SIXTH STREET VIADUCT
OVER THE LOS ANGELES RIVER
(Bridge No. 53C-1880)

FINAL SEISMIC RETROFIT STRATEGY REPORT

Submitted to

The City of Los Angeles
Bureau of Engineering

Prepared by

W. Koo & Associates, Inc.

June 2004

Final Seismic Retrofit Strategy Report

Sixth Street Viaduct over the Los Angeles River

State Bridge No. 53C-1880

FINAL SEISMIC RETROFIT STRATEGY REPORT

CITY OF LOS ANGELES
FOUNDED 1781

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Bridge Improvement Program
June 2004

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1. INTRODUCTION

The Sixth Street Viaduct is located East of downtown Los Angeles carrying four lanes of traffic over the Los Angeles River, Union Pacific Rail Road (UPRR) and Metrolink tracks and the US101 freeway (see vicinity map in Figure 1). Sixth Street Viaduct is the longest of the bridges crossing the Los Angeles River. This massive viaduct was constructed in 1932 using state-of-the-art concrete technology at that time and on-site mixing plants. Over the last 70 years, concrete of the viaduct has deteriorated as a result of an internal chemical reaction. A material testing program[1] confirmed that Alkali Silica Reaction (ASR) is the main cause of the concrete cracking. Today cracking is evident throughout the bridge with large cracks and concrete spalling on the columns, bent caps and girders. Analytical studies presented in this report show that the viaduct with its current state of deterioration has high vulnerability to collapse in moderate seismic events. Laboratory testing has concluded that concrete deterioration of the bridge due to ASR will continue to occur, which will further increase risk of collapse of the bridge in seismic events. Thus, there is a persisting need for seismic retrofitting of the viaduct. The age, size and architecture of the structure qualify the Sixth Street Viaduct to be eligible for the National Register of Historical Places. This requires that special protection and measures be taken to reduce impact of seismic retrofitting on the historical integrity of the structure.

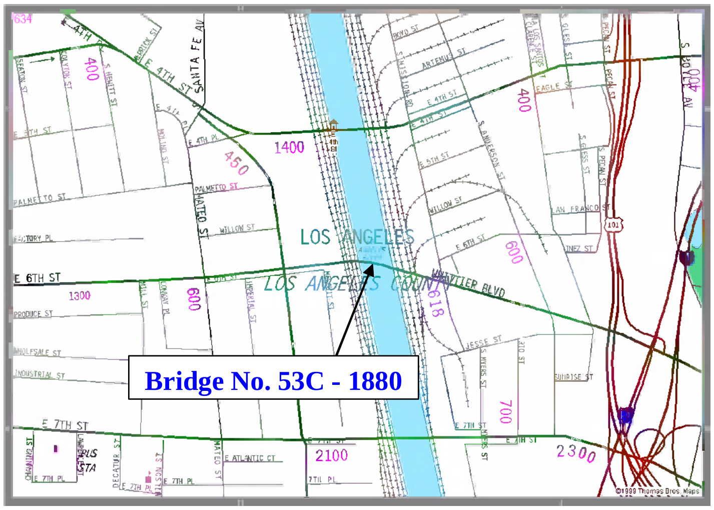

Figure 1. Map in vicinity of the Sixth Street Viaduct

2. DESCRIPTION OF THE BRIDGE

The Sixth Street viaduct (Bridge No. 53C-1880) is over 3500 feet long and is comprised of 43 concrete spans and two large steel through arch truss spans over the river. The majority of the structure sits on 58 ft high columns supported by spread footings. The viaduct can be divided into the following three segments: (1) Approach spans West of the Los Angeles River, (2) Steel through arch truss spans over the river (main spans), and (3) Approach spans East of the river. Table 1 summarizes information of the bridge.

Table 1. Summary of information of the Sixth Street Viaduct

Superstructure Type	Approach spans: cast-in-place concrete T-beams
	Los Angeles River spans: through steel overhead arch ribs with suspended deck
Substructure	Tapered concrete columns on concrete pedestals
Foundation	Approach Spans: spread footing, 15′ to 20' +/- below ground
	Los Angeles River spans: pile foundations (precast concrete piles)
Total Span Length	3,178′ (West Abutment to East Abutment)
Number of Spans	45 (43 concrete spans + 2 steel arch spans)
Spans within Caltrans Right-of-Way (ROW)	Bent 37 to East Abutment
Length within Caltrans ROW	235′
Average Span Length	71′
River Spans	2 Spans each is approximately 163′
Width	46′ curb-to-curb with 5′ wide raised walkways on both sides
	Total out-to-out width = 55′-10" (River spans & East Approach)
Average Column Height	West Approach spans: 30′ above ground
	East Approach spans: 55′ above ground
	Los Angeles River spans: 61′ above river

__West Approach Spans:__ The West Approach has a total of 12 spans. The reinforced concrete deck, longitudinal T-beams and diaphragm beams are supported on reinforced concrete bent caps. The bridge superstructure is supported on a seat type abutment on the West side. On the East end, the approach superstructure is supported on the West River Pier. Expansion joints exist at nearly every third span of the superstructure with the longitudinal T-beams of the superstructure continuous between the expansion joints. All piers are supported on spread footings except at Bent 11 where columns are supported on pile foundations.

__River Spans:__ The middle segment of the bridge crosses the Los Angeles River. It consists of a two-span continuous asymmetrical steel tied arch, as shown schematically in Figure 2. The arch ribs are comprised of built-up sections with varying depth that form a compression arch that rises gracefully above the deck from the East and West River Piers and then dives below the concrete deck just before reaching the Center River Pier, with the base of the arches supported at the center pier. Thus, the arch ribs are fixed to the Center River Pier while supported on segmental rockers on the West and East River Piers (see Figure 2).

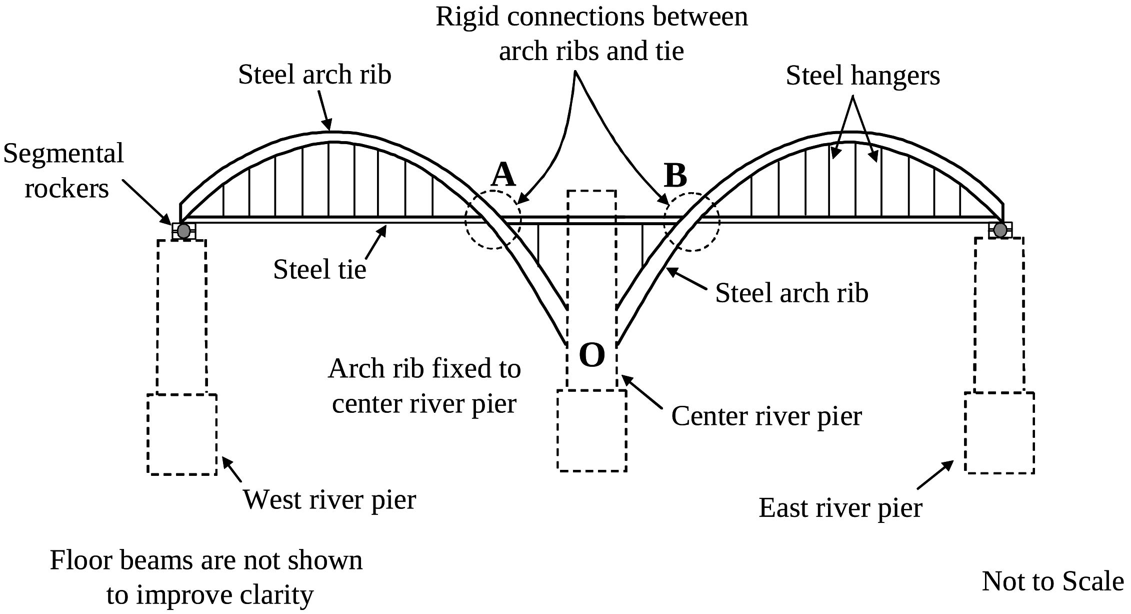

Figure 2. Steel arches in spans over the Los Angeles River

The outer ends of the steel arch ribs are tied together at the superstructure level by continuous steel tie that spans the entire length between the West and East River Piers. This steel tie member is rigidly connected to the arch ribs at locations "A" and "B" shown in Figure 2. The bridge deck slab is supported on steel floor beams that are suspended from the steel arches by steel hangers.

East Approach Spans: The East Approach is similar in construction to the West Approach. It has a total of 31 spans between the East River Pier and the East Abutment. The span lengths and skew angles to the bents vary to allow several local streets to pass underneath the bridge. Columns of Bent 12 are supported on pile foundations, whereas columns in all other bents are supported on spread footings.

3. BRIDGE CONDITION

In the 1940s, two large historic monuments at the center river bent were removed due to the poor condition of the concrete. Approximately 20 years ago the deck asphalt was stripped and a waterproof coating was applied to the concrete deck in an attempt to prevent future deck cracking. Today cracking is evident throughout the bridge with large cracks and spalling on the outer columns (see field photos in Appendix F). Over the past 70 years, concrete of the Sixth Street Viaduct has deteriorated as evidenced by map-type cracking throughout the structure. In the past, the City of Los Angeles has patched the cracks with epoxy injection leaving discolorations and honeycombs on surfaces of the entire structure. To cover the unsightly honeycomb effect of these repairs, cementitious coating has been applied to the surface, resulting in a loss of the historic appearance of the concrete. Due to the continuous cracking from ASR, the bridge requires epoxy injection and patching every 10 years.

4. PROJECT BACKGROUND AND RETROFIT STRATEGY OBJECTIVES

In 1989, the Whittier Narrows earthquake overturned rocker bearings, damaged shear keys and cracked a column at Bent 33. The structure has since been classified by Caltrans as Category I and is on the mandatory seismic retrofit list. The City of Los Angeles has been working on a retrofit program for the structure since then. A retrofit strategy including shear walls and steel restrainers was approved by the County of Los Angeles in the early 1990s. The City subsequently proceeded with the retrofit design. Responsibility for the retrofit is divided into two parts. The portion of the structure over the US101 freeway is owned by Caltrans, while the remaining structure is owned by the City of Los Angeles. In the mid 1990s Caltrans constructed a retrofit using infill walls from Bent 37 to the East Abutment. The City of Los Angeles retrofit design did not advance to construction due to concerns related to continuing concrete degradation of the bridge substructure due to Alkali Silica Reaction (ASR).

In late 2000, a material testing study was conducted to determine the current concrete properties and overall structure condition. This study revealed poor concrete condition of the structure and the possibility of a continuing chemical reaction that would further lead to the structure's deterioration. In January of 2002 an extensive material testing program revealed severe cracking throughout the structure due to ASR. The extent of internal cracking required a new investigation of possible retrofit schemes to ensure public safety and adequate performance of the structure.

W. Koo & Associates, under Consultant Agreement C-102112, conducted a material testing and survey of condition of the Six Street Viaduct in 2001. Results of the study indicated moderate to severe damage of the structure from ASR[1]. A total of 88 core samples were taken from the bridge for inspection and testing. Laboratory testing confirmed the presence of ASR and reduced strength capacities of the samples. The laboratory also found evidence of continuing chemical reaction possibly leading to further concrete degradation.

Based on the experimental material properties, seismic analyses of the As-Built condition were performed. A Seismic Retrofit Pre-Strategy Report was prepared by W. Koo & Associates and submitted to the City of Los Angeles Bureau of Engineering in June 2003[2] summarizing the findings. In the retrofit pre-strategy phase, linear and nonlinear analyses were conducted to determine seismic demands and capacities of the as-built approach spans of the structure. Seismic deficiencies of the as-built structure were determined from the analytical results[2]. The as-built analyses showed that the structure could collapse under the Maximum Credible Earthquake (MCE) event. This is evidenced by the high displacement Demand-to-Capacity (D/C) ratios of the structure under such loading. The analyses also showed that some columns of the existing structure could suffer shear failure under the MCE event due to concrete degradation. A seismic vulnerability study was also conducted in the retrofit pre-strategy phase showing high probability of collapse[2]. Summaries of the analytical models and analyses results of the as-built approach spans as well as the vulnerability study will be presented in this report for convenience. A seismic retrofit strategy task was subsequently authorized by Caltrans. The objectives of the retrofit strategy development are:

1. To complete the material sampling and testing program. In the retrofit strategy phase, additional concrete cores are extracted from the river piers and other columns and visually inspected for damage.
2. To complete analyses of the as-built structure. This includes seismic demand and capacity analyses of the steel arch spans.
3. To develop range of alternatives that will lead to an acceptable seismic retrofit.
4. To perform structural analyses to determine seismic demands and capacities of the retrofitted structure.
5. To develop replacement options for the existing viaduct.
6. To conduct quantities estimate and cost analysis of the different retrofit/replacement alternatives.
7. To discuss the seismic retrofit/replacement alternatives. This includes discussion of structural efficiency, cost and life expectancy of each alternative.

5. SUMMARY OF PREVIOUS WORK

5.1. SEISMIC SAFETY AND VULNERABILITY

When originally constructed in 1932, the Sixth Street Viaduct utilized then state-of-the-art concrete construction techniques. In order to cross the river, two spans were constructed using asymmetrical riveted steel arches. Literature written during construction indicates that the structure was designed for lateral forces equivalent to 10% of gravity loads, most probably to resist wind forces. Thus, most foundations consist of spread footings that do not have much larger plan dimensions than those of the columns. These small-size footings contribute little to overturning resistance. The column to footing connection was designed as a fixed connection. However, the footing lacks top reinforcement and thus cannot develop the plastic moment capacity of the column at the base. In addition, the column shear reinforcement is spaced at 12 to 18 inches, resulting in poor column ductility. Many of the construction details in 1932 lack ductility and strength required for seismic resistance. Because of the tall columns and the massive structure, the displacement demands are high. The poorly confined columns will fail before the high displacement demands are reached. Additionally, some of the bent caps at the cap-column interface are not sufficiently detailed to transfer the full plastic column moment. The vulnerability to collapse has kept the Sixth Street Viaduct on the County of Los Angeles and Caltrans Mandatory Seismic Retrofit List.

5.2. STRUCTURE WEAKENING AND DETERIORATION

A visual survey was conducted to assess the damage state of the Sixth Street Viaduct during the material testing program. Significant cracking was observed in most portions of the bridge. In general, severe surface cracks exist between Bents 12 to 30. Moderate to severe surface cracking was noted between Bents 1 to 11, whereas light to moderate surface cracking was noted between Bents 30 to 37. Samples of photos that show surface cracks of the viaduct are included in Appendix F. More photos can also be found in Reference 1.

Previous studies determined that Alkali Silica Reaction (ASR) is the main cause of concrete deterioration of the Sixth Street Viaduct[3]. ASR is caused by the presence of aggregate with high silica content. The silica reacts with the calcium, sodium, and potassium hydroxide alkalis in portland cement concrete to form a gel-like material that is potentially expansive. This gel undergoes extensive expansion in the presence of water or humidity (a relative humidity of 60 to 80 percent is usually required), resulting in development of cracks around the aggregate and expansion of the concrete.

Severity of cracking at the concrete surface could be evaluated from the visual survey. However, extension of the surface cracks inside the columns could be evaluated only using core samples taken from the concrete elements. Petrographic examination of concrete cores taken from the Sixth Street Viaduct was used to determine the presence of ASR and to verify that ASR was the cause of extensive concrete cracking observed in the visual survey.

In late 2000, the City of Los Angeles began a limited material sampling and testing program by extracting two core samples out of two columns from Bents 17 and 30 of the Sixth Street Viaduct to conduct petrographic and strength testing. The core samples exhibited wide cracks parallel to the surface at 4 to 6 inch depth intervals. Petrographic examination confirmed that alkali silica gel is present and is likely the main cause of concrete cracking. Results of the limited material testing were presented to the City of Los Angeles in a Report entitled "HBRR Study Report: Sixth Street Bridge over Los Angeles River"[4]. In October 2001, W. Koo & Associates (WKA), under contract with the City of Los Angeles, began a second more comprehensive phase of the material sampling and testing program on this bridge. WKA, in cooperation with Wiss, Janney, Elstner Associates, Inc. (WJE), performed a comprehensive sampling and testing program by collecting 88 core samples throughout the viaduct, including the Center Pier of the Los Angeles River spans. In addition, impact echo and pulse velocity tests were conducted by WJE to determine the presence of sub-surface cracks. The primary objectives of the material testing program were:

1. To confirm the extent of concrete deterioration by testing core samples taken from different structural members along length of the viaduct.
2. To determine the depth of cracking in representative elements of the bridge foundations, substructures, and superstructures.

3. To test representative core samples for compressive strength and elastic modulus for use in structural analyses.
4. To conduct petrographic testing to verify presence of ASR, identify the reactive aggregates, and assess the potential for future deterioration due to ASR.
5. To determine mechanical properties of the steel reinforcement used in construction of the viaduct. Samples of the steel reinforcement were taken from the viaduct's columns for this purpose.

Results of the material testing program indicated that there is significant cracking due to ASR throughout the Sixth Street Viaduct. This was evidenced from the petrographic tests, which indicated that most of the core samples collected from the columns and beams suffered from severe ASR due to presence of reactive aggregate and cement material[1]. Though the severity of cracking varies depending on location, cracking occurs throughout the entire length of the bridge. Cracking was observed in all bridge elements including the railings, deck, girders, bent caps, columns, and foundations. Severe cracking due to ASR was observed in numerous bridge elements. Based on laboratory observation of gel formation, it was also concluded that the reactive agents in the concrete remain highly reactive under moist conditions. Thus, ASR appears to be active and will likely continue to deteriorate the bridge.

Elastic modulus tests have shown a significant reduction of the Elastic modulus, E_c, when the samples under consideration showed significant ASR related damage[1]. The compression tests conducted on these samples have also shown weakening of the material in terms of reduced compressive strength[1]. More details of the experimental results can be found in Reference 1.

As mentioned earlier, visual survey of damage was conducted on all structural elements of the bridge. The visual survey rating of damage was compared to the damage rating of the core samples. A good correlation was generally observed between damage rating obtained from the visual survey of the existing structure and the core samples[1]. Based on visual survey of the structure and the core samples, different color codes have been assigned to different structural elements with varying degrees of concrete deterioration; these color codes have been presented in the retrofit pre-strategy report[2]. As part of the retrofit strategy phase, additional core samples were taken from the columns of the Sixth Street Viaduct to provide a minimum of one core sample per column. The color codes were updated based on inspection of damage in the extracted cores. Updated drawings with color codes that indicate state of concrete deterioration in all structural elements of the bridge are given in Appendix G.

Table 2 summarizes material properties for concrete with different degrees of deterioration, based on the laboratory tests on cores and the visual survey conducted in the pre-strategy phase. Compressive test results for uncracked or lightly distressed cores are very close to results of the moderately stressed cores. Thus, the compressive strength for concrete with light or moderate deterioration was taken as 3,500 psi (see Table 2). The values given in Table 2 for f_c' are the 10th percentile values. Six samples of the reinforcement bars used in construction of the viaduct were extracted from the column and tested to determine mechanical properties of the steel reinforcement. Yield strength, f_y, of the reinforcement bars was found to range between 41.5 ksi and 51.5 ksi. Ultimate tensile strength ranged between 61.6 and 80.7 ksi.

Table 2. Summary of the as-built concrete material properties in the Sixth Street Viaduct

Degree of Concrete Deterioration	**Compressive Strength (psi) (10th Percentile Value)**	**Elastic Modulus (psi) (Average Values)**
Light	3,500	3,360,000
Moderate	3,500	2,630,000
Severe	2,100	1,600,000

5.3. NEW SEISMIC RETROFIT STRATEGY REQUIREMENT

The Sixth Street Viaduct still remains on the List of the State Mandatory Seismic Retrofit Program. Previous analysis by WKA has shown that the seismic retrofit strategy of building infill walls between the columns, which was proposed by the City of Los Angeles Bureau of Engineering, would not be effective due to the poor material conditions[4]. A new seismic retrofit strategy needs to be developed that will consider the conditions and actual properties of materials determined from the material sampling and testing program[1].

6. ANALYSIS AND DESIGN CRITERIA

6.1. OBJECTIVES

Objectives of this study are:

1. Evaluation of the existing structure to determine vulnerabilities under the design earthquake loading (MCE event).
2. Development of retrofit strategies to eliminate potential for collapse under the MCE event.

6.2. EARTHQUAKE DESIGN CODES

Existing Elements: Caltrans MTD[5] 1994 20-4.

New Elements: Caltrans SDC[6] 1999, Version 1.1. Caltrans SDC[6] 2001, Version 1.3, will be used in the PS&E phase of the project.

Design of Concrete Elements: Caltrans Bridge Design Specifications[7], April 2000. The 2002 version of the Caltrans Bridge Design Specifications[7] will be used in the PS&E phase.

6.3. ANALYTICAL MODELS

Seismic Demands Analysis: Elastic dynamic and nonlinear time-history analyses using SAP2000[8] Nonlinear.

Seismic Capacity Analysis:

(1) Moment-curvature sectional analysis using XTRACT[9].

(2) Moment-curvature sectional analysis using ANDRIANNA[10].

(3) Nonlinear pushover analysis using SAP2000[8] Nonlinear.

(4) Soil-pile interaction analysis using LPILE[11].

6.4. GEOTECHNICAL DATA

Maximum Credible Earthquake (MCE): Magnitude 7.25 centered less than 1 kilometer away from site of the viaduct.

ARS curve: SDC 1999 Figure B.8 with 0.6g PGA and Soil Profile Type D. The ARS curve is modified to account for near-fault source and reverse type fault. Damping coefficient of 5% of critical damping was assumed to develop the ARS curve.

Input Ground Motions: Recommended input ground motions are used in nonlinear time-history analysis of the main spans (arch spans shown in Figure 2). The input ground motions used in the analysis were recommended by Earth Mechanics, Inc. (EMI) and are shown in Appendix H.

More details about geotechnical data can be also found in the geotechnical memos attached in Appendix H.

6.5. AS-BUILT MATERIAL PROPERTIES

As mentioned earlier, the concrete core samples taken from the existing concrete elements of the viaduct were tested to determine the concrete compressive strength, f_c', and the elastic modulus, E_c. Most of the material tests were performed in the pre-strategy phase of the project. Additional material sampling falls within the scope of the retrofit strategy development phase. Thus, additional concrete cores were extracted from columns of the river bents as well as from other columns from which no core samples have been taken in the pre-strategy phase. The material properties used in analyses of the as-built structure were based on the material test results reported in Reference 1 and supplemented by damage investigation of the recent additional core samples extracted in the retrofit strategy phase (49 additional core samples). Results of material tests can be found in Reference 1. The visual survey indicated that columns are the most severely damaged structural elements of the existing structure. Thus, core samples were taken from columns of all bents in the pre-strategy and strategy phases of the project except for columns in Bents 11-13 and the East River Pier due to difficult accessibility to extract core samples from these columns. As mentioned earlier, Appendix G includes drawings of the viaduct with color codes assigned to different structural elements with varying degrees of concrete deterioration. Color codes in the drawings shown in Appendix G are based on observed damage in all concrete cores extracted from the viaduct. Concrete material properties in the As-Built conditions are given in Table 2.

6.6. UTILITIES AND RIGHT-OF-WAY

Limited utility information is available at the retrofit strategy phase. Many of the utilities exist along the corridor have no as-built record, and most of them are believed to have been abandoned. Further site investigation is recommended in the PS&E phase to identify existing utilities at the site. Should utilities be identified, they will have to be relocated as required, and as permissible to accommodate construction of the new bridge.

Several industrial buildings are located immediately adjacent to the existing structure. Some of these buildings may have to be relocated in case of replacement of the existing structure. Adjacent to Mission Road a loading dock has been constructed below the existing bridge, which may also be impacted by retrofitting or replacement of the existing bridge. Other Right-of-Way (ROW) constraints may be imposed by railroad tracks underneath the bridge. Further ROW investigation should be conducted in the PS&E phase.

6.7. TRAFFIC HANDLING

The final seismic retrofit strategy should endeavor to minimize complete traffic closure, and to limit the total closure on the structure for only short duration on weekends and week nights. Traffic lanes may be reduced for extended duration, such as work on installation of new expansion joint seals or deck rehabilitation. Where necessary, full bridge closure will require traffic detours onto 4th street and 7th Street. These costs should be included in the cost estimates. Many businesses utilize the local streets below the Sixth Street Bridge. Of the three streets crossing under Sixth Street on the East side, only one should be closed at a time such that the other two can be used for detours. A detailed Traffic Management Plan (TMP) will be developed during the final PS&E phase of the project.

7. GEOTECHNICAL CHARACTERISTICS

The structure is located within 1 km of the Elysian Park Fault that is capable of producing a magnitude 7.0 earthquake with a peak ground acceleration of 0.6g. In addition, several other faults run nearby, increasing the probability of a large earthquake in the area.

The soil fill is immediately underlain by dense to very dense, native, alluvium comprising alternating layers of sands, gravelly sands and gravels. The alluvium is further underlain by firm and hard, dark gray clayey silt to the maximum depth explored, which was 175 feet. The fill soils are not expected within the Los Angeles River Channel. Based on borings done in 1997, the soil capacity for the spread footings is estimated at 10 ksf at service loads with an ultimate capacity estimated at 30 ksf. Pile driving may encounter resistance at relatively shallow depth. To improve the vertical and lateral pile capacity of the old foundations, steel piles or micro piles (as uplift piles) may be feasible. Field program will be required in the final retrofit PS&E phase to establish the subsurface characteristics of the viaduct, and to confirm the pile types and capacity.

The water table as measured by a nearby Los Angeles County monitoring well is approximately 150 feet below ground level. Liquefaction is thus considered a low potential in soil under the structure. Detailed information about geotechnical characteristics can be found in the geotechnical memos, which are attached in Appendix H. This includes data for the recommended ARS curve, recommended ground motions, probabilistic seismic hazard analysis, foundation bearing capacities and soil spring coefficients under vertical loads, p-y curves, soil spring coefficients for piles and logs of explanatory borings.

A soil contamination investigation was conducted in 1996. The subsurface investigation encountered no significant hydrocarbons with low hydrocarbon levels at some locations and low-moderate levels at other locations in the site. However, surface soil samples in other locations showed hazardous levels of diesel hydrocarbons. Hazardous levels of lead were measured in shallow soil of borings taken at bents near the railroad tracks. Based on findings of the soil contamination investigation report, excavated material during the retrofit will have to be tested, and properly disposed. A copy of the soil contamination investigation report was included in the HBRR study report[4].

Additional soil sampling and testing was also conducted in 2001 (see the geotechnical and environmental investigation report in Appendix I). Samples were taken from soil borings in the East Approach Spans and were tested for Total Petroleum Hydrocarbons (TPH) and metals. TPH associated with waste oils and other heavier fuels was detected in all soil borings, especially in shallow soil (see Appendix I). The tests also indicated elevated concentrations of lead in the surface samples collected at all soil borings. Soil in some borings was also impacted with Barium and Copper. Based on findings of this study, excavated material will have to be properly covered during excavation and be treated or disposed at a licensed facility. Impacts of contaminated soil may be managed by health and safety controls that include the use of appropriate personal protective equipment.

8. ANALYSIS OF APPROACH SPANS OF THE AS-BUILT STRUCTURE

Analyses to determine the seismic demands and capacities of the as-built structure were performed. Based on geometry and material deterioration, the structure was divided into the following four separate frames:

Frame 1: The first frame represents the West Approach Spans from the West Abutment to Bent 11.

Frame 2: The second frame represents the main spans over the Los Angeles River (see Figure 2). Frame 2 includes the West, Center and East River Piers and the steel through arch spans between the river piers. Analysis of the as-built main spans will be discussed in Section 9.

Frame 3: The third frame represents the East Approach Spans between Bents 12 and 22.

Frame 4: The fourth and last frame represents the East Approach Spans between Bents 23 and 37.

Figure 3 shows idealization of the as-built structure as modeled by SAP2000[8]. Frames 1 to 4 are shown in Figure 3. Each of the four frames was analyzed separately; however all the frames are shown in Figure 3 to provide an overall idea of the bridge global model. Analyses of the as-built approach spans were performed by WKA in the retrofit pre-strategy phase of the project[2]. Demands and capacities analyses of the as-built approach spans will also be summarized in this report for convenience. WKA, in cooperation with Dowell-Holombo Engineering, Inc. (DH Engineering), has performed analyses of the main spans (Frame 2) in the retrofit strategy phase. This section of the report is concerned with analyses of the approach spans (Frames 1, 3 and 4, see Figure 3), whereas analyses of the main spans (Frame 2, see Figure 3) will be presented in Section 9. This will be followed by discussion of deficiencies of the as-built structure in Section 10.

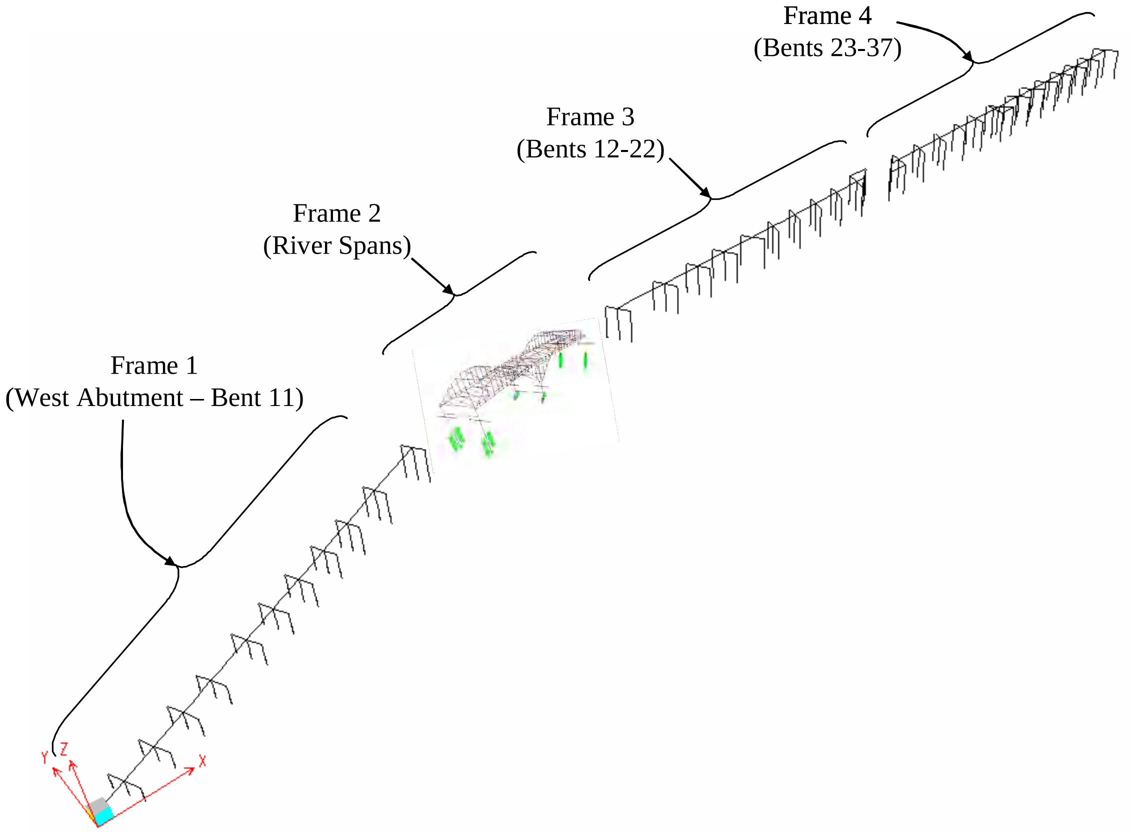

Figure 3. Global model of the Sixth Street Viaduct

8.1. SEISMIC DEMAND ANALYSIS

Simplified models of the structure were developed to investigate its seismic behavior. Analytical models were developed for Frames 1, 3 and 4 (see Figure 3) based on geometry and details shown on the 1932 as-built drawings. Seismic demands were obtained from elastic dynamic analyses using SAP2000. The recommended ARS curve (see geotechnical memos in Appendix H) was used in the dynamic analyses. Spectral method of analysis was employed with the Complete Quadratic Combination (CQC) procedure for statistical combination of maximum modal responses. For simplicity, each one of the frames shown in Figure 3 was assumed to act independently without the redundancy from interaction with the adjacent frames. Stand-alone analysis of each frame is expected to result in higher seismic demands than if all frames are combined and analyzed as one frame. Thus, stand-along analyses would result in more conservative results and would be preferred for design. Also, stand-alone analysis of each frame would substantially reduce computational costs. For these reasons, it was decided to conduct stand-alone analyses of the individual frames.

The analysis models utilized beam elements for all columns and bent caps, and one beam element along the superstructure to represent the concrete deck and longitudinal T-beams supporting the deck. Dimensions and shapes of columns and cap beams vary at each bent. Thus, varying sizes and cross sections prompted the need to develop detailed element properties for every column, bent cap and superstructure girder. The superstructure consists of concrete T-beams with variable depth along the span length. In each span, depths of the T-beams at their ends were also different. Thus, each span of the superstructure was non-symmetrical with respect to the midspan section. To account for these section variations, each frame element representing the superstructure in each span was divided into four segments of equal lengths. Section properties were calculated at span quarter points and the average properties were used in the superstructure beam elements.

Soil-structure interaction was modeled by means of three translational springs at the base of each column as shown in Figure 4. Stiffness of the vertical spring was obtained from the geotechnical data (see Appendix H). Interaction between soil and footings and columns was modeled by two lateral springs at the base of each column in two orthogonal directions (see Figure 4). Stiffness values of the lateral soil springs at each column were calculated based on the p-y curves (see the geotechnical data in Appendix H) and the as-built dimensions of the columns and footings. Stiffness of the lateral springs also accounted for friction between the footings and soil.

The as-built drawings showed that the footings do not have a top reinforcement mat. This indicates that footings will not be able to resist plastic moments that could develop at base of columns. Thus, no rotation restraints or rotational springs were modeled at base of all columns. In other words, the columns were assumed to have pinned ends at their bases.

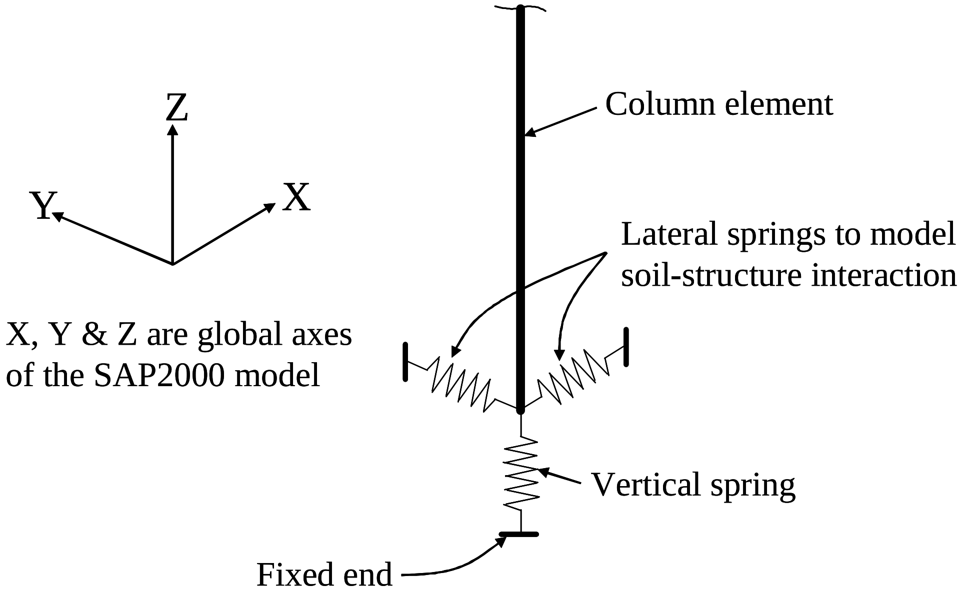

Figure 4. Modeling of soil-structure interaction in the elastic dynamic analyses models

Material properties used in the elastic dynamic analyses were based on average values for each frame. Results of the material sampling and testing program (Reference 1) provided the basis for average material properties used in the dynamic analyses. The material testing report furnished values for the concrete compressive strengths, f_c', and elastic modulus, E_c. Appendix G includes drawings with color codes to show where to apply the average values of f_c' and E_c. For analysis purpose and based on the material testing report[1], Frame 1 was assumed to have moderate degradation. Frame 3 was assumed to have severe degradation, whereas Frame 4 was assumed to have light degradation. Compressive strength and elastic modulus values used for different degrees of concrete degradation are given in Table 2.

Two elastic dynamic analysis cases were performed for each of Frames 1, 3 and 4. In each of the two cases, the recommended ground motion for the MCE (Maximum Credible Earthquake) event (see the ARS curve in Appendix H) was applied in two orthogonal directions along the global axes of the model (axes X and Y in Figure 3). In Case 1, response resulting from 100% of the longitudinal seismic loading (along the X-axis) was combined with response from 30% of seismic loading in the transverse direction (along the Y-axis). In Case 2, the response was determined for 100% of seismic loading in the transverse direction combined with 30% of seismic loading in the longitudinal direction. These seismic loading cases are based on the Caltrans Seismic Design Criteria (SDC)[6].

More details about elastic dynamic analyses models and results are given in the retrofit pre-strategy report[2], as well as in Appendices D and E of this report. The models are shown in Appendix E, whereas seismic displacement and shear demands on different elements of the as-built structure are summarized in Appendix D. These seismic demands are compared to capacities obtained from the capacity analyses.

8.2. CAPACITY ANALYSIS

A displacement-based approach was taken to determine capacity of the existing structure. Sectional analyses were performed to determine flexural capacities of different structural elements. Nonlinear pushover analyses using SAP2000 Nonlinear were performed to determine displacement capacities and plastic hinge mechanisms. Plastic hinges were modeled at critical sections in the columns, bent caps and superstructure elements. Data for moment-rotation and biaxial moment-axial force interaction surfaces were required as input to SAP2000 Nonlinear. These data were obtained from moment-curvature analyses of different sections in all structural elements of the bridge.

