Analytical Assessment of Cellular Foundations for The Seismic Retrofit of The Dumbarton Bridge

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ABSTRACT

The Dumbarton Bridge along State Route 84 is the southern most toll crossing over the San Francisco Bay in California. The bridge carries traffic to Silicon Valley where major high tech industry resides. The bridge consists of main channel crossing in addition to east and west approaches totaling 8600 feet in length. Superstructure is made of two steel box girders for the main channel crossing and precast prestressed delta girders for the approaches. Piers typically consisted of two hollow columns, pedestal footings and pile caps with several different types of piles. Two cellular foundation structures bound the main channel crossing. The bridge was designed and built in the late 1970's according to the seismic standard at the time. However the foundations were not specifically required to be designed for seismic loads, nor were all bridge components required to be designed such that they were capacity protected by ductile columns. Caltrans completed a seismic vulnerability study of the Dumbarton Bridge, which indicated the need for a retrofit strategy for the bridge to be compatible with the current knowledge in seismic safety.

In this paper, two seismic retrofit alternatives for the Dumbarton bridge are presented. Of particular interest in this paper is the seismic assessment of the cellular foundation structures located at the ends of the main channel crossing. Results from three dimensional finite element analysis and a retrofit strategy for this foundation structure are also presented.

Keywords: Seismic, Foundation, FEM, Retrofit

INTRODUCTION

The Dumbarton Bridge is the southernmost crossing of the San Francisco Bay (Fig. 1). It is an 8,600 ft. long structure on Route 84 connecting the cities of Newark and East Palo Alto. It was designed in the late 1970's and construction was completed in January 1982. Prior to 1971, the seismic design requirements for highway structures in California were minimal and generally only project specific. In 1971, following the San Fernando Earthquake, the California Department of Transportation (Caltrans) significantly revised its seismic design procedures to incorporate "Lessons Learned" from this event. Resulting code changes primarily focused on increasing column confinement to create more ductile columns and increasing the length of in-span hinges to avoid unseating.

The Dumbarton Bridge was designed and constructed subsequent to the San Fernando earthquake, and fully incorporated the seismic design code changes at the time. However the foundations were not specifically required to be designed for seismic loads, nor were all bridge components, other than columns, required to be designed such that they were capacity protected by ductile columns. Such requirements became part of the Caltrans Seismic Design Criteria[1] (SDC) released in 1999. One of the basic tenets of the SDC is that all structural members, including foundations, must be designed to either be ductile or must have adequate strength such that the member is capacity protected by a ductile member.

Consistent with the Seismic Advisory Board's formal recommendation in its December 2003 report[2] "The Race to Seismic Safety", Caltrans regularly reassesses existing bridges for seismic hazard. As part of this reassessment policy, Caltrans completed a seismic vulnerability study of the Dumbarton Bridge in November 2004, which indicated the need for a retrofit strategy for the bridge to be compatible with the current knowledge in seismic safety.

In this paper, two seismic retrofit alternatives for the Dumbarton bridge are presented. Of particular interest in this paper is the seismic assessment of the cellular foundation structures located at the ends of the main channel crossing. Results from three dimensional finite element analysis and a retrofit strategy for this foundation structure are also presented.

BRIDGE DESCRIPTION

The Dumbarton Bridge comprised of five main components: a Main Channel Crossing in the middle of the bridge, an Approach Structure adjacent to each end of the main crossing, and a Trestle structure at the end of each Approach Structure. It carries three lanes of traffic in each direction, and has a separate bike/pedestrian lane.

MAIN CHANNEL CROSSING

The Main Channel Crossing extends from bent 16 to bent 31 and is 3150 ft. in length. The spans vary in length from 175 ft. to a maximum of 340 ft. for the main span (span 23). spans

21 and 25 have expansion hinges. The superstructure consists of continuous twin steel box sections with a single concrete deck over the two boxes. The superstructure is 10 ft. 7½ in. deep and tapers down to 8 ft. 1½ in. at the end spans. The deck is 85 ft. wide. The substructure consists of two hollow concrete columns battered in a V-shape in the transverse direction. The steel boxes are supported on a single concrete cap beam that connects the two columns. Four 6 in. steel pipes connect each steel box to the cap beam. The columns are supported on a single concrete footing placed at Mean Sea Level. The footings are supported on vertical piles. The pile caps are placed either in water or very poor soil (bay type mud) and the piles extend down to an elevation of –40 ft. to –60 ft. See Fig. 2.

APPROACH STRUCTURES

The Approach Structures are 2850 ft. and 2600 ft. in length. The first or “West” Approach Structure begins at abutment 1 and ends at bent 16. The second or “East” Approach Structure begins at bent 31 and ends at abutment 44. The spans are 150 ft. in length with the exception of span 1 which is 100 ft. Expansion hinges are placed at abutments 1 and 44, and at bents 4, 8, 12, 16, 31, 35, 38 and 41. The abutments are closed bin type and are 50 ft. in length. The superstructure is 85 ft. wide, 8 ft. 3 in. deep, and consists of five pre-cast, pre-stressed bathtub girders with a single concrete deck spanning over the girders (Fig. 3). The bathtub girders are spliced together at the bents with a continuous connection, and are tied into a concrete bent cap with either a fixed or pin type connection except at the expansion joints. The columns are similar to those for the Main Channel Crossing. At bents 2 through 15, and 32 through 43, the columns are supported on concrete pedestals on top of concrete footings. The columns at bents 16 and 31 are supported on a relatively thin slab that overlays a cellular pedestal and footing. The footings are located below the mud line and are supported on vertical piles that extend through approximately 40 ft. of soft bay type mud.

