	1.000	0.115
0.095	0.119	1.000	1.000	0.119
0.1	0.122	1.000	1.000	0.122
0.11	0.130	1.000	1.000	0.130
0.12	0.137	1.000	1.000	0.137
0.13	0.143	1.000	1.000	0.143
0.133	0.145	1.000	1.000	0.145
0.14	0.149	1.000	1.000	0.149
0.15	0.155	1.000	1.000	0.155
0.16	0.160	1.000	1.000	0.160
0.17	0.165	1.000	1.000	0.165
0.18	0.169	1.000	1.000	0.169
0.19	0.173	1.000	1.000	0.173
0.2	0.177	1.000	1.000	0.177
0.22	0.181	1.000	1.000	0.181
0.24	0.185	1.000	1.000	0.185
0.25	0.187	1.000	1.000	0.187
0.26	0.188	1.000	1.000	0.188
0.28	0.190	1.000	1.000	0.190
0.29	0.190	1.000	1.000	0.190
0.3	0.191	1.000	1.000	0.191
0.32	0.189	1.000	1.000	0.189
0.34	0.187	1.000	1.000	0.187
0.35	0.186	1.000	1.000	0.186
0.36	0.185	1.000	1.000	0.185
0.38	0.183	1.000	1.000	0.183
0.4	0.181	1.000	1.000	0.181
0.42	0.180	1.000	1.000	0.180
0.44	0.179	1.000	1.000	0.179
0.45	0.178	1.000	1.000	0.178
0.46	0.178	1.000	1.000	0.178
0.48	0.177	1.000	1.000	0.177
0.5	0.176	1.000	1.000	0.176

0.55	0.170	1.000	1.000	0.170
0.6	0.165	1.000	1.000	0.165
0.65	0.161	1.000	1.000	0.161
0.667	0.159	1.000	1.000	0.159
0.7	0.157	1.000	1.000	0.157
0.75	0.153	1.000	1.000	0.153
0.8	0.148	1.000	1.000	0.148
0.85	0.144	1.000	1.000	0.144
0.9	0.140	1.000	1.000	0.140
0.95	0.136	1.000	1.000	0.136
1	0.132	1.000	1.000	0.132
1.1	0.125	1.000	1.000	0.125
1.2	0.119	1.000	1.000	0.119
1.3	0.113	1.000	1.000	0.113
1.4	0.108	1.000	1.000	0.108
1.5	0.103	1.000	1.000	0.103
1.6	0.097	1.000	1.000	0.097
1.7	0.092	1.000	1.000	0.092
1.8	0.088	1.000	1.000	0.088
1.9	0.084	1.000	1.000	0.084
2	0.080	1.000	1.000	0.080
2.2	0.073	1.000	1.000	0.073
2.4	0.067	1.000	1.000	0.067
2.5	0.064	1.000	1.000	0.064
2.6	0.061	1.000	1.000	0.061
2.8	0.057	1.000	1.000	0.057
3	0.053	1.000	1.000	0.053
3.2	0.049	1.000	1.000	0.049
3.4	0.046	1.000	1.000	0.046
3.5	0.045	1.000	1.000	0.045
3.6	0.043	1.000	1.000	0.043
3.8	0.041	1.000	1.000	0.041
4	0.039	1.000	1.000	0.039
4.2	0.037	1.000	1.000	0.037
4.4	0.035	1.000	1.000	0.035
4.6	0.033	1.000	1.000	0.033
4.8	0.032	1.000	1.000	0.032
5	0.030	1.000	1.000	0.030

PROBABILISTIC

Probabilistic Model
USGS Seismic Hazard Map(2008) 975 Year Return Period

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Not Applied)	SA(Final Spectrum)
0.01	0.279	1.000	1.000	0.279
0.02	0.328	1.000	1.000	0.328
0.022	0.336	1.000	1.000	0.336