The program XTRACT[9] was used for the moment-curvature analyses. Material properties used in moment-curvature analyses of different sections were based on experimental results and the visual survey performed in the material testing report[1]. A good correlation was found between the visual survey of cracking in the bridge element and condition of the concrete cores extracted from the interior of the structural elements (see Appendix G). Based on damage rating of any structural element (light, moderate or severe), appropriate concrete properties were assigned to the element (see Table 2). During the material conditions survey, ASR cracking in the columns and bent caps was found to be most severe in the outer layers of the elements, whereas cracking in the interior was moderate. Thus, for severely deteriorated elements, the outer 18 inches of concrete was assumed to be of low quality with $f_c' = 2{,}100$ psi and $E_c = 1{,}600{,}000$ psi. Material properties of concrete with moderate deterioration were assumed for the inner core, with $f_c' = 3{,}500$ psi and $E_c = 2{,}630{,}000$ psi (see Table 2). Figure 5 shows the XTRACT model for one of the exterior columns in Bent 10. Columns in Bent 10 were found to have severe or moderate-to-severe deterioration. Thus, the outer 18 inches of the column core was assigned weak material properties, whereas properties of moderate deterioration condition were assigned to the inner core as shown in Figure 5. Figure 6 shows the XTRACT model for the superstructure at Bent 6. Figure 6 indicates that the superstructure, modeled in SAP2000 by one frame element, was comprised of the deck slab and all five girders supporting the concrete deck.

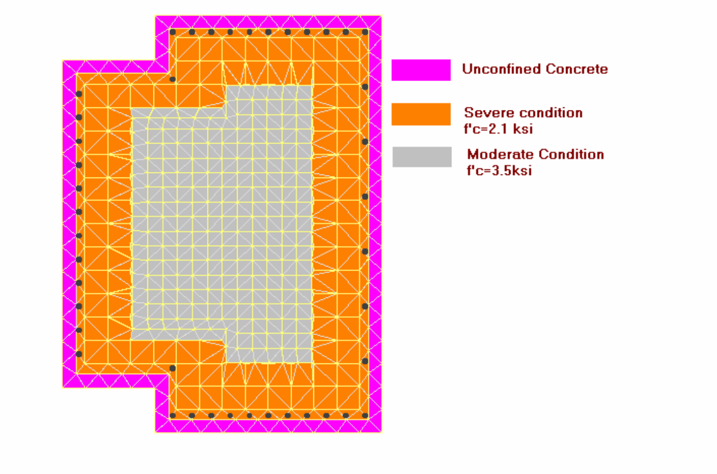

Figure 5. Model for moment-curvature analysis of an exterior column in Bent 10

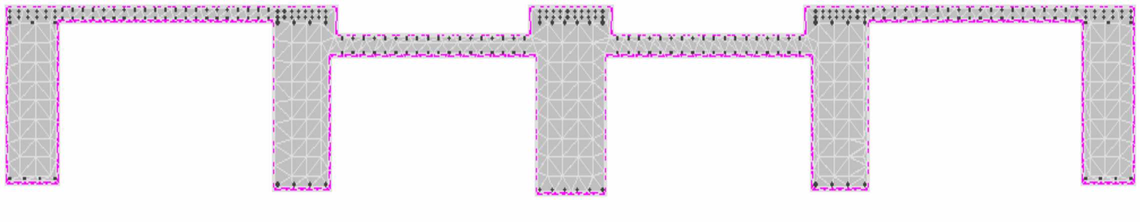

Figure 6. Model for moment-curvature analysis of the superstructure at Bent 6

For each pushover analysis model, the in-situ material conditions were applied element by element, creating an accurate model of the current bridge condition. To create the nonlinear pushover models such that they correspond to the linear elastic models, the same base SAP2000 model used in the seismic demand analyses (Section 8.1) was also used for the pushover analyses. Thus, direct comparison of the seismic displacement demands and capacities obtained from the elastic and nonlinear models, respectively, can be made. Once the geometry and material properties of the linear elastic model had been completed, nonlinear hinges were added at potential locations of plastic hinges. Beam elements with axial nonlinear hinge properties were also introduced to model the soil-structure interaction (see Figure 7); axial properties of the nonlinear hinges were determined based on the p-y curves (see geotechnical data in Appendix H) and dimensions of the columns and footings. Columns in Frames 1, 3 and 4 had lateral nonlinear soil elements that were spaced at 1 ft intervals in the vertical direction as shown in Figure 7.

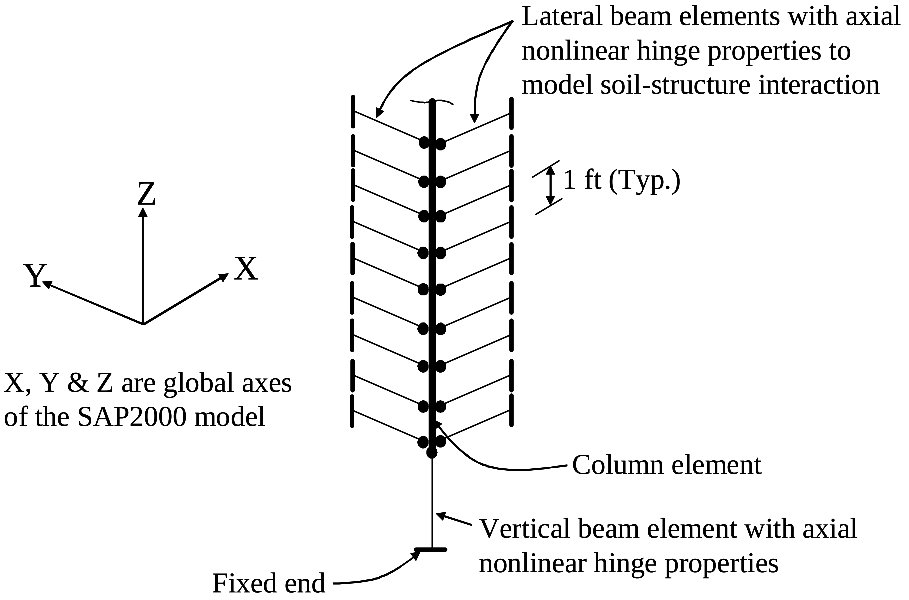

Figure 7. Modeling of soil-structure interaction in the nonlinear pushover analyses models

Nonlinear hinge properties were also introduced for the axial direction of the vertical soil beam element shown in Figure 7; the nonlinear hinge properties were also based on geotechnical data. The ultimate soil bearing pressure capacity is 30 ksf. The foundation vertical load capacity was calculated using the soil ultimate bearing capacity multiplied by a soil-footing contact area. The soil-footing contact area was determined at each column by projection of the column cross section to the footing base at an angle of 30° with the vertical direction. The pushover analyses showed that the vertical soil springs did not reach their ultimate capacities.

A 3-D model was created for each of Frames 1, 3 and 4 to determine displacement capacities of the structure under transverse and longitudinal lateral loadings. Each frame was subjected to static pushover in displacement control until failure was detected in some of the structural elements, usually the columns. At this stage the structure is determined to reach a “collapsible mechanism”.

The collapsible mechanism was determined based on flexural performance of the structural elements, or in other words shear failure was not simulated in the pushover analyses. However, it will be shown that some columns could experience shear failure before reaching the collapsible mechanism determined from the pushover analyses.

In Frame 1, a 3-D model was created for Bents 3 to 9 with the substructure and superstructure. The outer columns for the most part are un-symmetrically reinforced resulting in different moment capacities under positive and negative bending moments in the transverse direction as well as in the longitudinal direction. Thus, column interaction surfaces between axial forces and bending moments were calculated by XTRACT and the results were used as input for the nonlinear hinges in the SAP2000 model. The interaction surfaces were input for both the positive and negative bending directions. Similarly for the bent caps and superstructure, top and bottom reinforcement bars were not the same at any section, resulting in different moment-curvature performance under positive and negative bending moments. Also, reinforcement details were different along length of the bent cap. Thus, plastic hinges that were assigned to the bent cap in the nonlinear analysis models had different input data for positive and negative bending. Up to four plastic hinges were modeled in each bent cap since three columns exist in a typical bent. Pushover analysis was performed in the longitudinal direction of the bridge by application of increasing displacement at the superstructure level until the above-mentioned “collapsible mechanism” is reached. Similarly, transverse pushover analysis was performed by application of increasing transverse displacement at top of the columns. More details about the nonlinear pushover analyses can be found in Appendix E and the retrofit pre-strategy report[2]. These include undeformed shapes of the models, deformed shapes with locations of the plastic hinges that formed as a result of the pushover loads, bending moments, shears and axial forces. Maximum displacement capacities are also given in Appendix E. Figure 8 shows the base shear, or seismic force, versus top displacement in both longitudinal and transverse directions as obtained from the pushover analyses of Frame 1. Figure 9 shows the deformed shape of Frame 1 at longitudinal ultimate displacement capacity, or in other words when the collapsible mechanism was reached under longitudinal pushover loading. Locations of plastic hinges are also shown in Figure 9 as well as locations of first columns that would fail.

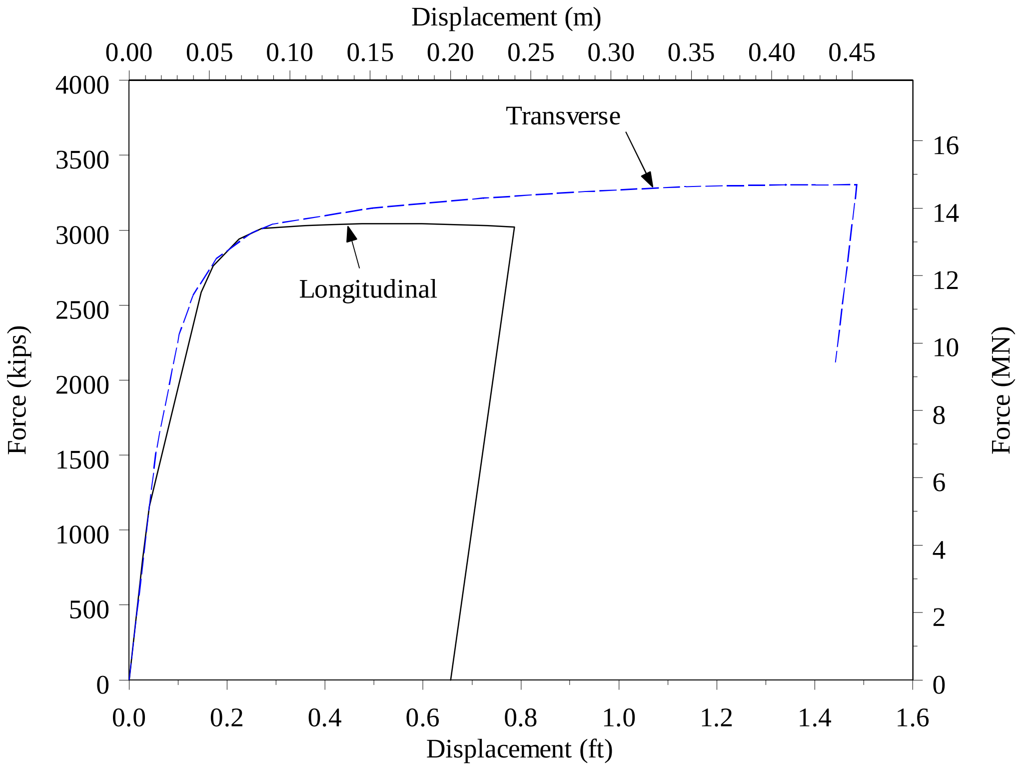

Figure 8. Longitudinal and transverse load-displacement response of Frame 1 (pushover analyses; displacement at deck level)

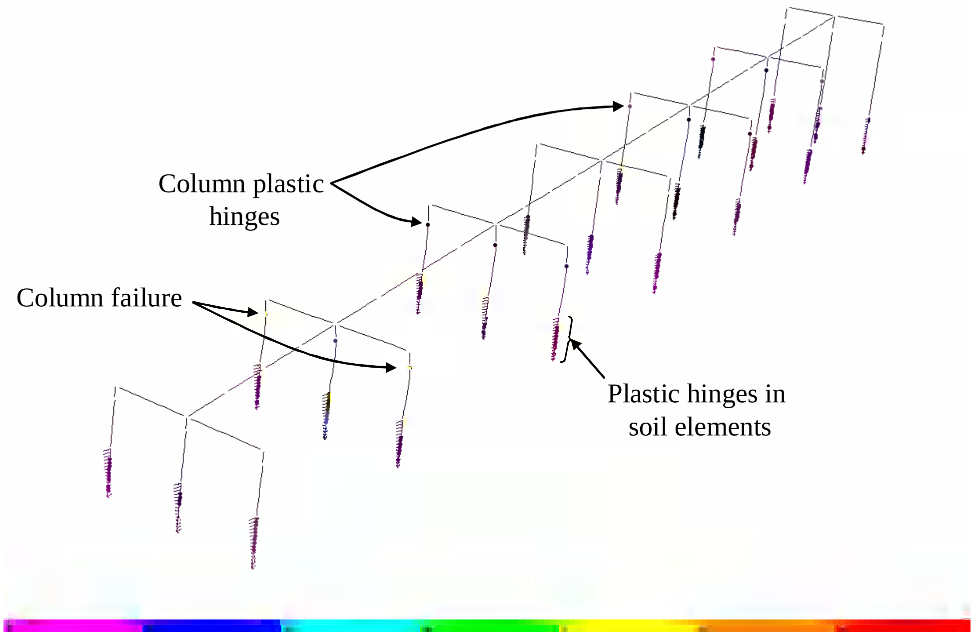

Figure 9. Deformed shape of Frame 1 at longitudinal ultimate displacement (pushover analysis)

As in Frame 1, longitudinal pushover analysis of Frame 3 was performed by application of increasing displacement at the superstructure level in longitudinal direction of the bridge. In transverse pushover analysis, lateral displacement was applied at top of the columns until a "collapsible mechanism" was reached. Failure occurred at some of the columns and bent caps. Deformed shapes of Frame 3 as well as locations of plastic hinges that formed under pushover loads are given in Appendix E. The pushover models included two types of bents. In Type A bents, exterior surfaces of the exterior columns are flush with the bent cap end as shown in Figure 10, whereas the bent cap extends beyond the column surface in Type B bents as shown in Figure 11.

Longitudinal and transverse pushover analyses of Frame 4 were done in a similar procedure to that of Frames 1 and 3. Deformed shapes of Frame 4 as well as locations of plastic hinges that formed under pushover loads are given in Appendix E.

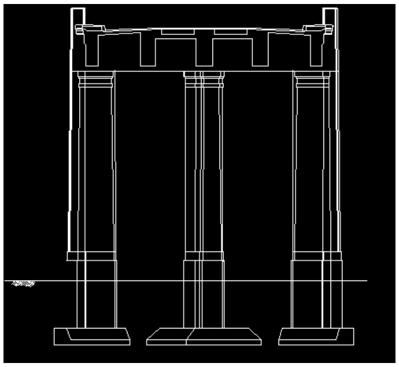

Figure 10. Bent 15 (Type A)

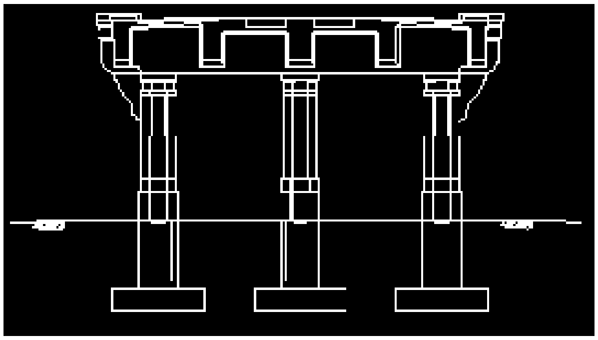

Figure 11. Bent 18 (Type B)

Shear capacities of the as-built columns were discussed in the retrofit pre-strategy report[2]. Columns in the West Approach Spans (Frame 1) are relatively short. Several columns in the as-built West Approach Spans will experience shear failure in the MCE event[2]. The shear capacity was roughly estimated during the retrofit pre-strategy phase by using a shear strength, $v_c = 2\sqrt{f_c^{'}}$ (lb and in. units) and assuming that only the concrete core is effective in resisting shear[2]. This means that for severely deteriorated columns, the outer 18-in. layer of the column was not considered effective in resisting shear. Also, shear resistance provided by the stirrups was ignored because of the following: (1) relatively wide spacing and poor detailing of stirrups, which may not have adequate anchorage by means of hooks and will not be able to develop their yield strengths, and (2) the outer concrete layer in most columns is severely damaged, which renders effectiveness of the stirrups questionable.

According to Attachment B of Memo 20-4 in Caltrans MTD[5], shear resistance provided by stirrups for rectangular (or non-circular) columns should be ignored if spacing of stirrups is equal to 12 inches or more, which is the case in the Sixth Street Viaduct columns. However, this may be conservative in case of the Sixth Street Viaduct since effective depths of columns substantially exceed the above-mentioned MTD stirrup spacing limit. Shear demands were obtained from the pushover analyses rather than from plastic moment capacities of the columns as mentioned in the MTD[5]. This is because plastic hinges will not develop in all columns and pushover analyses provide less conservative but more realistic values of shear demands. Results given in the retrofit pre-strategy report indicate that many columns will experience shear failure during the MCE event. Shear failure is brittle and will result in catastrophic collapse of the structure.

8.3. SEISMIC DEMANDS AND CAPACITIES

Summary of the seismic displacement demand-to-capacity (D/C) ratios in columns in some of the bents in the approach spans are given in Table 3. A D/C ratio less than 1.00 indicates that the capacity exceeds the maximum demand, whereas a D/C ratio more than 1.00 indicates that the seismic demand from the MCE event would exceed the capacity and failure of some structural elements would occur. The D/C ratios given in Table 3 clearly indicate that some structural elements, mostly columns, in the as-built structure would experience failure during the MCE event. Failure of these structural elements could result in collapse of the structure. It should be noted that displacement capacities were obtained from nonlinear pushover analyses, which modeled plastic hinging assuming no shear failures. As discussed below, some of the columns will experience shear failure before the displacement capacity from pushover analyses is reached, which will increase the D/C ratios above the values given in Table 3[2].

Table 3. Summary of seismic displacement D/C ratios in the approach spans

Frame #	**Displacement D/C Ratio**			
	Transverse Direction		**Longitudinal Direction**	
	D/C	**@ Bent #**	**D/C**	**@ Bent #**
1	**1.33**	8	**1.17**	4
	1.21	9		
3	**2.22**	19	0.38	16
	11.90	18	**4.40**	18
	(Shear failure)		(Shear failure)	
4	**1.95**	34	**2.44**	26

It was found that the demand shears in some of the columns would exceed their shear capacities, which were obtained as discussed earlier in Section 8.2. Assuming that the displacement capacity is the displacement when shear failure occurs, the displacement D/C ratio for the columns in Bent 18 (Frame 3) was found to be approximately 11.9 under transverse seismic loading. Similarly, the displacement D/C ratio for the same column under longitudinal seismic loading was found to be approximately 4.4 (see Table 3). This indicates that some of the columns could experience shear failure at displacements less than those obtained from the pushover analyses. Summary of the shear demands and capacities was presented in the retrofit pre-strategy report[2] and is given in Appendix D of this report for completeness. The D/C ratios given in Table 3 and Appendix D indicate the immediate need to retrofit the existing structure.

9. ANALYSIS OF MAIN SPANS OF THE AS-BUILT STRUCTURE

Analysis of main spans of the as-built structure (Frame 2, see Figure 3) was conducted in the retrofit strategy phase. This section is concerned with analysis of the main spans. The analytical models will be described and major results will be presented in this section.

9.1. SEISMIC DEMAND ANALYSIS

The main spans (arch spans, Frame 2 in Figure 3) were modeled in SAP2000 Nonlinear, Version 8. Steel arch members (rib, tie, hanger, bracing), steel transverse floor beams and concrete pier columns were modeled with beam elements. The concrete deck was modeled with 4-node shell elements that were tied to the transverse floor beams and represent the deck's in-plane stiffness through diaphragm action. Soil-structure interaction was represented by both translational and rotational springs at the foundations and translational springs along the buried height of the columns. Spring stiffness values were determined using the LPILE[11] program and soil layer information (nonlinear *p-y*, *t-z* and *q-u* curves, see Appendix H) provided by EMI. Translational spring stiffness values were found by applying a shear force to the top of the pile while maintaining zero rotation at the pile head.

Due to the complexity, scale and architectural significance of this bridge, nonlinear time-history analyses were conducted using ground motions that were developed by EMI. Three components of input ground motions were given, representing longitudinal, transverse and vertical time-history accelerations. Although unique ground motions were provided for each support along the bridge due to the length of the viaduct, the given input motions generated by EMI were identical at the three river piers. This is due to the relatively short length of the two river spans compared to the total length of the viaduct. The input ground motions are given in Appendix H. In addition to nonlinear time-history analysis, linear elastic time-history and modal analyses of the main spans were conducted.

A stand-alone analysis represented the arch spans having no interaction with the approach frames. In a separate analysis, approach frames were included as boundary elements on either side of the primary model. Adjacent approach spans (Frames 1 and 3, see Figure 3) were modeled as SDOF nonlinear elements with appropriate stiffness, strength, mass and damping. It was found that the added boundary frames slightly reduced displacements and demands of the main spans, resulting in less conservative results. Thus, the stand-alone analysis is the design case for the two arch spans. Nonlinear geometry and material behavior were included in both analyses. Significant nonlinear response includes plastic hinging of piers and arch ribs, as well as gapping and crushing of soil along the bottom regions of the piers, with about 33 ft of soil cover at the West and East River Piers and 15 ft of soil cover at the Center River Pier.

9.1.1. Analysis Models:

As discussed above, the global structure is modeled with a combination of beam and shell elements, as well as ground-to-node spring elements. Shell elements were used to model the concrete deck, providing in-plane stiffness through diaphragm action. All of the steel members were modeled as beam elements with nonlinear moment-rotation hinges at both member ends for elements that exceed yield. For arch rib members that vary in depth, beam elements with non-prismatic properties were used. A non-prismatic member definition consists of the member length, material properties and section definitions at left and right sides of the beam.

Initially, linear elastic time-history analyses were conducted to verify the model and to determine which regions of the model required nonlinear elements. Nonlinear moment-rotation hinges were added to provide biaxial bending and axial load interaction, or coupling, at the member ends for all members that exceed nominal moment. The SAP2000 built-in AISC moment-axial load interaction curves were used for moment-rotation plastic hinges of the structural steel members after finding that they agreed very closely with results from more detailed moment-curvature analyses (see Figure 12). This approach of selectively adding nonlinear behavior to the model greatly simplifies the analysis and significantly reduces computational times compared to including nonlinear behavior for all steel elements.

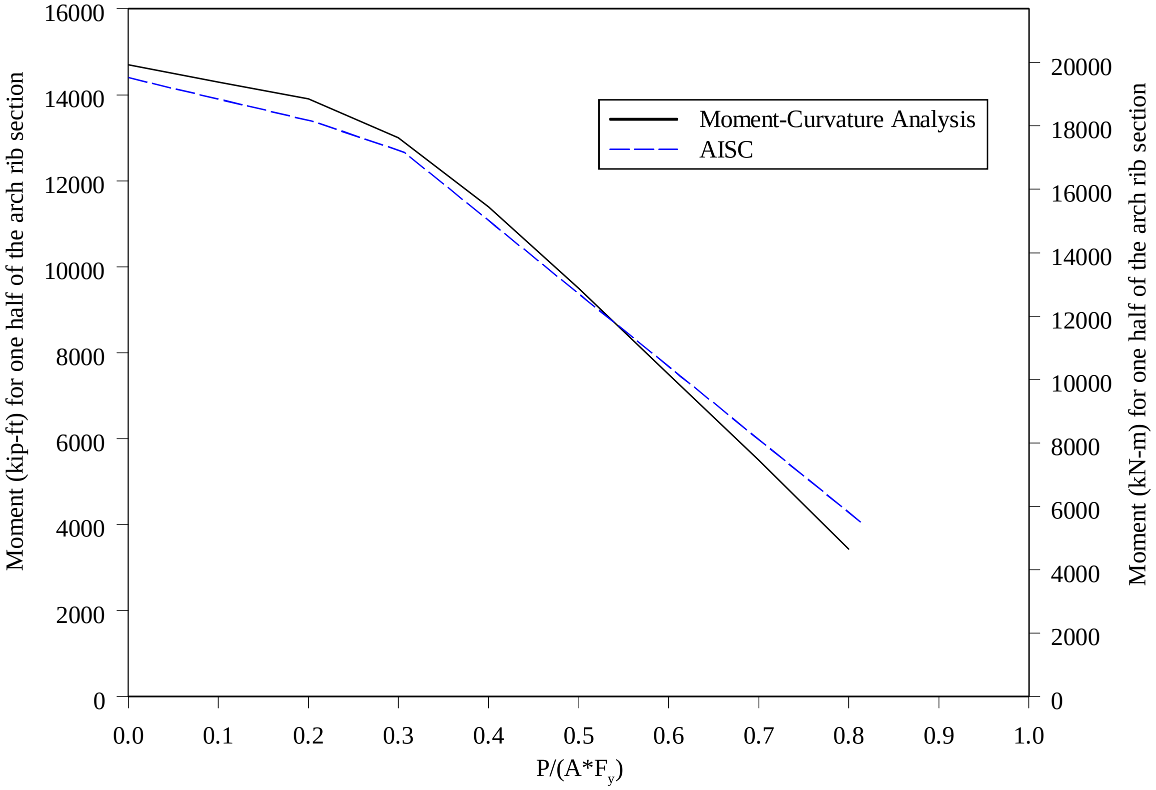

Figure 12. Moment-axial load interaction curves

9.1.2. Breakout Models:

As presented in the following, several breakout (detailed local) models were required in order to calibrate the behavior of various parts of the arch spans for inclusion in the global bridge model. Breakout models also allow modification of parameters to increase computing efficiency before they are added to the global model. This is especially important in the modeling of column plastic hinges, where very stiff axial load members are used to represent the concrete in compression and the primary reinforcement in tension and compression.

Laced Steel members: Many of the steel members are laced, with the lacing between primary steel plates providing shear capacity and stiffness, forcing members to behave compositely. Direct modeling of lacing of the built-up sections in the global model was beyond the scope of this project, and thus laced members were modeled as equivalent beams. One concern was that the lacing would not provide full composite action between the primary steel plates, resulting in additional flexibility, or shear deformation, that would not be captured in the model by beam elements with properties based on total area of flanges.

To determine the added shear flexibility that the lacing contributes to fully composite action for a typical laced member, a breakout model was developed of a cantilever member subjected to linear bending. The breakout model is shown in Figure 13. From this breakout model it was found that a typical laced member from the arch spans is 30% more flexible than the same member with fully composite behavior based on the flange areas. In the SAP2000 software, the local beam element stiffness matrix includes separate flexure and shear terms, with the amount of shear flexibility controlled by a shear area multiplier. For typical rolled or built-up members the shear area is based on the web area and most of the deformations are associated with flexure. However, with laced members the additional shear flexibility is significant and should be included in the model. A dimensionless shear area multiplier was determined based on analyses of the breakout model shown in Figure 13 and a second similar breakout model comprised of a single beam element. This shear area multiplier was applied to beam elements used in the global model of the main spans. Thus, all of the lacing was included in the breakout model and represented by a single beam element in the global model, but with the correct flexure and shear behavior of the detailed laced member. The same dimensionless shear multiplier was used for all laced members, providing additional shear flexibility to the beam members associated with deformation of the lacing. The above-mentioned second breakout model was developed of a single beam element, with shear modification, to confirm that its behavior under flexure, shear, and a combination of both flexure and shear, agreed with the more detailed laced model. Hand calculations were also used to verify the results for pure shear, pure bending and for linear bending. It should be noted that under pure bending the shear modification has no effect on the response of the member, with identical results to a fully composite section, as expected.

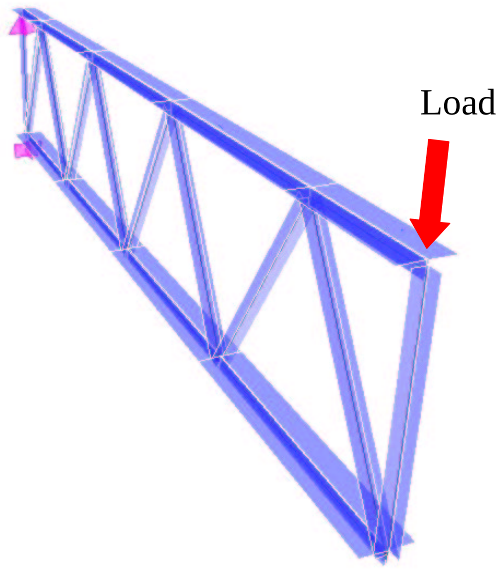

Figure 13. Breakout model of a laced steel member

Concrete Plastic Hinges: Column plastic hinge elements at the bottom of the West and East River Piers were not required for as-built analyses under longitudinal and transverse seismic loading because lap-splice failure in the column reinforcement, which will be discussed in a later section, would result in pinned column bases. Due to the existing infill wall at the lower portion of the Center River Pier, column plastic hinges can develop only under longitudinal seismic loading at the base of the columns and under transverse loading above the infill walls and at the top of the columns. Therefore, biaxial bending at locations of potential plastic hinges is not possible.

A breakout model was required to calibrate plastic hinges for the RC columns. At the time of running the models for this project, SAP2000 Nonlinear did not have a realistic moment-rotation plastic hinge element for RC members. However, at the time of writing this report, the latest interim version to SAP 2000, Version 8, indicates that some form of the Pivot Model has been added to the program, although all of the features are apparently not yet installed. This is encouraging and will probably be used in final design of the Sixth Street Viaduct as the Pivot Model realistically captures the hysteretic response of RC plastic hinges. The Pivot Model was developed by Dr. Dowell of DH Engineering.

The most significant nonlinear response affecting the behavior of the arch spans and deformations of the arch ribs is the hysteretic nature of plastic hinges that develop at the base of the concrete river pier columns. Reinforced concrete columns have definite stiffness degradation and pinching characteristics, with increasing ductility, that are well documented from large-scale structural testing and analysis. Currently, this behavior cannot be properly modeled in nonlinear time-history analyses using the standard SAP2000 Nonlinear plastic hinge elements. Thus, a series of elements were combined in such a way to mimic the nonlinear cyclic behavior of RC plastic hinges. In order to model RC plastic hinges as accurately as possible, a detailed model of a single cantilever member with a plastic hinge at the base was developed and loaded with various input time-history base motions to large ductility. One base motion used was a sine curve that increased linearly in magnitude. This resulted in symmetric displacement cycles with increasing ductility so that the shape and behavior of the hysteresis loops could be examined. It also allowed fine-tuning of the various parameters and convergence tolerances to produce the fastest, converged, solution for RC plastic hinge models. Such an effort increased the computation speed of the global model by a factor of 10 or more. This was an important consideration in developing the global model with plastic hinge elements included, allowing multiple analyses to be conducted within the available time.

The column plastic hinge model represents the steel reinforcement and the concrete as two vertical lines of elements. This 2-D plastic hinge model is used to model regions that will have plastic hinging in only one direction. An example of this is the behavior of plastic hinges that form at the base of the Center River Pier columns for the as-built structure under longitudinal seismic loading, the plastic hinges that form under transverse loading at top of columns in all three river piers and plastic hinges that form under transverse loading above the infill wall in columns of the Center River Pier. Rather than an elastic-perfectly-plastic cyclic moment-rotation response, a more realistic behavior was required to model the concrete plastic hinges. At the end of the member of interest a rigid member, which has no mass, extends perpendicularly in both directions to the vertical lines of elements that provide nonlinear axial behavior and allows for coupling of flexure and axial load.

The same approach can be used in 3-D, allowing proper coupling for biaxial bending and axial load. However, as discussed previously, the nonlinear analyses required plastic hinging in only one direction at a given plastic hinge location and, therefore, only the 2-D plastic hinge element is required and discussed here. Specifically, RC plastic hinge elements are needed in the longitudinal direction at the base of the Center River Pier columns and in the transverse direction at the top of the columns (at soffit of the bent caps) and just above the infill wall in columns of the Center River Piers. Figure 14 shows a schematic of the breakout model for RC plastic hinges. It should be noted that the vertical distance between the horizontal rigid members shown in Figure 14 is very small; the figure is not drawn to scale in order to easily visualize the breakout model.

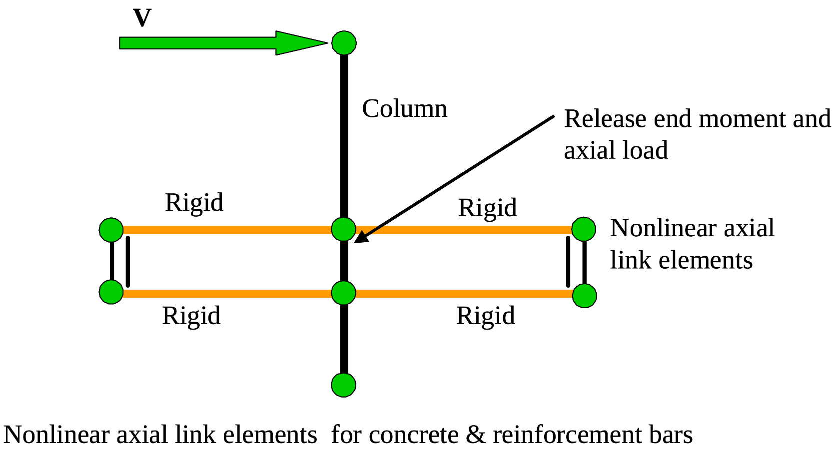

Figure 14. Breakout model for concrete plastic hinges

Nominal and ultimate moments and curvatures were determined from moment-curvature analyses using ANDRIANNA[10], with limiting curvatures found at a concrete compressive strain for the unconfined concrete of 0.005. Although the degraded concrete had reduced strength it also had a significantly reduced modulus of elasticity, making the limiting compressive strain of 0.005 a realistic value. For the idealized plastic hinge model the total reinforcement area, A_{stot}, was divided in two, with the equivalent tension and compression reinforcement, $A_s = A_{stot}/2$ concentrated at the two locations of the nonlinear link elements shown in Figure 14. Spacing between equivalent tension and compression steel elements or fibers is designated as w, with total tension force for the plastic hinge model at nominal moment, $T_{sn} = A_s f_y$, where f_y is the reinforcement yield strength. The distance w between steel fibers is found from statics in terms of the applied axial load P and nominal moment M_n of the concrete section (see Appendix E).

The equivalent rebar elements are modeled as 1-D link plasticity elements. A single tension steel element and a single compression steel element are used. Concrete elements are also provided at the same location as the two steel elements. While steel elements are connected to ground (for a plastic hinge model at column base), concrete elements are connected to gap elements that allow compression only, recognizing minimal tension capacity of cracked concrete. The stiffness values of the concrete, K_C, and gap, K_g, elements should be equally balanced to increase computing efficiency for the plastic hinge elements. Since concrete and gap element stiffness terms act in series, the total axial stiffness K_T for the two of them acting together is given as:

$$K_T = \frac{1}{\frac{1}{K_c} + \frac{1}{K_g}}$$

Thus, to produce an efficient nonlinear analysis scheme, while giving the correct overall concrete compression stiffness, both the gap and concrete stiffness terms should be set equal to twice the actual stiffness of the concrete. In flexure, only that portion of the section that is on the compression side of the neutral axis is contributing to the concrete response, and thus the concrete area attributed to the simple fiber model must recognize this. As presented below, a simple hand solution to determine the concrete area is warranted since the steel response dominates the plastic hinge behavior, so long as there is a realistic compression region at the right location to pivot about. Based on plastic analysis of the equivalent section, the tension and compression axial forces of the steel fibers cancel, as they have the same capacity, and so the compression force that the concrete resists, C_{cp}, is simply the axial load P on the section. This is shown in the following as:

$$P + T_{sp} = C_{sp} + C_{cp}$$

Therefore,

$$C_{cp} = P$$

At ultimate, the compression strength is found from the Whitney block assumption given in the ACI Code[12], with an average stress of $0.85f_c'$ acting over the concrete area, A_c. Thus, the concrete area is found as:

$$A_c = \frac{P}{0.85 f_c'}$$

Stiffness terms are based on an axial stiffness of $K = AE/L$, with length L being the equivalent plastic hinge length. For the columns on this project the plastic hinge length is defined by strain penetration of the primary longitudinal reinforcement extending in both directions from the critical section. The initial stiffness of the steel elements is based on the modulus of elasticity, E_s, steel area, A_s, and plastic hinge length, l_p. As discussed above, each of the two steel fibers contain half of the total steel area and the yield moment is calibrated to nominal moment found from moment-curvature analysis by determining, from statics, the distance w between fibers. The plasticity behavior in the SAP2000 model follows a Menegotto-Pinto type of strain-hardening curve that approaches a second modulus of elasticity as an asymptote, and has a reasonable representation for cyclic steel behavior. This second modulus was calibrated so that at ultimate curvature and rotation of the plastic hinge model, the ultimate moment found from moment-curvature analysis is reached.

A potential problem with modeling plastic hinges with only two lines of fibers (statically determinate) is that as the concrete member crushes, additional compression capacity is not available to balance any increase in axial load that may develop from frame action in the transverse direction and from vertical earthquake excitation. Of course the actual section has additional compression capacity from concrete that can be utilized with only a slight shift of the section neutral axis. If post-crushing stiffness is not given to the concrete fiber then large compressive strains will develop, with steel strain excursions of similar amplitude for both tension and compression. This is not consistent with actual behavior of reinforced concrete plastic hinge response where tension strains are far greater than compression strains due to the off-center location of the neutral axis.