TRESTLE STRUCTURES

The Trestle Structures are located at the ends of each of the Approach Structures and are 600 ft. in length. There are 20 spans in each trestle structure, each 30 ft. in length. The deck is a 1 ft. 5 in. concrete slab with three expansion joints in each structure. The deck slab is supported on seven 20 in. concrete pile extensions at each bent.

SEISMIC RETROFIT CRITERIA

SEISMIC PERFORMANCE

Seismic performance criteria for the Dumbarton Bridge was developed for two folds. A seismic safety for the 1000 year return period design earthquake, in which no collapse is expected but with acceptable damage in predetermined locations. A functional performance is specified for a repairable damage with the bridge open to traffic in 3 to 6 month.

SEISMIC HAZARD

The seismicity of the San Francisco Bay area is largely controlled by the northwest trending, right-lateral San Andreas fault system. The main trace of the San Andreas fault situated 9 miles west of the Dumbarton Bridge west end and the Hayward fault located approximately 8 miles east of its east end. Depending upon fault rupture in part or as a whole, USGS Working Group on Earthquake Probabilities in Northern California (WG, 2003) estimated that the San Andreas fault might be capable of generating earthquakes with estimated mean magnitude (M) varying from 7.0 to 7.7, and the Hayward fault as the most likely source of the next major earthquake in the Bay area could initiate earthquakes with estimated mean magnitude (M) varying from 6.7 to 7.0.

According to the source and attenuation models of "next generation attenuation" (NGA), uniform hazard spectra (UHS) at six return periods (100, 300, 475, 1,000, 1,500 and 2,000 years) and Maximum Credible Earthquake (MCE) rock spectra of the San Andreas and Hayward faults for the 50^{th} percentile (mean) and 84^{th} percentile (mean plus one standard deviation) were generated. From discussions with Caltrans and the Seismic Peer Review Panel for the toll bridges, the 1,000-year return period ground motion was considered appropriate and selected as the Safety Evaluation Earthquake (SEE) for seismic retrofit evaluation of the Dumbarton Bridge.

Two sets of ARS curves were developed for the retrofit design.

- ARS Curve 1: This ARS design curve covers Piers 1 through 16, and Piers 27 through 44, representing 20-inch diameter pipe piles with a buried pile-cap
- ARS Curve 2: This ARS design curve cover Piers 17 through 26, representing 54-inch diameter concrete piles with a long cantilever pile extending above mud-line

In addition to ARS design curves, pier-specific kinematic time histories were developed for use in multiple support time history analyses of the global bridge structure. Pier-specific kinematic time histories were developed for seven earthquake time histories; each pier has three-component kinematic motions for the given set of earthquake time history.

ANALYTICAL MODELS

LOCAL MODELS:

Local models or Stand-Alone models of seven representative piers were selected to conduct soil structure interaction analysis. These seven selected piers are: Piers 2, 9, and 15 within West Approach (20-inch diameter steel pipe piles with a buried pile cap); Piers 17 and 23 within Main Span (54-inch diameter concrete hollow piles with a long cantilever pile length above mud-line); Piers 30 and 43 within East Approach (20-inch diameter steel pipe piles with a buried pile cap). Given the limited generality of these local models, the goal was to select bents that represent a variety of column, footing and superstructure types and configurations as well as capture the varying soil conditions. In addition, because of the unusual configuration of the footing at bent 16, joint shear at the column/footing connection was also investigated at this location.

3D Local models were developed using SAP2000 to model the bent cap, hollow columns, piles and soil. Soil was model using depth varying linear *p-y, t-z, q-z* curves. These models were used to perform non-linear static pushover analysis to predict the sequence of failure of each component of the bent leading to the failure mechanism. These model were used to compute the capacities of the components of the bent and hence the capacity of each pier system. In addition non-linear static pushover analysis was also performed on the frames to compute displacement capacities of columns, force demands on super-structure. These models included the interaction of the super-structure in the longitudinal direction of the bridge.

GLOBAL MODELS OF AS-BUILT STRUCTURE:

Response Spectra Analysis:

The seismic response evaluation was carried out using SAP2000 Response spectra/Linear Dynamic Modal analysis of the entire structure was performed to capture the overall dynamic response of the bridge. These model included East and West Trestle structure, East and West Approach structure and the Main Channel crossing.

Site specific ARS curves were used for the response spectra analysis. Displacements at pile cap, column drifts were calculated in the two directions. These displacement demands were used to in the local models to calculate the response of substructure. The soil-structure interaction was captured with 6x6 stiffness matrices at each pier in the global bridge model.

Non-Linear Time History Analysis:

Global 3D non-linear model for the Main Channel Crossing was developed for Isolation Bearing Retrofit Alternative 2. Non-linear Direct Integration Time History Analysis with multi-support excitation was performed due to the complex behavior of the isolators. Two different types of Isolation devices, Friction Pendulum Bearing Isolator and Flat Spherical Bearing Isolator, respectively were investigated.

AS-BUILT SEISMIC VULNERABILITIES

Brief description of the seismic deficiencies of various component of the bridge is discussed in this section.

TRESTLES:

Results of the SAP dynamic analysis shows existing piles are deficient in displacement capacity. The Demand/Capacity ratios exceeds 1 at most piles and pile extensions.

APPROACHES:

Results of the response spectra analysis and global pushover analysis indicated that column shear capacities were slightly exceeded at few locations for tension columns. As-Built columns have couplers at base designed to carry little over yield stress of the longitudinal rebars. Under seismic loadings, these couplers are vulnerable to plastic hinging resulting in a catastrophic failure.