0.025	0.346	1.000	1.000	0.346
0.029	0.358	1.000	1.000	0.358
0.03	0.361	1.000	1.000	0.361
0.032	0.366	1.000	1.000	0.366
0.035	0.374	1.000	1.000	0.374
0.036	0.377	1.000	1.000	0.377
0.04	0.386	1.000	1.000	0.386
0.042	0.390	1.000	1.000	0.390
0.044	0.395	1.000	1.000	0.395
0.045	0.397	1.000	1.000	0.397
0.046	0.399	1.000	1.000	0.399
0.048	0.403	1.000	1.000	0.403
0.05	0.407	1.000	1.000	0.407
0.055	0.416	1.000	1.000	0.416
0.06	0.425	1.000	1.000	0.425
0.065	0.433	1.000	1.000	0.433
0.067	0.436	1.000	1.000	0.436
0.07	0.440	1.000	1.000	0.440
0.075	0.447	1.000	1.000	0.447
0.08	0.454	1.000	1.000	0.454
0.085	0.461	1.000	1.000	0.461
0.09	0.467	1.000	1.000	0.467
0.095	0.473	1.000	1.000	0.473
0.1	0.479	1.000	1.000	0.479
0.11	0.494	1.000	1.000	0.494
0.12	0.509	1.000	1.000	0.509
0.13	0.523	1.000	1.000	0.523
0.133	0.527	1.000	1.000	0.527
0.14	0.536	1.000	1.000	0.536
0.15	0.548	1.000	1.000	0.548
0.16	0.560	1.000	1.000	0.560
0.17	0.572	1.000	1.000	0.572
0.18	0.583	1.000	1.000	0.583
0.19	0.593	1.000	1.000	0.593
0.2	0.604	1.000	1.000	0.604
0.22	0.604	1.000	1.000	0.604
0.24	0.603	1.000	1.000	0.603
0.25	0.603	1.000	1.000	0.603
0.26	0.603	1.000	1.000	0.603
0.28	0.603	1.000	1.000	0.603
0.29	0.603	1.000	1.000	0.603
0.3	0.603	1.000	1.000	0.603
0.32	0.592	1.000	1.000	0.592
0.34	0.582	1.000	1.000	0.582
0.35	0.577	1.000	1.000	0.577
0.36	0.572	1.000	1.000	0.572
0.38	0.563	1.000	1.000	0.563
0.4	0.555	1.000	1.000	0.555
0.42	0.547	1.000	1.000	0.547
0.44	0.540	1.000	1.000	0.540

0.45	0.536	1.000	1.000	0.536
0.46	0.533	1.000	1.000	0.533
0.48	0.526	1.000	1.000	0.526
0.5	0.520	1.000	1.000	0.520
0.55	0.496	1.000	1.000	0.496
0.6	0.475	1.000	1.000	0.475
0.65	0.457	1.000	1.000	0.457
0.667	0.451	1.000	1.000	0.451
0.7	0.440	1.000	1.000	0.440
0.75	0.425	1.000	1.000	0.425
0.8	0.406	1.000	1.000	0.406
0.85	0.389	1.000	1.000	0.389
0.9	0.374	1.000	1.000	0.374
0.95	0.360	1.000	1.000	0.360
1	0.347	1.000	1.000	0.347
1.1	0.321	1.000	1.000	0.321
1.2	0.299	1.000	1.000	0.299
1.3	0.281	1.000	1.000	0.281
1.4	0.265	1.000	1.000	0.265
1.5	0.250	1.000	1.000	0.250
1.6	0.238	1.000	1.000	0.238
1.7	0.226	1.000	1.000	0.226
1.8	0.216	1.000	1.000	0.216
1.9	0.207	1.000	1.000	0.207
2	0.199	1.000	1.000	0.199
2.2	0.179	1.000	1.000	0.179
2.4	0.163	1.000	1.000	0.163
2.5	0.156	1.000	1.000	0.156
2.6	0.149	1.000	1.000	0.149
2.8	0.137	1.000	1.000	0.137
3	0.128	1.000	1.000	0.128
3.2	0.118	1.000	1.000	0.118
3.4	0.109	1.000	1.000	0.109
3.5	0.105	1.000	1.000	0.105
3.6	0.102	1.000	1.000	0.102
3.8	0.095	1.000	1.000	0.095
4	0.089	1.000	1.000	0.089
4.2	0.085	1.000	1.000	0.085
4.4	0.082	1.000	1.000	0.082
4.6	0.079	1.000	1.000	0.079
4.8	0.076	1.000	1.000	0.076
5	0.073	1.000	1.000	0.073