Three solutions to the above-mentioned problem have been considered and are discussed in the following. The first approach is to use two concrete springs at the same location, one nonlinear and the other linear elastic with a relatively small stiffness. The sum of the two initial stiffness values must equal the total concrete stiffness. A second approach, which has the same effect, is to merge the two springs into one spring, and provide a post-elastic stiffness that is equal to the stiffness of the relatively weak elastic spring in the prior arrangement. This second stiffness, after yield, provides the increase in compression capacity needed when varying axial loads are present. The third approach is to provide all of the initial compressive concrete stiffness to the gap element, thus reducing the number of required elements at a plastic hinge to 4 (see Figure 14); the four elements are comprised of two steel elements and two concrete elements, represented by compression stiffness of the gap elements. In this last scheme the concrete elements remain linear in compression. It was found that good overall hysteretic response was possible with this simplified scheme and that the solution time was superior to the other methods investigated. Thus, the third approach was finally chosen for the global bridge modeling. In other words, two nonlinear link elements exist at each end of the rigid members shown in Figure 14; one element to represent steel reinforcement in tension or compression and the other element (gap element) to represent concrete in compression, with no tensile force capacity. Figure 15 shows a comparison of moment-rotation response of the plastic hinge at the base of the column in the Center River Pier, obtained from a breakout model (hysteretic response) and from a detailed moment-curvature analysis using ANDRIANNA[10] (envelope curve). The comparison indicates that the plastic hinge model realistically captures the behavior of concrete plastic hinges.

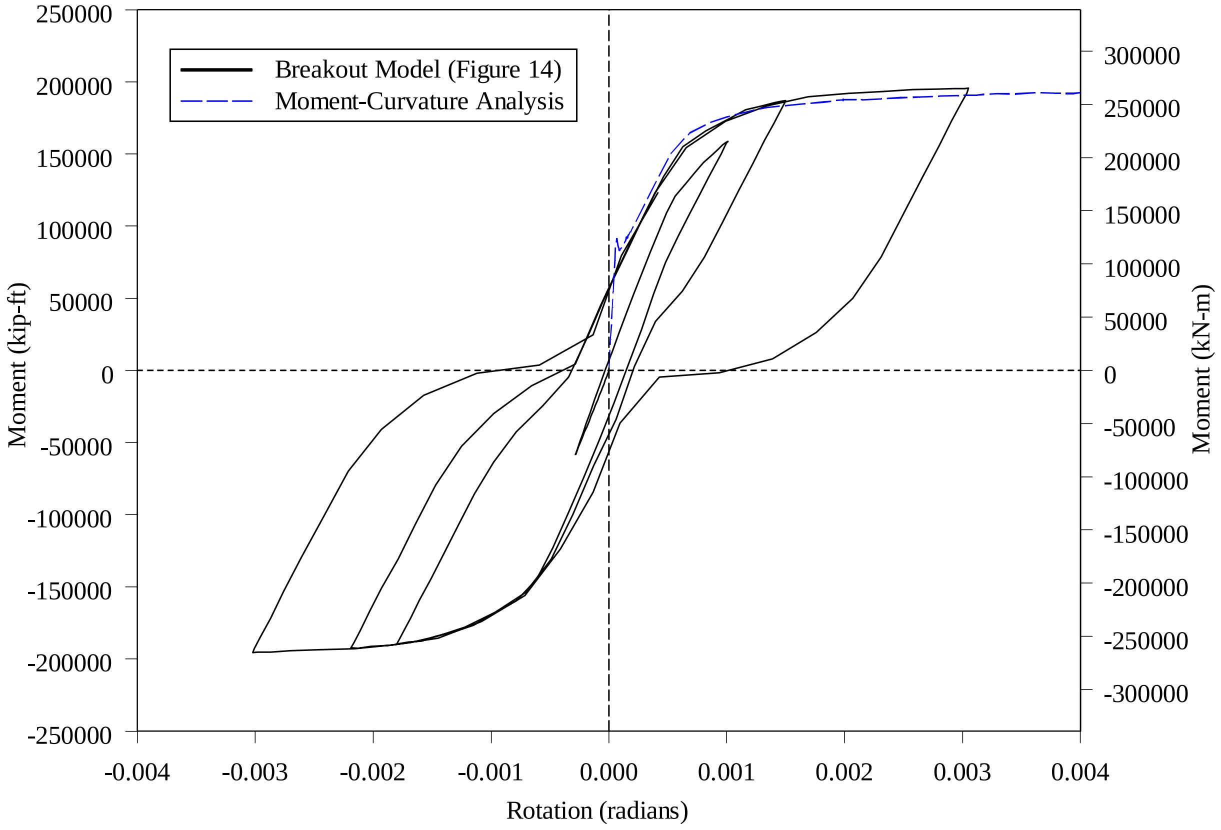

Figure 15. RC Plastic hinge response from the breakout model and moment-curvature analysis

<u>Gusset Plates and Rivets:</u> Steel members are connected to each other by gusset plates and rivets. Additional flexibility of the members, associated with deformation of the connections, was based on local rivet slip from holes that are oversized by 1/16 in. It was assumed that the total slip occurs at first yield, allowing a dimensionless multiplier to be included in the global model, based on a cantilever member derivation. Each member of the global model was given its own multiplier. Since some rivets will be positioned on the right side of the hole and others on the left side, only one-half of the oversize was used in the calculation. This resulted in additional flexibility of between 7% and 30% for the different members. Note that the strength of the gusset plate connections was not determined to be critical. No additional member flexibility was provided for deformation of the gusset plates since the members were modeled to the centerline of connecting members, providing reasonable member-end deformations.

9.1.3. General Analysis Approach and Model Description:

Initially, the global bridge model was developed with only linear elastic elements. For the as-built analysis, columns at the West and East River Piers were pinned at their bases to represent lap-splice failure when plastic hinges begin to form and compressive strains exceed 0.002, followed by reversed loading[13]. Prior to running nonlinear time-history analyses it was important to fully develop a linear elastic model of the structure and conduct elastic time-history analyses to look for any anomalies in the behavior of the model. Modal analyses were also performed and compared to elastic time-history results in terms of deformed shapes and natural periods. From these models, discrepancies and errors in boundary conditions, stiffness and mass will often be identified and corrected before adding any nonlinear elements to the model.

After validation of the elastic model, nonlinear soil springs, coupled with nonlinear gap elements, were included over the depth of the buried portions for the columns of the three river piers. Buried depths are 33 ft for the West and East River Piers and 15 ft for the Center Pier. Spring stiffness values were given for both longitudinal and transverse directions. Gap elements were provided to let the column move away from the soil after crushing it in one direction, resulting in increased spaces between the column and the surrounding soil as the motion continues. Following initial nonlinear time-history analyses, RC plastic hinge models (see Figure 14) were added at the locations indicated in Figure 16.

With these limited number of nonlinear elements included, nonlinear time-history analyses were conducted and design check features of SAP2000 were used to determine which steel members exceeded yield. For all steel elements that yielded, plastic hinges were provided at the member ends and new nonlinear time-history analyses were conducted. Coupling between biaxial bending moments and axial load was included (Figure 12).

Figure 16 shows the global model used in seismic demand analysis of the main spans. Shell elements representing the concrete deck are shaded to distinguish them from beam elements used in modeling the steel arch ribs, tension ties, floor beams, bracing, hangers, concrete columns and bent caps. The nonlinear spring elements that are coupled with gap elements to represent soil-structure interaction are also shown in Figure 16 at the columns' bases. Figure 16 also shows locations of the RC plastic hinge models (see Figure 14) in the global model. As discussed earlier, these RC plastic hinge elements are needed in transverse direction at top of columns in all three river piers as well as in columns of the Center River Pier above the existing infill wall. Plastic hinge elements are also included in the longitudinal direction at base of the Center River Pier. Figure 17 shows the extruded SAP2000 model of the main spans, which gives a clear indication of the size of different structural elements.

In analysis of the main spans, concrete strength and elastic modulus were taken as 3,200 psi and 1,500,000 psi, respectively. These values were taken directly from material tests of sample cores extracted from the Center River Pier. Yield strength of reinforcement was taken as 44 ksi. Yield strength of structural steel was assumed to be 36 ksi (Grade A36 Steel).

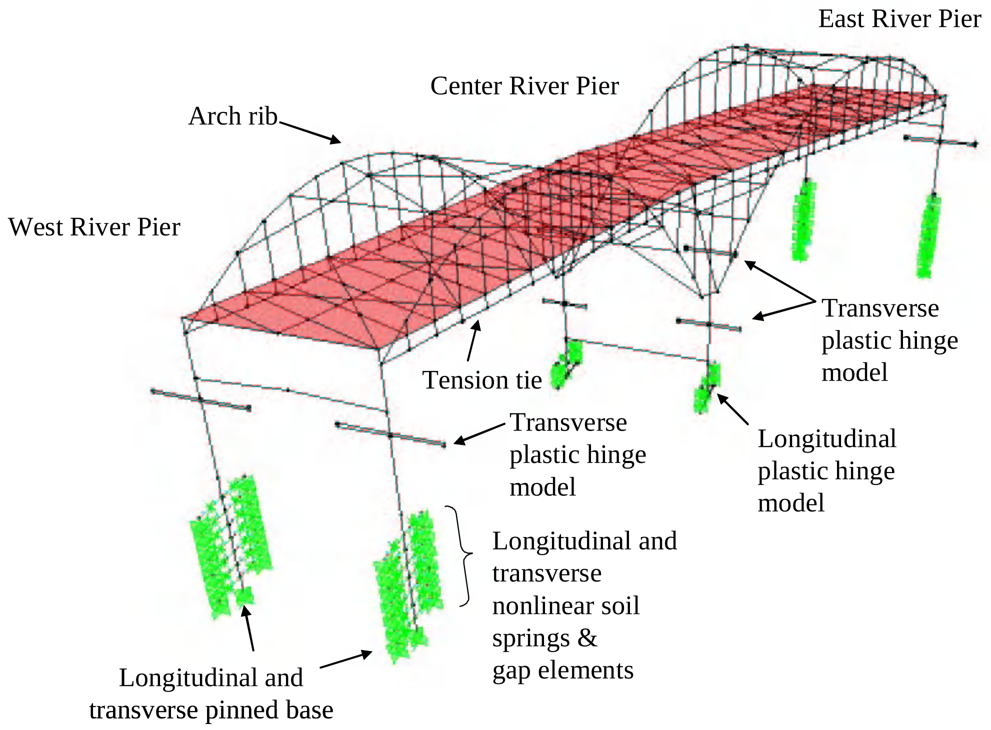

Figure 16. Global model for arch spans of the as-built structure

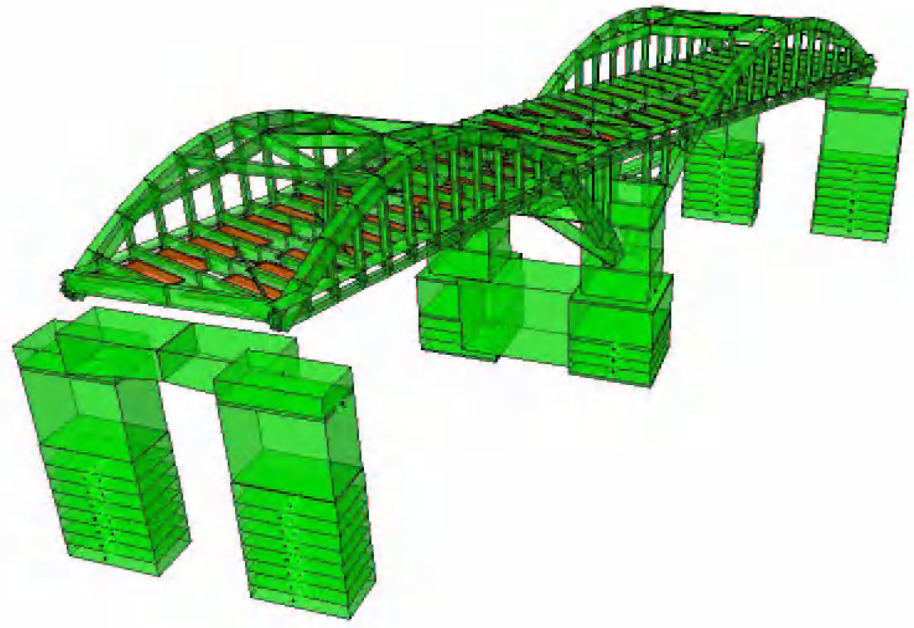

Figure 17. Extruded SAP2000 model for arch spans of the as-built structure

9.1.4. Results:

Modal analysis indicates that the first natural period of the as-built structure is 0.422 seconds. Figure 18 and Figure 19 show, respectively, the longitudinal and transverse displacement time-history results for the first 15 seconds of the earthquake, which includes the strong motions. Figure 18 indicates that longitudinal top-of-column displacements are nearly equal in all bents with maximum displacements of approximately 1.18 ft, 1.14 ft and 1.13 ft for the West, Center and East River Piers, respectively. As mentioned earlier, lap-splices of longitudinal reinforcement at the column bases for the West and East River Piers are inadequate and will fail prematurely; thus, these columns were assumed to have pinned ends. The analysis also indicates that plastic hinges will form at the bottom of the Center River Pier columns. Thus, the as-built main spans would experience large longitudinal seismic displacements and the triangles designated “OAB” in Figure 2 would tend to rotate as a rigid body. This rotation results in overloading of the arch ribs at their connections with the tension tie members (i.e., Points “A” and “B” in Figure 2).

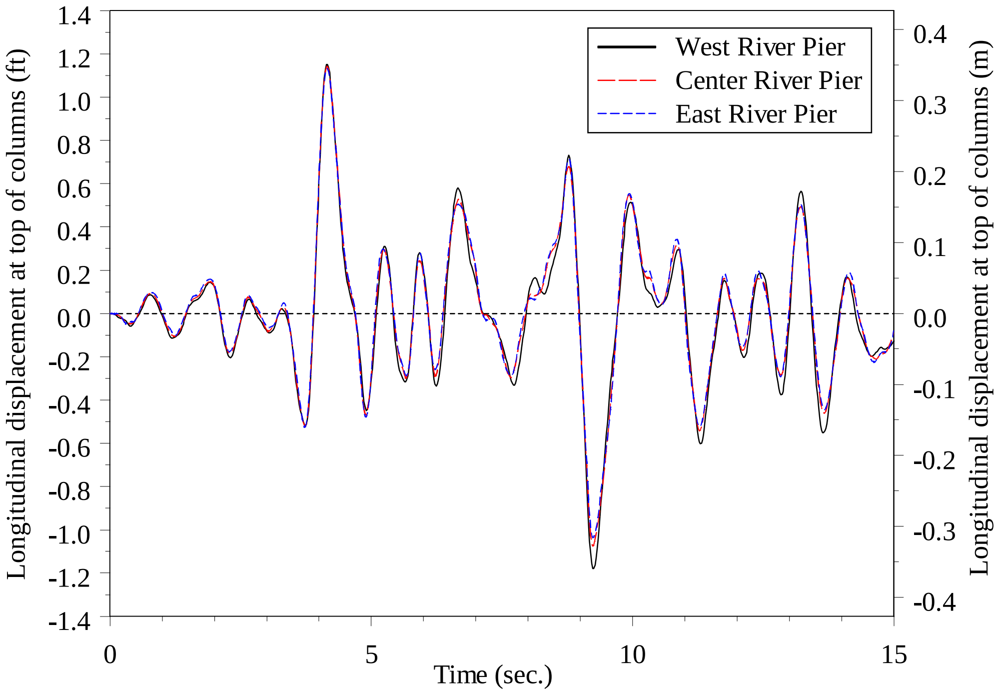

Figure 18. Longitudinal top-of-column displacements of the river piers for the as-built structure

Figure 19 indicates that maximum transverse displacements are approximately 1.01 ft, 0.69 ft and 1.03 ft for the West, Center and East River Piers, respectively. The results show that in the transverse direction, the Center River Pier is stiffer than the West and East River Piers due to the existing infill wall in the lower portion of the Center River Pier. Seismic demands are given in Appendix D, whereas analytical results are presented in Appendix E, including different views of the analytical model and deformed shapes.

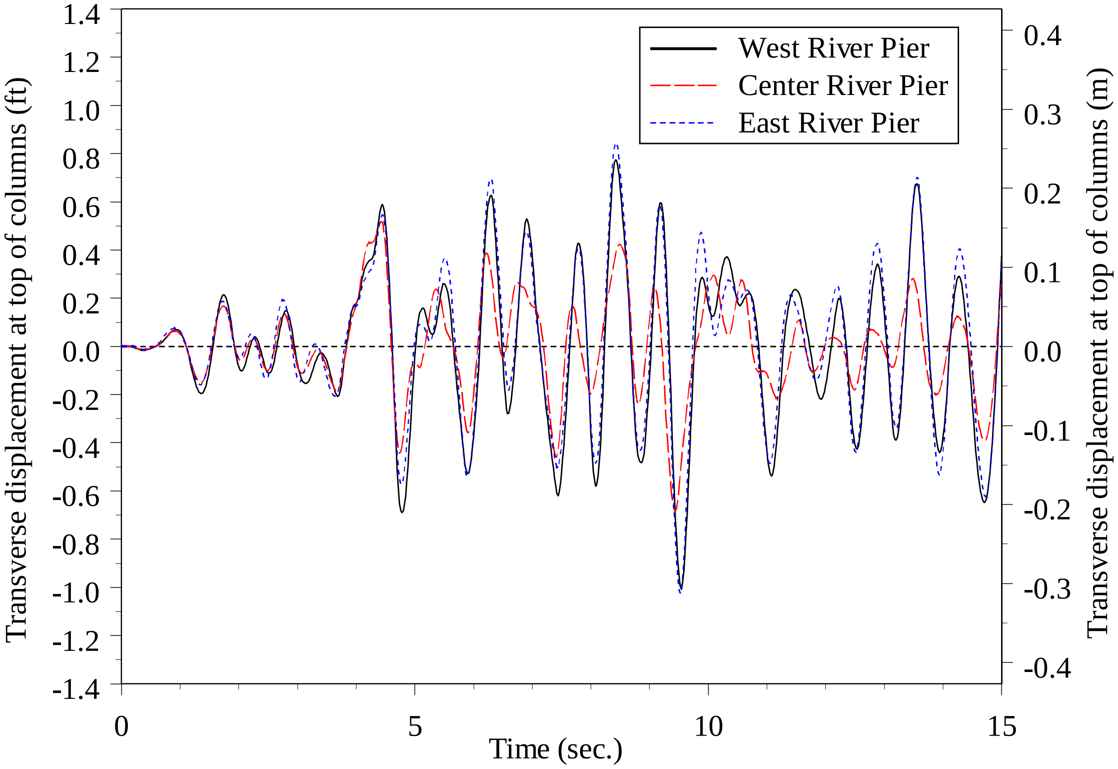

Figure 19. Transverse top-of-column displacements of the river piers for the as-built structure

9.2. CAPACITY ANALYSIS

As for the approach spans, nonlinear pushover analyses were conducted for the main spans. The model shown in Figure 16 was used for pushover analyses. As discussed earlier, refined models for concrete plastic hinges (see Figure 14) were used for the columns. Transverse plastic hinge elements were also introduced at the ends of the bent caps; however as the rotation demands in the bent cap hinges were relatively small, the SAP2000 Nonlinear elastic-perfectly-plastic moment-rotation model was adopted rather than the more accurate plastic hinge model shown in Figure 14. This would have minimal effects on the results since most of the plastic deformations come from plastic hinging of the columns, steel arch ribs and tension ties.

Pushover analyses of the arch spans were different from pushover analyses of the approach spans (Section 8.2) in that the arch spans model was pushed only to the maximum displacements obtained from nonlinear time-history analyses (Figure 18 and Figure 19). The objective of the pushover analyses was to verify the plastic hinge mechanism found from time-history analyses. Another objective of the pushover analyses was to show that more plastic hinges form in the as-built structure than the retrofitted structure, which will be discussed later. If the model of the arch spans is pushed to failure, the obtained displacement capacity cannot be directly compared to maximum displacement demands obtained from time-history analyses. Therefore, it was decided to conduct pushover analyses of the arch spans to maximum displacement levels obtained from nonlinear time-history analyses. Pushover analyses were performed separately for the longitudinal and transverse global directions of the bridge.

Figure 20 shows the deformed shape of the model at the ultimate longitudinal displacement of 1.18 ft. The deformed shape clearly indicates the relatively rigid triangle defined in Figure 2 as “OAB”. Figure 20 also shows the location of plastic hinges that have formed in the steel arch ribs and tension tie members, indicated by solid circles with different color codes for different limit states. The color code ranges from onset of yielding to fracture limit state; the color code is also shown in Figure 20. Plastic hinges that develop in the steel arch ribs and tension ties are concentrated in the vicinity of the arch rib-tie junction, as was discussed before. This is because of the relatively large longitudinal displacements of the as-built main spans, which results in rigid body rotations of the stiff triangle “OAB” (shown in Figure 2). This rigid body rotation results in high rotation, curvature and strain demands in the steel members at Points “A” and “B” shown in Figure 2.

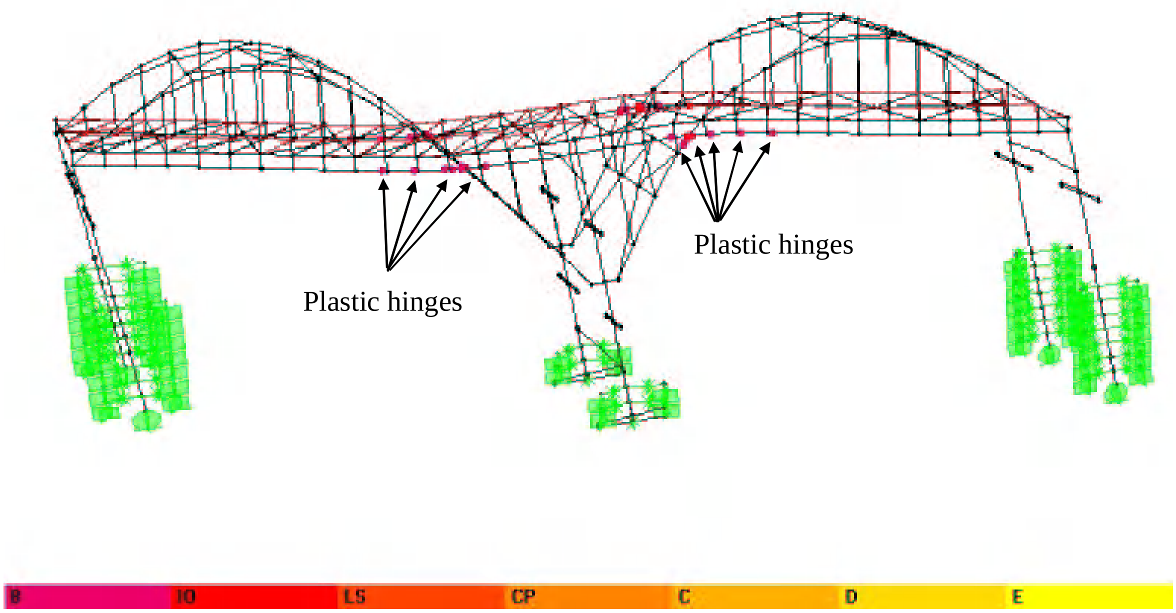

Figure 20. Longitudinal pushover model for arch spans of the as-built structure

Similarly, transverse pushover analysis was conducted on the main spans up to a maximum displacement of approximately 1.01 ft. Deformed shape and plastic hinge mechanism from the transverse pushover analysis are given in Appendix E.

Moment capacities of different structural elements were obtained from sectional moment-curvature analyses using the computer program ANDRIANNA[10]. Rotation capacities were determined for different sections of the columns and bent caps of the river piers as well as for the steel arch ribs. Ultimate curvatures were directly obtained from the moment-curvature analyses; rotation capacity is the ultimate curvature multiplied by the plastic hinge length. Rotation capacities of the columns and bent caps are given in Appendix D. Also, rotation capacities of the arch ribs are given in Appendix D at critical arch-deck interface locations (i.e., Points “A” and “B” in Figure 2).

9.3. SEISMIC DEMANDS AND CAPACITIES

Pushover analyses of the main spans were not conducted to failure as the displacement capacities obtained from pushover analyses will not be directly comparable to displacement demands from time-history analyses. This is because in the time-history analyses, the structure was subjected to simultaneous ground motions in the longitudinal, transverse and vertical directions. Thus, it was decided to compare seismic demands to capacities in terms of rotations at critical sections of the columns, bent caps and steel arch ribs. Seismic demands and capacities as well as demand-capacity (D/C) ratios are given in Appendix D. Seismic plastic rotation D/C ratios for the columns of the river piers are summarized in Table 4. The table indicates that the D/C ratios are zero under longitudinal seismic loading at the top of the West and East River Pier columns, since the arch ribs are supported on rockers on top of the piers. Rotation D/C ratios for the Center River Pier columns have non-zero values under longitudinal seismic loading only at the bottom; this is because the plastic hinges form at bottom of the columns. Under transverse seismic loading, no rotations were expected at the bottom of the Center River Pier columns because of the existing infill wall. Rotation D/C ratios at the bottom of the West and East River Piers columns were not available since these columns have pinned ends due to failure of lap-splices in the as-built condition, as discussed earlier. The D/C ratios given in Table 4 clearly indicate high vulnerability of the columns in all bents under transverse seismic loading, which may result in collapse of the structure under the MCE event.

Table 4. Plastic hinge rotation D/C ratios for columns of the as-built river piers

River Pier	**Rotation D/C Ratio**											
	Transverse Direction						**Longitudinal Direction**					
	North Column			**South Column**			**North Column**			**South Column**		
	Top	**Mid***	**Bot.**	**Top**	**Mid***	**Bot.**	**Top**	**Mid***	**Bot.**	**Top**	**Mid***	**Bot.**
West Pier	**1.63**	NA	NA	**2.47**	NA	NA	0.00	NA	NA	0.00	NA	NA
Center Pier	0.06	**4.27**	0.00	**1.42**	**5.31**	0.00	0.00	0.00	0.77	0.00	0.00	0.68
East Pier	**1.82**	NA	NA	**2.39**	NA	NA	0.00	NA	NA	0.00	NA	NA

* Section above existing infill wall in the Center River Pier (NA for the West and East River Piers).

Plastic rotation D/C ratios for the bent caps are given in Table 5. The rotation D/C ratios are given at the bent cap-column interface under positive and negative bending moments. Shear D/C ratios for the bent caps are also given in Table 5. The table indicates that bent cap plastic rotation demands do not exceed plastic rotation capacities under positive and negative bending. However, it was found that bent cap shearing force demands of the Center River Pier exceed shear capacity during the MCE event. Shear capacities of the bent caps were calculated using the shear strength model developed at the University of California, San Diego[14].

Table 5. Plastic hinge rotation and shear D/C ratios for bent caps of the as-built river piers

River Pier	**Rotation D/C Ratio**				**Shear D/C Ratio**
	At North Column		**At South Column**		
	Positive Bending	**Negative Bending**	**Positive Bending**	**Negative Bending**	
West Pier	0.00	0.57	0.12	0.32	0.49
Center Pier	0.00	0.18	0.00	0.66	**1.12**
East Pier	0.36	0.41	0.00	0.37	0.51

The as-built analyses showed that the arch ribs would also be overloaded during the MCE event leading to catastrophic collapse of the main spans. Concentration of plastic hinges in the arch ribs and steel ties at all locations of rib-tie junctions is shown in Figure 20. Ductility demand was assessed for a given member length as total rotation divided by yield rotation. By assuming that the moment is constant over the short member length, which is approximately correct, yield rotation is defined as yield curvature multiplied by the member length. Since yield curvature is the yield moment divided by rigidity *EI*, the rotation at yield is:

$$\theta_y = \frac{M_y}{EI} l$$

Total rotation of steel members is found from the model as the absolute difference in rotations at the member ends, and ductility demand is the total rotation divided by the yield rotation defined above. From the literature it was found from large-scale tests of structural steel members that are similar to the arch ribs, with flexure and high axial loads, that ductility demands (as defined above) should be limited to 2 to prevent catastrophic collapse[15]. Table 6 summarizes yield rotations, rotation capacities and demand rotations as well as rotation D/C ratios at the critical arch rib locations (Points "A" or "B" in Figure 2). Rotation capacities assume a ductility of 2 based on experimental research[15]. The rotation D/C ratios clearly indicate that the arch rib members would fail during the MCE event.

Table 6. Plastic hinge rotation D/C ratios for arch ribs at critical arch-deck interface (as-built)

Arch Rib Location (Figure 2)	Yield Rotation (rad.)	Rotation Capacity (rad.)	Demand Rotation (rad.)	Ductility Demand	Rotation D/C Ratio
A-South	0.00141	0.00282	0.00570	4.04	**2.02**
A-North	0.00141	0.00282	0.00560	3.97	**1.99**
B-South	0.00164	0.00328	0.00466	2.84	**1.42**
B-North	0.00164	0.00328	0.00340	2.07	**1.04**

10. SEISMIC DEFICIENCIES OF THE EXISTING STRUCTURE

Results of the elastic dynamic, linear elastic and nonlinear time-history and pushover analyses indicate that the as-built structure has high seismic vulnerability and the structure would collapse under the MCE event. Thus, immediate seismic retrofitting is needed. This is clear from the D/C ratios given in Table 3 to Table 6 and Appendix D. Seismic deficiencies of the as-built structure are discussed in this section for Frames 1 through 4 based on results of demand and capacity analyses.

10.1. FRAME 1 (WEST APPROACH SPANS)

Frame 1 has a unique set of bents of the bridge partly due to the varying height of columns from the first bent after the West Abutment (Bent 1) with column height of 18 ft to Bent 11, with column height of 57 ft. All bents in Frame 1 are perpendicular to the bridge longitudinal axis with no skew.

Results of the pushover analysis indicate the following deficiencies in Frame 1 under longitudinal seismic forces:

1. Plastic hinges could form at top of the short columns at displacements as low as approximately 0.5 inch.

2. Ultimate rotation capacity of the columns will be reached at longitudinal displacement of approximately 9.4 inches. The MCE event will result in a longitudinal displacement of approximately 11.0 inches, which means that the concrete sections will be stressed beyond their ultimate capacities. Thus, failure of plastic hinges at top of columns will occur in the MCE event.

3. Shear failure of several columns will occur before reaching the ultimate displacement capacity of the structure.

4. Footing stability problems are expected as evidenced from the deformed shape of Frame 1 under longitudinal pushover loads (see Appendix E). The deformed shape of the columns indicates that rotations of footings have significant contribution to displacements at top of columns. Spread footings are not much larger than the supported columns in terms of plan dimensions. This is because the footings were not designed to resist seismic forces. Thus, the spread footings may not have sufficient overturning resistance and would result in large rotations at bases of the columns. Footing retrofitting is thus needed to avoid any stability problems and to assure adequate transfer of the seismic forces to the ground.

Results of the pushover analysis indicate the following deficiencies in Frame 1 under transverse seismic forces:

1. Plastic hinges will form at top of the long columns at a relatively low displacement of approximately 1.2 inches.

2. Bent caps lack continuous bottom reinforcement over all three columns within some bents. Bent caps also have inadequate top reinforcement. Yielding of reinforcement in the bent cap will occur at transverse displacement of approximately 0.7 inch in most bents. Bent caps will fail when plastic hinges reach capacities.

3. The columns and bent caps will reach their ultimate capacities at displacements less than demand displacements from the MCE event. For example, failure of bent caps at Bents 8 and 9 will occur at transverse displacements of approximately 12.0 and 17.8 inches, respectively. The MCE transverse displacement demands are approximately 15.9 and 21.4 inches at Bents 8 and 9, respectively. Thus, the MCE event will cause failure of the bent caps at the column-cap interface. Columns and bent caps in Bent 11 (the tallest bent in the frame) are expected to reach failure condition first.

4. Short columns will fail in shear at transverse displacements that are significantly less than ultimate displacement.

10.2. FRAME 2 (ARCH SPANS)

Analyses of Frame 2 (Section 9) indicate the following seismic deficiencies:

1. The bottom of the West and East River Pier columns have inadequate lap-splices that will fail as plastic hinges begin to form and compressive strains exceed 0.002, followed by reversed loading. This results in pinned conditions and reduces the overall bridge strength and stiffness in the longitudinal direction and increases displacements, subsequently overloading the steel arch ribs at the arch-deck interface.

2. Demand rotations at column plastic hinges of all three river piers exceed rotation capacities under transverse loading, indicating failure of the column sections and collapse of the main spans in the MCE event.

3. Under transverse loading, Center River Pier's bent cap will be overloaded in shear. This results in shear failure of the bent cap and complete collapse of the structure.

4. The variable depth arch ribs are the most significant structural members for the performance and capacity of the two arch spans. At the location that the steel arch ribs cross the deck and steel tie, the ribs are loaded to a rotation ductility of over 4. Based on research[15], the non-compact built-up arch rib sections are expected to have a ductility capacity of 2. This indicates that failure of the arch ribs and complete loss of the structure will occur under the MCE event.

5. Existing foundations lack adequate top mat reinforcement, which renders them unable to resist plastic moments that will develop in the columns. This could result in premature failure of the foundations and excessive displacements of the bridge under longitudinal seismic loading. Thus, the existing foundations require the addition of perimeter piles and a top mat of reinforcement to transmit the column plastic moments to the ground. Perimeter piles are required due to the limited capacity of the existing piles.

10.3. FRAME 3 (EAST APPROACH SPANS BETWEEN BENTS 12 AND 22)

The pushover analyses indicate that the displacement demands do not exceed the capacities under longitudinal seismic loading. However, column shear failure is expected under longitudinal seismic loading. Results from the transverse pushover analysis indicate the following deficiencies in Frame 3:

1. Plastic hinges will form at top of the columns at relatively low displacements.

2. Bent caps lack continuous bottom reinforcement over all three columns within a bent and inadequate top reinforcement. Thus, bent caps will experience flexural failure when plastic hinges reach capacities.

3. The MCE event will cause failure of the columns and bent caps at the column-cap interface. This is evident from the MCE displacement demands that exceed the ultimate displacement capacities at several sections in columns and bent caps. For example, a transverse displacement

of approximately 46.0 inches is expected at Bent 19, whereas transverse pushover analysis showed that columns in Bent 19 will reach ultimate capacity at a displacement of approximately 20.7 inches.

4. Shear capacity of the severely deteriorated columns may be very low. It was found that the columns may fail in shear at low displacements. The shear failure mode has a brittle nature as failure occurs suddenly and would result in collapse of the structure at relatively low displacements.

5. Large displacements may cause P-Δ stability problems. In fact the relatively low displacement capacity is partly due to the P-Δ effect.

10.4. FRAME 4 (EAST APPROACH SPANS BETWEEN BENTS 23 AND 37)

As in Frame 3, the columns of Frame 4 rise approximately 60 ft above the ground and most of the bents consist of three columns. Results of pushover analyses indicate the following deficiencies in Frame 4:

1. Because of the relatively good condition of columns in Frame 4, compared to columns of Frames 1 through 3, plastic hinges will form in bent caps prior to formation of plastic hinges in columns. Onset of plastic hinge formation in bent caps will occur at approximately 0.35 inches and 0.70 inches of transverse and longitudinal displacements, respectively.

2. Bent caps lack continuous bottom reinforcement over all three columns within a bent and inadequate top reinforcement. Thus, the MCE event will result in failure of the bent caps at the column-cap beam interface. This is evident from the MCE displacement demands that exceed the displacements at which ultimate capacities is reached in several bent caps. For example, the transverse and longitudinal displacement D/C ratios are 1.95 and 2.44, respectively. Failure of the bent caps at Bents 34 and 26 will occur under transverse and longitudinal loading, respectively.

3. Plastic hinges will form in grade beams prior to formation of plastic hinges in columns. Grade beams' plastic hinges will form at top displacements of approximately 3.9 inches and 2.9 inches under transverse and longitudinal loadings, respectively. Plastic hinges will form in the columns at higher displacements of approximately 5.9 inches and 6.0 inches under transverse and longitudinal loadings, respectively.

11. VULNERABILITY STUDY

A vulnerability study was conducted in the retrofit pre-strategy phase. Major findings will be briefly presented in this report for completeness purpose. The intent of the vulnerability study is to correlate the lowest level of earthquake where a failure could occur along the length of the viaduct with a simplified probabilistic analysis of an earthquake event.

From the frame analysis, the structure is determined to reach a "collapsible failure" when one of the following conditions is satisfied:

1. Failure of the superstructure when reaching bending moments that could cause failure of either the superstructure girders in the longitudinal direction, or the cap beams in the transverse direction.
2. Ultimate rotation is reached in the columns.
3. Shear failure occurs in the columns.

Determination of the failure condition is an iterative process. Three sets of ARS curves were used as input. The three earthquakes have return periods of 72 years (50% probability of recurrence in 50 years), 475 years (10% probability of recurrence in 50 years) and 950 years (5% probability of recurrence in 50 years). Data of the ARS curves are given in the geotechnical memos (see Appendix H). The sensitive structural components as noted in the frame analysis were verified for capacity limitation in the seismic analyses. An approximate interpolative ARS curve was developed to determine the capacity threshold of the components and the recurrence interval was determined.

The analyses indicated that the structure will reach the “collapsible failure” state in Frame 1 at 0.36g or 0.24g accelerations in the longitudinal or transverse directions, respectively. For Frames 3 and 4, the analyses indicated that collapse is reached at 0.17g acceleration under longitudinal or transverse loading. Based on the geotechnical memos (see Figure 5 in the URS Geotechnical Memo, Appendix H), the “collapsible failure” state will be caused by an earthquake with a return period of approximately 40 years (corresponding to 0.17g ground acceleration); the corresponding probability of recurrence in 50 years is 71%. Thus, the probability that the Sixth Street Viaduct will experience significant failure, and possibly collapse, under seismic events exceeds 70% in 50 years. Poor detailing and severe concrete deterioration are major factors that increase collapse vulnerability of the viaduct. Thus, immediate retrofitting is needed.

12. ANALYSIS OF THE RETROFITTED MAIN SPANS

This section presents analysis results of the retrofitted main spans (Frame 2) of the Sixth Street Viaduct. Two retrofit alternatives are possible for the main spans but only one of them is recommended and included in Section 13 in discussion of the seismic retrofit alternatives of the existing structure.