Bent caps to column connection were vulnerable to high joint shear stresses. Joints were classified as week joints according to Caltrans Memo To Designer (MTD) 20-4. Bent cap flexural capacities were also exceeded. Shear keys capacities were limited and were not able to transfer forces through cap to superstructure. Superstructure flexural capacities were exceeded at few piers. Joint shear stresses at the connection between columns and pile caps were highly exceeded.

MAIN CHANNEL CROSSING:

Results of the response spectra analysis and global pushover analysis of the As-Built structure indicated that column flexural capacities were slightly exceeded at few locations. This is due to the low concrete ultimate strain values used for the hollow columns. Test on these hollow columns is under progress at UC San Diego to verify ductility and joint shear capacities. . As-Built columns have couplers at base designed to carry little over yield stress of the longitudinal rebars. Under seismic loadings, these couplers are vulnerable to plastic hinging resulting in a catastrophic failure.

Bent caps to column connection were vulnerable to high joint shear stresses. Joints were classified as week joints according to MTD 20-4. Bent cap flexural capacities were also exceeded.

Existing hinges are not adequate for the large longitudinal seismic displacement leading to catastrophic failure. These hinges need more seat width to avoid unseating of the spans. Existing cross bracing in steel box superstructure at the piers are deficient due to buckling of the compression brace under design seismic demands. Also, the weld connections are questionable for these high axial demand forces.

Foundation displacements are more than the capacity of the existing piles. The limiting displacements for these piles were computed from the local models and verified by time history analyses.

Buckling of the thin web panels of the steel box girder was observed at sections close to support. Yielding of the vertical stiffeners was also observed.

RETROFIT STRATEGY

The retrofit strategies of various sections of the bridge are as follows:

TRESTLES

As-built model were modified to reduce the displacement demands in both longitudinal and transverse direction. The analysis models concluded the following strategy:
a) Strengthen existing slab bridge frames with 4' diameter CISS (Cast in Steel Shell) driven pipe piles connected to the existing slab bridge with pile cap extension.
b) Install steel hinge seat extenders.
c) Replace existing deck joints with seismic joint system to accommodate increased joint movements at Pier 1 and Pier 44 from the main bridge. Provide concrete seat extender/corbel with new footing with piles.

APPROACHES

Retrofit of west approach from Pier 5 to Pier 15 and east approaches from Pier 32 – Pier 40 will be done in shallow water. The analysis models concluded the following strategy as shown in Figure 4:

a). Strengthen pile cap to column connection by casing with concrete bolsters around the existing footing pedestals.
b). Strengthen bent cap by providing P/S concrete bolsters on the sides of the existing bent caps
c). Strengthen existing column (with couplers) with jackets and replace existing deficient column longitudinal rebar couplers with HRC ultimate couplers.
d) Strengthen superstructure to meet flexural demands at Piers 10, 11, 13, 14, 15, 32, 33, 34, 36 and 37.
e) Strengthen bent cap to superstructure connection by providing shear keys.

MAIN CHANNEL CROSSING

Retrofit Alternative 1:

As-built model were modified to account for new added piles in order to reduce the foundation displacement demands in both longitudinal and transverse direction. Foundation springs were modified to reflect the increase in stiffness due to new battered piles. Retrofit of the main channel from Pier 16 to Pier 31 will be done in deep water. The analysis models concluded the following strategy:

a) Retrofit with new footing around the existing pile caps with new 3’ diameter steel pipe piles driven (battered and vertical) along the perimeter of the existing footing.
b) Strengthen pile caps.
c) Strengthen column to bent cap connection by providing P/S concrete bolsters on the sides of the existing bent caps
d) Strengthen existing column (with couplers) with jackets and replace existing deficient column longitudinal rebar couplers with HRC ultimate couplers.
e) Strengthen existing cross frames at piers.
f) Strengthen existing steel box superstructure with vertical and longitudinal stiffeners along the bridge spans.
g) Retrofit existing hinges to accommodate large seismic movements.

Retrofit Alternative 2:

The use of Isolation Bearings eliminated the need of new piles at Pier 17 through Pier 26. As-built model were modified to incorporate isolator bearings with the objective of reducing foundation displacement demands in both longitudinal and transverse direction, column drifts, column moments, bent cap demands. The analysis models concluded the following strategy:

a) Strengthen existing steel box superstructure cross frames at piers for Isolation bearing installation. See Fig. 5
b) Strengthen pile caps by providing footing overlay.
c) Strengthen column to bent cap connection for joint shear by providing P/S concrete bolsters on the sides of the existing bent caps. Bent cap flexural capacities seem adequate.
d) Strengthen existing column with concrete jacket.
e) Retrofit existing hinges to accommodate large seismic movements.
f) Strengthening of steel box superstructure may not be needed. Finite element analysis verified the adequacy of the box under the reduced demand.
g) Replace existing deck joints with special seismic joint system to accommodate seismic movements at Pier 16 and Pier 31. Retrofit bent cap to accommodate Isolator bearings.

CONSTRUCTABILITY DETAILS

All the work within deep water in the main channel piers and superstructure will be done using barges. All the work within shallow water will be done by constructing the temporary trestles to access the piers. The temporary trestles will consist of a set of two piles of 3’ diameter at 50’ spacing. On the east approach the existing old bridge will remain in use as a fishing pier. A temporary trestle will be constructed between this pier and the bridge.

Construction feasibility for strengthening of the existing footing in the main channel crossing from Pier 16 to Pier 31 was also investigated. The footing extension will be built using precast panel slab and wall around the existing footing.

PIER 16 & PIER 31 CELLULAR FOUNDATION SPECIAL STUDY

The two end piers of the main channel (Pier 16 and Pier 31) represent a different foundation structure than the rest of the main channel footings. To reach the soft bay mud underneath and utilize piles that are embedded in the ground rather than free standing in the water, an 18 ft and 4 ft of additional heights are needed below the typical footing that are used in the entire main channel.