MINIMUM DETERMINISTIC SPECTRUM

Period	**SA**
0.01	0.226

0.02	0.228
0.022	0.231
0.025	0.234
0.029	0.238
0.03	0.239
0.032	0.242
0.035	0.247
0.036	0.249
0.04	0.255
0.042	0.258
0.044	0.262
0.045	0.263
0.046	0.265
0.048	0.269
0.05	0.272
0.055	0.284
0.06	0.296
0.065	0.309
0.067	0.314
0.07	0.321
0.075	0.333
0.08	0.346
0.085	0.358
0.09	0.370
0.095	0.382
0.1	0.394
0.11	0.414
0.12	0.431
0.13	0.448
0.133	0.452
0.14	0.462
0.15	0.474
0.16	0.481
0.17	0.487
0.18	0.492
0.19	0.496
0.2	0.500
0.22	0.499
0.24	0.497
0.25	0.495
0.26	0.493
0.28	0.489
0.29	0.487
0.3	0.484
0.32	0.477
0.34	0.470
0.35	0.466
0.36	0.462
0.38	0.454
0.4	0.447

0.42	0.437
0.44	0.428
0.45	0.424
0.46	0.419
0.48	0.411
0.5	0.403
0.55	0.376
0.6	0.354
0.65	0.334
0.667	0.328
0.7	0.316
0.75	0.300
0.8	0.285
0.85	0.271
0.9	0.258
0.95	0.246
1	0.235
1.1	0.215
1.2	0.198
1.3	0.182
1.4	0.169
1.5	0.157
1.6	0.146
1.7	0.136
1.8	0.127
1.9	0.119
2	0.112
2.2	0.099
2.4	0.089
2.5	0.084
2.6	0.080
2.8	0.073
3	0.067
3.2	0.061
3.4	0.056
3.5	0.054
3.6	0.052
3.8	0.049
4	0.045
4.2	0.043
4.4	0.040
4.6	0.038
4.8	0.036
5	0.034

Envelope Data

Period	SA
0.01	0.279
0.02	0.328

0.022	0.336
0.025	0.346
0.029	0.358
0.03	0.361
0.032	0.366
0.035	0.374
0.036	0.377
0.04	0.386
0.042	0.390
0.044	0.395
0.045	0.397
0.046	0.399
0.048	0.403
0.05	0.407
0.055	0.416
0.06	0.425
0.065	0.433
0.067	0.436
0.07	0.440
0.075	0.447
0.08	0.454
0.085	0.461
0.09	0.467
0.095	0.473
0.1	0.479
0.11	0.494
0.12	0.509
0.13	0.523
0.133	0.527
0.14	0.536
0.15	0.548
0.16	0.560
0.17	0.572
0.18	0.583
0.19	0.593
0.2	0.604
0.22	0.604
0.24	0.603
0.25	0.603
0.26	0.603
0.28	0.603
0.29	0.603
0.3	0.603
0.32	0.592
0.34	0.582
0.35	0.577
0.36	0.572
0.38	0.563
0.4	0.555
0.42	0.547