12.1. MAIN SPANS RETROFIT ALTERNATIVES

Two alternatives are possible for retrofit of the main spans of the Sixth Street Viaduct. The alternatives will be discussed in the following sections.

12.1.1. Alternative A (Foundations Retrofit and Infill Walls):

The following presents the Alternative A retrofit strategy to alleviate structural deficiencies of the main spans of the as-built structure (see Section 10.2).

__Columns of River Piers:__ Foundation overlays are proposed in the West and East River Piers to confine the poor lap-splices of the longitudinal column reinforcement and to allow column bases to develop their full plastic moment capacities. This reduces longitudinal displacements of the bridge and lowers ductility demands on the arch ribs to acceptable levels below 2. As discussed earlier, the ductility capacity of structural steel members similar to the arch ribs is approximately 2[15]. With this foundation retrofit, curvature demands at the base of the columns are small, resulting in compressive strains that are below the unconfined concrete strain capacity of 0.005. Therefore, no column retrofit is required.

However, infill walls are required between columns at the West and East River Piers to reduce transverse displacements and shear demands on the bent caps. Infill walls are 7 ft thick and extend from the top of footing to just below the ground level. The Center River Pier does not require the addition of an infill wall as an existing cross member of the as-built structure already acts as an infill wall. An advantage to this infill wall strategy is that it will not be visible to the public and, thus, will not change the architectural character of this historically significant bridge.

__Bent Caps:__ As discussed above, recommended infill walls reduce shear demands of the bent caps so that no bent cap retrofit is required. The infill walls also reduce transverse plastic hinge demands at all columns to acceptable levels below capacities of the bent caps.

__Arch Ribs:__ By anchoring the column lap-splices and allowing dependable plastic hinges to form at the bottom of columns under longitudinal seismic loading, displacements are smaller and rotation ductility demands of the arch ribs are reduced from 4.04 for the as-built structure to 1.65 for the retrofitted structure, at the critical arch-deck junctions. This limited level of ductility demand for the retrofitted structure is below the suggested ductility capacity of 2 for this type of built-up members, as discussed in the literature[15] (recommendations based on large-scale tests and analysis results).

In final design of the Sixth Street Viaduct it is recommended that a detailed nonlinear finite element analysis be conducted for a portion of the arch rib to confirm that the ductility capacity is 2. Such a model would include the different plates of the built-up section and the large axial loads of the arch.

It is anticipated that nonlinear shell elements would be used, with nonlinear material and geometry modeled. This type of detailed local nonlinear model was beyond the scope for development of this strategy report.

Foundations: Foundation overlays at the West and East River Piers consist of a top mat of reinforcement and drill-and-bond dowels to anchor the new overlay to the existing foundations. Perimeter piles are also required to transfer the column plastic moment to the ground.

12.1.2. Alternative B (Foundations and Bent Caps Retrofit):

An alternative to the infill wall retrofit strategy, discussed in Alternative A, is to strengthen the bent caps by increasing their size with concrete bolsters and added prestressing steel to both sides of the bent cap. Alternative B includes bent cap retrofit in addition to the foundations retrofit discussed above for Alternative A. This retrofit would be more visible than the infill wall approach of Alternative A and, thus, is probably not the best choice for the site due to the architecturally significant nature of this structure. The cost of Alternative B was also estimated to be higher than the cost of Alternative A.

Alternative A is recommended for retrofit of the arch spans since it results in the least visible modifications of this historically significant structure and is the cheaper alternative. In all subsequent discussions in this report, Alternative A is assumed for retrofitting of the main spans.

12.2. MODEL DESCRIPTION

The model used for analysis of the as-built main spans (see Figure 16 and Figure 17) was modified and used for analysis of the retrofitted structure. Shell elements were included between columns in the West and East River Piers to represent the proposed infill walls. In addition, plastic hinge models, similar to that shown in Figure 14, were added at the bottom of the West and East River Pier columns as foundation retrofit confines lap-splices, allowing plastic hinges to develop under longitudinal seismic loading. Figure 21 shows the model used in modal, linear elastic and nonlinear time-history analyses, as well as pushover analyses of the retrofitted arch spans.

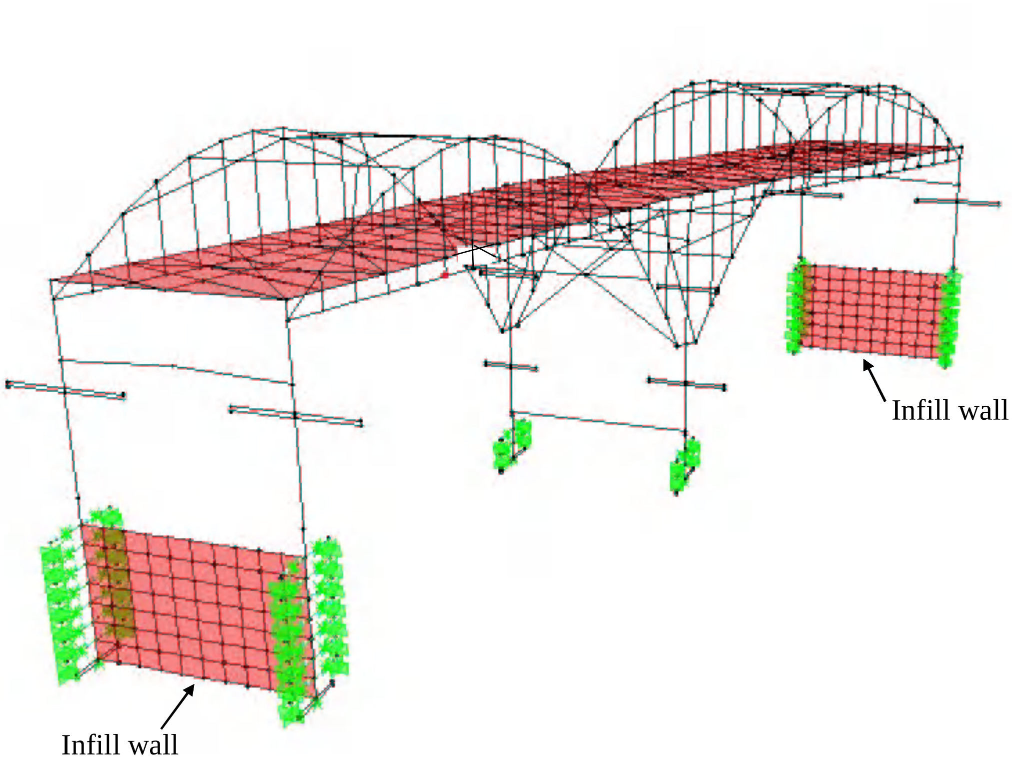

Figure 21. Global model for arch spans of the retrofitted structure

12.3. ANALYSIS RESULTS AND DEMAND-CAPACITY RATIOS

Figure 22 and Figure 23 show, respectively, the longitudinal and transverse seismic displacements at the top of the three river piers of the retrofitted structure. Longitudinal displacement demands for the retrofitted structure are less than demands for the as-built structure (compare Figure 18 and Figure 22). Considerable reduction of transverse displacement demands is achieved by addition of the infill walls (compare transverse displacements in Figure 19 and Figure 23). Figure 24 shows transverse displacements of the East River Pier for both as-built and retrofitted structures. The figure clearly shows that the infill wall retrofit approach considerably stiffens the structure and reduces displacement demands in the transverse direction. This is also clear from modal analysis results, which shows that natural period of the retrofitted structure is 0.276 seconds compared to 0.422 seconds for the as-built structure.

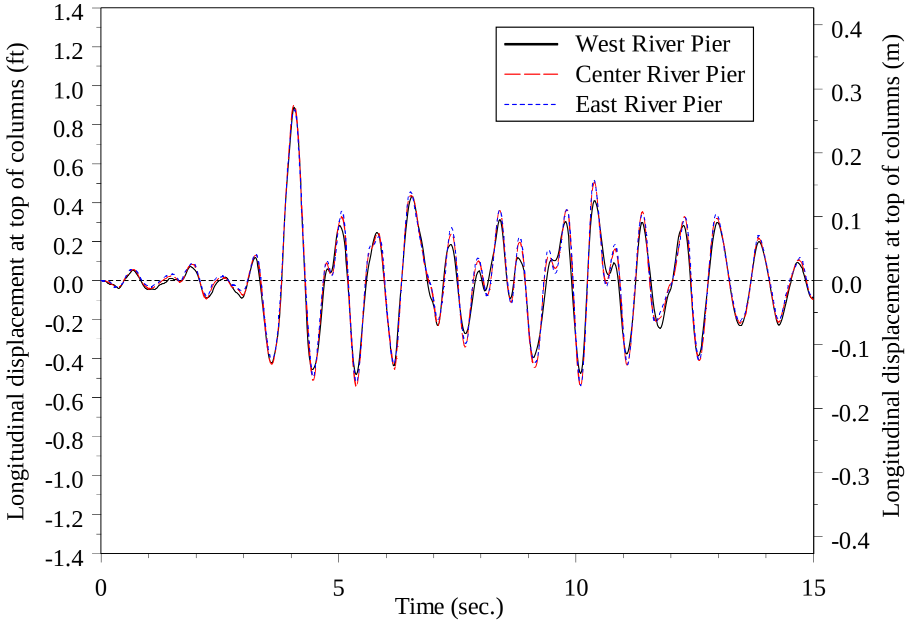

Figure 22. Longitudinal displacements at top of river piers for the retrofitted structure

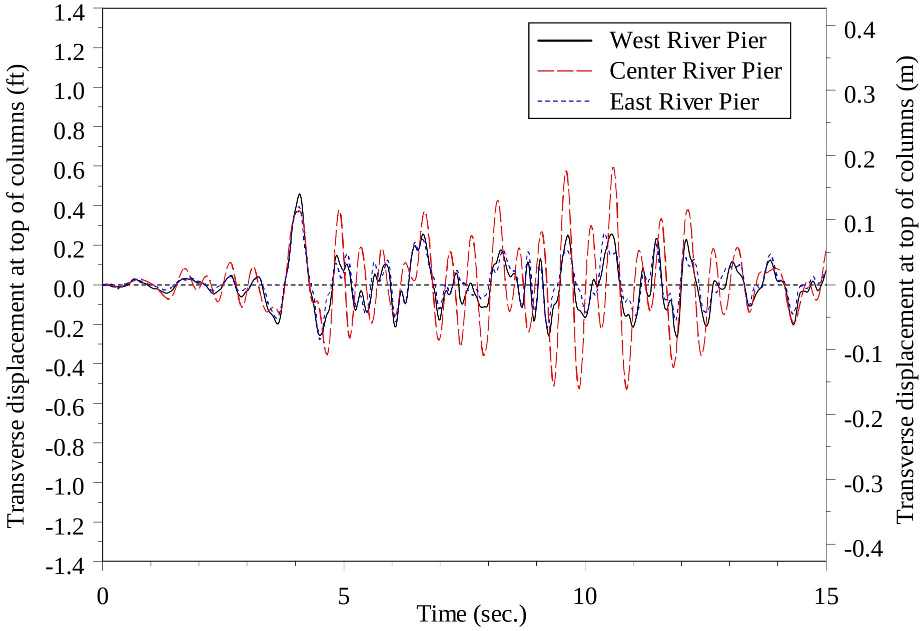

Figure 23. Transverse displacements at top of river piers for the retrofitted structure

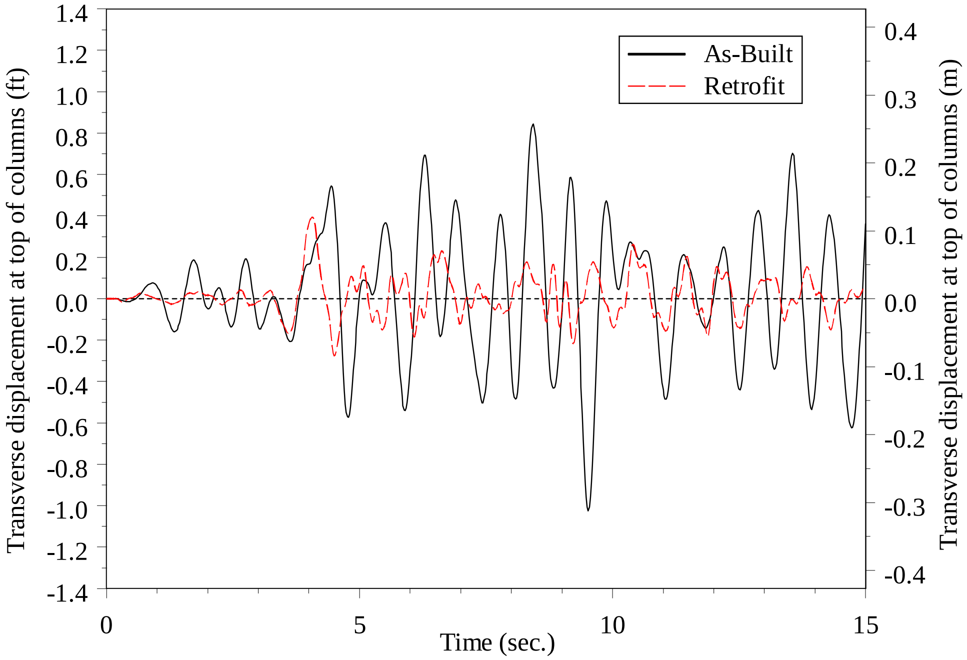

Figure 24. Transverse displacements at top of the East River Pier for the as-built and retrofitted structures

Figure 25 shows the deformed shape from the pushover model of the retrofitted structure at maximum longitudinal displacement demand of 0.89 ft from time-history analysis. As for the as-built structure, the model was pushed to maximum displacement demand levels rather than to failure displacements. The objectives of pushover analyses of the arch spans were to verify the plastic hinge mechanism found from nonlinear time-history analyses and to show that fewer plastic hinges develop in the retrofit model compared to the as-built model. Comparison of plastic hinges shown in Figure 20 and Figure 25 clearly indicates that fewer plastic hinges form in the arch rib and tension tie members for the retrofitted structure compared to the as-built structure at maximum displacement demands. Also, time-history analyses indicate reduced ductility demands in these members for the retrofitted structure.

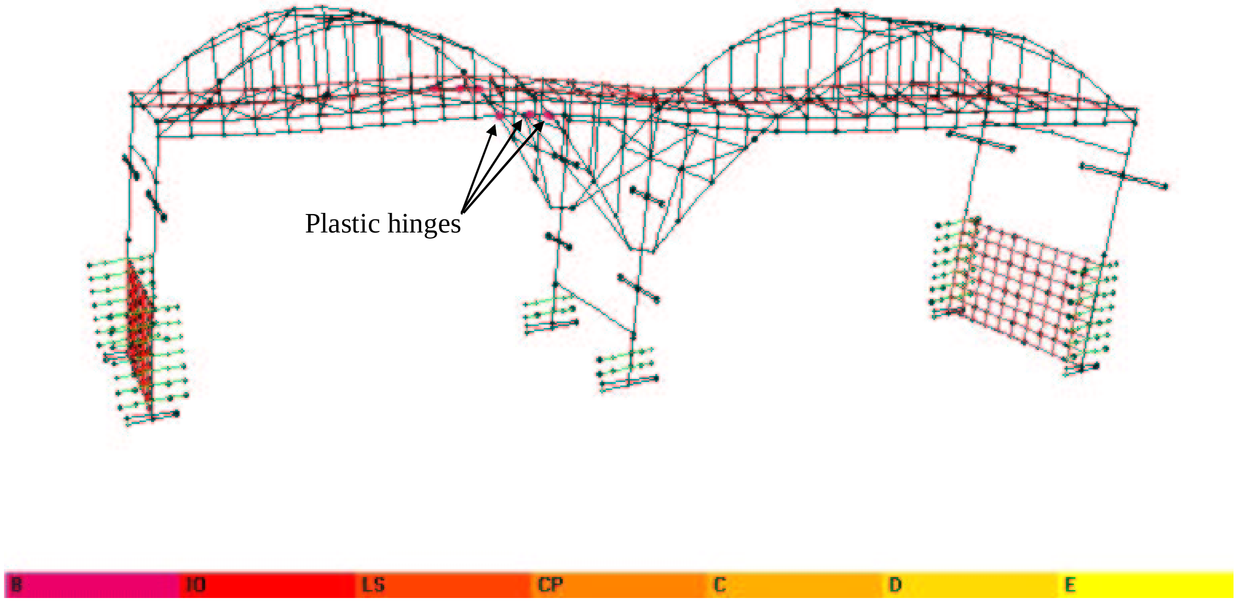

Figure 25. Longitudinal pushover model for arch spans of the retrofitted structure

Figure 26 shows load-displacement curves obtained from pushover analyses of the as-built and retrofitted arch spans. The purpose of this figure is to illustrate effects of the proposed retrofit in increasing stiffness and strength of the arch spans. Comparison of the curves shown in Figure 26 indicates that the proposed retrofit would significantly reduce seismic displacements and result in less inelastic strains in different elements of the retrofitted structure. More analysis results can be found in Appendix E.

Demand-capacity ratios in columns, bent caps and critical locations of the arch ribs in the retrofitted structure are given in Table 7 to Table 9, respectively. Seismic demands and capacities as well as D/C ratios are also given in Appendix D. The D/C ratios demonstrate that the proposed retrofit alternative will protect the arch spans during the MCE event while maintaining the architectural significance of this historic structure.

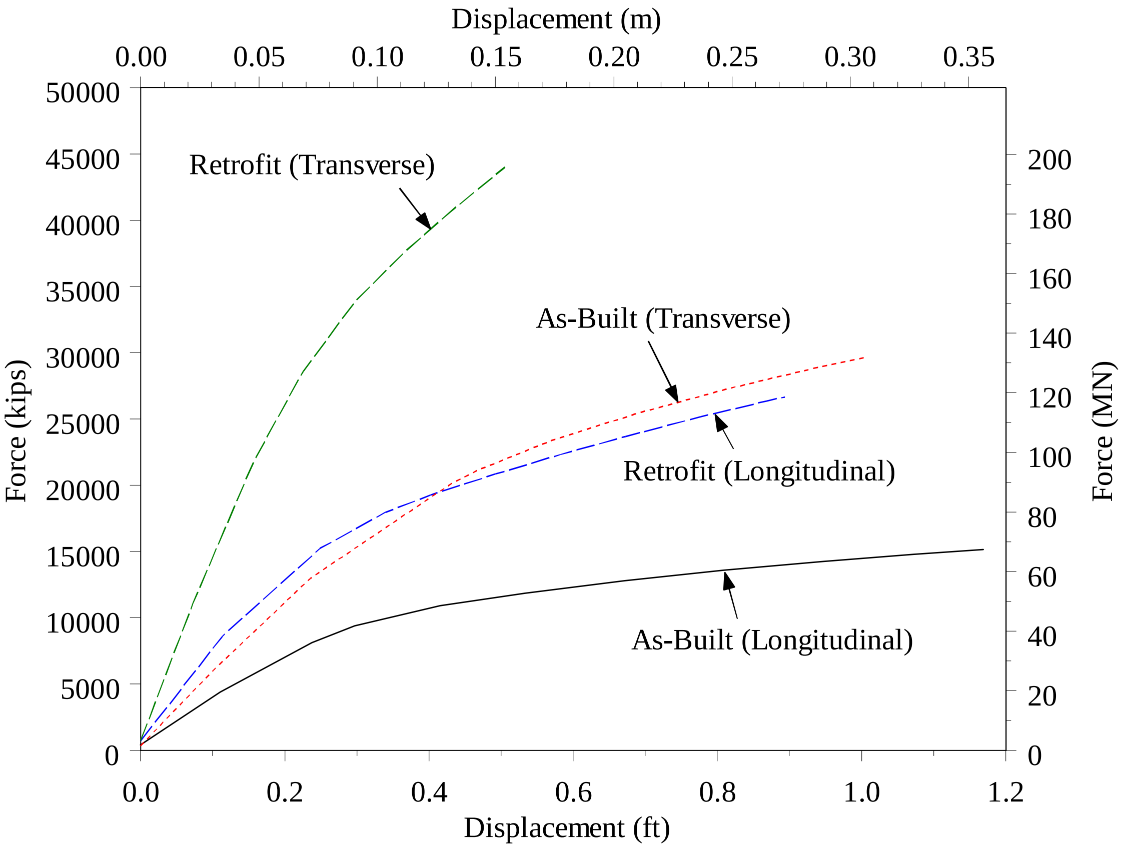

Figure 26. Load-displacement response of the main spans (pushover analyses)

Table 7. Plastic hinge rotation D/C ratios for columns of the retrofitted river piers

River Pier	**Rotation D/C Ratio**											
	Transverse Direction						**Longitudinal Direction**					
	North Column			**South Column**			**North Column**			**South Column**		
	Top	**Mid***	**Bot.**	**Top**	**Mid***	**Bot.**	**Top**	**Mid***	**Bot.**	**Top**	**Mid***	**Bot.**
West Pier	0.03	NA	NA	0.06	NA	NA	0.00	NA	0.25	0.00	NA	0.25
Center Pier	0.05	0.74	0.00	0.43	0.88	0.00	0.00	0.00	0.66	0.00	0.00	0.57
East Pier	0.03	NA	NA	0.05	NA	NA	0.00	NA	0.27	0.00	NA	0.27

* Section above existing infill wall in the Center River Pier (NA for the West and East River Piers).

Table 8. Plastic hinge rotation and shear D/C ratios for bent caps of the retrofitted river piers

River Pier	Rotation D/C Ratio				Shear D/C Ratio
	At North Column		At South Column		
	Positive Bending	Negative Bending	Positive Bending	Negative Bending	
West Pier	0.00	0.02	0.00	0.00	0.41
Center Pier	0.00	0.16	0.00	0.24	0.95
East Pier	0.00	0.00	0.00	0.00	0.31

Table 9. Plastic hinge rotation D/C ratios for arch ribs at critical arch-deck interface (retrofit)

Arch Rib Location (Figure 2)	Yield Rotation (rad.)	Rotation Capacity (rad.)	Demand Rotation (rad.)	Ductility Demand	Rotation D/C Ratio
A-South	0.00141	0.00282	0.00208	1.48	0.74
A-North	0.00141	0.00282	0.00233	1.65	0.83
B-South	0.00266	0.00532	0.00215	0.81	0.40
B-North	0.00266	0.00532	0.00245	0.92	0.46

13. SEISMIC RETROFIT ALTERNATIVES

The purpose of this report is to study different alternatives for seismic retrofit or replacement of the Sixth Street Viaduct. The study includes retrofit preliminary design, evaluation of structural efficiency of the retrofit, cost estimates and life expectancy of the retrofitted structure. Analyses of seismic demands and capacities of the as-built and retrofitted river spans (Frame 2 in Figure 3) have been presented in Sections 9 and 12 of this report. Similar to the as-built structure, elastic dynamic analyses of the retrofitted approach spans (Frames 1, 3 and 4 in Figure 3) were conducted to determine seismic demands. Nonlinear pushover analyses were not conducted for the retrofitted approach spans, but capacities were obtained from moment-curvature analyses of the retrofitted structural members; capacities were calculated by this procedure for only the retrofit alternative with steel casings and infill shear walls (Alternative 2). The seismic displacement D/C ratios for the steel casings alternative are summarized in Appendix D. The shear D/C ratios for the steel casings retrofit alternative (Alternative 2) are also summarized in Appendix D. Information about the elastic dynamic analysis models are given in Appendix E.

As discussed later, catcher walls are provided in Alternative 3 to enhance seismic safety, but the catcher walls will not alter seismic performance of the existing structure; thus, seismic demands and capacities of Alternative 3 are similar to those of the as-built structure (see Section 8.3). Infill shear walls identical to those used in Alternative 2 will be used in Alternative 5, but with heavier steel casing of more columns. Thus, seismic demands in Alternative 5 will be similar to those of Alternative 2, but capacity of the retrofitted structure in Alternative 5 will be higher than capacity of the retrofitted structure in Alternative 2. In other words the D/C ratios will be higher in Alternative 2 than Alternative 5; thus seismic analysis of only Alternative 2 was conducted. Alternative 4 uses concrete casings of columns and bent caps, which would increase stiffness and reduce seismic displacement demands compared to the existing structure. Demand and capacity analyses were not conducted for Alternative 4 since it is believed that this alternative is not economic as will be seen later in the cost analysis.

Retrofit strategy for the river spans have been described and discussed in Section 12. Thus, discussions in this section focus more on retrofit alternatives of the approach spans. The retrofit alternatives are described in this section. Structural efficiency is also discussed for different retrofit alternatives, as well as satisfaction of other requirements for historical aesthetics and environmental mitigation. Cost estimates are discussed in Section 14 of this report. The retrofit alternatives are summarized in Appendix A.

13.1. RETROFIT DESIGN CRITERIA AND OBJECTIVES

The selected criteria for retrofit design depend on deficiencies of the as-built structure. Two basic approaches may be adopted for seismic retrofitting. In the first approach, seismic demands can be reduced. In the second approach, strength or ductility of structural elements can be improved by different means of retrofitting. In the second approach, retrofit is generally provided for inadequate flexural ductility or capacity, inadequate shear strength and lack of structural integrity of lap-splices. As discussed in Section 10, analyses of the as-built structure indicate that:

1. Seismic displacement demands exceed ultimate displacement capacities.
2. Excessive seismic displacements will result in yielding of arch ribs and tension tie members in the main spans.
3. Rotation demands in columns substantially exceed capacities, which will result in failure of columns.
4. Shear failure may occur in columns with severely deteriorated concrete as well as in the Center River Pier bent cap.
5. Failure of bent caps will occur due to lack of continuous bottom reinforcement and inadequate top reinforcement in the cap beams over the columns.
6. Stability problems could be encountered in tall columns because of small spread footings and the resulting excessive column displacements.

Based on the above-mentioned deficiencies of the as-built structure, the retrofit should be designed to satisfy the following objectives:

1. Reduction of the seismic displacement demands on the structure. Seismic displacements can be reduced by the following:
   (a) Construction of infill shear walls between columns in selected bents. This will reduce seismic displacements in transverse direction of the bridge.
   (b) Construction of grade beams to reduce seismic longitudinal displacements.
   (c) Closure of some expansion joints, which will enhance stiffness of the structure under longitudinal seismic loading and reduce longitudinal displacements.
   (d) Increasing stiffness of the existing columns, such as by use of concrete column casings.

   (e) Footing retrofit by overlays and additional piles or construction of new footings. The retrofitted footings will have adequate flexural capacities, which will enable the columns to develop plastic hinges at their bases. The columns will be in double bending because of fixed top and bottom ends, which will increase stiffness and reduce seismic displacements.

2. Reduction of seismic displacements in the main spans will protect the steel arch ribs from excessive rotations, which will prevent failure.
3. Enhancement of ductility and displacement capacities of columns by steel or concrete casings.
4. Enhancement of shear capacity of the severely deteriorated columns by steel or concrete casings.
5. Enhancement of flexural capacity of some bent caps to ensure that plastic hinges will form in the columns. This is because bent caps do not have adequate ductility capacity due to lack of continuous reinforcement over the columns. In the final design of the retrofit, special detailing should be considered at top of the columns to reduce strain penetration of reinforcement into the superstructure.
6. Enhance stability by construction of new footings at selected bents. As mentioned earlier, the new footings will provide fixity at bases of columns, which will increase stiffness of the structure and reduce seismic displacements.

In addition, a comprehensive retrofit design should take into account future deterioration of concrete caused by Alkali Silica Reaction (ASR). Based on the above-mentioned design criteria and objectives, efficiency of different retrofit alternatives will be discussed below.

Seven alternatives for retrofit or replacement of the existing structure are discussed in the following sections. The goal of retrofit Alternatives 2 through 4 is to seismically retrofit the existing structure to meet the current public safety requirements. These retrofit alternatives account for the current material degradation, but do not provide any measures to arrest future degradation and thus may require future seismic retrofits. The goal of Alternative 5 is to seismically retrofit the existing structure with taking into account future deterioration of concrete due to ASR. The goal of Alternatives 6A and 6B is to replace the existing structure with a new one. Retrofit Alternatives 1, 2, 4, 5 and 6A try to maintain historical integrity and visual aesthetics of the Sixth Street Viaduct, whereas Alternatives 3 and 6B do not. Advantages and disadvantages of each retrofit/replacement alternative will be discussed in the following sections.

13.2. ALTERNATIVE 1 (INFILL WALLS)

13.2.1. Scope:

This retrofit alternative was designed by the City of Los Angeles Bureau of Engineering (BOE) in 1995, and was approved by the County of Los Angeles and Caltrans in 1998. The City of Los Angeles has requested, and subsequently received an authorization for construction from Caltrans in 2000 for the sum of $18.2 million. The design consists of construction of infill walls between columns at a total of 17 bents, and construction of 6 grade beams and two footings. The retrofit design also includes restrainers at the West and East River Piers and concrete-filled steel pipes at the West Abutment to enhance capacity of shear keys under seismic forces. Though this alternative does not meet the technical standards for Life Safety, it is presented herein as a basis for cost comparison.

13.2.2. Infill Walls:

The major component of the retrofit design is construction of the infill walls between columns. These infill walls will be connected at their top ends to the bent caps, or superstructure girder at some bents, and to the columns along their sides. The infill walls will extend below the ground surface and will be connected to either the existing grade beams, new grade beams, or to the footings. The infill walls will be constructed with extensive use of drill-and-bond dowels inserted into the existing concrete surfaces of the bent caps, girders, columns, footings and grade beams. These drill-and-bond dowels typically consist of pairs of #8 bars that extend 12 inches inside the existing structural elements and are spaced at 12 inches. The drill-and-bond dowels are lapped with the new infill wall reinforcements.

13.2.3. Foundations Retrofit:

The design includes retrofit of foundations at Bents 26 and 32 (see drawings in Appendix B). At each of the two bents, one spread footing will be constructed to support all three columns. The new footings will be thicker than the existing spread footings. The new footings will be bonded to the existing footings by drill-and-bond dowels. A total of 6 new grade beams will be constructed and connected to the existing footings and columns by drill-and-bond dowels. At all other bents, no retrofit work was designed for foundations of the bents with the proposed infill walls. Locations of the infill walls, grade beams and footing retrofits are shown in Appendix B.

13.2.4. River Spans:

No retrofitting was proposed for the arch spans over the Los Angeles River (main spans). The as-built analyses using current standards indicate that retrofitting of the river spans is required.

13.2.5. Discussion:

This design was completed by the Los Angeles City BOE before commencement of the material sampling and testing program. The material testing report concluded that concrete has severe deterioration at significantly more bents than those at which the infill walls are proposed to be constructed. Appendix G includes drawings of the as-built structure with color codes corresponding to different levels of concrete deterioration. As can be seen from the drawings of Appendix G, many columns are in poor condition and the infill wall retrofit design does not take into account the current deterioration of concrete.

Assuming that the new infill walls would perform as desired, seismic displacement demands would be reduced and overall stability of the structure would be enhanced. Deterioration of concrete as found in the material sampling and testing program makes the functionality and structural viability of this retrofit alternative questionable. As documented in the material sampling and testing report[1], concrete core samples from columns with severe surface distress showed severe cracking over 18-in. deep inside the columns. Extensive cracking was observed throughout the length of the 48-in. long cores. These cracks would significantly reduce the ductility, flexural and shear capacities of columns. The high shear stresses expected in short columns would render the deteriorated columns vulnerable to shear failure.

Analysis of the infill walls under transverse seismic forces can be simplified with a strut-and-tie model as illustrated in Figure 27. Figure 27 shows that the seismic force, F, is transferred to the foundation by means of diagonal compression struts in the infill walls. In fact, tension ties would also exist in the shear wall, but only compression struts are assumed for simplification purpose. To satisfy force equilibrium, high shears would be transferred between the infill walls and columns. It has been shown in the material testing report that force transfer mechanisms, similar to that shown in Figure 27, could result in transfer of shear stress as high as 175 psi between columns and the adjacent infill walls[1]. The deteriorated concrete in the columns could prevent transfer of such high shear stresses and thus result in extension and widening of existing cracks in the columns. This could result in further weakening of the structure to resist seismic forces, or even gravity loads after the earthquake.

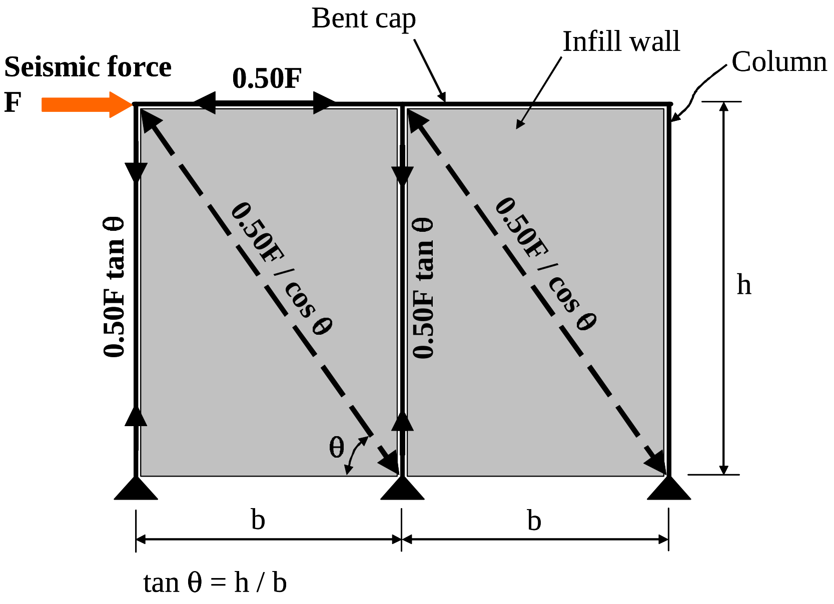

Figure 27. Strut-and-tie seismic force transfer mechanism inside infill walls (Alternative 1)

In addition to the above-mentioned structural deficiencies, this retrofit design alternative does not include any measures for future protection of the structure. As discussed in the material testing report, concrete deterioration due to ASR will continue to occur in the future. This may result in seismic vulnerability in the future even if assuming that the structure is seismically safe immediately after retrofitting. Retrofit life expectancy is defined here as the number of years after retrofitting and before a significant investment in a new retrofit or rehabilitation is required to maintain seismic and operational safety of the structure. Because of the above-mentioned structural deficiencies and because of continued degradation of concrete due to ASR, life expectancy of the structure with this retrofit alternative is extremely short. It can be assumed that the retrofitted structure would have no life expectancy.

Another deficiency of this retrofit design alternative is that it does not meet historical aesthetic requirements. Although the new infill walls will include indentations to add visual effects to the structure, the retrofitted structure will not be visually consistent with the existing historical one. The retrofit design also does not meet requirements of the State Historic Preservation Office (SHPO) and environmental mitigation is not available in this alternative.

The Los Angeles City held back the advertising and bidding of this project on the concern of aesthetics and historical preservation expressed by Caltrans in 2001. The design was subsequently re-evaluated by the Los Angeles City BOE after discovering the severe concrete damage due to continuing ASR. The design was also deemed incomplete because of the extensive foundation damage caused by ASR as uncovered in the material sampling and testing report[1]. This would require foundation retrofit in addition to the infill walls.

The nonlinear seismic analyses of the arch spans over the Los Angeles River (see Section 9) also indicated that columns of the river piers will not have adequate rotational capacities to withstand the MCE event and that shear capacity of the Center River Pier cap beam is not adequate. The analyses indicated that the arch ribs will fail during the MCE event, which may result in catastrophic collapse of the structure. Based on the above discussion, this retrofit alternative is not a viable retrofit alternative and is not considered, but serves as the basis for comparison with other alternatives.

13.3. ALTERNATIVE 2 (STEEL CASINGS)

Alternative 2 proposes construction of infill shear walls to reduce seismic displacements and the use of steel plates to provide encasement to the columns. Steel casings and infill shear walls are also combined with construction of new foundations, grade beams, retrofitting of bent caps and closure of some expansion joints in the superstructure. Locations of structural elements to be retrofitted are shown in Appendix B.

13.3.1. Scope:

The as-built analyses indicate large displacements in the structure. To limit the displacements, infill shear walls are added similar to Alternative 1. However, to improve performance of the columns, steel casings are added to columns in the bents with infill shear walls in addition to other columns at some of the bents with no infill walls. The steel casings will enhance confinement, ductility and shear strength of the existing columns. The steel casings will also improve shear force transfer capacity between the infill walls and the deteriorated columns. The major component of retrofit Alternative 2 proposes construction of infill shear walls at a total of 14 bents in addition to the use of steel plates to provide encasement to a total of 29 columns (see plans in Appendix B). Since ductility and displacement capacity of the retrofitted columns will be enhanced, it is necessary to increase flexural strength of some of the bent caps to assure that plastic hinges will not form in the bent caps after retrofitting of the columns, but plastic hinges would rather form in the columns. This is because of limited ductility capacity of bent caps due to lack of continuous bottom reinforcement and inadequate top reinforcement in the cap beams at locations of the columns.

The infill shear walls will reduce seismic transverse displacements in the existing structure. It is proposed to close two expansion joints in the superstructure and construct new grade beams to reduce seismic longitudinal displacements. The as-built analyses showed that stability problems may be encountered in the existing structure because of the small-size footings. Thus, new footings are also proposed to reduce displacements and enhance stability of the structure since the existing footings were, according to literature, sized to resist gravity plus 0.10g lateral loads. Also, retrofitting of the existing footings is necessary because of degradation due to ASR.

13.3.2. Columns Retrofit:

The steel plate encasements are proposed for the columns with “Moderate-Severe” to “Severe” damage that will be connected to new infill shear walls, as well as columns with displacement D/C ratios greater than 1.0. Most of the existing columns that will be steel encased have severe concrete degradation as found from the visual survey and laboratory experiments of the concrete cores[1] (see photos in Appendix F). It was found that at some bents, only one of the exterior columns needs to be retrofitted; however both exterior columns in those bents will be retrofitted to create visual balance and consistency for the two exterior columns.