The additional heights are achieved through a cellular type structure with voided cells (chambers) that are filled with water to balance the hydrostatic pressure of the surrounding water. The cellular structure is then supported on a pile cap with 43 Cast in Steel Shells 20 in. diameter piles. This foundation, which has a 24' X 52' horizontal footprint, consists of solid concrete footing supporting the two hollow columns. The 5 feet deep footing is rested on a grid of 1.5' thick walls creating 8 chambers. Each chamber is approximately 11' X 11' in size with a height of 18'. The hollow chambers are provided with vent holes located on the exterior walls to allow for sea water to flow in and out of the chambers (Fig. 6). The grid of walls is supported by 4' deep pile cap with a total of 43 concrete filled 20" diameter steel pipe piles.

In a seismic event, the walls of the cellular structure are subjected to seismic loading due to the plastic hinging of the compression and tension columns. In addition to the seismic loading, the cellular walls are subjected to the hydrodynamic effects from the water inside and outside the structures.

The water inside the cells is modeled using Housner's method[4]. This method assumes that hydrodynamic effects due to seismic loading can be evaluated as the sum of Impulsive water which represents the portion of the water which moves with the structural walls, and the convective water which represents the sloshing effect.

In addition, the immersed cellular structure is subjected to an additional force from the outside water[5]. The surrounding water responds with the cellular structure increasing their effective weight and the corresponding inertia forces. The effective weight of outside water, per lineal foot of height of the walls, is added to the weight of the structure in addition to the weight of the inside water to determine the total hydrodynamic forces. The immersed structure is also subjected to a drag force due to the sloshing of the outside water. This force is added to the hydrodynamic forces and can be determined as per AASHTO LRFD[6] guidelines.

In this study, a linear elastic finite element analysis is used to determine the level of compressive, tensile and shear stresses in the structural walls of the cellular structure and evaluate the demand to capacity ratio of the concrete members. Soil non-linearities are modeled with P-Y, T-Z and Q-Z springs. Analysis was carried out both in the longitudinal and transverse direction, and results are shown in Fig. 7.

Shear and normal stresses were acceptable in the cellular walls. Piles were excessively overstressed in shear and axial capacity with demand to capacity ratio exceeding 2.0. The retrofit strategy utilized new footing around the existing pile caps at pier 16 and 31 with new 3 ft diameter steel pipe piles driven (battered and vertical) along the perimeter of the existing footing.

CONCLUSION

Based on several local and global analyses, two seismic retrofit alternatives were deduced for the Dumbarton toll bridge. Seismic assessment of the cellular foundation structures located at the ends of the main channel crossing were presented considering the hydrodynamic effects. Results from three dimensional finite element analysis and a retrofit strategy for this foundation structure were also presented.

REFERENCES

1. California Department of Transportation, “Seismic Design Criteria,” Version 1.4, Sacramento, CA, 2006.
2. California Department of Transportation Seismic Advisory Board, “Race to Seismic Safety”, Sacramento, CA, 2003.
3. California Department of Transportation, Memo To Designer 20-4, “Joint shear in existing structures”, Sacramento, CA, 2008.
4. Munshi J. A., “Design of liquid-containing concrete structures fort earthquake forces”, Portland Cement Association, Skokie, IL, 2002.
5. Westergaard, H. M. 1933. "Water Pressures on Dams During Earthquakes", Transaction, American Society of Civil Engineers, 98:418-433.
6. AASHTO LRFD Bridge Design Specifications, third edition, American Association of State highway and Transportation Officials, Washington, DC, USA