0.44	0.540
0.45	0.536
0.46	0.533
0.48	0.526
0.5	0.520
0.55	0.496
0.6	0.475
0.65	0.457
0.667	0.451
0.7	0.440
0.75	0.425
0.8	0.406
0.85	0.389
0.9	0.374
0.95	0.360
1	0.347
1.1	0.321
1.2	0.299
1.3	0.281
1.4	0.265
1.5	0.250
1.6	0.238
1.7	0.226
1.8	0.216
1.9	0.207
2	0.199
2.2	0.179
2.4	0.163
2.5	0.156
2.6	0.149
2.8	0.137
3	0.128
3.2	0.118
3.4	0.109
3.5	0.105
3.6	0.102
3.8	0.095
4	0.089
4.2	0.085
4.4	0.082
4.6	0.079
4.8	0.076
5	0.073

Appendix G – Caltrans Inspection Report

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Bridge Number : 38C0023
Facility Carried: SEVENTH STREET
Location : 10-STA-214-0-MOD
City : MODESTO
Inspection Date : 10/13/2011

Bridge Inspection Report

Inspection Type

Routine	FC	Underwater	Special	Other
X				

STRUCTURE NAME: TUOLUMNE RIVER

CONSTRUCTION INFORMATION

Year Built : 1916
Year Widened: N/A
Length (m) : 356.6

Skew (degrees): 0
No. of Joints : 6
No. of Hinges : 6

Structure Description: A series of "Canticrete" type trusses supported on RC piers and abutments all founded on concrete or timber piling.
A "Canticrete" truss frame consists of a longitudinal continuous truss span and cantilevered arms (half span), connected transversely with truss configured floor beams; longitudinal and transverse members are then encased in concrete.

Span Configuration : 1 @ 16.5 m, 1 @ 30.8 m, 2 @ 30.5 m, 9 @ 25.6 m, and 1 @ 16.5 m

LOAD CAPACITY AND RATINGS

Design Live Load: UNKNOWN
Inventory Rating: 6.5 metric tonnes   Calculation Method: ALLOWABLE STRESS
Operating Rating: 11 metric tonnes   Calculation Method: ALLOWABLE STRESS
Permit Rating : XXXXX
Posting Load : Type 3: 4 U.S. Tons   Type 3S2: 4 U.S. Tons   Type 3-3: 4 U.S. Tons

DESCRIPTION ON STRUCTURE

Deck X-Section: 0.7 m r, 1.1 m sw, 7.4 m, 1.1 m sw, 0.7 m r

Total Width: 10.9 m   Net Width: 7.4 m   No. of Lanes: 2
Rail Description: Concreted Truss Panel   Rail Code : 0000
Min. Vertical Clearance: Unimpaired

DESCRIPTION UNDER STRUCTURE

Facility Name	Func Class	Lanes	Horiz Clr (m)	Vert Clr (m)
River Road	19	2	16.00	3.40

Channel Description: Unlined - sand and silt

INSPECTION COMMENTARY

INSPECTION ACCESS:

At the time of this investigation, there was approximately 10 FT (3.0 m) deep of water in the main channel under Span 3. The rest of the spans were dry. Pier 3 and half of Pier 4 in the southern edge was submerged in the channel. A complete investigation of the soffit and the remaining substructure was performed.

CONDITION OF STRUCTURE:

At Abutment 1, the AC along the joint has a pothole of 15 FT long x 8 IN wide x 3 IN deep (4.5 m x 0.2 m x 75 mm) near the centerline and the northbound lane. See attached photo #1.

INSPECTION COMMENTARY

At Abutment 15, the previous AC patches had broke up at the wheel lines. The joint along the whole bridge width has potholes of 3 FT long x 12 IN wide x 4 IN deep (0.9 m x 0.3 m x 0.1 m). There is also up to 3 IN (75 mm) of settlement of the approach AC and uneven surface on both the north and southbound lanes. See attached photos #2 and 3.

There are various length concrete spalls with exposed reinforcement on the outer bottom rail above most spans on both sides. Some also on the top rail and inner face near the hinge lines. See attached photos #4 through 6.