Figure 28 shows a schematic elevation view of the steel plate encasement proposed in Alternative 2. All surfaces of a retrofitted column will be encased by 5/8" thick steel plates. The columns are not circular or rectangular, but have complicated geometry; an example of an exterior column with steel casing is shown in Appendix B (see typical details of steel casings).

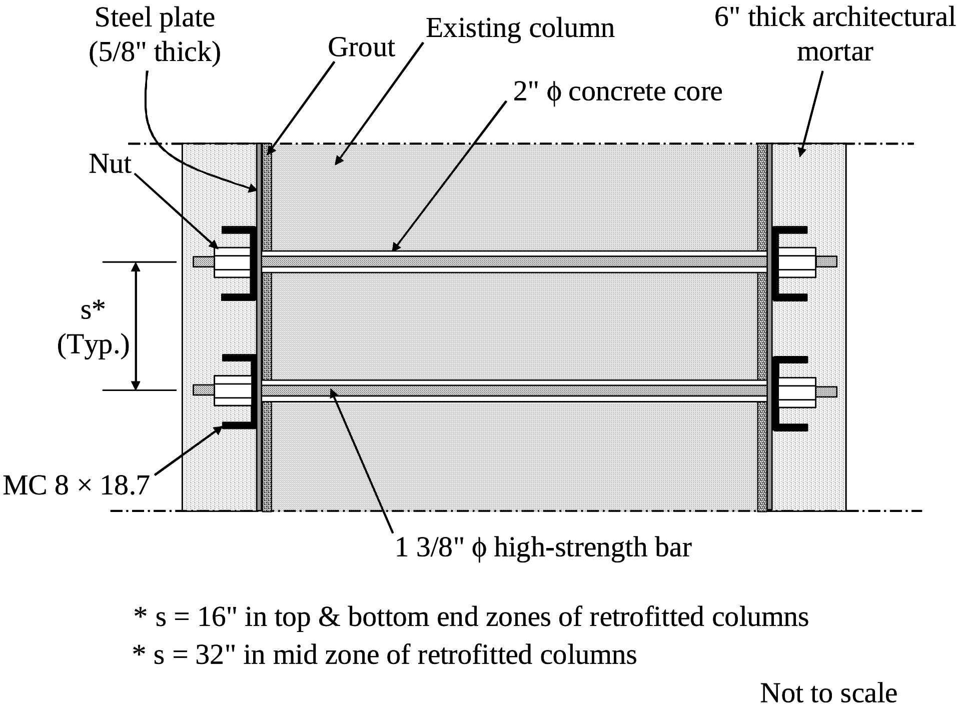

Figure 28. Steel encasement of columns

The steel plates will be welded together, with longitudinal welds at corners of the column and along its height. Most of these plates will be welded in the shop to have better control on quality of welds. The plates will be welded in the shop such that the complete steel casing of a column would consist of two halves, or two parts, that will be welded together in the field. Before placement of the steel casing, concrete cover of the columns would be removed at some locations. The two halves of the steel casing will then be assembled and joined together by longitudinal welding along height of the column. The steel plates are also tied together by means of 1 3/8" ϕ high strength bars that will run through pre-drilled cores in the column (see Figure 28 and Appendix B). These high strength bars will be tightened after assembly of the two steel encasement halves. Gaps between concrete surfaces of the existing column and the steel encasement will be filled by pressure grout to ensure full contact between the existing concrete surface and the steel casing, so that the encasement will be effective in resisting lateral dilation of the concrete core under seismic loading.

Ideally, steel jackets should have a circular or elliptical shape to be most efficient in confinement of the column. For this project circular or elliptical casings would drastically increase the column size and be aesthetically unappealing. The proposed steel encasement instead matches the existing column shape. The flat steel plates along the column faces by themselves will be fully effective in enhancement of shear strength, but not ductility. Enhancement of flexural ductility is achieved by restraining lateral dilation of the column core that would be expected under seismic loading. With the column cross section shape and only the steel plate casing, confinement will be fully provided by the steel plates only at column corners, whereas confinement would be less effective between corners because of the little restraint of the core provided by bending of the steel plate. Thus, use of the steel plate by itself will not be effective for enhancement of ductility. To improve confinement efficiency of the steel casing, the 5/8" steel plate will be stiffened by means of structural steel MC8×18.7 sections that run along perimeter of the column as shown in Figure 28 and Appendix B. The steel channels will enhance bending stiffness of the steel casing and would restrain lateral dilation of the concrete core. These peripheral MC8×18.7 steel channels will be welded at corners of the columns and will be placed along the column height. The steel channels will be closely spaced at top and bottom ends of the column in which higher confinement is needed to the existing column at plastic hinge zones, whereas the spacing is increased at mid-zones of the columns (see Figure 28). Bending stiffness of the steel casing and consequently confinement efficiency will be enhanced furthermore by the high strength bars shown in Figure 28. The column casings terminate below the bent cap and above the footing. A space should be left between the steel encasement and the footing or bent cap to avoid the possibility of the jacket acting as compression reinforcement by bearing against the footing or bent cap at large lateral drifts. To improve the visual aesthetics of this retrofit option, a mortar finish will be added to the outside of the casings. Steel plates, channels and high strength bars that are exposed will be concealed with a 6" thick architectural mortar coating (see Figure 28 and Appendix B). The mortar will be applied by air-blown technique, and the surface of the mortar will be hand finished to simulate the texture of the existing columns.

Appendix D includes D/C ratios for displacement and shear in all columns of the retrofitted structure. The tables given in Appendix D indicate that the displacement D/C ratios are below 1.0 for all columns, indicating that with the infill walls and steel casings, displacement capacity of the structure will exceed the displacement demand from the MCE event. However, the shear D/C ratios for few columns in the West Approach Spans (Frame 1) exceed 1.0. Frame 1 has relatively short columns, which will be more susceptible to shear failure than the tall columns in the East Approach Spans. Shear capacities of the Frame 1 columns are estimated based on the as-built drawings and using only 70% of concrete area for the deteriorated columns. Shear capacities were also calculated assuming that the full concrete area contributes to the shear resistance. In both cases, the shear D/C ratios for some uncased columns exceed 1.0. The shear D/C ratios are high only under longitudinal seismic loading, whereas all D/C ratios are below 1.0 under transverse loading. To reduce the retrofit cost, no steel casings are proposed for those columns with shear D/C ratios exceeding 1.0 (Columns in Bents 1, 4, 5 and 6). However, it is proposed to cut some of the longitudinal reinforcing steel bars to reduce plastic moments of these columns under longitudinal seismic loading, which will consequently reduce the shear demands and the likelihood of shear failure. The reinforcing bars to be cut will be strategically selected such that the moment capacity will be reduced under longitudinal loading without significant reduction in moment capacity under transverse loading. This is because no shear problems are expected in these columns under transverse loading as mentioned earlier and as indicated by the shear D/C ratios given in Appendix D.

13.3.3. Infill Walls:

Seismic transverse displacement demands are reduced by construction of new infill shear walls between the columns at a total of 14 bents. Locations of the infill shear walls are shown on the plans of Appendix B. The shear walls will be constructed in both the transverse and longitudinal directions of the bridge between interior columns in Bents 21-22 and Bents 23-24. The shear walls were laid by an iterative process in order to reduce seismic displacements, so that the displacement D/C ratios of the retrofitted structure would be kept below 1.0. The plans of Appendix B show that the walls are closely spaced in the portions with tall columns in the West Approach Spans (Bents 7-11) and Frame 3 (Bents 12-22) of the East Approach Spans. This is because of the severe deterioration of concrete, which would significantly reduce stiffness of the structure in these zones. Walls are placed at wider spacing in Frame 4 (Bents 23-37) because of the relatively good concrete condition as found in the material testing study (see Appendix G).

All columns with “Moderate-Severe” to “Severe” concrete degradation that coincide with the proposed shear walls will be cased. The shear walls will be connected to the retrofitted columns by means of the 1 3/8" ϕ high strength bars that act as cross ties to the steel encasement; some of these high strength bars will be embedded inside the walls with sufficient length to provide resistance to the shear stresses that would be transferred between the walls and the columns (see typical details of steel encasement in Appendix B). Some of the infill walls will be constructed between un-retrofitted columns; however, these columns have good concrete condition as evidenced from the visual survey and concrete core tests. Also, the seismic force transfer mechanism in the shear walls of Alternative 2 would be different than what is shown in Figure 27 for Alternative 1. In Alternative 1, no footings will be constructed below the shear walls, but the shear walls would be supported on grade beams; thus all seismic forces will be transferred to the ground only at locations of the existing footings, which are represented by the hinged supports in Figure 27. In Alternative 2, new footings will be constructed to support the new infill walls. Thus, the seismic force will be transferred to the ground by a series of compression struts and tension ties in the walls as shown in Figure 29. This would reduce the shear stresses along the interface between the new infill walls and the existing columns.

This was confirmed by linear elastic finite element analyses of two models of a bent with shear walls. In the first model, no footing was assumed below the shear walls (Alternative 1), whereas a footing was assumed to exist below the wall in the second model. The bent was subjected to a seismic force as shown in Figure 27 and Figure 29. These analyses were conducted to only compare the effect of presence of footing underneath shear walls on shears transferred between the columns and the shear walls; these qualitative analyses were simplified and not intended for design purposes. Assuming a seismic force F = 6,000 kips, the analyses showed that the total shear transferred between the columns and the walls would be approximately 5,300 kips and 4,000 kips in Alternatives 1 and 2, respectively. Finite element models, reactions and contour plots of vertical forces per unit width, as obtained from SAP2000, are shown in Appendix E. With reduced shear stress demands at wall-column interface and with steel encasement of significantly deteriorated columns, the high strength bars that connect the infill walls and existing columns, as proposed in Alternative 2 (see typical steel casing details in Appendix B), should be structurally efficient.

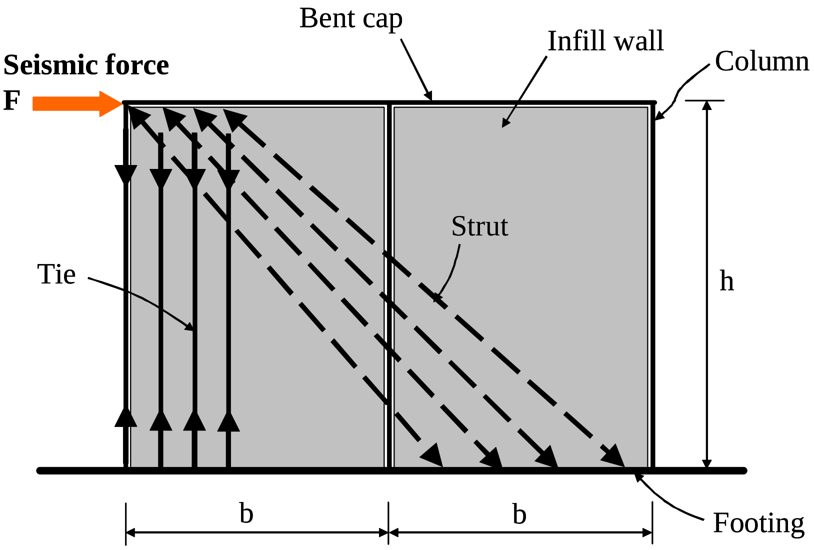

Figure 29. Strut-and-tie seismic force transfer mechanism inside infill walls (Alternative 2)

13.3.4. Foundations Retrofit:

Alternative 2 proposes construction of new foundations at a total of 19 bents in the approach spans. These include foundations to support the infill walls and retrofitted columns in addition to foundations to support new grade beams. The new foundations will have adequate reinforcement to resist plastic moments that could develop in the columns at their bases. Thus, plastic hinges can develop at the top and bottom sections of the columns compared to plastic hinging at only top of columns in the existing structure. At bents with new foundations, columns will have fixity at their top and bottom sections. Thus, the columns will deform in double bending, which will increase stiffness of the structure and subsequently reduce seismic displacements, especially in the longitudinal direction of the bridge. Locations of the new foundations are shown in Appendix B. Top surface of the new footings would be 2 ft below the ground level. The new foundations will be constructed with the placement of new concrete piles, or steel pipe piles, around the existing column foundations. To improve stability of the footings, uplift tie-downs (soil anchors) may be required at some columns where there are large uplift demands on the foundation that could result in rocking response and excessive displacements of the superstructure. Additional 4 longitudinal and 2 transverse grade beams are also proposed in Alternative 2. Locations of the new grade beams are shown in Appendix B. The new grade beams are strategically located in the approach spans in order to reduce seismic displacements of the retrofitted structure, especially under seismic longitudinal loading.

13.3.5. Bent Caps Retrofit:

As mentioned earlier, steel casing of the columns would also require increasing flexural strength of some bent caps to ensure that plastic hinges would form in the columns under seismic loading. This is because the existing bent caps have limited ductility capacity due to poor detailing. Clearly, no retrofitting is needed for bent caps at locations of the infill walls. However, cap beams at bents with no infill walls, but with retrofitted columns or existing columns in relatively good conditions would need to be retrofitted based on displacement D/C ratios and locations of potential plastic hinges in the bent caps (as obtained from analyses of the as-built structure). Alternative 2 includes retrofitting of three bent caps (Bents 8, 16 and 32) by means of concrete bolsters constructed on both sides of the bent cap as shown in Figure 30. These bolsters will be constructed on sides of the bent cap and around the longitudinal superstructure T-girders. The bolsters will be bonded to the existing bent caps by dowels that run through pre-drilled cores in the existing bent cap. Continuity of the concrete bolsters along length of the bent cap would be achieved by post-tensioning of high strength bars that would run through pre-drilled cores in the superstructure girders (see Figure 30). The post-tensioning bars would be anchored at their ends by exterior steel plates; these exposed plates and the bars would be concealed by mortar as for the column steel jackets.

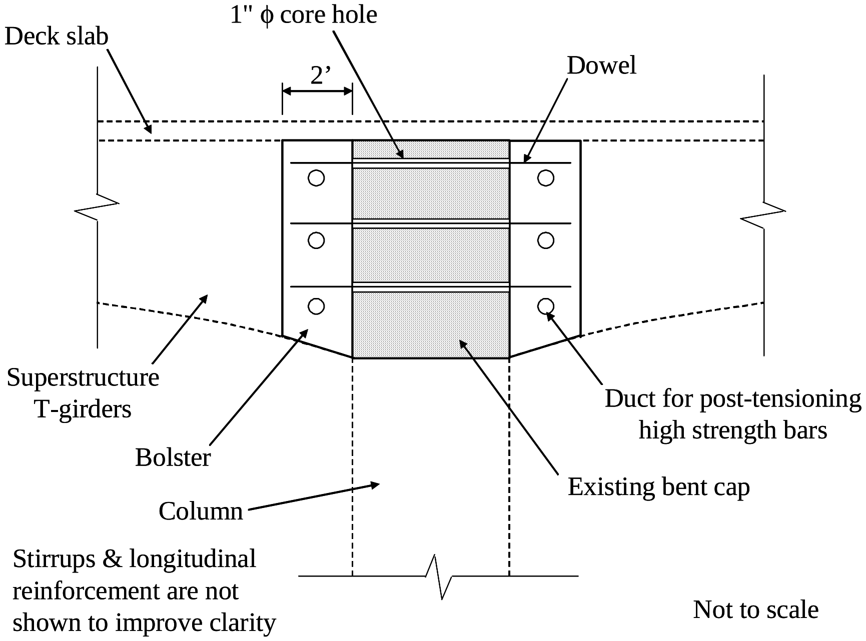

Figure 30. Retrofitting of bent caps by concrete bolsters

13.3.6. Closure of Expansion Joints:

Alternative 2 also proposes closing two existing expansion joints and retrofitting the bent caps at Bents 27 and 33 in order to reduce seismic displacement demands in longitudinal direction of the bridge. Determination of expansion joints that are proposed for closure was done though an iterative process during the preliminary design of retrofit Alternative 2. Locations of all expansion joints are shown in the plans of Appendix B in which "E" indicates an expansion joint, or pinned end of the superstructure, whereas "F" indicates that the superstructure has a fixed end, or is monolithic with the bent cap. Expansion joints at Bents 27 and 33 will be closed. At Bent 27, both spans of the superstructure are simply supported on the bent cap, whereas only one of the two adjoining spans is simply supported on the bent cap at Bent 33. Figure 31 shows a proposed retrofit for the cap beam at Bent 27. An infill wall will be constructed underneath the cap beam; thus there is no need to increase flexural capacity of the bent cap at Bent 27. Concrete bolsters will be constructed at the two sides to improve resistance of the superstructure girders against sliding during the seismic event. New concrete will be poured to fill the gaps between the existing superstructure girders, the existing bent cap and the new concrete bolsters.

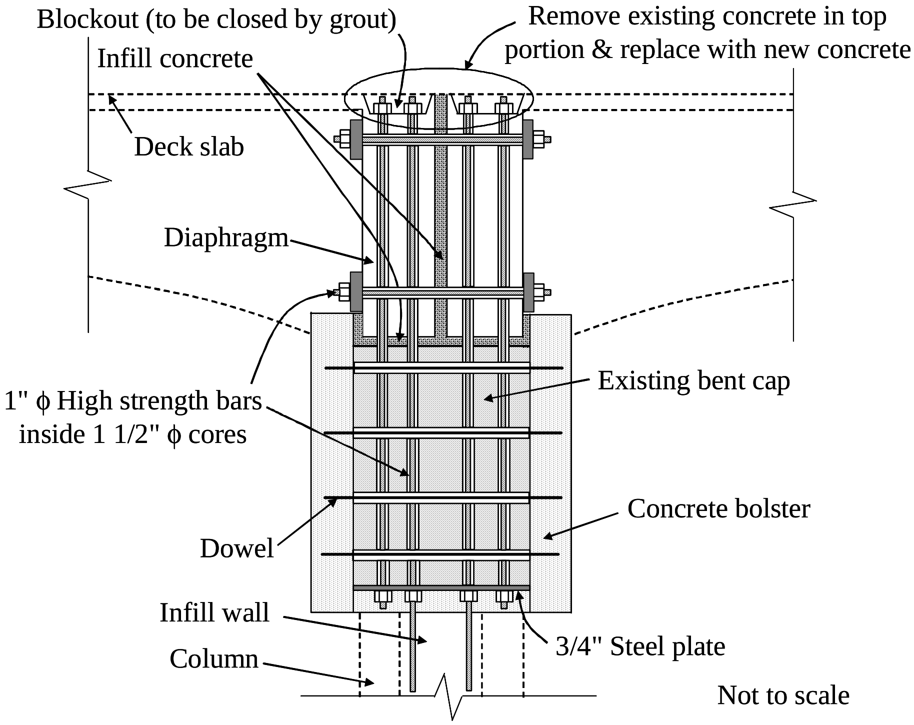

Figure 31. Bent cap retrofit and closure of expansion joint with two simply supported superstructure spans (Bent 27)

The top portion of the existing diaphragm beams at the locations of columns will be removed; this will be followed by placement of vertical high strength bars that connect the diaphragm beams to the bent cap as shown in Figure 31; some of these vertical high strength bars will be embedded in the infill wall to ensure good bond between the wall and the bent cap.

The cap beam at Bent 27 has severe concrete deterioration; thus, a steel plate will be placed at the soffit of the existing cap beam to improve shear transfer between the bent cap and the new infill wall. The steel plate will be bonded to the existing concrete by means of the high strength bars that run through concrete cores as shown in Figure 31. The exposed steel plate and bars at soffit of the existing bent cap will be concealed by architectural mortar similar to the column encasements.

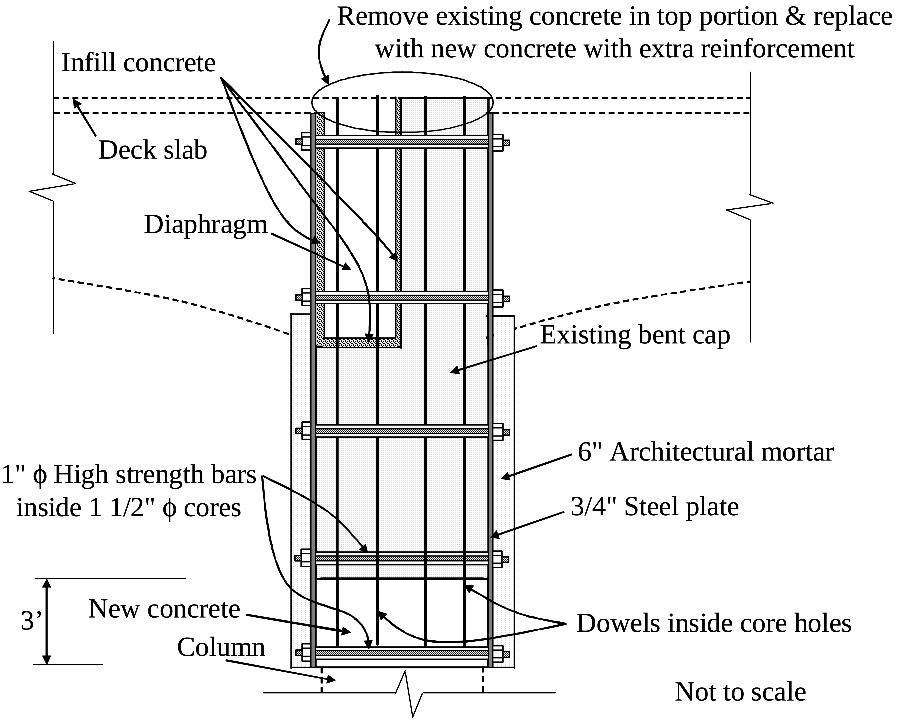

Figure 32. Bent cap retrofit and closure of expansion joint with one simply supported superstructure span (Bent 33)

Similarly, the expansion joint with only one simply supported superstructure span is closed. Figure 32 is a schematic of the proposed retrofit for the expansion joint at Bent 33. No shear walls exist underneath Bent 33; thus, positive flexural moment capacity will be increased by addition of a drop cap at soffit of the bent cap. The negative flexural moment capacity will be enhanced by removal of top portion of the existing bent cap and diaphragm, placement of more top reinforcement and pouring new concrete that will also fill the gaps at the expansion joints. Also, steel plates will be placed along vertical side faces of the bent cap, which will enhance flexural strength as well as resistance to horizontal shears transferred between the new drop cap and the existing bent cap.

13.3.7. River Piers Retrofit:

Retrofit of the river spans have been discussed in detail in Section 12. As mentioned earlier, Alternative A of river spans retrofit is the proposed alternative. Thus, infill walls will be placed between columns in the West and East River Piers and new pile foundations will be also constructed around the existing foundations at the West and East River Piers. The retrofit concept for the main arch spans will be similarly applied to Alternatives 2 through 5 proposed below.

13.3.8. Discussion:

The steel encasement (see Figure 28) is designed to provide sufficient lateral confining pressure to resist dilation of the concrete cores under seismic forces. Based on the material sampling and testing study, it is believed that ASR will continue to occur due to highly reactive aggregates in concrete used throughout the existing structure. ASR will result in increased dilation of the concrete core and consequently would increase the required lateral confining pressure substantially. Based on experimental data[16], it is estimated that prevention of concrete dilation due to only ASR would require lateral confinement pressure of approximately 435 psi, which is about 145% of the confinement required to resist effects of seismic loading. This high level of internal pressure caused by ASR could damage the column casings and their anchoring high strength bars, thus reducing effectiveness of the column casings. It is expected that significant retrofitting of the bridge would be required in approximately 10 years after Alternative 2 retrofitting in order to maintain seismic and operational safety of the structure.

The retrofitted columns would look similar to the existing columns. Thus, Alternative 2 could meet historical aesthetics requirements. However, Alternative 2 design by itself does not meet all SHPO requirements and the structure would require some form of environmental mitigation. Possible environmental mitigation measures that may be required by SHPO include replacement of barrier rail, rehabilitation of electroliers and restoration of the Center River Pier obelisks that have been removed because of severe concrete deterioration.

13.4. ALTERNATIVE 3 (CATCHER WALLS)

13.4.1. Scope:

The objective of this retrofit design is to increase seismic safety by preventing collapse of the viaduct during an earthquake with no regard to historical integrity. The design consists of constructing catcher walls at locations of all bents except Bent 12. This bent is excluded because of the tight room available for construction of the catcher walls due to proximity of railroad tracks. These catcher walls provide a secondary support system to the bridge that supplements the existing columns and foundations in the event of column collapse.

13.4.2. Structural System Description:

The catcher wall system will serve to catch the superstructure, girders and bent cap in case of column failure. Drawings of the catcher wall system and their foundations at a typical bent are given in Appendix B. At each typical bent, the catcher wall system consists of two shear walls that are parallel to the bent cap centerline. One shear wall will be constructed on each side of the bent cap as shown in the plan view in Appendix B (see plans for Alternative 3). These two walls will be referred to here as primary shear walls.

Short secondary shear walls will connect the above-mentioned two primary shear walls. The secondary walls will be constructed perpendicular to the primary walls as shown in the plan view in Appendix B. The objectives of secondary walls are to catch the bent cap and provide stiffness for the catcher wall system in longitudinal direction of the bridge. The catcher walls will be supported on new pile caps and new piles, which will be sized adequately to support self weights of the walls and dead load of the superstructure in case of collapse during an earthquake. The pile caps and piles will support only the primary walls as shown in Appendix B, whereas the secondary walls will only connect the two primary walls as mentioned above. A horizontal cantilever slab will be constructed at top of each primary wall as shown in Appendix B (see Section B-B). These two horizontal slabs at top of the catcher wall system serve as a seat extender to contribute to carrying the bridge superstructure in case of collapse during an earthquake. At locations of skewed bents, such as Bents 21-22, Bents 23-24 and Bents 29-30, the catcher wall system will consist of three primary walls and a set of secondary walls.

The catcher wall system is not attached to the existing structure. Thus, the catcher walls will not modify seismic response, nor will eliminate seismic deficiencies of the existing structure. However, the catcher walls are designed to support the superstructure in case of a catastrophic failure during an earthquake. In other words, these walls will “catch” the superstructure and prevent its collapse during an earthquake and thus would increase seismic safety.

Each catcher wall system at each bent must consist of at least two primary walls. This is because of framing type of the existing structure. The superstructure consists of continuous spans and moment-resisting bent caps. The superstructure has expansion joints at about every third bent cap. With two spans joining the bent at expansion joints, a single catcher wall constructed at the center of the bent would be ineffective. This is because the superstructure could experience seismic longitudinal displacements in excess of 30 inches and the single catcher wall will not be able to support the adjoining spans if the cap beam is drifted off the bent centerline.

13.4.3. River Piers Retrofit:

The river piers will be retrofitted as for Alternative 2 and as discussed in Section 12.

13.4.4. Discussion:

The catcher wall system will completely alter aesthetic appearance of the structure since the two primary walls at each bent will cover up the existing columns. Thus, this retrofit design alternative does not meet historical aesthetic requirements of the bridge. As a result from the drastic change, environmental mitigation is not available for the Alternative 3 design. The catcher wall retrofit design also does not include any preventive measures to protect the structure from future concrete degradation due to ASR.

In summary, this alternative will only increase seismic safety by prevention of structural collapse, but will not improve seismic performance of the existing structure. Thus, seismic damage will be high, with almost no chance for repair available after a large seismic event. Life expectancy of the structure if retrofitted with catcher walls would be approximately 10 years since ASR degradation will continue. Construction costs will be high and the structure will not meet historical aesthetic requirements. For these reasons, it is believed that this alternative will not be acceptable to the Los Angeles City BOE.

13.5. ALTERNATIVE 4 (CONCRETE CASINGS)

13.5.1. Scope:

This alternative utilizes concrete column casings to increase the ductility and stiffness of the existing structure. Alternative 4 is similar to Alternative 2 in that the existing columns will be encased to provide additional confinement to resist lateral dilation of the core. Alternative 4 proposes retrofitting of all columns, bent caps and construction of new foundations at bents with "Moderate-Severe" to "Severe" concrete column degradation, based on results of the material sampling and testing study[1]. No infill shear walls are proposed in this alternative since the concrete column casings and the bent cap retrofit will increase the stiffness of the structure and consequently reduce seismic displacements. The new foundations will also provide fixity at base of columns, which would consequently enable the columns to deform in double bending and reduce seismic displacements furthermore. Retrofit is proposed at a total of 28 bents along the viaduct (see Appendix B). Bent 12 is excluded from retrofitting because of the tight room available for retrofit construction; however, it should be noted that seismic displacement demands of the retrofitted structure are less than displacements of the existing structure because of the higher stiffness provided by the concrete casings. In most of these bents, concrete degradation was more significant in columns compared to concrete degradation in the bent caps.

13.5.2. Columns Retrofit:

Alternative 4 proposes encasement of columns by concrete jackets, which would enhance flexural strength, ductility and shear strength of the columns. This retrofit design will also increase stiffness of the structure, which would reduce seismic displacements. The concrete jacket consists of either a circular or elliptical 12-in. thick concrete ring that has two layers of longitudinal reinforcement and transverse closed hoops; this reinforced concrete ring is referred to as the confinement ring throughout this report. The shape of the confinement ring, whether circular or elliptical, depends on dimensions of the existing column to be encased by the concrete jacket. Many interior (center) columns have similar out-to-out dimensions in two orthogonal directions; thus the confinement ring used in retrofitting of these columns would be circular. However, elliptical confinement rings are used for exterior columns. Typical details for concrete jackets are shown in Appendix B.

Use of circular or elliptical concrete jackets with closed hoops for transverse reinforcement increases efficiency of the design in perspective of confinement to the concrete core and would thus enhance flexural ductility of the existing columns. Clearly, the concrete jacket with its transverse reinforcement will also significantly increase shear strength of the columns. Use of longitudinal reinforcement in the concrete jacket will also increase flexural capacity of the columns and would require retrofitting of the bent caps and foundations. Appearance of the columns would be significantly altered if the concrete jackets have circular or elliptical shapes. To preserve the historical appearance of the bridge, outer surface of the concrete jacket will be similar to that of the existing column as shown in Appendix B. Only the confinement ring would provide the required confinement to the core of an existing column, whereas the outer layer of the jacket consists of architectural concrete. The architectural concrete layer will be lightly reinforced and a space should be provided between the architectural concrete and any supporting member such as the footing or bent cap to avoid premature spalling of the architectural concrete under large seismic lateral drifts.

With this design, out-to-out dimensions of the existing column cross section will increase by as much as 40 inches. The concrete encasement will be constructed along full length of the columns. To increase the stiffness of the structural system, the longitudinal reinforcement in confinement ring of the concrete encasement will be embedded into the bent cap and will also be embedded inside new pile caps that will be constructed to support the retrofitted columns. With these details, flexural capacity of the column at top and bottom ends will increase and will require retrofitting of the bent cap. Design of the new pile cap takes into account the increase in flexural strength of the columns.

In Alternative 2, the steel encasement consists of welded steel plates. The encasement is fabricated in two halves that will be assembled in the field. Although prefabrication of the steel encasement without sacrificing structural efficiency is possible, this may not be possible for concrete encasements. This is because if the concrete encasement is composed of two precast halves, continuity of the transverse hoops will not be achieved at the interface between the two encasement halves, unless the two halves are tied together by a set of high strength bars that run perpendicular to the interface between the two precast halves in addition to another set of high strength bars in the orthogonal direction. Continuity of the transverse hoops is essential for the design to be effective. Thus, if the concrete encasement consists of two precast halves, high strength bars should be carefully designed and detailed to achieve continuity of the hoop reinforcement. Another concern with precast encasement is the total length of columns, which reaches 60 ft in the East Approach Spans. This would require construction of several precast segments along height of the column; these precast segments need to be erected together such that splices of longitudinal reinforcement will be adequate. This may be achieved by having a gap between the vertical precast segments; the longitudinal bars will protrude from each of the two precast segments and they will be spliced within the gap zone. The gap will be closed later by cast-in-place concrete. Despite the higher concrete quality control and faster construction, the above discussion indicates the construction difficulties associated with use of precast concrete encasements.

An alternative to precast concrete casings is cast-in-place (CIP) casings. Reinforcement of the concrete encasement will be placed around the existing column. Transverse hoops of the confinement ring will be welded in the field. Since longitudinal reinforcement will be embedded in the bent cap as well as in the new pile cap, construction of CIP casings will clearly be simpler. Thus, CIP casings are assumed for this alternative.

As mentioned earlier, the outer layer of the concrete encasement will consist of architectural concrete with the same cross-sectional shape as that of the existing column. Special texture can be applied to the formwork to simulate fabrics of the existing columns and to create a similar appearance of the existing structure. A total of 80 columns will be retrofitted with concrete encasements. With this encasement design, a close match to the historical appearance of the existing bridge columns can be maintained.

13.5.3. Foundations Retrofit:

As mentioned earlier, all of the columns in all bents with “Moderate-Severe” to “Severe” concrete column deterioration along the viaduct, except in Bent 12, will be encased (total of 28 bents). New foundations will be constructed to support the encased columns because of the higher flexural capacity of the columns provided by the concrete encasement. As discussed earlier, the new foundations also provide fixity to the columns at their bases. Thus, the columns will be in double bending, which will increase stiffness and reduce seismic displacement demands. The new foundations will include a new pile cap around the columns and existing footings supported on new precast concrete piles. The new pile cap will be bonded to the existing footing by means of drill-and-bond dowels. Longitudinal reinforcement in the confinement rings of the column encasement will be embedded in the new pile caps.

13.5.4. Bent Caps Retrofit:

Bent caps need also to be retrofitted at bents with encased columns. This is because of the increased flexural capacity of the encased columns, which would require increase of flexural capacity of bent caps at same locations to ensure that plastic hinges would form in the columns, rather than in the existing bent caps. Again, plastic hinges should be avoided in bent caps due to poor detailing and inadequate ductility of the existing cap beams. Retrofitting of 28 bent caps (see Appendix B) is proposed in Alternative 4. Retrofitting of bent caps will be achieved by concrete bolsters that will be bonded to the existing cap beams by dowel reinforcement. The bolsters will be post-tensioned along length of the bent cap. Typical details of bolsters at locations of bent caps with no expansion joints are shown in Appendix B as well as in Figure 30.

Expansion joints exist in the two adjoining superstructure girders at locations of some bents; these are designated by “E/E” in the drawings given in Appendix B. Figure 33 shows the proposed retrofitting of bents caps at locations of expansion joints with two simply supported superstructure spans. Figure 33 shows that retrofitting of the existing bent caps can be achieved by bolsters along sides of the existing cap beams. The bolsters are bonded to the existing bent cap by dowels as shown in Figure 33. Figure 34 shows bent caps with an expansion joint at only one of the two adjoining spans (bent caps designated by “E/F” in drawings of Appendix B). These bent caps will be retrofitted by concrete bolsters and dowel reinforcement as shown in Figure 34. Negative flexural moment capacity will be increased by removal of the top portion of the existing bent cap; this will be followed by placement of more top reinforcement and new concrete (see Figure 34).

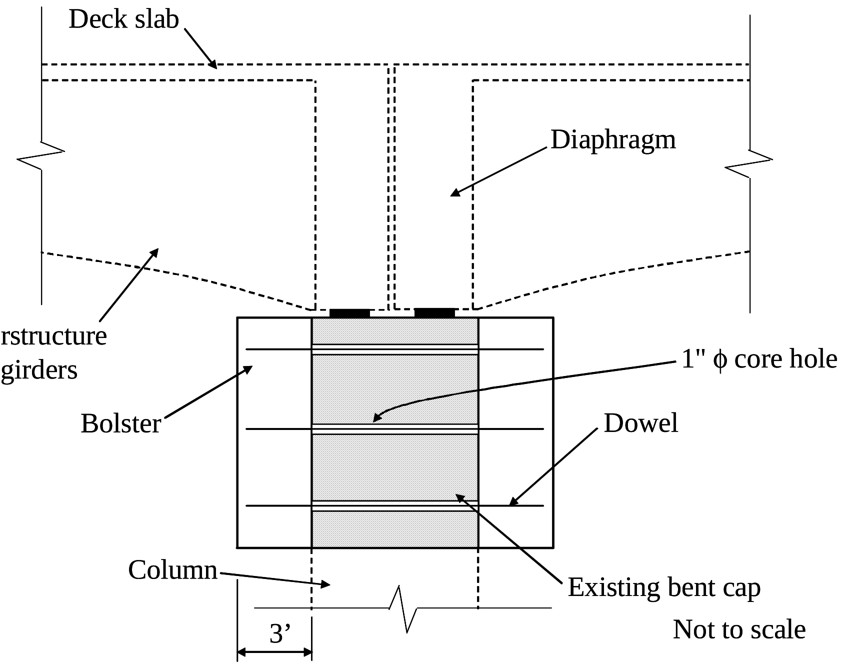

Figure 33. Alternative 4 retrofit of bent caps at expansion joints (two simply supported spans)

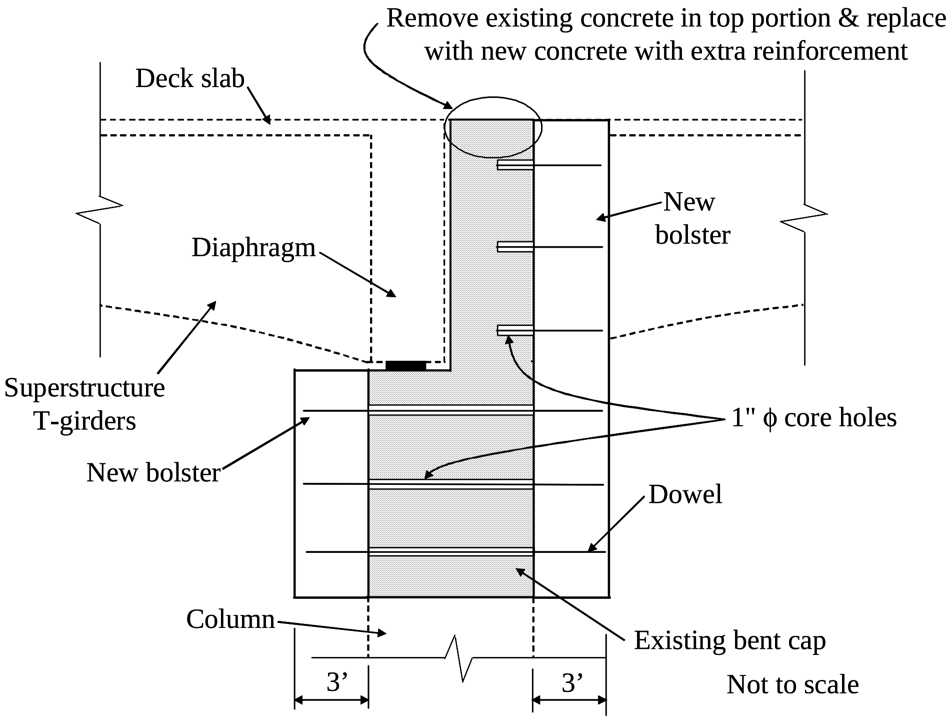

Figure 34. Alternative 4 retrofit of bent caps at expansion joints (one simply supported span)

13.5.5. River Piers Retrofit:

The river piers will be retrofitted as for Alternative 2 and as discussed in Section 12.