Fig. 1 The Dumbarton Bridge

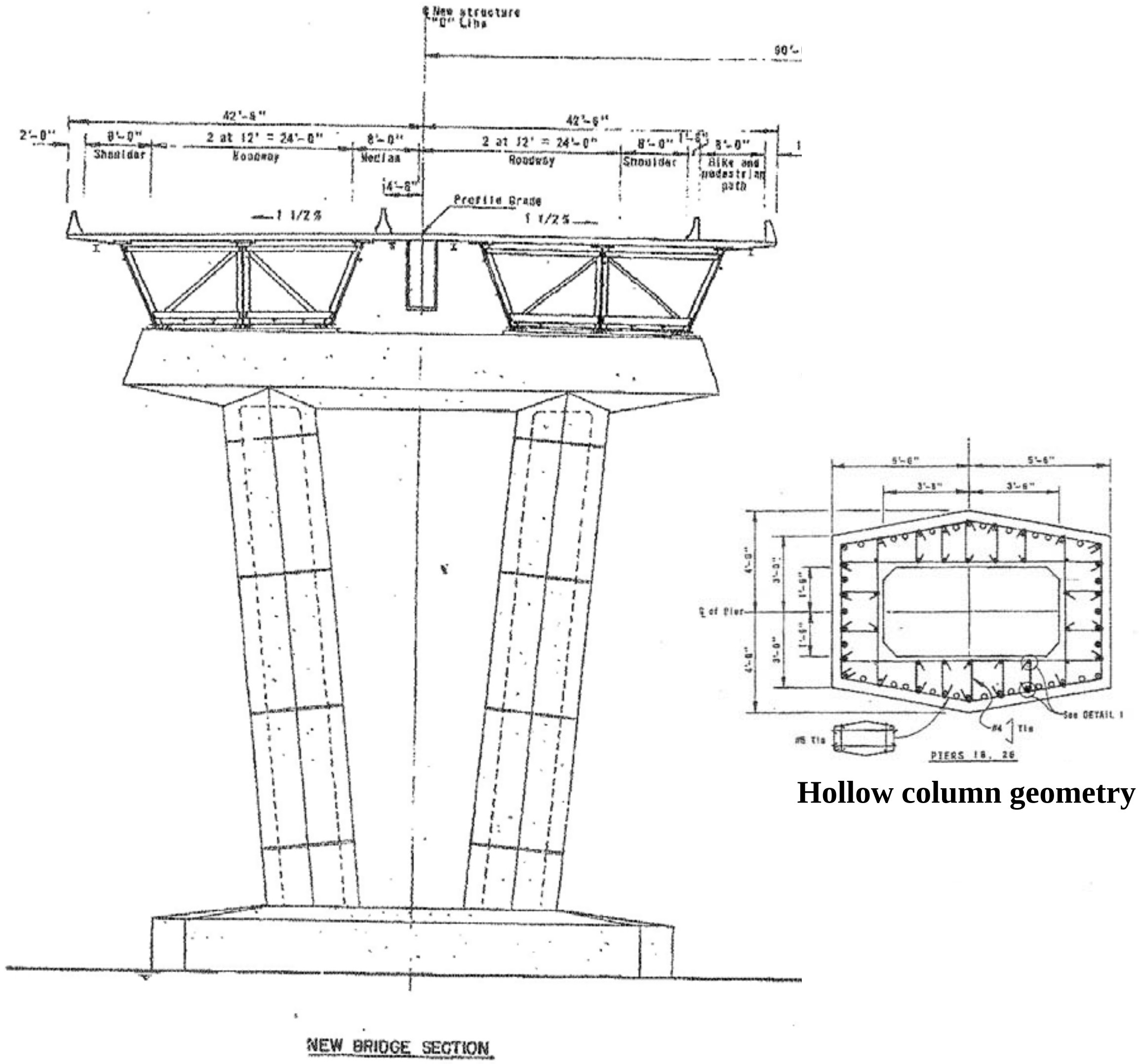
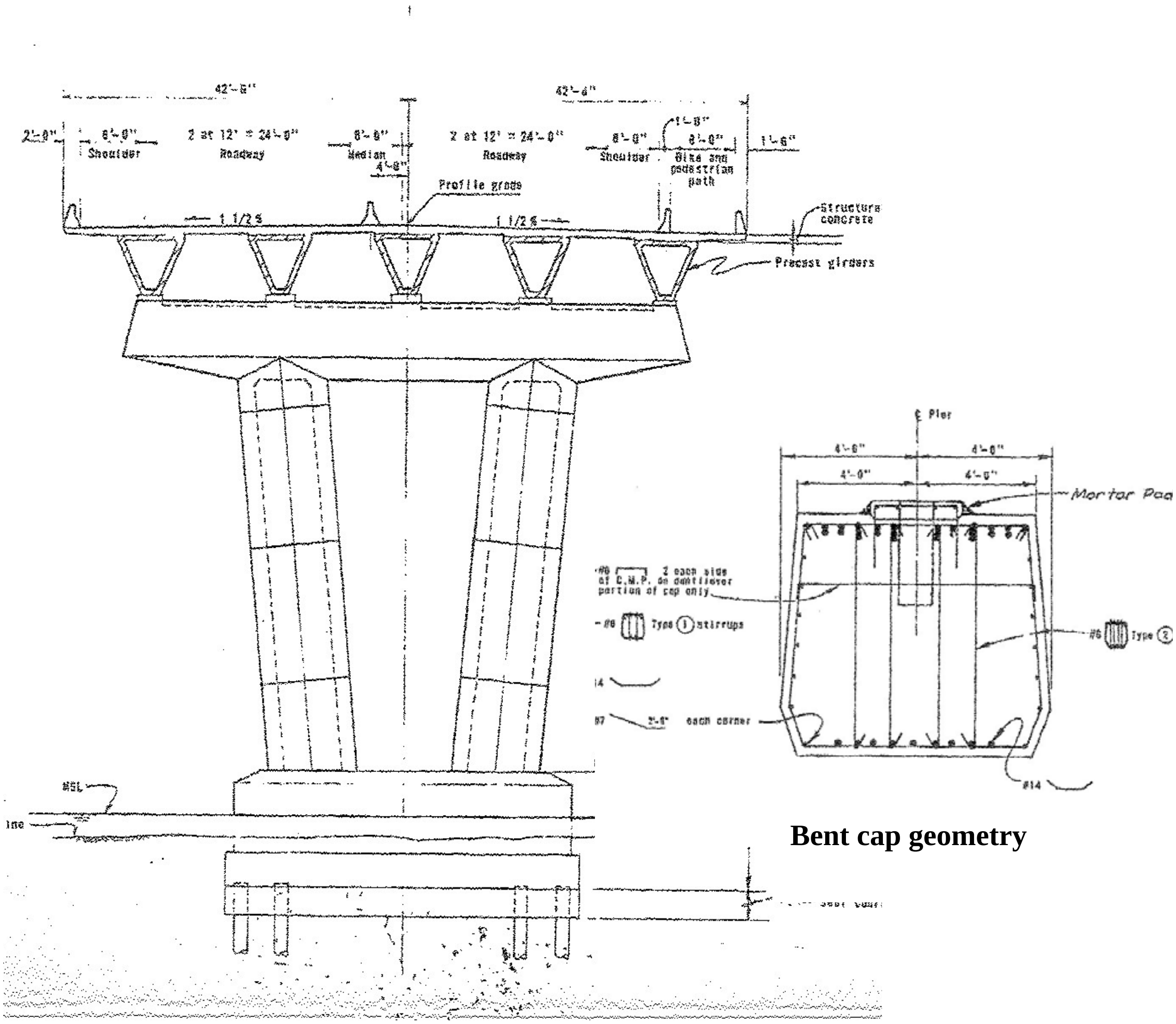