At the inner vertical face of both concrete sidewalks, there are 0.08 IN (2 mm) wide horizontal cracks and spalls due to the joint compression and deck deflection near the expansion joints. There are also spalls with some exposed reinforcement on the concrete sidewalk along these hinge lines. See attached photos #7 and 8.

In Span 1, there is a 12 IN (0.3 m) wide concrete edge spall with no exposed reinforcement on the downward projection of the east arch near Abutment 1, which resulted from a previous high load hit. See attached photo #9.

The deck soffit edges have up to 3 FT (1.0 m) long spalls with longitudinal reinforcements being exposed in most of the spans. Areas of soffit efflorescence were also observed under most spans. See attached photo #10.

The bottom concrete of many floor beams has spalled off with exposed steel sections at several locations in most spans, with the most severe in Span 13, which had an inverted V steel beam support at bottom of midspan. See attached photos #11 and 12.

At Piers 3 and 4, the bottom concrete portions of the pier curtain walls have spalled with some exposed reinforcements, measuring up to 5 FT (1.5 m) high from bottom at center, between the footings of the pier wall. This condition was not visible for inspection during this time. See attached photo #13.

There are vertical and pattern cracks with areas of rock pocket and exposed reinforcements on the bottom 5 FT (1.5 m) of the curtain wall in the middle of Pier 4.

There is a concrete spall of 4.6 FT wide x 12 IN high (1.4 m x 0.3 m) with exposed reinforcements at the center bottom of Abutment 15. See attached photo #14.

There is heavy graffiti observed on most pier walls and abutments.

The condition of the area at the open expansion joints, at approximately 70 degrees Fahrenheit, are as follows:

Span 3: No concrete spall at the header. Moderate spalls on the sidewalk, top of railing, and interior railing. The vertical offset at top of right rail was 0.25 IN (6 mm) and 1.0 IN (25 mm) on the left rail. There was moderate vertical deck deflection observed under passing traffic. See attached photo #15.

Span 5: The previous 4.0 FT long x 8 IN wide (1.2 m x 0.2 m) edge spall on the south concrete header of the northbound lane with exposed reinforcements had AC patch, 12 IN long has broken up adjacent to the sidewalk. There was 2 IN (50 mm) vertical railing offset on both sides. The deck along this joint also has 1 IN (25 mm) vertical offset. See attached photo #16.

Span 7: There is a 12 IN long x 4 IN wide x 3 IN deep edge spall on the south header of the southbound lane. The vertical offset at top of the rail was 1.2 IN (30 mm) at right and 0.4 IN (10 mm) at left. Moderate deck deflection observed. See attached photo #17.

INSPECTION COMMENTARY

Span 9: There are up to 3.0 FT long x 7 IN wide x 3 IN deep (1.0 m x 0.18 m x 75 mm) spalls on the south header with exposed reinforcement of both the north and southbound lanes near the centerline. The vertical offset at top of both rails is not significant at this time. Moderate deck deflection was observed. See attached photo #18.

Span 11: 3.0 FT long x 4 IN wide x 3 IN deep(1.0 m x 0.1 m x 75 mm) edge spall at north header with exposed reinforcement on the northbound lane and 16 IN long x 3 IN wide x 3 IN deep (0.4 m x 75 mm x 75 mm) edge spall on the south header on the southbound lane with no exposed reinforcement. No significant vertical offset on both rails. There is a 3 FT long x 4 IN wide spall on the right sidewalk. See attached photo #19.

Span 13: 20 IN (0.5 m) long shallow header spall with no exposed reinforcement on the southbound lane. There was 3.5 IN (89 mm) railing offset on both sides. There is a 6 IN (0.15 m) wide through hole on the left sidewalk (southbound). See attached photo #20.

SCOUR:

The exposed footings around Piers 3 and 4 was not measured during this investigation due to the high water depth in Span 3. No visible scour condition was observed at this time.