13.5.6. Discussion:

Alternative 4 has similar shortcomings as Alternative 2. Design of the concrete encasement will not meet the required strength to withstand the high internal pressure from future ASR that will occur after construction of the concrete encasements. Because ASR would only occur when moisture is introduced to the concrete, water tight seals will be an important detail to implement to the concrete encasements. Construction of the concrete encasement will take place with rigorous water and moisture control of the existing concrete to prevent trapped moisture inside the encased sections of columns. Alternative 4 by itself does not meet all SHPO requirements and the structure would require some form of environmental mitigation as discussed earlier for Alternative 2 (see Section 13.3.8). If Alternative 4 is chosen for retrofitting, the life of the retrofitted structure is estimated to be 20 years. This means that major retrofitting would be needed after 20 years of retrofitting of the existing structure in order to maintain its seismic and operational safety.

13.6. ALTERNATIVE 5 (HEAVY STEEL CASINGS)

13.6.1. Scope:

Alternative 5 is similar to Alternative 2 in that columns will be retrofitted by steel casings and infill walls will be constructed at some of the bents. As discussed in Section 13.3, design of steel casings in Alternative 2 does not account for internal pressure resulting from lateral dilation of the concrete core caused by ASR, which could result in damage of the steel casings in the future. Effect of ASR on future concrete degradation of columns with no retrofitting is expected to shorten life expectancy of the retrofitted structure. However, Alternative 5 will fully address the ASR concerns of Alternative 2.

13.6.2. Columns Retrofit:

Compared to Alternative 2, Alternative 5 proposes steel casing of more columns. Except for Bent 12, all columns that are currently identified to have “Moderate-Severe” to “Severe” damage ratings will be encased to reduce the possibility of further deterioration. Additionally, the steel casings will be designed to withstand the high level of internal pressure due to ASR-induced lateral dilation of the encased column. Bent 12 is excluded from retrofitting because of the tight room available for construction of the column encasement due to proximity of railroad tracks. However, it should be kept in mind that seismic displacement demands can be significantly reduced compared to displacements of the as-built structure by the proposed infill shear walls, new foundations, new grade beams and expansion joints closures (see Appendix B). All exterior columns with “Light” or “Moderate” damage ratings will also be encased to account for future concrete degradation due to ASR. Encasement of all exterior columns will also maintain visual balance and consistency for the retrofitted structure. In addition to the above mentioned columns, the interior columns in Bents 1, 4 and 5 will be encased to enhance their shear strengths; this is because seismic demand and capacity analyses of the Alternative 2 retrofit demonstrated that the shear D/C ratios for these columns exceed 1.0. A total of 76 columns will be encased by steel jacket in Alternative 5.

Research conducted by the Transtec Group, Inc. for the FHWA indicated that a volumetric expansion exceeding 0.6% could occur to concrete as a result of ASR, when the concrete member is allowed to expand freely[16]. This was based on experiments conducted by Le Roux et al.[16] in which the volumetric expansion was measured over a one year period. The same experiments indicated that external lateral pressure as high as 725 psi would completely restrain dilation of the concrete member due to ASR. The experiments also indicated that volumetric expansion of approximately 0.08%, which is practically no expansion, would result in internal dilation pressure of approximately 435 psi[16]. Thus, for design of Alternative 5 steel casings, an internal pressure of 435 psi was assumed to occur due to ASR. This internal pressure was added to internal pressure induced by seismic forces, which was approximately 300 psi. As a result of this, steel encasements of Alternative 2 were re-designed to resist internal pressure of 735 psi. The re-design indicated that thickness of the steel plates should be increased to 7/8" (compared to 5/8" steel plates in Alternative 2; see Figure 28). Thus, even if internal stress in the column casing increases due to continued ASR, the seismic retrofit will perform adequately.

Steel MC8×18.7 channel sections will also be used (see Figure 28) to increase bending stiffness of the steel casing to resist lateral dilation of the column. The number of 1 3/8" ϕ high strength bars that function as cross-ties for the steel encasement will also be increased compared to Alternative 2, because of higher internal pressure. Steel casings with 7/8" plates and increased number of high strength bolts will be used for a total of 26 columns (see Appendix B).

A total of 50 additional columns will also be steel encased with 5/8" steel plates as proposed in Alternative 2, which will reduce future concrete degradation. Casings of these additional columns are designed to resist internal pressure caused by ASR. The steel casing of these columns consist of 5/8" plates with MC8×18.7 channels and 1 3/8" ϕ high strength bars similar to those shown in Figure 28. In all 76 columns that will be retrofitted by steel casings, the exposed plates, channels and bars will be concealed by a 6-in. layer of architectural mortar. As discussed in Section 13.3 for Alternative 2, a space should be provided between the encasement and surfaces of the footing and bent cap. Locations of columns that will be retrofitted by steel casings are shown in Appendix B. The columns with heavy steel casings (7/8" plates) are distinguished in the plans from columns with 5/8" plate casings (see Appendix B).

13.6.3. Infill Walls, New Foundations, Grade Beams and Closure of Expansion Joints:

Alternative 5 proposes construction of new infill shear walls as for Alternative 2 to reduce transverse seismic displacements. The new infill walls will be constructed over new pile foundations. As in Alternative 2, new grade beams will be constructed to reduce seismic displacements, especially in the longitudinal direction (see Appendix B). Expansion joints in the superstructure at Bents 27 and 33 (see Appendix B) will also be closed as for Alternative 2 in order to reduce seismic longitudinal displacement demands for the East Approach Spans (see Figure 31 and Figure 32).

13.6.4. Bent Caps Retrofit:

Because more columns are retrofitted in Alternative 5, compared to Alternative 2, more bent caps would need to be retrofitted for enhancement of flexural strength. This is to ensure that no plastic hinges will form in the bent caps after enhancement of ductility and displacement capacity of the columns. Retrofitting of bent caps for flexural strength enhancement is proposed at 16 bents; this does not include the two bent caps where expansion joints will be closed (Bents 27 and 33). As discussed in previous sections, bent cap retrofit will be achieved by means of concrete bolsters (see Figure 30). The bolsters will be bonded to the existing bent caps by dowel reinforcement bars that run through 1" ϕ cores. The concrete bolsters will be post-tensioned along length of the bent cap by means of high strength bars. Proposed retrofitting of bent caps in all bents with no expansion joints is similar to that shown in Figure 30.

Bent caps at locations of expansion joints will be retrofitted as shown schematically in Figure 35 and Figure 36. The positive flexural moment capacity will be enhanced by adding drop caps at soffit of the existing bent caps as shown in Figure 35 and Figure 36. The new drop caps will be bonded to the existing bent cap by dowels. Steel plates will be placed along sides of the bent caps and bonded to the concrete by means of high strength bars inside core holes. The steel plates contribute to enhancement of flexural capacity and also would enhance bond and resistance to horizontal shears transferred between the new drop cap and the existing bent cap.

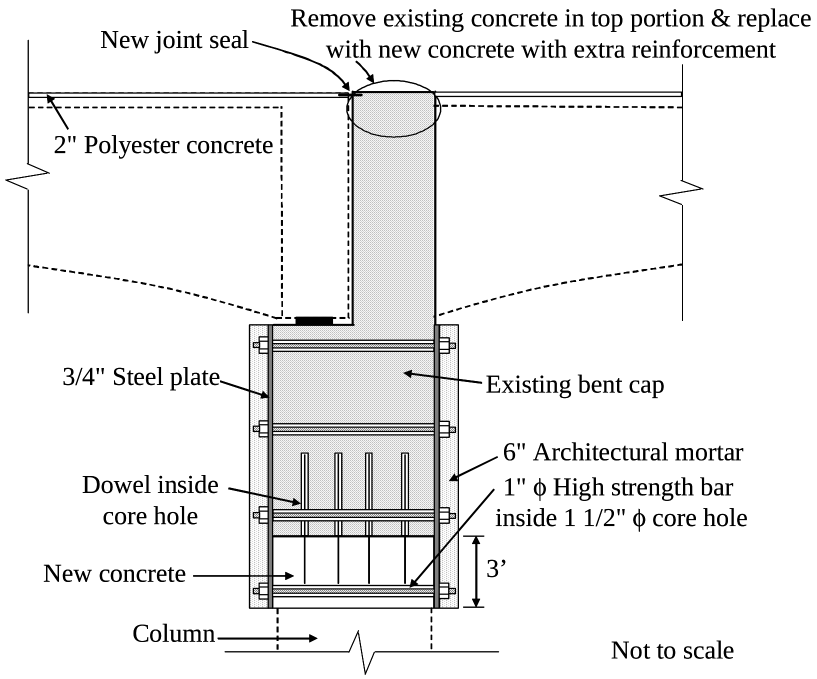

Figure 35. Alternative 5 bent cap retrofit at expansion joints (one simply supported span)

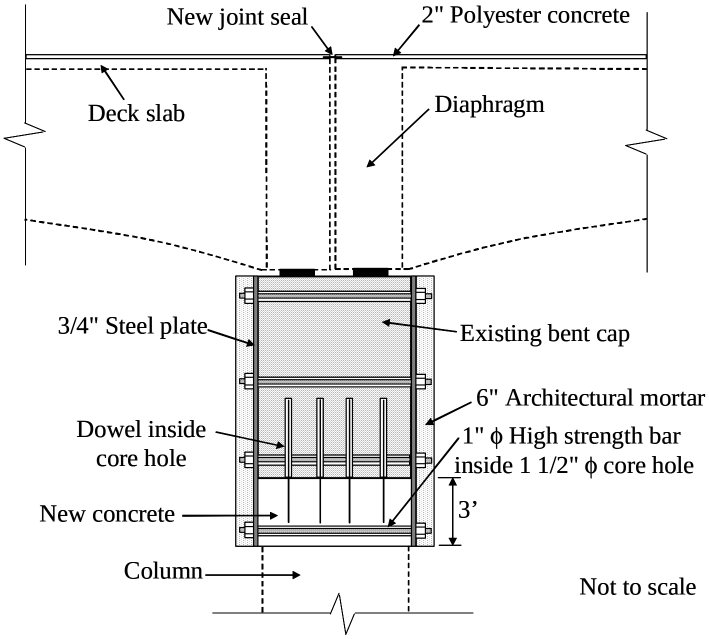

Figure 36. Alternative 5 bent cap retrofit at expansion joints (two simply supported spans)

13.6.5. River Piers Retrofit:

The river piers will be retrofitted as for Alternative 2 and as discussed in Section 12.

13.6.6. New Expansion Joint Seals:

Alternatives 2 to 5 are concerned with retrofitting of the Sixth Street Viaduct to meet seismic safety requirements. Alternative 5 is the only one that considers the effects of future deterioration of concrete on seismic safety of the retrofitted structure. To ensure long-term efficiency of the Alternative 5 design, installation of new expansion joint seals is essential. The objective of the new expansion joint seals is to protect the substructure from moisture, and consequently reduce further concrete degradation caused by ASR. Thus, cost for installation of new expansion joint seals is included in cost analysis for seismic retrofit in Alternative 5 (Section 14.1). Figure 35 and Figure 36 show the proposed new expansion joint seals.

Figure 35 and Figure 36 show proposed rehabilitation for the deck in Alternative 5. Deck rehabilitation is not part of the seismic retrofit Alternative 5 and it will be discussed in more details in Section 14.2 (life cycle cost analysis).

13.6.7. Discussion:

Alternative 5 is the only one that provides effective seismic retrofit of the existing columns, with full considerations given to the current as well as future conditions of concrete of the Sixth Street Viaduct. Since effects of ASR on material degradation is considered in Alternative 5 design, future retrofitting to maintain seismic and operational safety of the structure may not be required before 30 years after retrofitting as proposed in Alternative 5.

As in Alternative 2, visual appearance of the existing structural elements will be maintained in Alternative 5 and the design will meet historical aesthetic requirements. However, environmental mitigation may be needed as discussed earlier for Alternative 2 (see Section 13.3.8).

13.7. ALTERNATIVE 6A (BRIDGE IN-KIND REPLACEMENT)

13.7.1. Objective:

Bridge replacement alternative is introduced in the seismic retrofit strategy for the purpose of providing a comparative study based on cost and functionality of the structure. The objective of Alternative 6A is to propose replacement of the existing structure with a new one that maintains the historic prominence of the Sixth Street Viaduct. Alternative 6A proposes in-kind replacement on the same horizontal and vertical profiles of the existing structure, which would result in a long structure with tall columns on the East side of the Los Angeles River.

The new structure will have a 65.5 ft curb-to-curb distance in addition to 5 ft side walks; thus, total width of the new structure is 75.5 ft and the total width of the deck slab is 77.5 ft. The new structure is wider than the existing one, which would also increase traffic safety. The wider structure is required according to the FHWA EBL. Preliminary plans of the proposed in-kind replacement are given in Appendix B (Alternative 6A).

13.7.2. Design Considerations:

Recognizing the importance of the structure to the City of Los Angeles, and its historical significance, conceptual design of the replacement structure has to address the following:

1. Respecting the importance of the "feel and setting" of the structure, and the high quality and ornate architectural details used in the original construction, which should either be replicated or replaced with equally high quality construction details.
2. This viaduct has unique steel through double arch spans, which is the only one that exists over the Los Angeles River. The replacement structure has to either preserve or replicate the unique arch spans over the Los Angeles River.
3. Small commercial and industrial tenants occupy areas around the structure. Construction of the replacement structure, as well as its function has to preserve the commercial uses of the properties alongside the viaduct.
4. Sixth Street serves as a critical transportation link between the Downtown and East Los Angeles. The structure is also the main route to several transit bus lines. Construction of the replacement structure has to address the traffic impact, and full traffic management plan will be required.

13.7.3. Structure Description and Type:

Preliminary drawings of the new replacement bridge are given in Appendix B. Lengths of spans of the new structure will be longer than the existing structure. The longer spans will reduce the construction impact of the substructure and foundation, and avoid much of the interference with the existing structure and the foundations, as well as reduce foundation costs.

The new replacement structure will have seven spans on the West Approach between the West Abutment and the West River Pier. The East Approach will consist of 14 spans between the East River Pier and Bent 37. Span length would vary between 80 ft and 156 ft, with average span length of 130 to 140 ft. The superstructure will be constructed with cast-in-place (CIP) concrete multi-cell box girder. The box girder will have a parabolic soffit as shown in Appendix B with a variable girder depth between 4.5 ft and 6.5 ft in a typical span. Depth of the box girder may reach up to 8 ft at some of the bents. The parabolic soffit of the superstructure will preserve visual appearance of the existing structure. The bent cap overhang will be constructed with similar details to those of the existing structure. Concrete barrier rails Type T-80 will be used to replace the existing railing and sidewalk.

The steel arches over the Los Angeles River will be preserved in the new replacement structure. The superstructure over the Los Angeles River consists of a CIP box girder as described above. However, the steel arches will be moved and reset on the exterior sides of the new superstructure to maintain the visual appearance of the existing bridge. The steel arches will not participate in load carrying capacity of the new bridge portion over the Los Angeles River. With this proposal, the steel arches will carry only their self weight as well as self weights of the vertical hangers and bracing members.

Circular columns with diameters ranging from 6 ft to 7 ft are proposed for the new structure. The circular columns will be covered by architectural precast concrete casings that have similar exterior shape as that of the existing columns. The architectural casing is not coupled with the circular column and the casing will not participate in load carrying capacity of the columns. The precast architectural casing can be only 6 inches thick. The objective of the architectural concrete casing is to maintain the visual appearance of the existing columns. The circular columns will be constructed first. This will be followed by placement of the architectural precast casings around the columns and before construction of the superstructure. The architectural casings will be precast segmental and they will be bonded together by means of shear keys and epoxy bonding at the segment-to-segment joints. Light vertical post-tensioning may also be provided to improve bond between the segments. The columns and the architectural casings will be supported on pile foundations with Class 100 piles.

On its West end, the superstructure will be supported on a seat type abutment with pile foundations. On the East end, the superstructure will be supported on Bent 37. No replacement is proposed in

Alternative 6A for the bridge portion between Bent 37 and the East Abutment (above the US101 Freeway), which is owned by Caltrans.

13.7.4. Traffic Handling During Construction:

One goal of the retrofit is to minimize traffic impacts on Sixth Street. One scheme studied to maintain traffic during construction of the new structure was to stage construct the bridge by splitting the bridge down the middle. Traffic would be changed from four lanes to two lanes and the outside column removed. Half of the new structure could be constructed without sidewalks to allow two lanes of traffic after completion of this half. Once the first stage is completed, traffic could be moved to the new structure and the remaining portion of the old structure removed and reconstructed. While this scheme allows for traffic to use the structure during the approach retrofits, traffic would still be required to be closed during replacement of the main spans since both arches must be in place to support the roadway.

Stage constructing the bridge in the manner above works geometrically, however it is strewn with technical difficulties and represents a large risk for the City. Demolishing the outer column changes the service load path within the bent cap of the structure. Currently the bent cap spans between three columns with the maximum negative moment over the center column. By removing the outer column, the bent cap will experience large positive moments between the center column and remaining edge column. In addition, the more slender center column will be eccentrically loaded with a larger unbalanced moment introduced by both dead load and live load. Since the concrete in this structure is severely deteriorated, it is unknown how these load path changes will affect the structure. Several bents consist of two column bents. At these locations temporary shoring would be required to support half of the existing structure and the live load. This carries substantial risk and is not agreeable to the City. Additionally it is more expensive to stage construct the bridge than to detour traffic around.

Because of the concerns discussed above and based on consultation with the Los Angeles City BOE, it is strongly recommended that stage construction for Alternative 6A by removal of half of the existing structure is not technically feasible. Alternatively, a detour route should be selected and the bridge should be completely closed during construction of the new bridge. A detailed Traffic Management Plan should be prepared in the PS&E phase of the project if this alternative is selected.

13.7.5. Discussion:

The new bridge will be constructed using modern materials and construction details. Thus, life expectancy of the new structure is estimated to be 75 years. The new structure will meet historical aesthetic requirements since it is an in-kind replacement of the existing structure. Environmental mitigation is already included in this alternative. Additionally, this alternative provides a wider roadway width that will meet the goal of removing the structure from the FHWA Eligible Bridge List for HBRR funding.

13.8. ALTERNATIVE 6B (REPLACEMENT WITH CIP BOX GIRDER BRIDGES ON A REVISED ALIGMENT)

13.8.1. Objective:

The objective of Alternative 6B is to propose a replacement of the existing structure with minimum construction costs. Though this alternative does not maintain the historic prominence or many of the existing features, it serves as least cost replacement alternative if this was a new route to be constructed today. As discussed above, Alternative 6A proposes in-kind replacement on the same profile of the existing structure, which would result in a long structure with tall columns on the East side of the Los Angeles River. As a result of this, seismic demands on columns and pile foundations of Alternative 6A would be relatively high. The revised alignment serves to minimize the design demands and consequently the construction costs.

The replacement structure is proposed on the same horizontal profile as the existing structure to minimize right-of-way costs. However, the vertical profile is modified to minimize the Sixth Street roadway height above the ground below. This serves two purposes. First the column heights are minimized, reducing both the size of columns and foundations. In addition, the lower profile allows the roadway to be constructed on retained fill structure, which will further reduce construction costs. Thus, four relatively short bridges and 4 ft to 36.5 ft fill structures will be constructed between the

East and West Abutments. The total length of the four bridge structures in Alternative 6B is substantially shorter than total length of the bridge in the existing structure or Alternative 6A, giving a considerable reduction in construction costs compared to Alternative 6A.

13.8.2. Design Considerations:

Alternative 6B is developed with revised profile geometry that meets the Caltrans geometric design requirements as provided in the Highway Design Manual[17]. Geometry of the vertical curves satisfies the Highway Design Manual requirements for stopping sight distance based on a speed limit of 50 mph. Also, the new profile is developed to satisfy minimum clearances of 16.5 ft and 15 ft above the freeways and city streets, respectively. Temporary and permanent minimum clearances over the railroad tracks of 21.5 and 24.5 feet, respectively, are met as well. Due to these restrictions, after the Sixth Street moves East and crosses the river, it does not make it all the way down to grade before rising again to cross the freeway.

13.8.3. Description of the Replacement Structure:

As mentioned above, the replacement structure consists of four bridges in addition to a concrete slab built on soil backfill and retaining walls for portions of the structure between the four bridges. Plans and developed profile of the replacement structure are shown in Appendix B. As in Alternative 6A, curb-to-curb width is 65.5 ft with 5 ft sidewalk and 1 ft standard barrier on each side of the roadway. The maximum drop in elevation of the replacement structure with respect to the existing structure is 36.98 ft.

Bridge 1 (Above the US101 Freeway and Anderson Street; East of the Los Angeles River):

The first bridge (Bridge 1) begins at the East Abutment of the existing structure (Sta 8+05.83) and ends at Sta 14+83.83 with a total length of 678 ft measured along the station line. The bridge crosses over the US101 Freeway as well as Clarence and Anderson streets. The slope of the superstructure is -4.6% (i.e. downhill as one travels from the East to the West side of the bridge). The minimum vertical clearance above the US101 Freeway is 24.5 ft on the Eastern freeway shoulder. The bridge consists of 7 spans with span lengths ranging between 95 ft and 101.5 ft. Abutment 8 (West Abutment of Bridge 1) is 36.5 ft tall.

Bridge 2 (Above Mission Road; East of the Los Angeles River):

The second bridge (Bridge 2) crosses over Mission Road. The East end of the second bridge (Bridge 2) is at Sta 23+03.78 and its West end is at Sta 24+06.78. Length of this single span bridge is 103 ft with a superstructure slope of +2.00% (i.e. uphill going from East to West) and a minimum vertical clearance to Mission Road of 21.97 ft at its East Abutment (Sta 23+03.78).

Bridge 3 (Above the Los Angeles River and Railroad Tracks):

Bridge 3 crosses over the Los Angeles River as well as the railroad tracks on the East and West river banks. The East end of Bridge 3 is at Sta 25+57.28 and its West end is at Sta 33+20.34 with a total length of 763.06 ft measured along the station line (see Appendix B). Slope of the superstructure changes from +2.00% on the East portion of the bridge to -2.80% on the West portion. The bridge has a total of six spans. The two center spans are 160 ft each and cross the Los Angeles River. The two spans on the East side of the Los Angeles River are 125 ft and 88.10 ft long, whereas the two spans on the West side of the river are 117.91 ft and 112.05 ft long (see Appendix B). The minimum vertical clearances above the railroad tracks on the East and West river banks are 28.4 ft (at Sta 25+90) and 28 ft (at Sta 32+90), respectively.

Bridge 4 (Above Santa Fe Avenue; West of the Los Angeles River):

Bridge 4 crosses over Santa Fe Avenue with a 104 ft long single span structure. The East end of Bridge 4 is at Sta 36+42.29, whereas its West end is at Sta 37+46.29 (see Appendix B). Slope of the superstructure is -2.80% (i.e. downhill going from East to West). The minimum vertical clearance at the West Abutment of Bridge 4 (Sta 37+46.29) is 17.78 ft.

Concrete Slab on Soil Backfill and Retaining Walls:

As mentioned above, the replacement structure between the proposed four bridges will comprise of a concrete slab constructed above soil backfill and retaining walls. The retaining walls and soil backfill will be constructed at the following locations (see Appendix B):

1. From Sta 14+83.83 to Sta 23+03.78 (Between Bridges 1 and 2).
2. From Sta 24+06.78 to Sta 25+57.28 (Between Bridges 2 and 3).

3. From Sta 33+20.34 to Sta 36+42.29 (Between Bridges 3 and 4).
4. From Sta 37+46.29 (end of Bridge 4) to the West end of the replacement structure.

The height of the retained fill structures varies from 4 ft to 36.5 ft. The walls and soil backfill are proposed in order to minimize the total length of bridges to be constructed.

13.8.4. Structure Type:

All of the four bridges will be comprised of CIP multi-cell box girder superstructure with a constant depth of 5 ft for Bridge 1 and a constant depth of 4.5 ft for Bridges 2 and 4. Superstructure depth in Bridge 3 is 6.5 ft in the center two spans that cross over the Los Angeles River. Superstructure depth in the exterior spans of Bridge 3 is 5 ft, whereas the superstructure depth between the river spans and the exterior spans varies from 6.5 ft to 5 ft. The steel arch ribs of the existing structure will be preserved and reset on the outer edges on the new structure. However, the arch ribs will not contribute to load carrying capacity of the superstructure and it will support its self-weight only.

The superstructure will be supported on multi-column bents. CIP octagonal columns with flare at the top are proposed for the new structure to improve bridge aesthetics. The columns will be supported on pile foundations. The superstructure of Bridges 2 and 4 as well as ends of the superstructure of Bridges 1 and 3 will be supported on seat type abutments with pile foundations.

As mentioned earlier, retained fill structures will be built between the proposed four bridges. A concrete slab will be constructed on top of the retaining walls and soil backfill between the walls. The retaining walls will be supported on Class 400C piles.

13.8.5. Utilities and Right-of-Way:

Further site investigation is recommended in the PS&E phase to identify existing utilities at the site. Should utilities be identified, they will have to be relocated as required, and as permissible to accommodate construction of the new bridge. Several above ground electrical and telephone lines cross under the structure at Clarence and Anderson streets. These will require relocation prior to demolition of the existing structure.

Currently water on the Sixth Street roadway runs into holes formed in the deck, pouring to the ground below. The new vertical alignment will change the drainage of the Sixth Street Roadway, creating a low point for water collection. Possibly a new RCB will be required to carry water from the roadway low point to the Los Angeles River.

Several industrial buildings are located immediately adjacent to the existing structure. There are four buildings on the North side that may be impacted by the wider roadway required. These buildings may have to be relocated for construction of the replacement structure. Adjacent to Mission Road a loading dock has been constructed below the existing bridge. Removal of this structure is likely required. Further Right-of-Way investigation should be conducted in the PS&E phase.

13.8.6. Construction Stages and Traffic Handling:

The new bridge is wider and below the existing superstructure. Thus, much of the retaining walls, and most foundations and columns of the new structures can be constructed prior to demolition of the existing bridge. During this phase, there will be no interruption of traffic on the existing structure. As discussed in Alternative 6A, stage construction of the bridge by removal of half the existing structure while opening the second half for traffic is not technically feasible. Thus, it is proposed to construct the new structure as follows:

Stage 1: Construction of Retaining Walls and Abutments (Bridge Remains Fully Open):

Construct retaining walls, new bridge abutments (Except Bridge 1 East Abutment and Bridge 4 West Abutment), new foundations and columns (except in freeway and river spans).

Stage 2: Demolition of the Existing Structure and Construction of the New Bridge:

The existing structure will be closed during this stage until construction of the new bridge is completed. An alternative detour will be selected and mitigation measures will be implemented. A detailed Traffic Management Plan should be prepared during the PS&E phase of the project if this alternative is selected for engineering.

13.8.7. Architectural Considerations:

As this alternative proposes to replace a significant landmark in the City of Los Angeles, some architectural details are brought over from the original design. These include salvaging the existing river arches to flank the outside of the river spans, providing a dental detail at the top of the retaining walls and detailing architectural railings. Though other aesthetic detailing or environmental mitigation may be required, no other considerations have been included in the study at this time.

13.8.8. Discussion:

Alternative 6B proposes replacement of the existing bridge with several smaller structures. The major advantages of this structure are the cost reduction and increased life expectancy. Alternatives 2 through 5 provide current seismic safety, but the bridge will continue to deteriorate and require replacement in the not too distant future. A complete replacement will entirely eliminate any concerns related to ASR. Additionally, this alternative provides a wider roadway width that will meet the goal of removing the structure from the FHWA Eligible Bridge List for HBRR funding. Although the steel arches in the river spans will be preserved and used in the replacement structure, the new structure will not have the same visual appearance and historical aesthetics of the existing bridge. Thus, historical and environmental considerations may be the main disadvantage of this alternative.

13.9. OTHER ALTERNATIVES

There could be more alternatives for either retrofitting of the existing structure, or replacement with a new one. One possible alternative is replacement of the existing structure with a signature bridge over the Los Angeles River, which means that the new bridge will not meet the historical aesthetic requirements and will not replicate the existing bridge. The West and East Approach Spans will be comprised of CIP multi-cell box girder supported on octagonal columns with flare at the top, as in Alternative 6B.

13.9.1. Cable-Stayed Bridge:

An alternative for the signature spans over the Los Angeles River would be construction of a cable-stayed bridge with a single pylon that will be built at location of the Center River Pier. The existing structure has a horizontal curvature in the spans over the Los Angeles River. This means that the cable-stayed bridge spans will have a horizontal curvature, which would result in large overturning moments in the pylon just under dead and live loads. This may result in uneconomic design of the pylon and its foundation. Only if horizontal alignment of the bridge is altered, the cable-stayed portion can have a straight horizontal profile over the Los Angeles River, which would render the design more efficient. In that case, the superstructure can be suspended from the pylon by means of the stay cables. Structural fuse elements, such as shear keys, and dampers should be used in the transverse direction between the deck of the cable-stayed spans and the center pylon as well as the West and East River Piers. Dampers may need to be installed between the deck and the river piers in longitudinal direction of the bridge. This alternative is not recommended unless alteration of horizontal alignment of the bridge is possible. It is also believed that the cable-stayed bridge alternative may not be acceptable to the Los Angeles City BOE.

13.9.2. Arch Bridge:

A more acceptable alternative would be construction of steel arches, which could have similar appearance to the existing steel arches. Alternatively, the arches could span the Los Angeles River below the superstructure deck. This alternative would need additional study if selected.

13.10. SUMMARY OF SEISMIC RETROFIT ALTERNATIVES

Table 10 summarizes features of the retrofit alternatives discussed in this section. The retrofit alternatives are also summarized, with more details, in Appendix A. The information presents whether each retrofit alternative has preventive measures for future damage due to ASR, and whether it may meet historical aesthetic requirements. Environmental mitigation requirements are also summarized in the table in which “No” means that the retrofit alternative by itself does not meet all SHPO requirements and it would require some form of mitigation as discussed earlier in Section 13.3.8; “Yes” means that the retrofit alternative includes environmental mitigation.

Table 10 also gives the estimated life expectancy for each alternative. Costs of each alternative are discussed in Section 14. Except in Alternative 1, the approach spans as well as the main spans over the Los Angeles River will be retrofitted in all alternatives. Existing material properties are incorporated in design of all retrofit alternatives except in Alternative 1.

Table 10. Summary of seismic retrofit alternatives for the Sixth Street Viaduct

Alt. No.	Retrofit Description	Prevention of ASR	Historical Aesthetics	Environmental Mitigation	Life Expectancy
1**	Infill shear walls	No	No	NA*	0 years
2	Steel column casings	No	Yes	No	10 years
3	Catcher walls	No	No	NA*	10 years
4	Concrete casings	No	Yes	No	20 years
5	Heavy steel column casings (for ASR)	Yes	Yes	No	30 years
6A	In-kind replacement	Yes	Yes	Yes	75 years
6B	Replacement with CIP box girder bridges & retaining walls (revised alignment)	Yes	No	No	75 years

* Not Available.

** Alternative 1 does not provide a technical solution and is presented for comparison only.

14. COST ANALYSIS

14.1. SEISMIC RETROFIT COST ANALYSIS

Table 11 summarizes cost analysis for the different retrofit alternatives. Detailed breakdown of costs for all alternatives is given in Appendix C. As discussed before, seismic retrofit costs given in Table 11 do not include costs of rehabilitation, except for installation of new expansion joint seals in

Alternative 5. Cost of installation of new expansion joint seals in Alternative 5 is included as part of seismic retrofit costs because Alternative 5 is the only seismic retrofit proposal that considers effects of ASR on future deterioration of concrete and seismic safety of the retrofitted structure. The substructure must be protected against moisture to minimize future concrete deterioration caused by ASR, so that Alternative 5 design would be efficient.

The construction as well as total project costs are given in Table 11 for each alternative. Construction costs per square ft are also given in Table 11. One-to-one cost comparison of all alternatives is difficult since each alternative has a different life expectancy and meets differing historical ideals. Also, width and length of Alternatives 6A and 6B are different from those of the existing structure to meet minimum roadway design standards. For these reasons, both the project cost and life expectancy should be considered in economic cost analysis of different alternatives. The project cost is divided by expected life in years and the results are given for the different alternatives in the last column of Table 11. Alternative 1 was designed before material testing, which has subsequently determined that the design does not work due to concrete degradation and the resulting weak bond between the new shear walls and the existing columns. Thus, Alternative 1 has no life expectancy and the Cost/Life value for Alternative 1 is not available. The Cost/Life values given in Table 11 indicate that although the replacement of the existing structure (Alternative 6B) would be more expensive than the minimum retrofit, it is the most long-term economic alternative. Although Alternative 6B would cost less than Alternative 6A, it may not meet historical aesthetic requirements as discussed earlier.

A portion of the replacement structure in Alternative 6B over the US101 Freeway is owned by Caltrans, whereas the remaining portion of the new structure is owned by the City of Los Angeles. Thus, the State is responsible for a portion of the total project costs on their Right-of-way. Table 12 provides the total costs of Alternative 6B to the City of Los Angeles and to the State. All other alternatives apply only to the City owned portion of the structure with no impact to the State’s portion of the structure.

Table 11. Summary of costs of seismic retrofit alternatives for the Sixth Street Viaduct

Alt. No.	**Retrofit Description**	**Construction Cost**	**Design & Administration Cost**	**Project Total Cost**	**Construction Cost per ft^2**	**Cost/Life ($/years)**
1*	Shear walls	$19,752,691	$2,707,731	**$22,460,422**	$121	NA
2	Steel casings	$37,507,696	$13,046,155	**$50,553,851**	$230	$5,055,385
3	Catcher walls	$41,806,807	$14,541,499	**$56,348,306**	$256	$5,634,831
4	Concrete casings	$61,727,544	$21,470,450	**$83,197,994**	$378	$4,159,900
5	Heavy steel casings	$59,956,834	$20,854,551	**$80,811,385**	$367	$2,693,713
6A	Replacement in-kind	$71,908,905	$26,511,793**	**$98,420,698**	$316	$1,312,276
6B	Replacement with new bridges & retaining walls	$56,163,443	$21,035,111**	**$77,198,554**	$207	$1,029,314

* Alternative 1 does not provide a technical solution and is presented for comparison only.

** Includes $1,500,000 for Right-of-Way.

Table12. Alternative 6B cost breakdown

Responsibility	**Construction Cost**	**Design & Administration Cost**	**Project Total Cost**
City Portion	$53,769,989	$20,202,605*	**$73,972,594**
State Portion	$2,393,454	$832,506	**$3,225,960**

* Includes $1,500,000 for Right-of-Way.

14.2. LIFE CYCLE COST ANALYSIS

The existing Sixth Street Viaduct is over 70 years old with extensive cracking present in many of the structural elements. The ongoing concrete deterioration cannot be arrested completely even with extensive rehabilitation and maintenance. Even with careful maintenance the expected life of the structure is estimated at 30 years (Alternative 5). In contrast, a new replacement structure will have little maintenance costs in the next fifty years. To compare actual costs of the retrofit and replacement options, a life cycle cost comparison is presented below.

The expected life of the retrofit/replacement alternatives are given in Table 10. From the above discussion it is concluded that Alternative 1 is not a viable retrofit option. Alternative 5 is the only retrofit proposal that addresses future ASR effects on concrete deterioration. In light of this, rehabilitation of Alternatives 1 through 4 is ineffective in prolonging the life span of the structure. Alternative 5, however, provides added steel casings to seismically protect columns that may become vulnerable in the near future without retrofit. Additional rehabilitation of the bridge will increase its serviceable life span to fully utilize the expected life of the seismic retrofit. The following rehabilitation items for the Sixth Street Viaduct are anticipated to provide longer service life for the bridge:

1. Repair of cracks in un-retrofitted structural elements by epoxy injection. This includes:
    (a) Cracks in columns with no steel or concrete casings.
    (b) Cracks in bent caps with no retrofit.
    (c) Cracks in superstructure girders.
    (d) Cracks in the deck slab.
2. Repair of the concrete barrier.
3. Removal of asphalt on the deck, crack sealing and addition of a protective coat of polyester concrete.

Figure 35 and Figure 36 show the proposed rehabilitation of the deck. This includes removal of the existing asphalt concrete and placement of 2-in. polyester concrete overlay. The new expansion joint seals, which are part of the seismic retrofit Alternative 5 only, are also shown in Figure 35 and Figure 36. The objective of the deck rehabilitation is to protect the deck and superstructure from moisture, and consequently reduce further concrete degradation due to ASR. Repair of the barrier rail includes partial replacement, repair of the electroliers, and epoxy injection along the portions that will not be replaced. Significant cracks in un-retrofitted columns and bent caps will be repaired by epoxy injection. Significant cracks in the deck and superstructure girders will also be repaired.