Fig. 2 Geometry of the Main Channel Crossing (Piers 17 – 30)



Bent cap geometry

Fig. 3 Geometry of the East and West Approach Structures

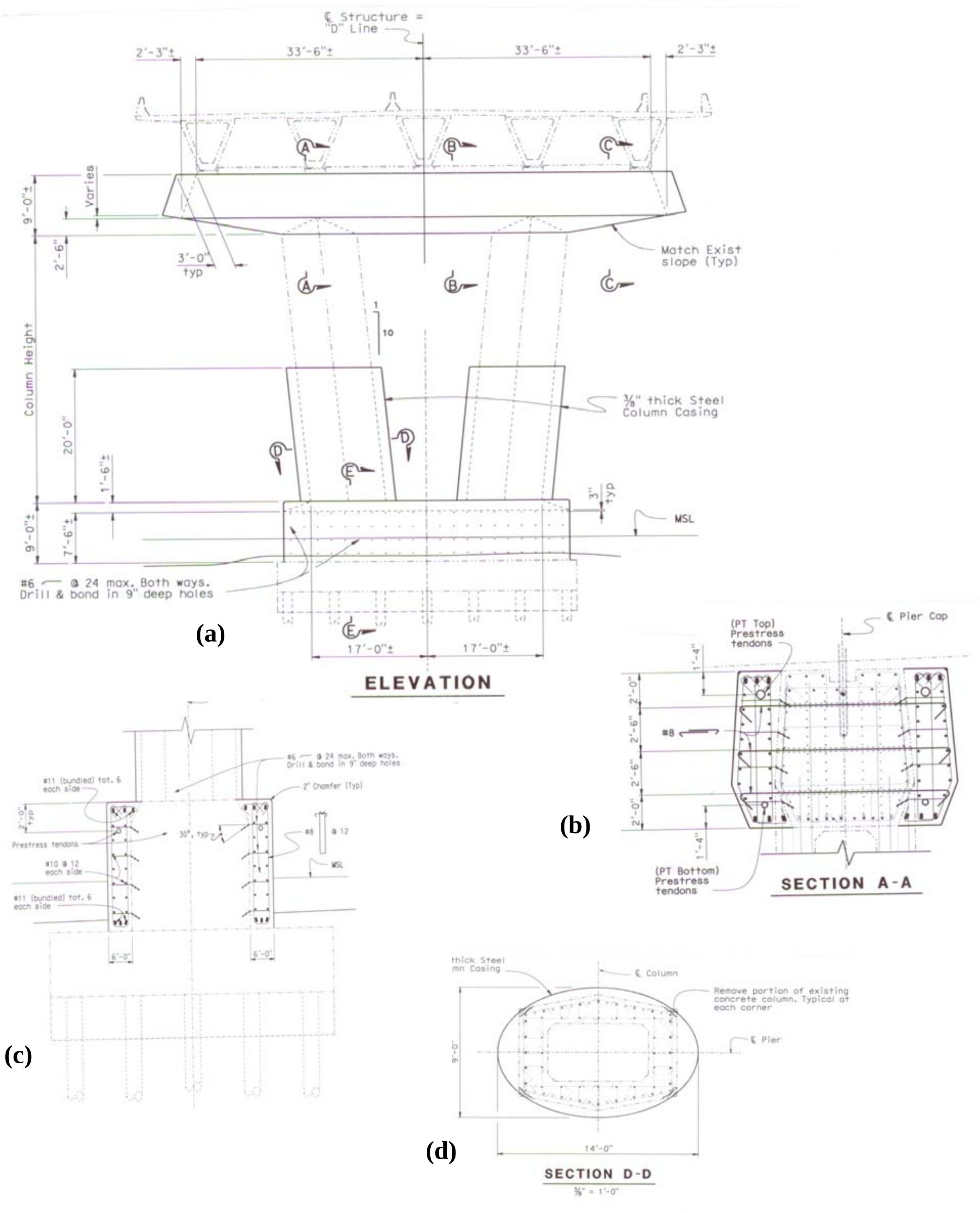


Fig. 4: Retrofit Strategy for the East and West Approach Structures a) General Elevation, b) Bent Cap Strengthening, c) Foundation Pedestal Retrofit, d) Column Retrofit.