SIGNS:

In Span 1, there are current vertical clearance signs of 13 FT-10 IN on the center right side and 11 FT-02 IN on the downward curve of left side near Abutment 1.

Appropriate load limit signs "4 TONS GROSS LOAD" are in place.

EXISTING POSTING:

This bridge was posted by Order of Director dated September 6, 1979 for the following load restrictions:

Weight Limit: 4 Tons Gross Load

LOAD CAPACITY:

The stress analysis conducted on May 4th, 1994, indicates that controlling members, the floor beams and deck slab, are capable of sustaining larger loads, namely 11 tons for Type 3, 18 tons for Type 3S2, and 22 tons for Type 3-3 vehicles, than those limited by the current existing posting.

However at these higher loads, the cantilevered half spans present excessive vertical deflection at the expansion joints resulting in pavement and rail damage.

The decision to post the bridge for 4 TONS GROSS LOAD was made based on the bridge report dated on June 1, 1979 according to the following two reasons:

First, the speed posting can not be considered as a basis for increasing the weight limit in areas where enforcement will be difficult and frequent violation can be anticipated. Therefore, 10 TON GROSS LOAD with 10-MILES SPEED LIMIT has been reduced to 4 TONS GROSS LOAD without speed regulation.

Second, the City and County would like to reduce maintenance costs which are mainly being generated by the larger vertical deflection at the end of half span cantilever and

INSPECTION COMMENTARY

hopefully extend the useful life of the bridge.

The current load capacity is applicable only as long as this structure remains in essentially the same condition as it was during this investigation.

RECOMMENDED POSTING:

Retain existing posting

ELEMENT INSPECTION RATINGS

Elem No.	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4	St. 5
					Qty in each Condition State				
13	Concrete Deck - Unprotected w/ AC Overlay	2	2610	sq.m.	2610	0	0	0	0
144	Reinforced Conc Arch	2	726	m.	725	1	0	0	0
155	Reinforced Conc Floor Beam	2	771	m.	0	0	771	0	0
210	Reinforced Conc Pier Wall	2	139	m.	117	0	22	0	0
215	Reinforced Conc Abutment	2	22	m.	21	0	1	0	0
227	Reinforced Conc Submerged Pile	2	1	ea.	1	0	0	0	0
304	Open Expansion Joint	2	66	m.	33	33	0	0	0
331	Reinforced Conc Bridge Railing	2	726	m.	0	0	726	0	0
359	Soffit of Concrete Deck or Slab	2	1	ea.	0	0	1	0	0
361	Scour	2	1	ea.	1	0	0	0	0

WORK RECOMMENDATIONS

RecDate: 10/30/2001
Action : Sub-Patch spalls
Work By: LOCAL AGENCY
Status : PROPOSED
EstCost:
StrTarget: 2 YEARS
DistTarget:
EA:
Repair the concrete spall at the center bottom of Abutment 15.

RecDate: 08/31/1999
Action : Super-Patch spalls
Work By: LOCAL AGENCY
Status : PROPOSED
EstCost:
StrTarget: 2 YEARS
DistTarget:
EA:
Clean the exposed steel sections and repair the spalled area in the floor beams and deck soffit with new concrete.

RecDate: 08/31/1999
Action : Appr. Roadway-Repair
Work By: LOCAL AGENCY
Status : PROPOSED
EstCost:
StrTarget: 2 YEARS
DistTarget:
EA:
Repair the cracks, AC potholes, and settlement on the approach roadway to both abutments.

RecDate: 01/04/1996
Action : Deck-Patch spalls
Work By: LOCAL AGENCY
Status : PROPOSED
EstCost:
StrTarget: 2 YEARS
DistTarget:
EA:
Repair the concrete header spalls at the open expansion joint in Spans 5, 7, 9, 11, and 13.