Based on available information on previous repairs of the Sixth Street Viaduct, it is expected that crack repair by epoxy injection would need to be repeated approximately every 10 years. For life cycle cost analysis, it is assumed that barrier repair is needed every 15 years for the retrofitted structure (Alternatives 2 through 5) or every 30 years for the replacement Alternatives 6A and 6B. Based on expected life of different retrofit/replacement alternatives, the following rehabilitation work is assumed for life cycle cost analysis:

<u>Alternatives 2 (Steel Casings) & 3 (Catcher Walls):</u> Cracks and barrier rails will be repaired only once (at time of retrofit construction). Deck rehabilitation is not included for these alternatives since the expected life of these alternatives is only 10 years.

<u>Alternative 4 (Concrete Casings):</u> Cracks will be repaired at time of retrofit construction as well as 10 years after retrofitting. Barrier rails will be also repaired during retrofitting as well as 15 years after retrofitting. Since the expected life of Alternative 4 is longer than expected life of Alternatives 2 and 3, deck rehabilitation is proposed during retrofit construction. No further deck rehabilitation is assumed throughout the expected life of 20 years for Alternative 4.

<u>Alternative 5 (Steel Casings with ASR Protection):</u> To prolong the structure life, epoxy crack repair and barrier rail repair will be completed during retrofit construction. Since, the expected life of Alternative 5 is 30 years, it is assumed, for life cycle cost analysis, that cracks will be repaired every 10 years, resulting in 3 rounds of crack repair (including initial crack repair during retrofitting). Also, barrier rail repair is assumed to be repeated 15 years after retrofitting of the structure. Deck rehabilitation is proposed during retrofit construction to protect the superstructure from moisture.

With 30 years life expectancy of Alternative 5 retrofit, deck rehabilitation may not need to be repeated within the expected life of the Alternative 5 retrofit.

Replacement Alternatives 6A & 6B: No crack or barrier rail repairs are needed for Alternatives 6A and 6B throughout their expected life since the existing bridge will be replaced with a new one using state-of-the-art materials and detailing. However, for the life cycle cost analysis it is assumed that deck rehabilitation may be needed after 50 years.

Costs for each rehabilitation item are given in Table 13. Costs for crack repairs are based on estimated significant cracks in un-retrofitted structural elements in Alternative 5. The costs of crack repairs in Alternatives 2 through 4 are higher than those given in Table 13 (see details of life cycle cost analysis in Appendix C); this is because of the more un-retrofitted elements of the structure and consequently the more significant cracks that need epoxy injection repair. Breakdown of rehabilitation cost for Alternative 5 is given in Appendix C as an example. It should be noted that the costs given in Table 13 are given for one repair occurrence. Thus, the number of repair occurrences, as discussed above, must be considered in life cycle cost analysis.

Table 13. Rehabilitation costs

Item	Construction Cost	Design & Administration Cost	Total Cost
Crack Repair*	$3,404,000	$1,184,000	$4,588,000
Barrier Rail Repair	$721,000	$251,000	$972,000
Deck Rehabilitation**	$2,730,000	$949,000	$3,679,000
Total Cost	**$6,855,000**	**$2,384,000**	**9,239,000**

* Estimated based on significant cracks in un-retrofitted structural elements in Alternative 5.

** For Alternatives 4, 5, 6A and 6B only.

To compare costs of retrofit and replacement of the existing structure, the life cycle analysis is prepared by determining the present value of all future maintenance costs for Alternatives 2 through

5, 6A and 6B. The cost of money, or discount rate, for a governmental agency is determined using the current long term yield on the U.S. Treasury bonds and the inflation rate for the past 20 years. The discount rate is given by the bond yield minus inflation, providing the true cost of money. Using this value of 2.01%, all future projected maintenance costs over the next 50 years can be translated into 2004 Dollars. In addition, the present value of replacing the bridge in 10 years for Alternatives 2 and 3, in 20 years for Alternative 4 or in 30 years for Alternative 5 is also calculated. Each of the initial capital costs, present value maintenance costs and present value future replacement costs are added together. Thus, the alternative with the lowest present value is the most economical option. This represents the actual Dollars, in 2004 prices, that will need to be spent on the structure in the next 50 years.

This method does not fully reflect the true situation at the end of fifty years since, for example, the replacement bridges in Alternatives 6A and 6B are two thirds through their expected life of 75 years, while the Alternative 5 replacement bridge is not even one third through its expected life. To capture this disparity, the residual value of each bridge is calculated 50 years from now. This residual value is translated into 2004 Dollars and subtracted from the total above. The final number gives the relative true cost between the alternatives. Again, the alternative with the lowest cost is the most economical solution. The life cycle cost results are presented in Table 14. For the full analysis, see Appendix C.

Table 14. Comparison of life cycle costs of retrofit and replacement alternatives (In thousands $)

Alt. No.	Retrofit Description	Initial Capital Investment*	Maintenance & Improvement Costs*	Total Capital Outlay*	Residual Value*	Life Cycle Cost*
2	Steel casings	$57,411	$80,660	**$138,071**	-$16,981	**$121,091**
3	Catcher walls	$64,547	$80,660	**$145,207**	-$16,981	**$128,227**
4	Concrete casings	$91,577	$69,881	**$161,458**	-$21,832	**$139,626**
5	Heavy steel casings	$90,050	$61,739	**$151,789**	-$26,684	**$125,105**
6A	Replacement in-kind	$98,421	$1,895	**$100,316**	-$12,129	**$88,187**
6B	Replacement with new bridges & retaining walls	$77,199	$1,895	**$79,094**	-$9,514	**$69,580**

* In 2004 Dollars.

Table 14 clearly indicates that the most cost efficient alternative is to replace the structure on a revised vertical alignment (Alternative 6B). Constructing Alternative 6B has a cost savings of 74% over Alternative 2 (minimum retrofit), 80% savings over Alternative 5 and 27% savings over Alternative 6A. The In-kind replacement (Alternative 6A) is also more cost efficient alternative than retrofitting of the existing structure as evident by the 37% cost savings compared to Alternative 2 and 42% cost savings compared to Alternative 5. Comparing the retrofit alternatives, Alternative 2 turns out to be the most cost effective retrofit strategy as shown by the life cycle cost analysis. Alternative 5 is only 3.5% more expensive than Alternative 2, which is within the margin of error in the life cycle cost estimating method.

14.3. ENVIRONMENTAL MITIGATION COSTS

Environmental mitigation is not available for Alternative 3 due to the complete change in visual aesthetics. Alternatives 2, 4 and 5 would need some form of environmental mitigation. A few mitigation measures are presented in Table 15 with their estimated costs. The mitigation measures in Table 15 are just given for reference purpose and do not represent all possible measures that may be required by SHPO. The environmental mitigation costs (as those shown in Table 15) should be added to the project costs given in Table 11 for Alternatives 2, 4 and 5 and to the rehabilitation costs of Alternative 5 (see Table 13).

Table 15. Possible environmental mitigation measures for retrofit Alternatives 2, 4 and 5

Environmental Mitigation Measure Description	**Cost**
Barrier Rail Replacement*	$9,600,000
Electrolier Restoration	$1,300,000
Pylon/Obelisk Restoration	$2,000,000

* Not needed in Alternative 5 in case of barrier rehabilitation.

15. CONCLUSIONS

Based on the engineering analysis presented in this report, the following are concluded:

1. Seismic retrofitting of the Sixth Street Viaduct is needed to meet present design requirements, and to slow the degradation of the structure.
2. The least cost seismic retrofit solution is Alternative 2 (steel casings and infill shear walls) when considering initial cost as well as life cycle cost analysis. This alternative does not address the continuing material degradation.
3. The second least cost seismic retrofit solution is Alternative 3 (Catcher Wall System) when considering initial cost. This alternative would carry a much higher life cycle cost compared to retrofit alternatives 2 and 5. This alternative does not address the continuing material degradation.
4. Alternative 5 (Heavy steel casing) is the only seismic retrofit solution that would address the problems with continuing material degradation. It is however one of the most costly alternatives in both initial cost and in life cycle cost analysis.

5. Alternative 6B (New Alignment Replacement) is the most economical solution as shown in the life cycle analysis. Alternative 6B is also the least cost alternative that meets the requirement for seismic performance, serviceability and stable concrete material.
6. Alternative 6A would provide all the benefit of Alternative 6B in terms of structure performance and serviceability requirement. It is also likely the most architecturally and institutionally acceptable replacement alternative. This alternative is however more costly than Alternative 6B in initial cost and in the life cycle analysis.

16. RETROFIT STRATEGY MEETING DISCUSSION

A retrofit strategy meeting was held on April 26, 2004. A full account of the meeting minutes can be found in Appendix J of this report. A summary of the seismic retrofit strategy developed for the Sixth Street Viaduct was presented to Caltrans during the meeting. The presentation included a summary of previous work completed, as-built analysis and five retrofit alternatives (Alternatives 1-5). Also, the two replacement alternatives (Alternatives 6A and 6B) were presented for comparison with the retrofit strategy costs.

The discussion during the strategy meeting presentation is summarized in Appendix J. Caltrans concurred with the Los Angeles City BOE and W. Koo & Associates, Inc. (WKA) that only Alternatives 5, 6A and 6B are feasible since they meet all seismic criteria and address the continuing ASR deterioration. The other retrofits are not practical alternatives as they will become obsolete within a few years due to ASR and thus would be a poor use of resources. This was subsequently borne out by the life cycle analysis showing Alternative 5 is less expensive than Alternatives 3 or 4. However, Alternative 2 has been subsequently shown to be slightly cheaper over the long term than Alternative 5. It was agreed that the retrofit Alternative 5 and replacement Alternatives 6A and 6B will be presented to the FHWA and a final decision, whether retrofit or replacement of the existing viaduct, will be made in conjunction with FHWA.

The selective column casing was questioned by Caltrans since the in situ behavior of columns affected by ASR has not been investigated. Thus, Alternative 2 is not recommended due to the minimum column casings provided in the retrofit. Subsequent to the strategy meeting, Caltrans recommended to provide steel encasement at a minimum of 2 columns in each bent for alternative 5. Caltrans also recommended that column casing should take precedence over foundation or infill shear wall retrofit in other locations. The plan for Alternative 5 was revised to incorporate the additional column casings requested. Subsequently, all of the exterior columns in the existing structure will be retrofitted by steel casings, except columns in Bent 12 due to the tight room available for construction as a result of proximity of railroad tracks to Bent 12. The plans and cost estimates are revised in this report to reflect these changes. During the final design, preference will be given to maintaining all of the column casings while reducing the number of foundation retrofits and infill shear walls. However, many foundation retrofits will still be required to help reduce longitudinal seismic displacement demands (see Appendix J).

Caltrans also requested that rehabilitation and maintenance costs associated with the retrofitted structure be included in the analysis to provide comparable costs with the replacement options. Both current and future rehabilitation and maintenance work and costs are included in this report for Alternatives 5, 6A and 6B. Caltrans requested that the report should discuss the options of closing the bridge to traffic versus construction staging and keeping part of the bridge open to traffic. After consultation with the Los Angeles City BOE, it has been concluded that given the risk, technical difficulty and added cost of construction staging, it is more practical to close the bridge during construction. Full impact of closing the structure will be addressed in the environmental documentation to be prepared later.

17. REFERENCES

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15. Uang, C.M., and Kleiser, M., Cyclic Performance of As-Built Latticed Members for the San Francisco-Oakland Bay Bridge, Structural Systems Research Project Report SSRP-97/01, Department of Structural Engineering, University of California San Diego, La Jolla, California, June 1997.

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APPENDIX A

SUMMARY OF RETROFIT ALTERNATIVES

June 2004

Sixth Street Viaduct Seismic Retrofit Strategy Alternatives

Alternative	Description	Estimated Construction Cost (US $)	Estimated Design & Administration Cost (US $)*	Estimated Project Cost (US $)	Seismic Retrofit of Approach Spans	Seismic Retrofit of River Spans	Incorporates Existing Material Properties in Design of Retrofit	Preventative Measures for Future ASR Damage	Design May Meet Historical Aesthetic Requirements	Environmental Mitigation	Retrofit Life Expectancy
1	Shear Wall Design by City BOE	19,752,691	2,707,731	22,460,422	✓	✗	✗	✗	✗	NA	0 years
2	Steel Casing	37,507,696	13,046,155	50,553,851	✓	✓	✓	✗	✓	✗	10 years
3	Catcher Walls	41,806,807	14,541,499	56,348,306	✓	✓	✓	✗	✗	NA	10 years
4	Concrete Casing	61,727,544	21,470,450	83,197,994	✓	✓	✓	✗	✓	✗	20 years
5	Steel Casing & ASR Protection	59,956,834	20,854,551	80,811,385	✓	✓	✓	✓	✓	✗	30 years
6A	Replace in Kind	71,908,905	26,511,793	98,420,698	✓	✓	✓	✓	✓	✓	75 years
6B	Replace with CIP Box Girder Bridges & Retaining Walls (Revised Alignment)	56,163,443	21,035,111	77,198,554	✓	✓	✓	✓	✗	✗	75 years

* This also includes Righ-of-Way costs for Alternatives 6A & 6B.

LEGEND:

✓	Yes
✗	No

Seismic Retrofit of Approach Spans
Designed for prevention of structure collapse. Assumes that the existing material is in good condition.

Seismic Retrofit of River Spans
Includes retrofit of the center river spans to prevent collapse of the steel tied arch during a seismic event.

Incorporates Existing Material Properties in Design of Retrofit
The seismic design takes into account the existing material properties of each element. The analysis uses material properties gained from the material testing phase of the work, and proposes appropriate design and construction methods for the material state.

Preventative Measures for Future ASR Damage
The design includes measures to protect the structure into the future. The analysis assumes that severely damaged columns will get worse and require seismic protection at a later time. Columns and bent caps with severe ASR damage are rehabilitated to prevent continued degradation and failure.

Design May Meet Historical Aesthetic Requirements
Visual aesthetics can be added to these alternatives to make them visually consistent with the original historical aesthetics. This may include adding reveals, additional concrete detailing, or architectural coatings. The net change in member sizes and openings have been minimized and may meet the Secretary of Interior Guidelines.

Environmental Mitigation
Yes(✓): Environmental mitigation not needed, or already included.
No(✗): In itself, the alternative does not meet all SHPO requirements, requiring some form of mitigation. A few mitigation measures are presented below for reference and by no means shall be considered as all possible measures that will be required by SHPO.

POSSIBLE ENVIRONMENTAL MITIGATION MEASURES		
Barrier Rail Replacement	$	9,600,000
Electrolier Restoration	$	1,300,000
Pylon/Obelisk Restoration	$	2,000,000

Retrofit Life Expectancy
Estimated number of years until a significant investment in a new retrofit or rehabilitation is required to maintain seismic and operational safety of the structure.

Description of Work for each Alternative:

Alternative 1	Includes shear walls at 17 bents, 6 grade beams, 2 footing retrofits, restrainers at the West and East River Piers and retrofitting of shear keys at the West Abutment.
Alternative 2	Includes shear walls at 14 bents, 29 steel plate column casings, new footings at 19 bents, 6 grade beams, 3 bent cap retrofits, 2 river pier retrofits and closure of 2 expansion joints in the superstructure.
Alternative 3	Includes 35 catcher walls with pile foundations and 2 river pier retrofits.
Alternative 4	Includes 80 concrete column casings with footings, 28 bent cap rerofits and 2 river pier retrofits.
Alternative 5	Includes shear walls at 14 bents, 76 steel plate column casings, new footings at 19 bents, 6 grade beams, 16 bent cap retrofits, 2 river pier retrofits and closure of 2 expansion joints in the superstructure.
Alternative 6A	Replace with similar looking structure along the same alignment.
Alternative 6B	Replace with a new structure comprised of 4 CIP concrete bridges, soil backfill and retaining walls. The new structure will be built along a different vertical alignment with respect to the existing structure.

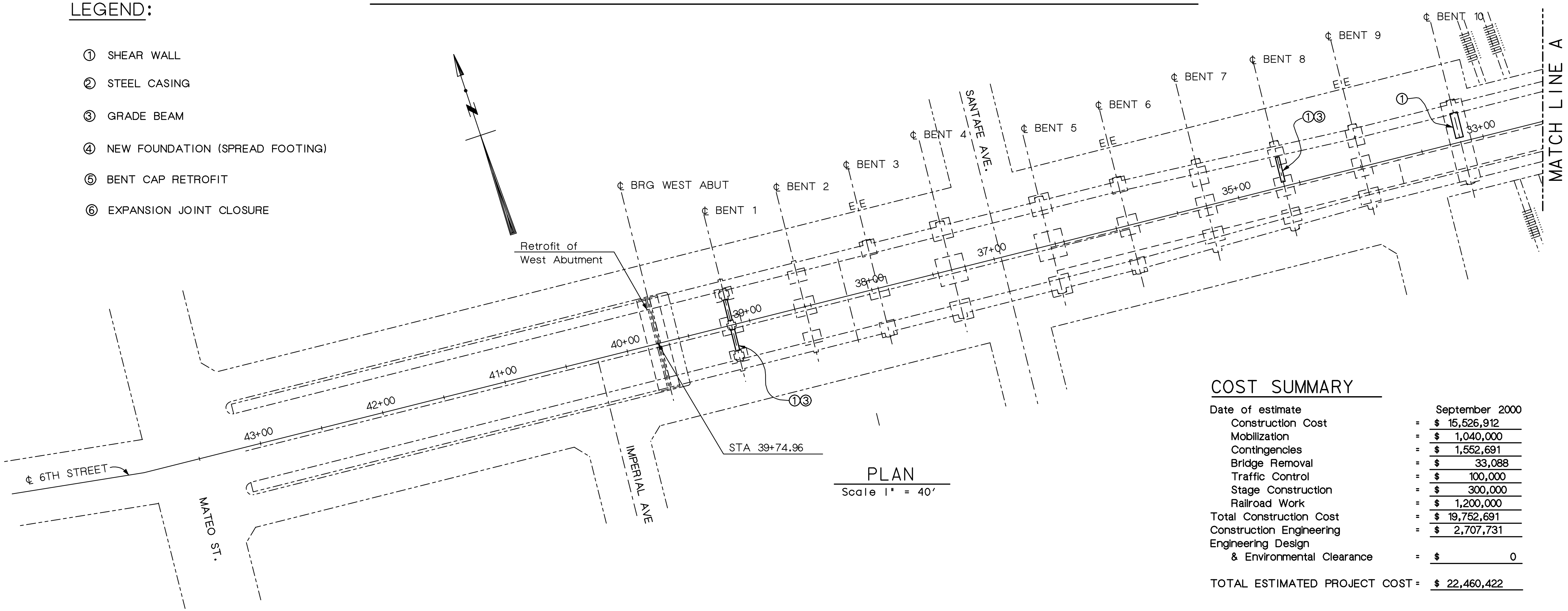
APPENDIX B

GENERAL PLANS

SEISMIC RETROFIT STRATEGY ALTERNATIVE I - EXISTING SHEAR WALL DESIGN

RETROFIT LEGEND:

① SHEAR WALL
② STEEL CASING
③ GRADE BEAM
④ NEW FOUNDATION (SPREAD FOOTING)
⑤ BENT CAP RETROFIT
⑥ EXPANSION JOINT CLOSURE

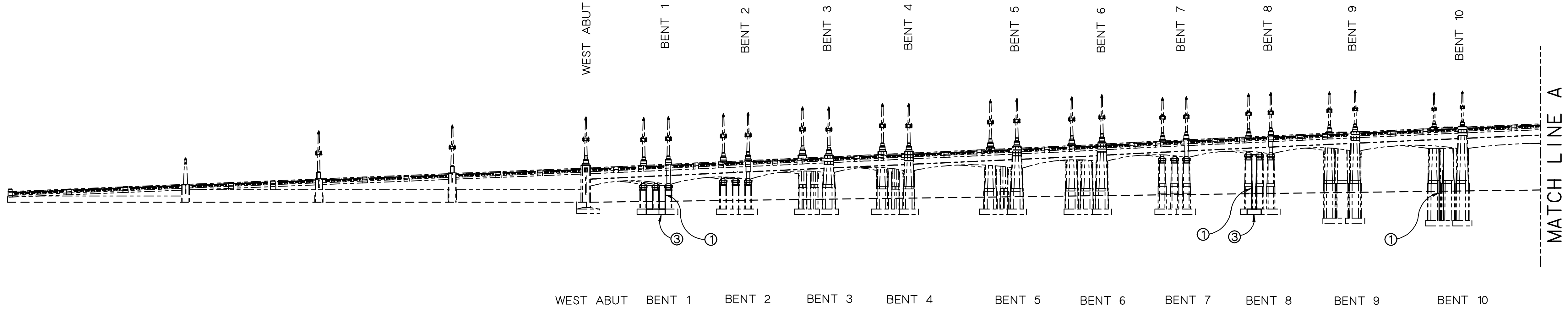


PLAN

Scale 1" = 40'

COST SUMMARY

Date of estimate		September 2000
Construction Cost	=	$ 15,526,912
Mobilization	=	$ 1,040,000
Contingencies	=	$ 1,552,691
Bridge Removal	=	$ 33,088
Traffic Control	=	$ 100,000
Stage Construction	=	$ 300,000
Railroad Work	=	$ 1,200,000
Total Construction Cost	=	$ 19,752,691
Construction Engineering	=	$ 2,707,731
Engineering Design & Environmental Clearance	=	$ 0
TOTAL ESTIMATED PROJECT COST	=	$ 22,460,422



ELEVATION

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SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
GENERAL PLAN NO. 1

NO.	REVISION DESCRIPTION	DIV./DIST. ENGR.	DATE

CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER

DATE ______ 20 ___

DIV./DIST.ENGR. R. E. NO.

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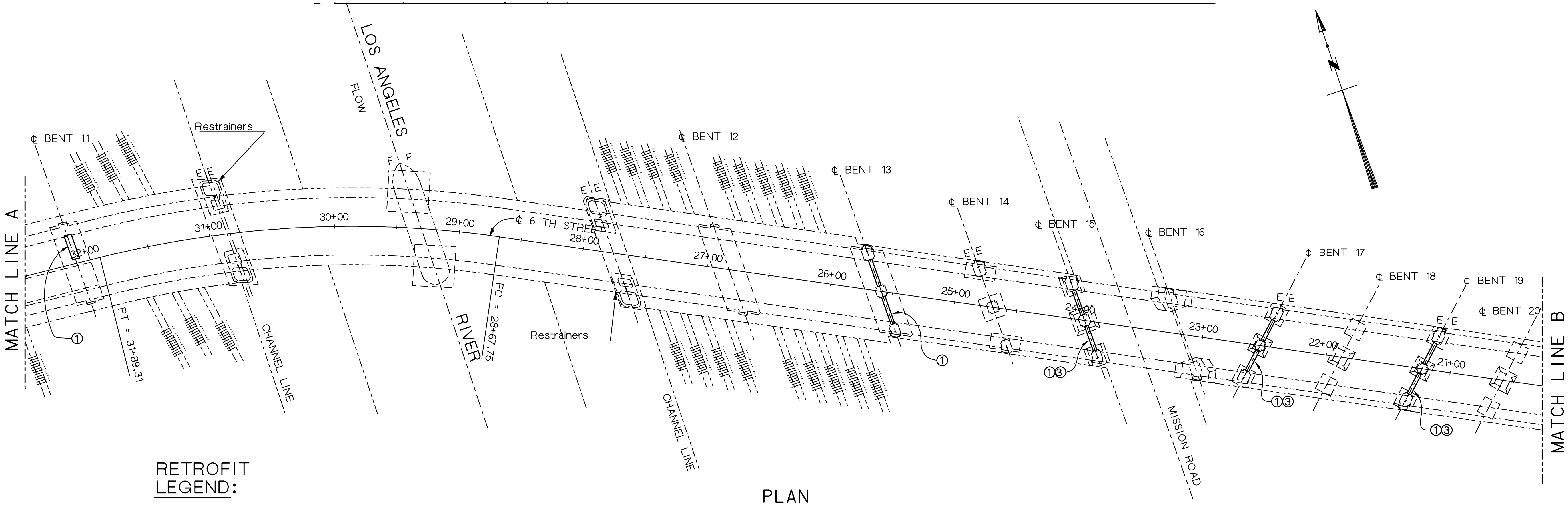
SEISMIC RETROFIT STRATEGY ALTERNATIVE I - EXISTING SHEAR WALL DESIGN



RETROFIT LEGEND:

① SHEAR WALL

② STEEL CASING

③ GRADE BEAM

④ NEW FOUNDATION (SPREAD FOOTING)

⑤ BENT CAP RETROFIT

⑥ EXPANSION JOINT CLOSURE

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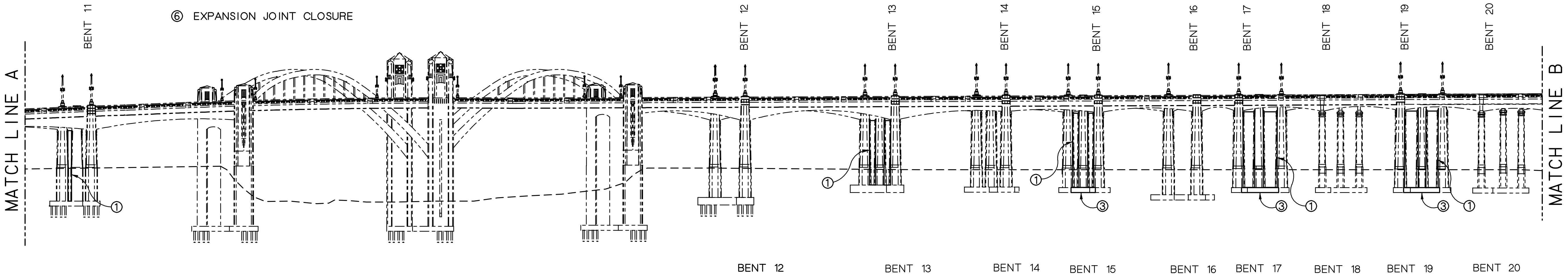

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SIXTH STREET VIADUCT
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GENERAL PLAN NO. 2

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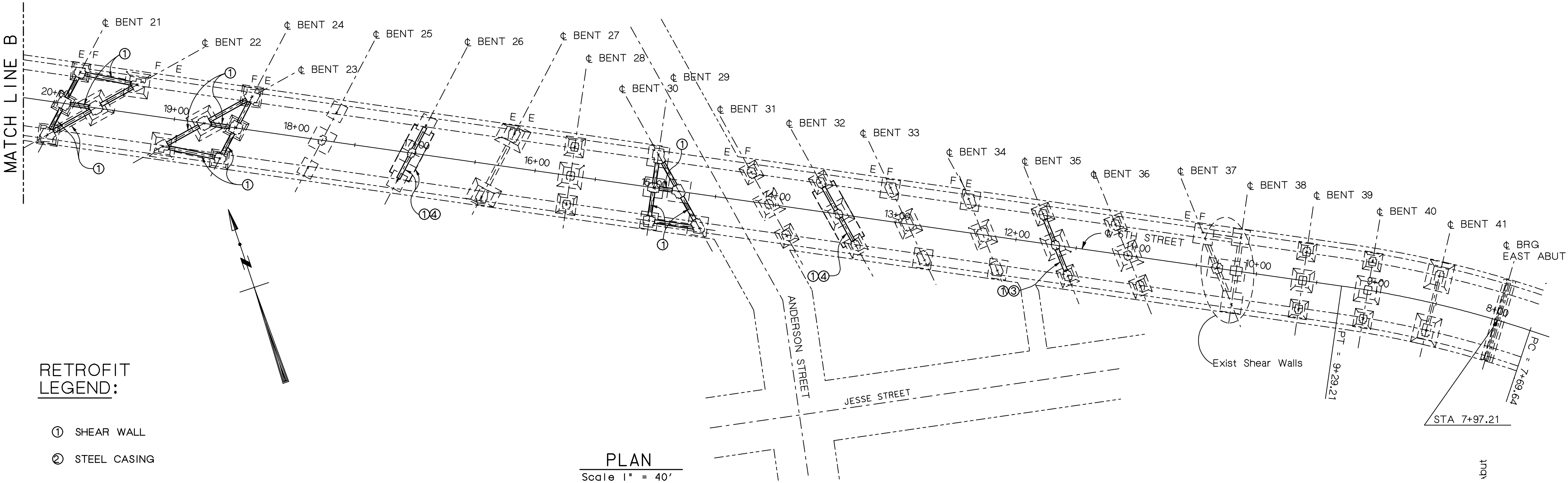
SEISMIC RETROFIT STRATEGY ALTERNATIVE I - EXISTING SHEAR WALL DESIGN



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RETROFIT LEGEND:

① SHEAR WALL

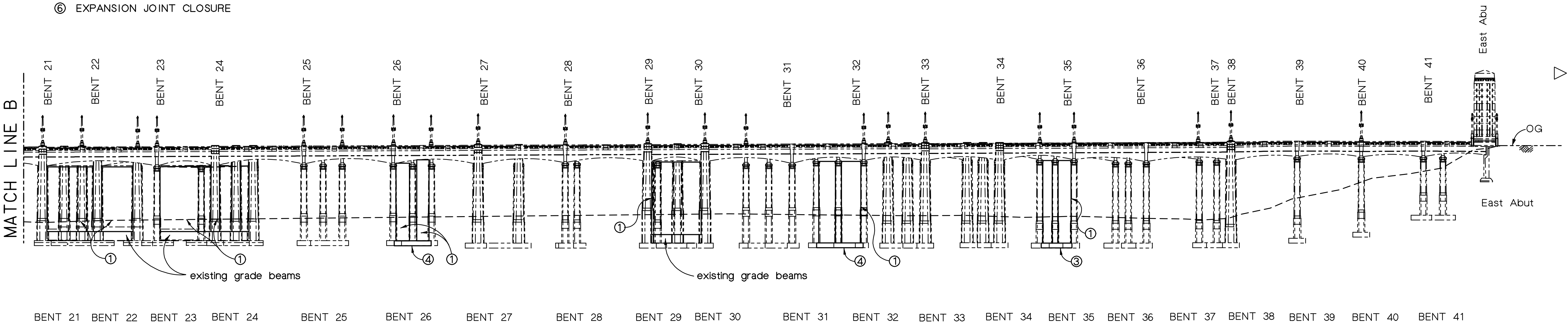
② STEEL CASING

③ GRADE BEAM

④ NEW FOUNDATION (SPREAD FOOTING)

⑤ BENT CAP RETROFIT

⑥ EXPANSION JOINT CLOSURE



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SIXTH STREET VIADUCT
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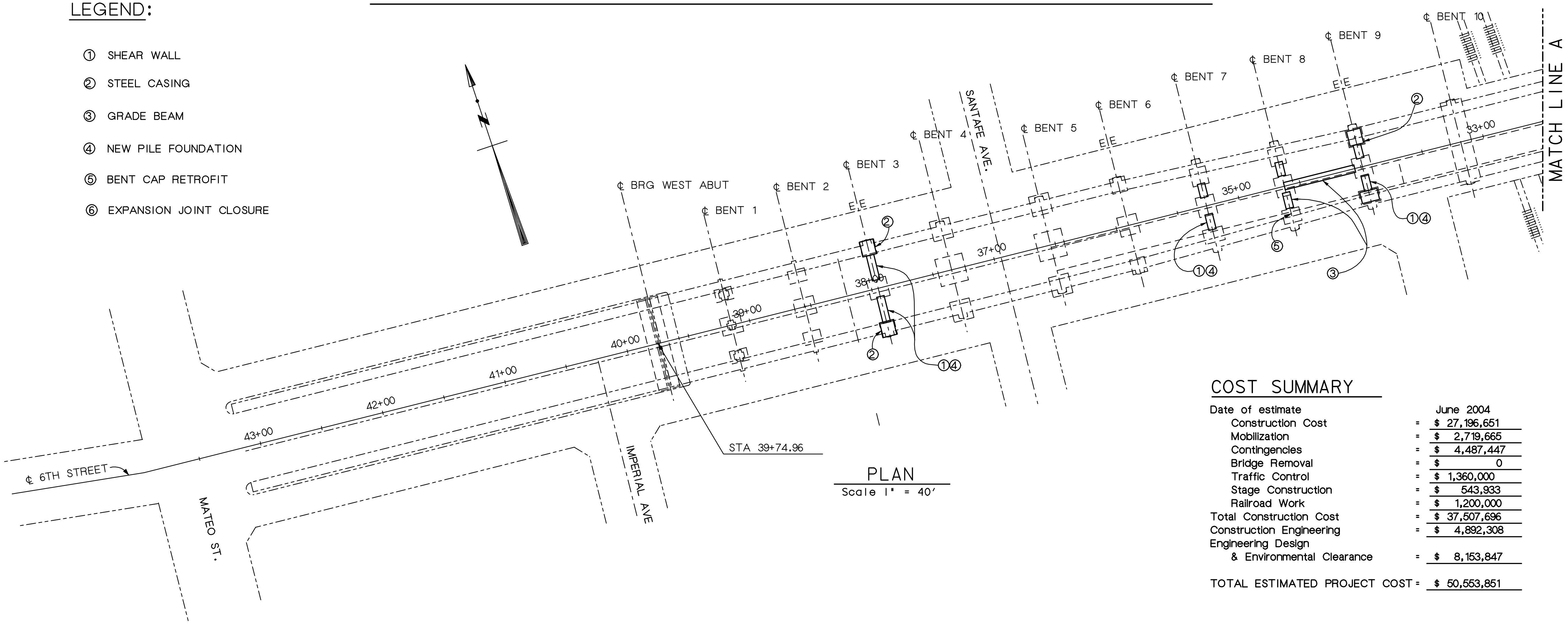
SEISMIC RETROFIT STRATEGY ALTERNATIVE 2 - SHEAR WALL AND STEEL CASING

RETROFIT LEGEND:

- ① SHEAR WALL
- ② STEEL CASING
- ③ GRADE BEAM
- ④ NEW PILE FOUNDATION
- ⑤ BENT CAP RETROFIT
- ⑥ EXPANSION JOINT CLOSURE



PLAN
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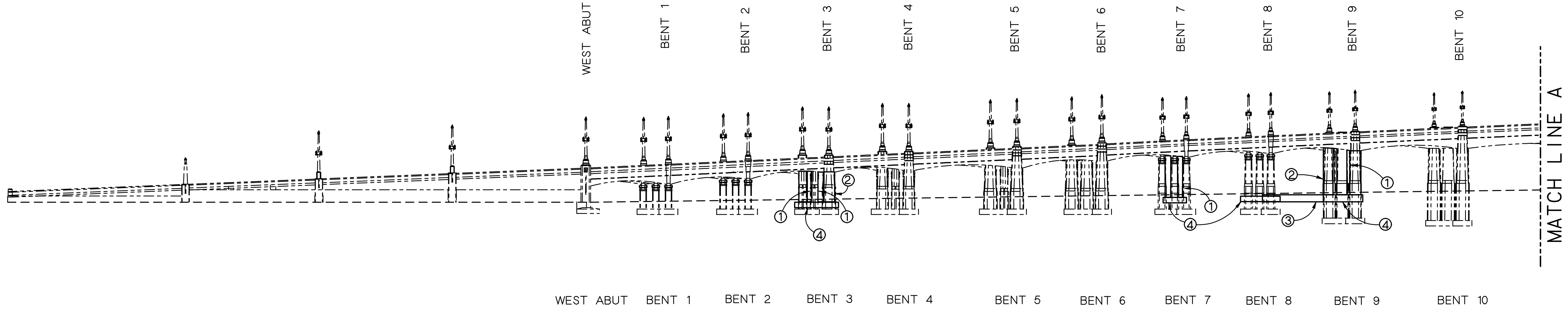
COST SUMMARY

Date of estimate		June 2004
Construction Cost	=	$ 27,196,651
Mobilization	=	$ 2,719,665
Contingencies	=	$ 4,487,447
Bridge Removal	=	$ 0
Traffic Control	=	$ 1,360,000
Stage Construction	=	$ 543,933
Railroad Work	=	$ 1,200,000
Total Construction Cost	=	$ 37,507,696
Construction Engineering	=	$ 4,892,308
Engineering Design & Environmental Clearance	=	$ 8,153,847
TOTAL ESTIMATED PROJECT COST	=	$ 50,553,851



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GENERAL PLAN NO. 1

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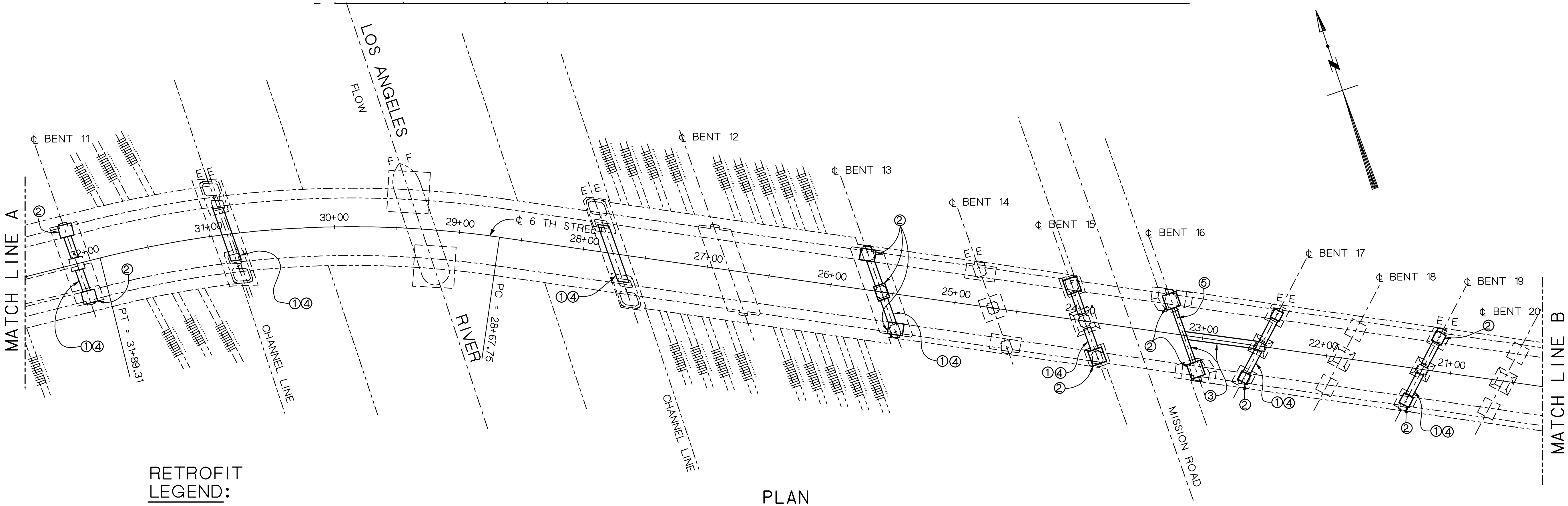
SEISMIC RETROFIT STRATEGY ALTERNATIVE 2 - SHEAR WALL AND STEEL CASING