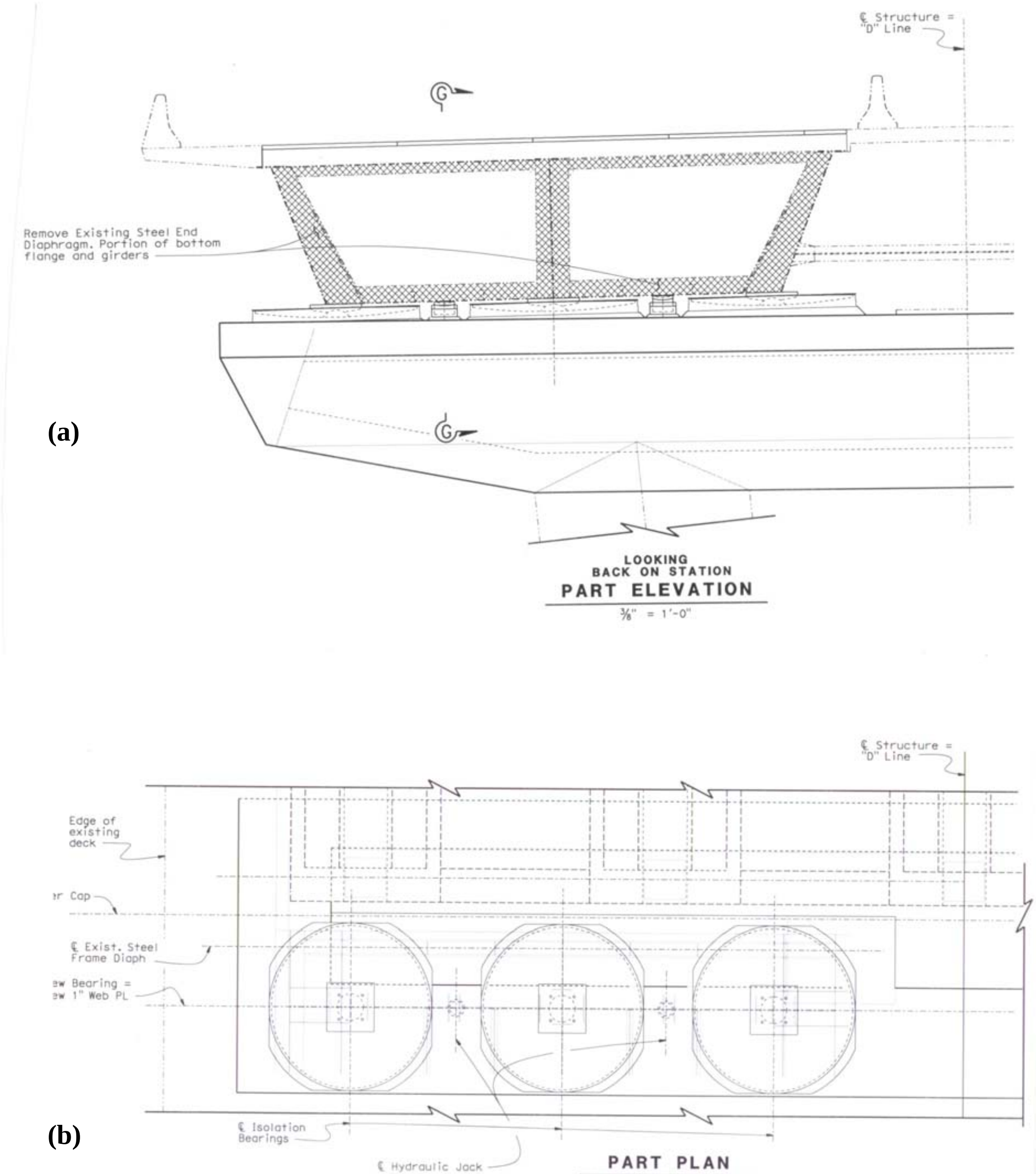


Fig 5: Retrofit of the Steel Box Girder of the Main Channel Crossing a) Strengthening of the End Diaphragm, b) Isolator Bearings - 3 per Each Box.