Inspected By : RH.Le/AG.Groess

John Andrew Gillis (Registered Civil Engineer)

REGISTERED PROFESSIONAL ENGINEER
JOHN ANDREW GILLIS
No. C 47311
Exp. 12/31/2013
CIVIL
STATE OF CALIFORNIA

STRUCTURE INVENTORY AND APPRAISAL REPORT

**************** IDENTIFICATION ***************

(1)	STATE NAME- CALIFORNIA	069
(8)	STRUCTURE NUMBER	38C0023
(5)	INVENTORY ROUTE(ON/UNDER)- ON	1500F2140
(2)	HIGHWAY AGENCY DISTRICT	10
(3)	COUNTY CODE 099 (4) PLACE CODE	48354
(6)	FEATURE INTERSECTED-	TUOLUMNE RIVER
(7)	FACILITY CARRIED-	SEVENTH STREET
(9)	LOCATION-	10-STA-214-0-MOD
(11)	MILEPOINT/KILOMETERPOINT	0
(12)	BASE HIGHWAY NETWORK- NOT ON NET	0
(13)	LRS INVENTORY ROUTE & SUBROUTE	
(16)	LATITUDE	37 DEG 37 MIN 37 SEC
(17)	LONGITUDE	120 DEG 59 MIN 38 SEC
(98)	BORDER BRIDGE STATE CODE % SHARE	%
(99)	BORDER BRIDGE STRUCTURE NUMBER	

******** STRUCTURE TYPE AND MATERIAL *********

(43)	STRUCTURE TYPE MAIN:MATERIAL-	CONCRETE CONT
	TYPE- TRUSS - DECK	CODE 209
(44)	STRUCTURE TYPE APPR:MATERIAL-	OTHER/NA
	TYPE- OTHER/NA	CODE 000
(45)	NUMBER OF SPANS IN MAIN UNIT	14
(46)	NUMBER OF APPROACH SPANS	0
(107)	DECK STRUCTURE TYPE- CIP CONCRETE	CODE 1
(108)	WEARING SURFACE / PROTECTIVE SYSTEM:	
A)	TYPE OF WEARING SURFACE- BITUMINOUS	CODE 6
B)	TYPE OF MEMBRANE- NONE	CODE 0
C)	TYPE OF DECK PROTECTION- NONE	CODE 0

*************** AGE AND SERVICE ***************

(27)	YEAR BUILT	1916
(106)	YEAR RECONSTRUCTED	0000
(42)	TYPE OF SERVICE: ON- HIGHWAY-PEDESTRIAN	5
	UNDER- HIGHWAY-WATERWAY	6
(28)	LANES:ON STRUCTURE 02 UNDER STRUCTURE	02
(29)	AVERAGE DAILY TRAFFIC	15719
(30)	YEAR OF ADT 2001 (109) TRUCK ADT	2 %
(19)	BYPASS, DETOUR LENGTH	2 KM

*************** GEOMETRIC DATA ****************

(48)	LENGTH OF MAXIMUM SPAN	30.8 M
(49)	STRUCTURE LENGTH	356.6 M
(50)	CURB OR SIDEWALK: LEFT 1.1 M	RIGHT 1.1 M
(51)	BRIDGE ROADWAY WIDTH CURB TO CURB	7.4 M
(52)	DECK WIDTH OUT TO OUT	10.9 M
(32)	APPROACH ROADWAY WIDTH (W/SHOULDERS)	7.3 M
(33)	BRIDGE MEDIAN- NO MEDIAN	0
(34)	SKEW 0 DEG (35) STRUCTURE FLARED	NO
(10)	INVENTORY ROUTE MIN VERT CLEAR	99.99 M
(47)	INVENTORY ROUTE TOTAL HORIZ CLEAR	7.4 M
(53)	MIN VERT CLEAR OVER BRIDGE RDWY	99.99 M
(54)	MIN VERT UNDERCLEAR REF- HIGHWAY	3.40 M
(55)	MIN LAT UNDERCLEAR RT REF- HIGHWAY	1.2 M
(56)	MIN LAT UNDERCLEAR LT	3.1 M