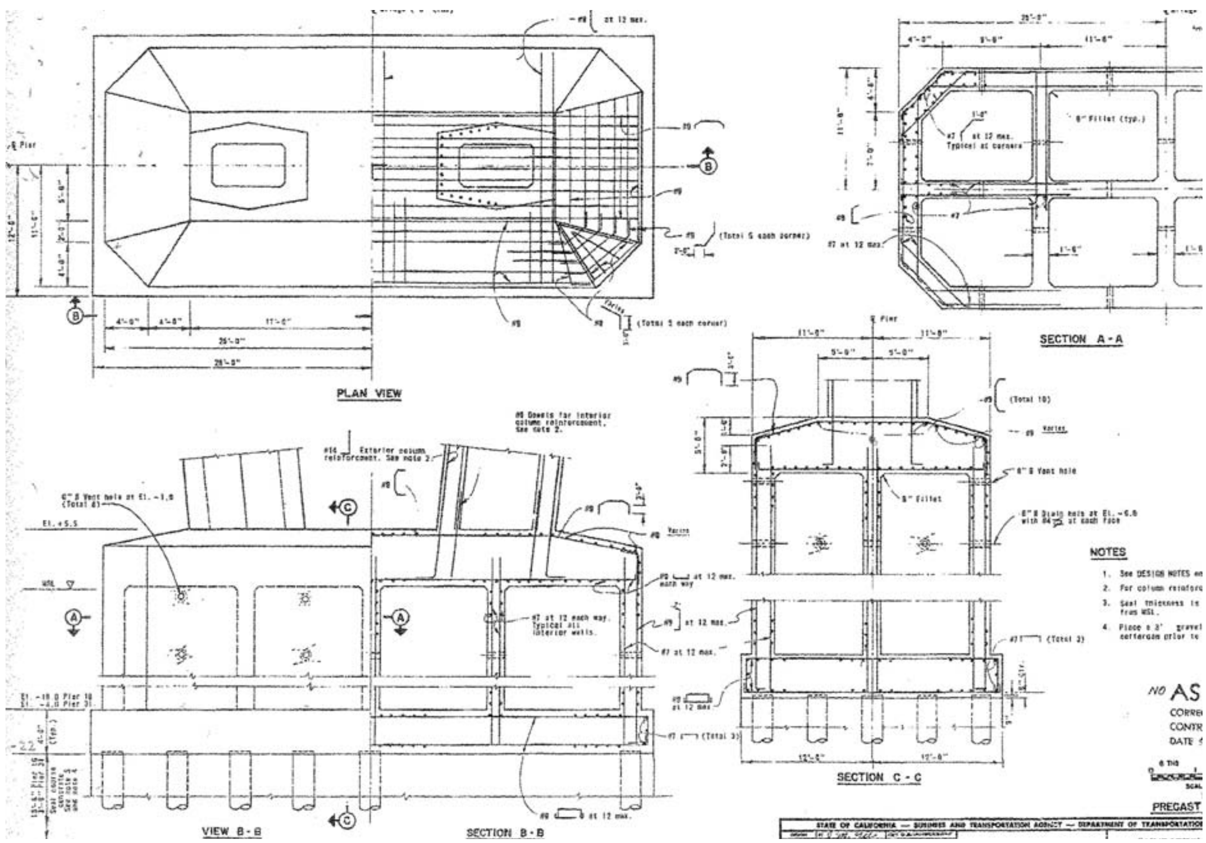
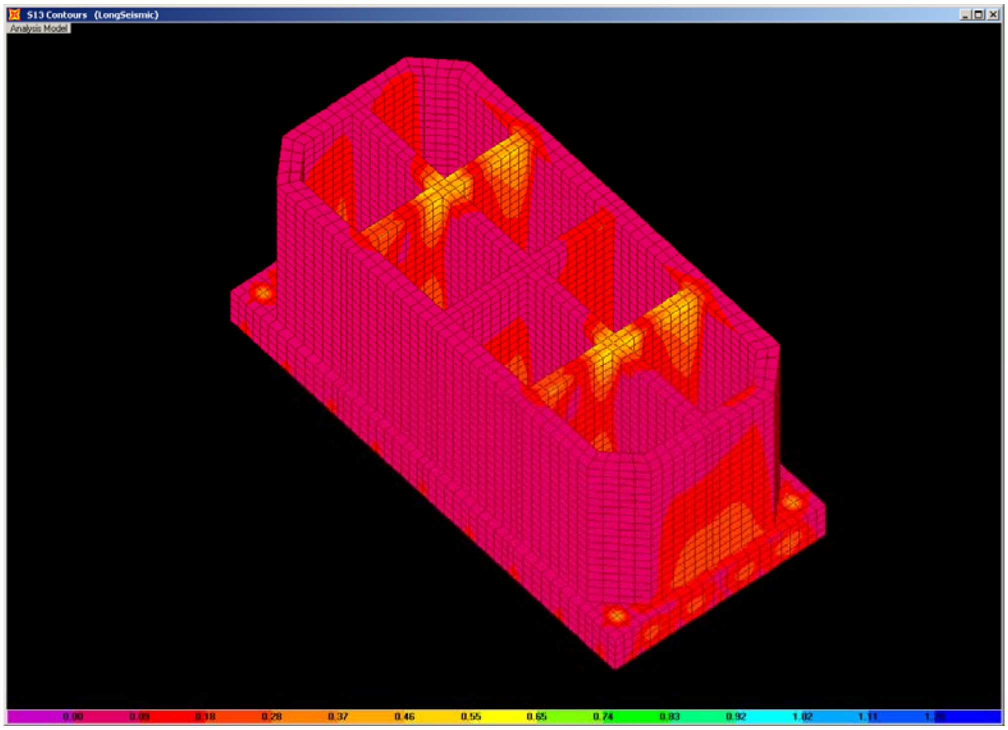
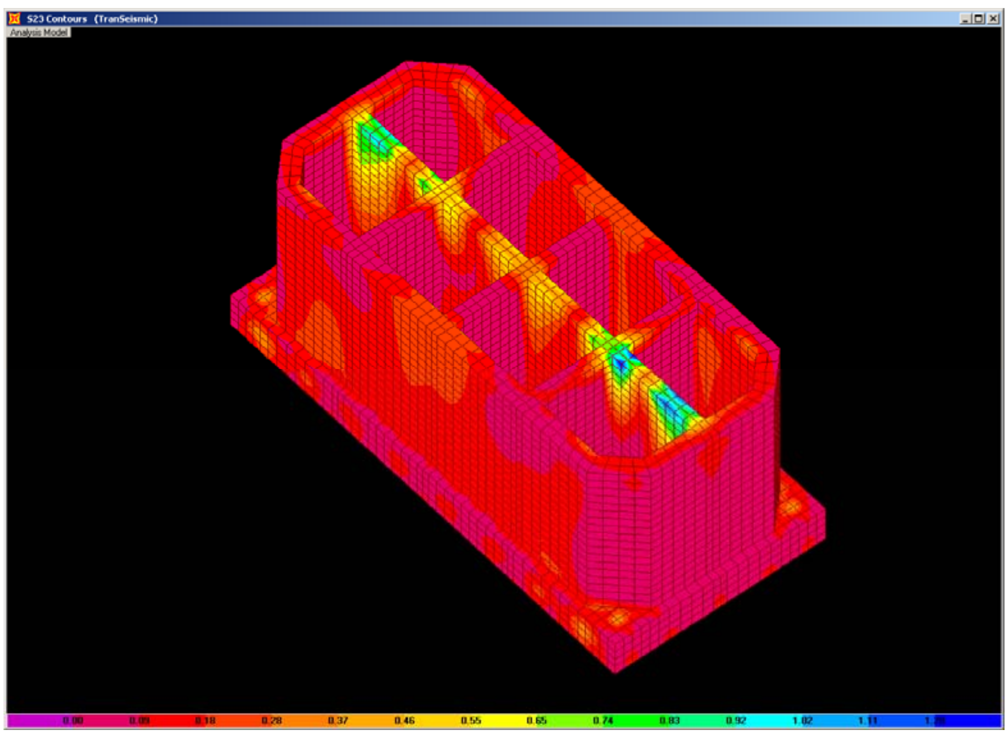


Fig. 6: Cellular Foundation Structure at Piers 16 and 31.



(a)



(b)

Fig. 7: Shear Stresses in the Cellular Foundation: a) Longitudinal Analysis, b) Transverse Analysis.