*************** NAVIGATION DATA ***************

(38)	NAVIGATION CONTROL- NO CONTROL	CODE 0
(111)	PIER PROTECTION-	CODE
(39)	NAVIGATION VERTICAL CLEARANCE	0.0 M
(116)	VERT-LIFT BRIDGE NAV MIN VERT CLEAR	M
(40)	NAVIGATION HORIZONTAL CLEARANCE	0.0 M

****

SUFFICIENCY RATING =	2.0
STATUS	STRUCTURALLY DEFICIENT
HEALTH INDEX	75.5
PAINT CONDITION INDEX =	N/A

************* CLASSIFICATION ************* CODE

(112)	NBIS BRIDGE LENGTH- YES	Y
(104)	HIGHWAY SYSTEM- NOT ON NHS	0
(26)	FUNCTIONAL CLASS- COLLECTOR URBAN	17
(100)	DEFENSE HIGHWAY- NOT STRAHNET	0
(101)	PARALLEL STRUCTURE- NONE EXISTS	N
(102)	DIRECTION OF TRAFFIC- 2 WAY	2
(103)	TEMPORARY STRUCTURE-	
(105)	FED.LANDS HWY- NOT APPLICABLE	0
(110)	DESIGNATED NATIONAL NETWORK - NOT ON NET	0
(20)	TOLL- ON FREE ROAD	3
(21)	MAINTAIN- CITY OR MUNICIPAL HIGHWAY AGENCY	04
(22)	OWNER- CITY OR MUNICIPAL HIGHWAY AGENCY	04
(37)	HISTORICAL SIGNIFICANCE- ELIGIBLE	2

**************** CONDITION **************** CODE

(58)	DECK	5
(59)	SUPERSTRUCTURE	4
(60)	SUBSTRUCTURE	5
(61)	CHANNEL & CHANNEL PROTECTION	6
(62)	CULVERTS	N

********* LOAD RATING AND POSTING ********* CODE

(31)	DESIGN LOAD- UNKNOWN	0
(63)	OPERATING RATING METHOD- ALLOWABLE STRESS	2
(64)	OPERATING RATING-	11
(65)	INVENTORY RATING METHOD- ALLOWABLE STRESS	2
(66)	INVENTORY RATING-	6.5
(70)	BRIDGE POSTING- > 39.9% BELOW	0
(41)	STRUCTURE OPEN, POSTED OR CLOSED-	P
	DESCRIPTION- POSTED FOR LOAD	

**************** APPRAISAL **************** CODE

(67)	STRUCTURAL EVALUATION	2
(68)	DECK GEOMETRY	2
(69)	UNDERCLEARANCES, VERTICAL & HORIZONTAL	2
(71)	WATER ADEQUACY	9
(72)	APPROACH ROADWAY ALIGNMENT	6
(36)	TRAFFIC SAFETY FEATURES	0000
(113)	SCOUR CRITICAL BRIDGES	U

********** PROPOSED IMPROVEMENTS **********

(75)	TYPE OF WORK- REPLACE FOR DEFICIENC CODE	31
(76)	LENGTH OF STRUCTURE IMPROVEMENT	356.6 M
(94)	BRIDGE IMPROVEMENT COST	$8,949,300
(95)	ROADWAY IMPROVEMENT COST	$1,789,860
(96)	TOTAL PROJECT COST	$15,034,824
(97)	YEAR OF IMPROVEMENT COST ESTIMATE	2010
(114)	FUTURE ADT	14144
(115)	YEAR OF FUTURE ADT	2029

*************** INSPECTIONS ***************

(90)	INSPECTION DATE 10/11 (91) FREQUENCY	24 MO		
(92)	CRITICAL FEATURE INSPECTION:		(93) CFI DATE	
A)	FRACTURE CRIT DETAIL-	NO	MO A)	
B)	UNDERWATER INSP-	NO	MO B)	01/96
C)	OTHER SPECIAL INSP-	NO	MO C)	