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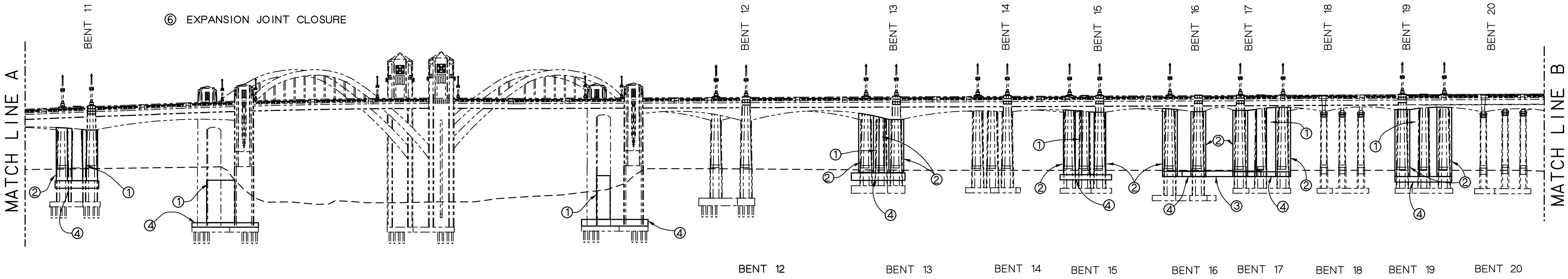
RETROFIT LEGEND:

- ① SHEAR WALL
- ② STEEL CASING
- ③ GRADE BEAM
- ④ NEW PILE FOUNDATION
- ⑤ BENT CAP RETROFIT
- ⑥ EXPANSION JOINT CLOSURE



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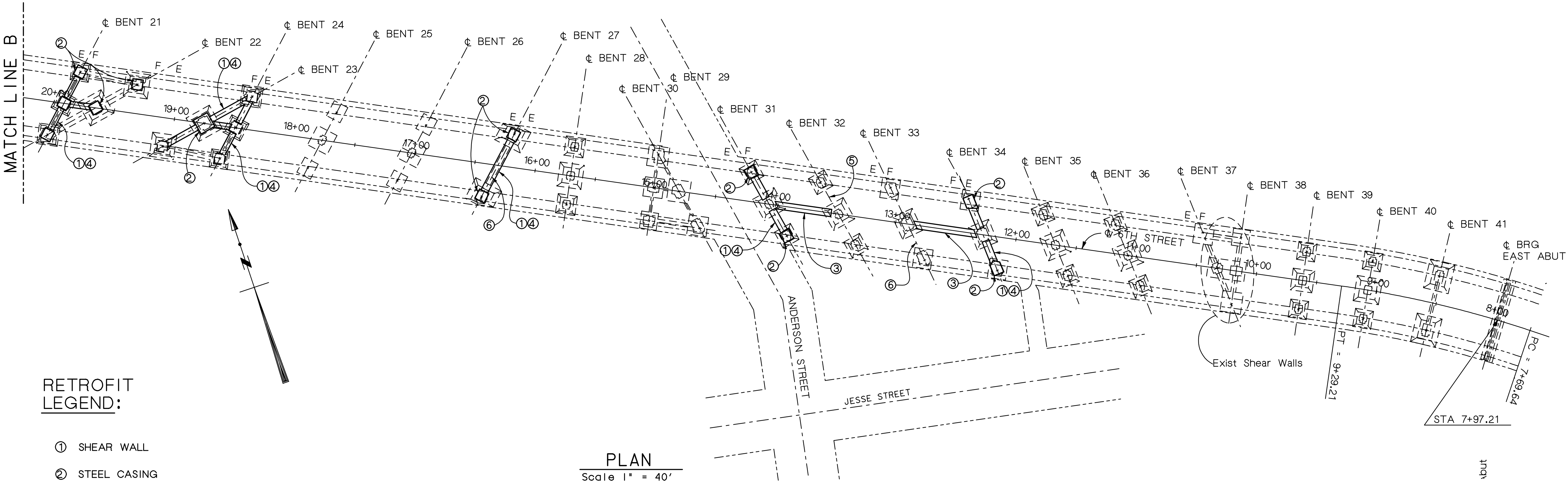
SEISMIC RETROFIT STRATEGY ALTERNATIVE 2 - SHEAR WALL AND STEEL CASING



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RETROFIT LEGEND:

① SHEAR WALL

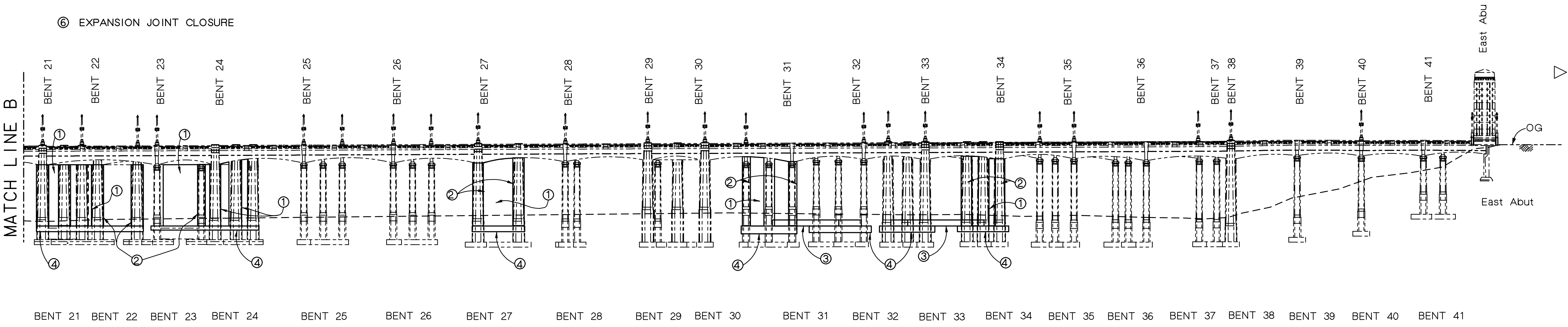
② STEEL CASING

③ GRADE BEAM

④ NEW PILE FOUNDATION

⑤ BENT CAP RETROFIT

⑥ EXPANSION JOINT CLOSURE



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SIXTH STREET VIADUCT
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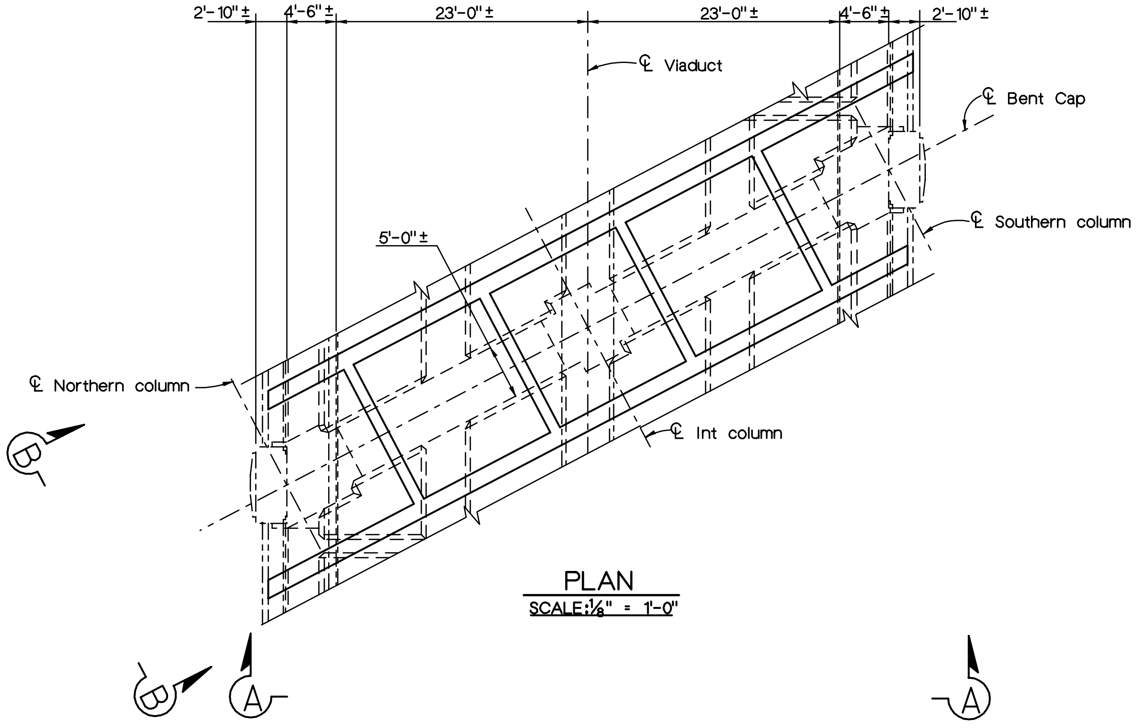
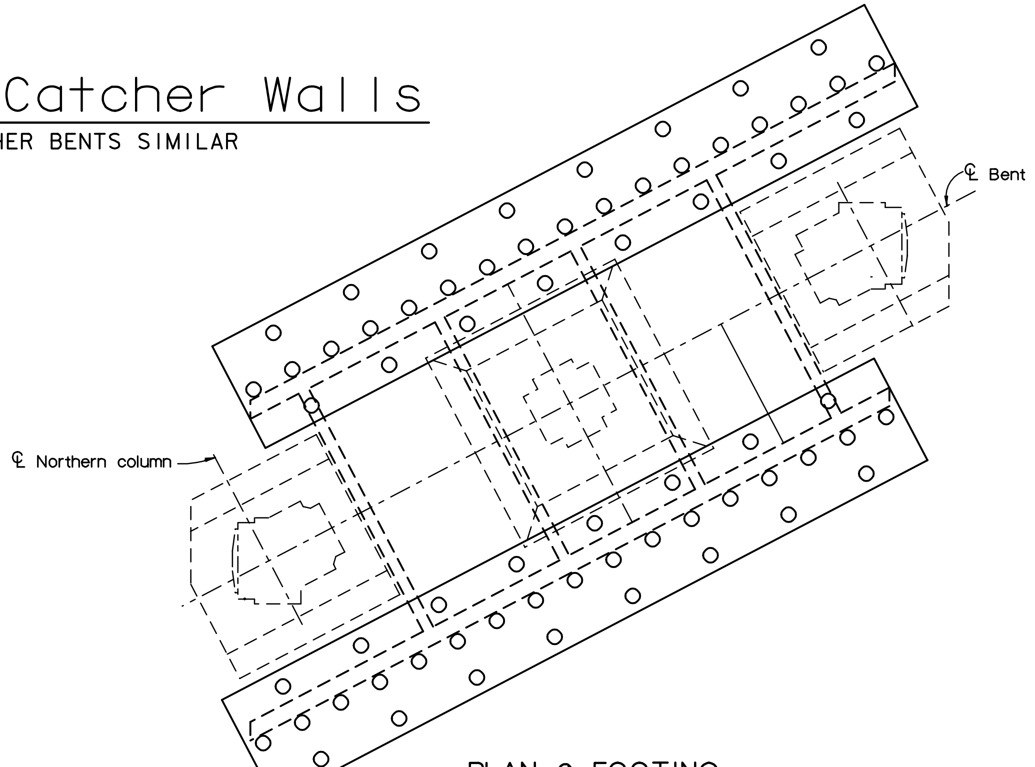
Alternative 3 - Catcher Walls

NOTE: BENT 15 SHOWN, OTHER BENTS SIMILAR



PLAN

SCALE: 1/8" = 1'-0"



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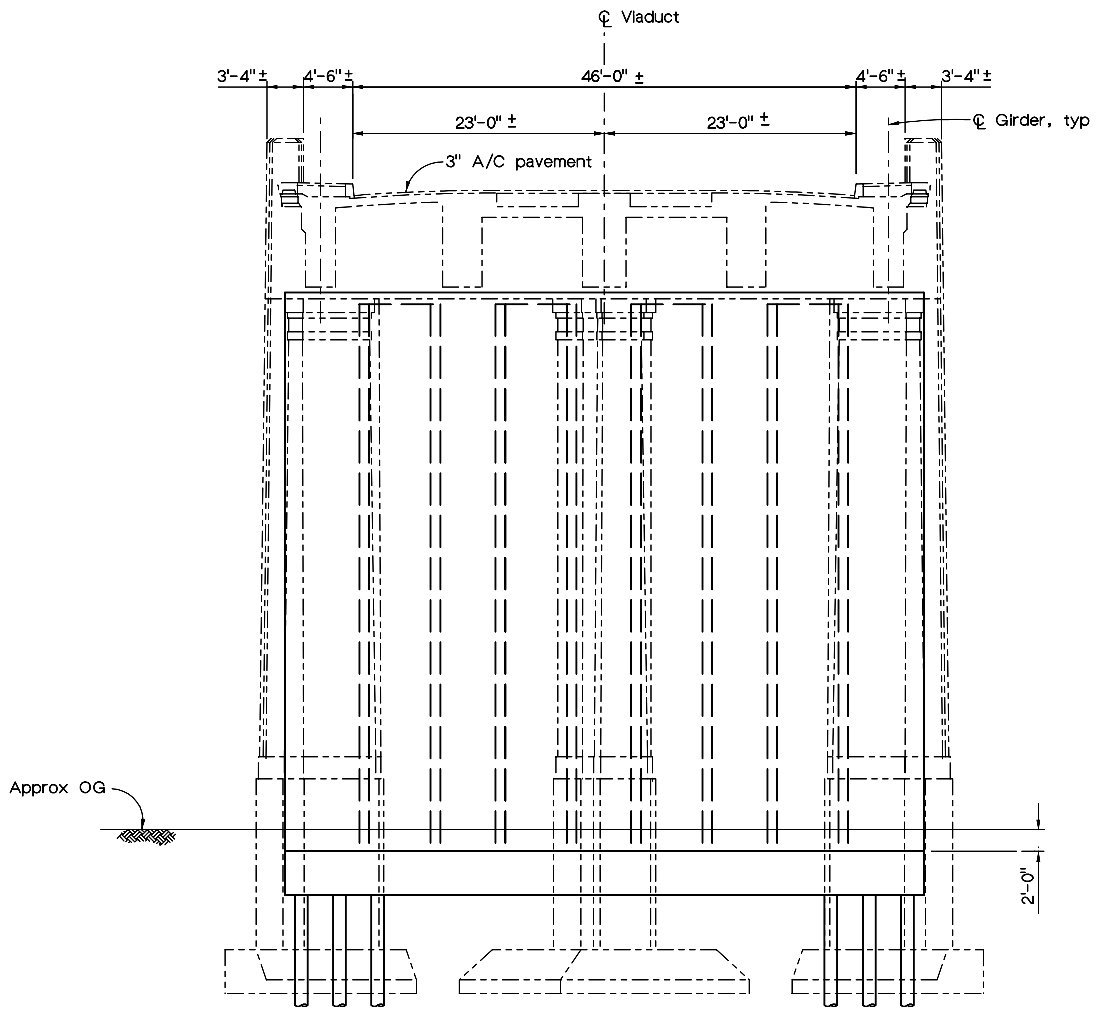
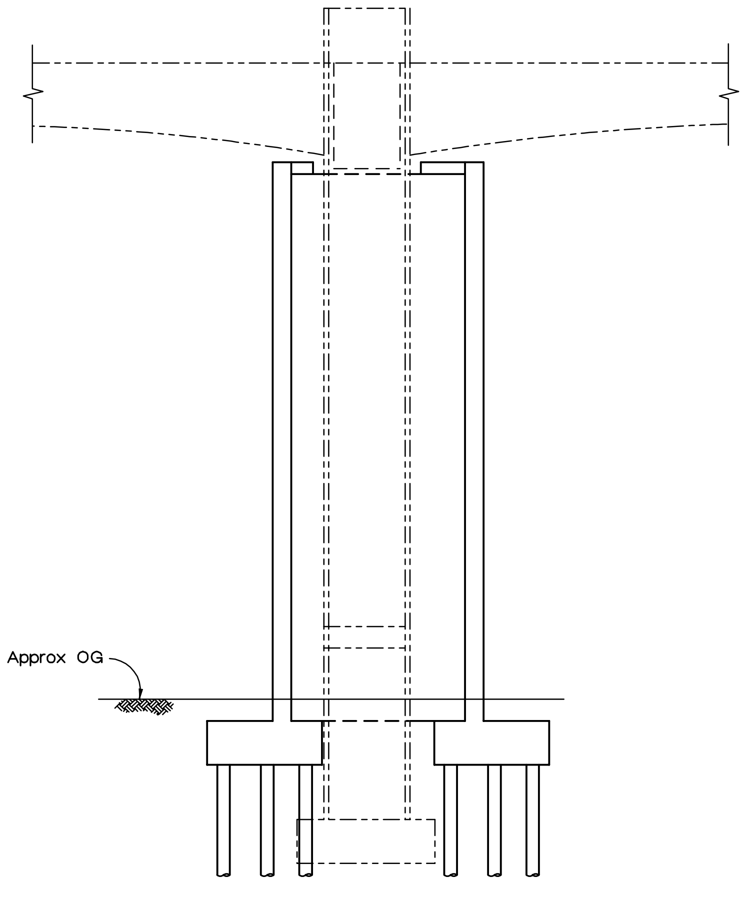


ELEVATION SECTION A-A

SCALE: 1/8" = 1'-0"



SECTION B-B

SCALE: 1/8" = 1'-0"

COST SUMMARY

Date of estimate		June 2004
Construction Cost	=	$ 29,967,437
Mobilization	=	$ 2,996,744
Contingencies	=	$ 4,944,627
Bridge Removal	=	$ 0
Traffic Control	=	$ 1,498.000
Stage Construction	=	$ 0
Railroad Work	=	$ 2,400,000
Total Construction Cost	=	$ 41,806,807
Construction Engineering	=	$ 5,453,062
Engineering Design & Environmental Clearance	=	$ 9,088,436
TOTAL ESTIMATED PROJECT COST	=	$ 56,348,306

NOTE: CATCHER WALLS ARE PROVIDED AT ALL BENTS (BENT 1 TO BENT 36) EXCEPT AT BENT 12.

SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
ALTERNATIVE 3 C WALL

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NO.	REVISION DESCRIPTION	DIV./DIST. ENGR.	DATE

CITY OF LOS ANGELES
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DATE ________ 20 ____

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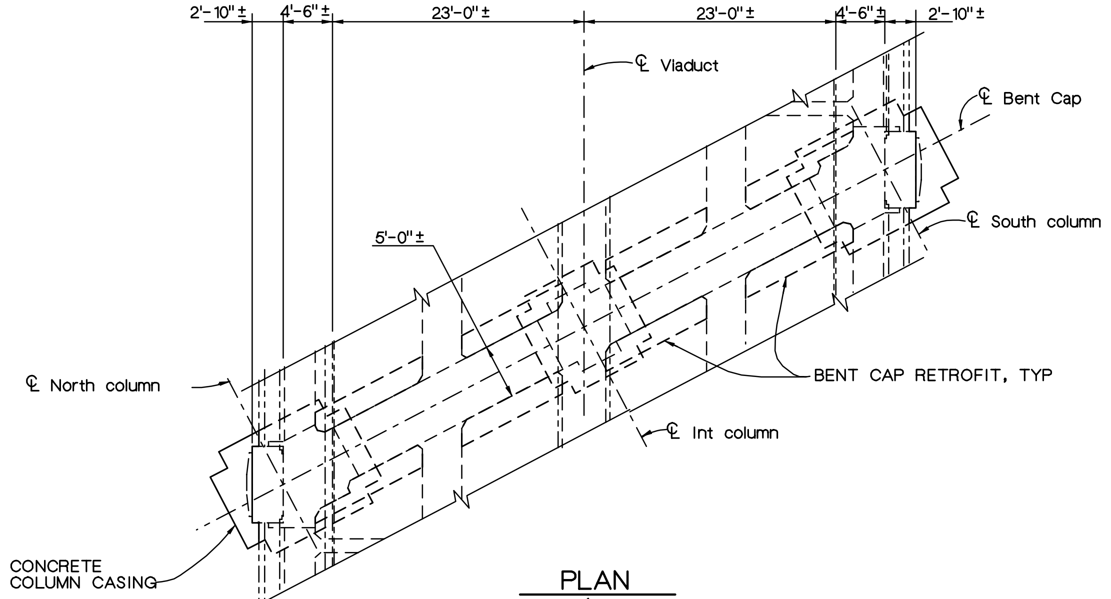
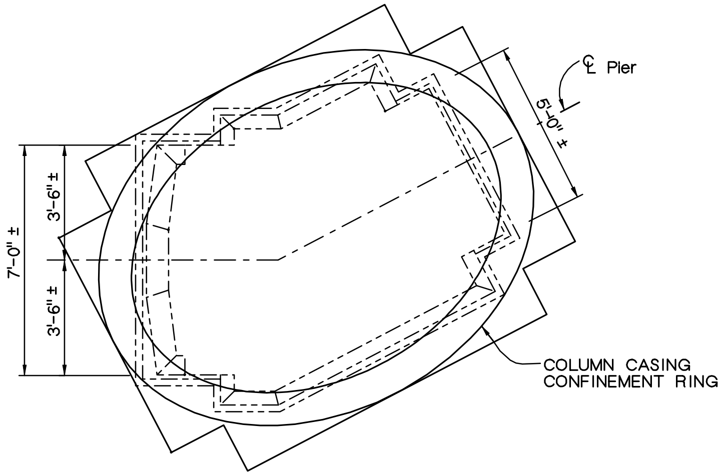
ALTERNATIVE 4 - CONCRETE CASING

NOTE: PIER 15 SHOWN, BENTS 3-6, 9-11, 13-14, 16-28, 30-32, 34 and 35 ARE SIMILAR (TOTAL OF 28 BENTS)



PLAN

SCALE: 1/8" = 1'-0"

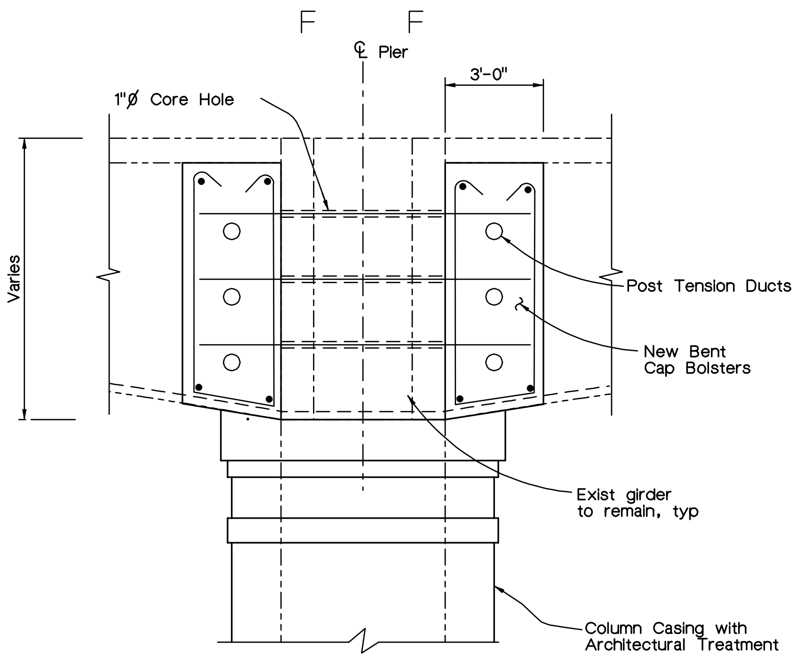


SECTION A-A

SCALE: 3/8" = 1'-0"

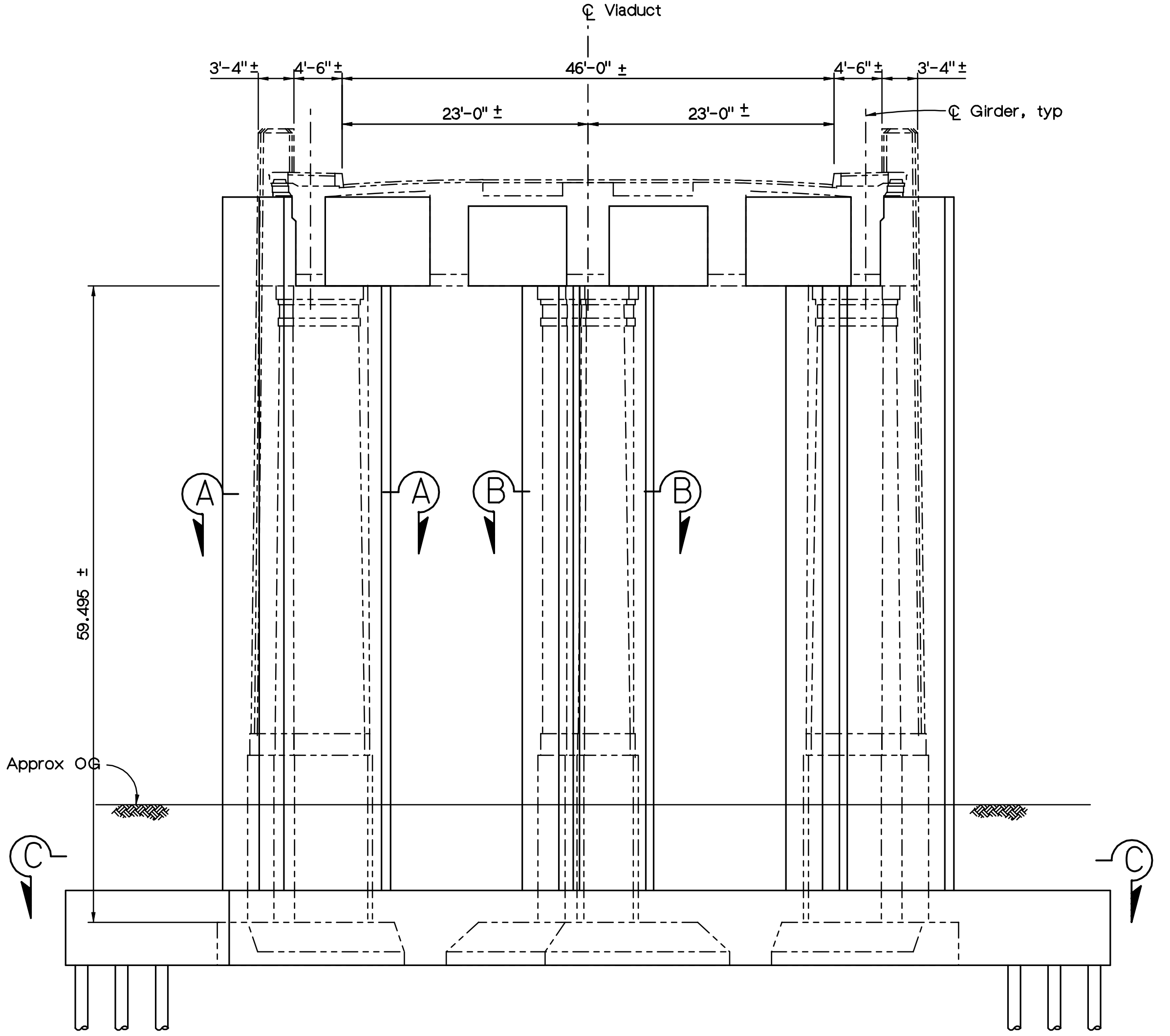
COST SUMMARY

Date of estimate		June 2004
Construction Cost	=	$ 45,338,945
Mobilization	=	$ 4,533,894
Contingencies	=	$ 7,480,926
Bridge Removal	=	$ 0
Traffic Control	=	$ 2,267,000
Stage Construction	=	$ 906,779
Railroad Work	=	$ 1,200,000
Total Construction Cost	=	$ 61,727,544
Construction Engineering	=	$ 8,051,419
Engineering Design & Environmental Clearance	=	$ 13,419,031
TOTAL ESTIMATED PROJECT COST	=	$ 83,197,994



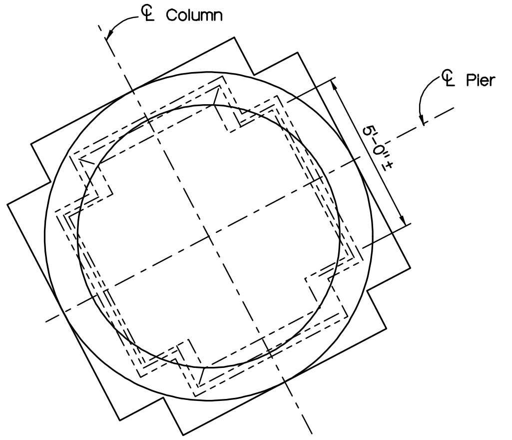
Bent Cap Bolster

SCALE: 5/32" = 1'-0"



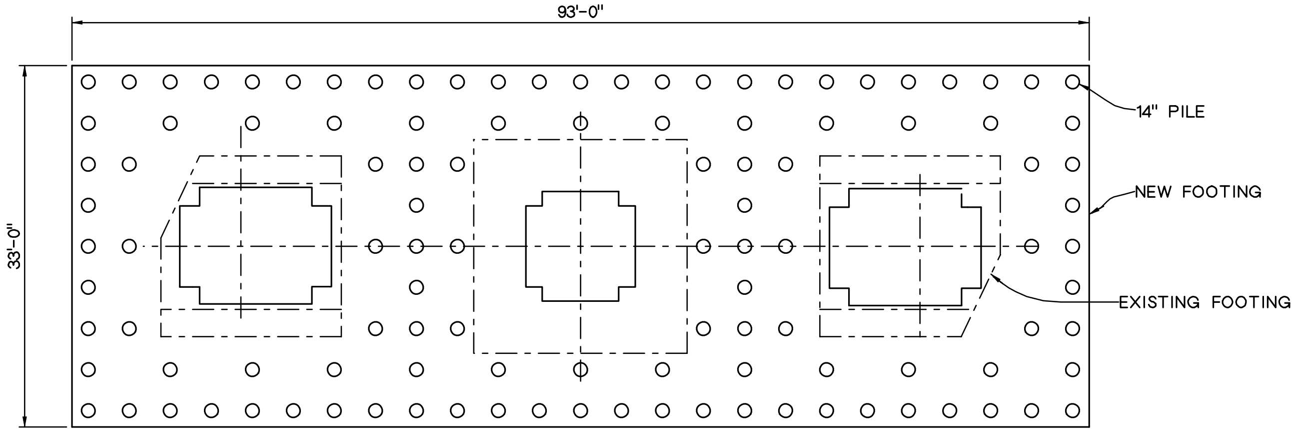
ELEVATION

SCALE: 1/8" = 1'-0"



SECTION B-B

SCALE: 3/8" = 1'-0"



SECTION C-C

SCALE: 3/8" = 1'-0"

SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
ALTERNATIVE 4 CONC

DATE

DESIGNED
DRAWN
CHECKED
SUPERVISED
PROJECT ENGR.
R. E. NO.
ASST.DIV./DIST.ENGR.
R. E. NO.

NO. | REVISION DESCRIPTION | DIV./DIST. ENGR. | DATE

CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER

DATE ______ 20 ___

DIV./DIST.ENGR. R. E. NO.

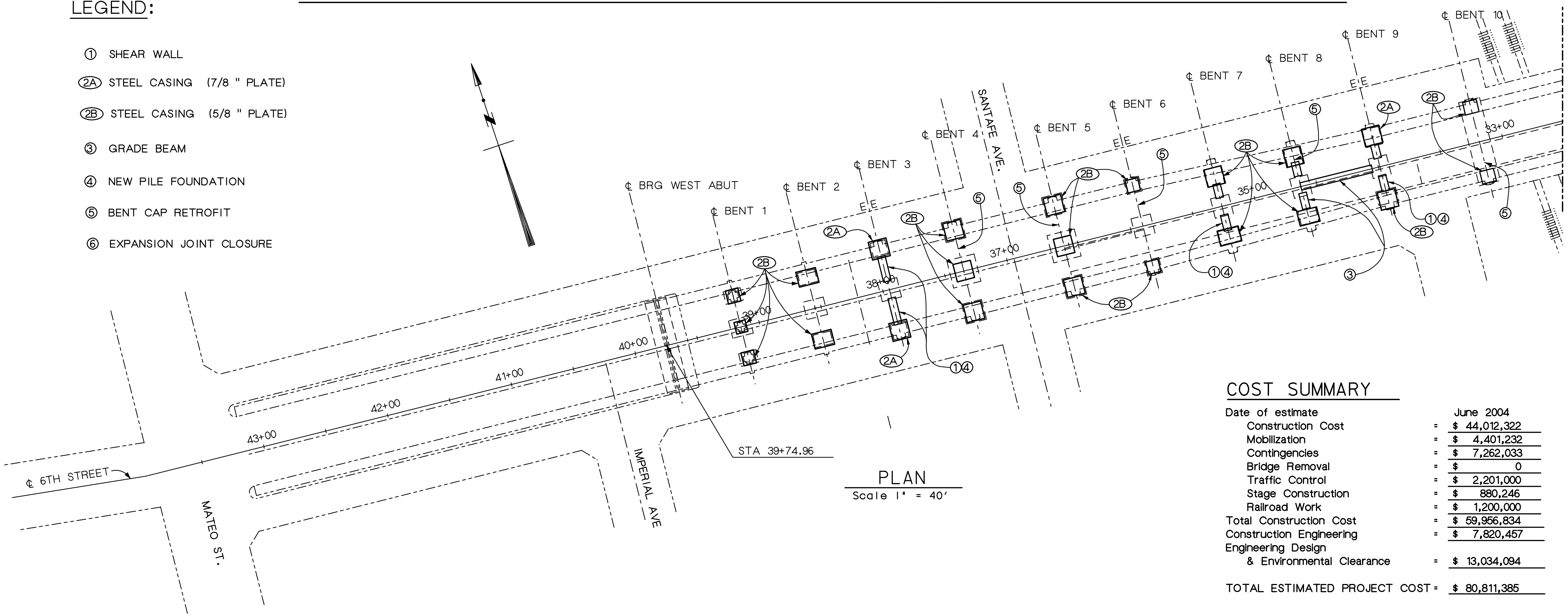
SCALES HORIZ. = VERT. = | SHEET OF SHEETS | INDEX NUMBER

$DATE$ $TIME$ $FILE$

SEISMIC RETROFIT STRATEGY ALTERNATIVE 5 - SHEAR WALL, STEEL CASING AND ASR PROTECTION

RETROFIT LEGEND:

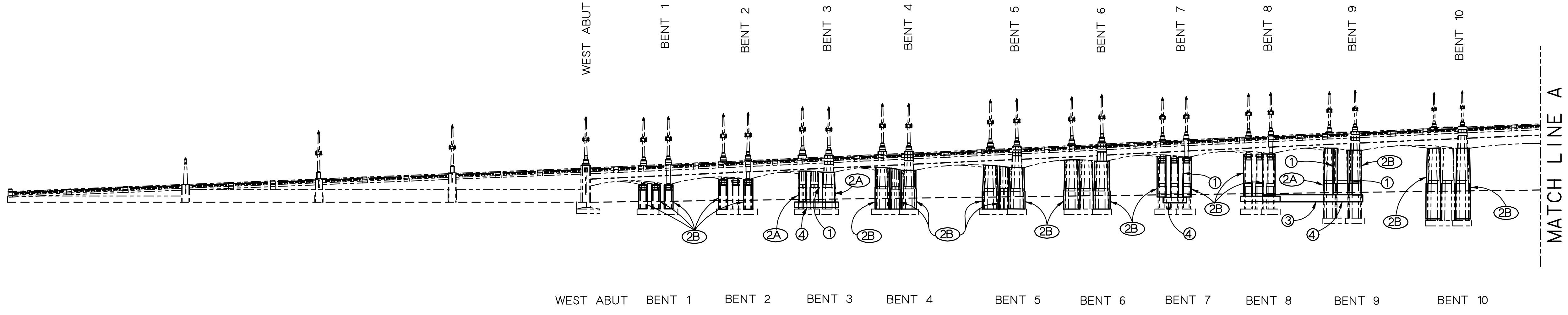
① SHEAR WALL
(2A) STEEL CASING (7/8 " PLATE)
(2B) STEEL CASING (5/8 " PLATE)
③ GRADE BEAM
④ NEW PILE FOUNDATION
⑤ BENT CAP RETROFIT
⑥ EXPANSION JOINT CLOSURE



COST SUMMARY

Date of estimate		June 2004
Construction Cost	=	$ 44,012,322
Mobilization	=	$ 4,401,232
Contingencies	=	$ 7,262,033
Bridge Removal	=	$ 0
Traffic Control	=	$ 2,201,000
Stage Construction	=	$ 880,246
Railroad Work	=	$ 1,200,000
Total Construction Cost	=	$ 59,956,834
Construction Engineering	=	$ 7,820,457
Engineering Design & Environmental Clearance	=	$ 13,034,094
TOTAL ESTIMATED PROJECT COST	=	$ 80,811,385

PLAN
Scale 1" = 40'



ELEVATION
Scale 1" = 40'

NO.	REVISION DESCRIPTION	DIV./DIST. ENGR.	DATE

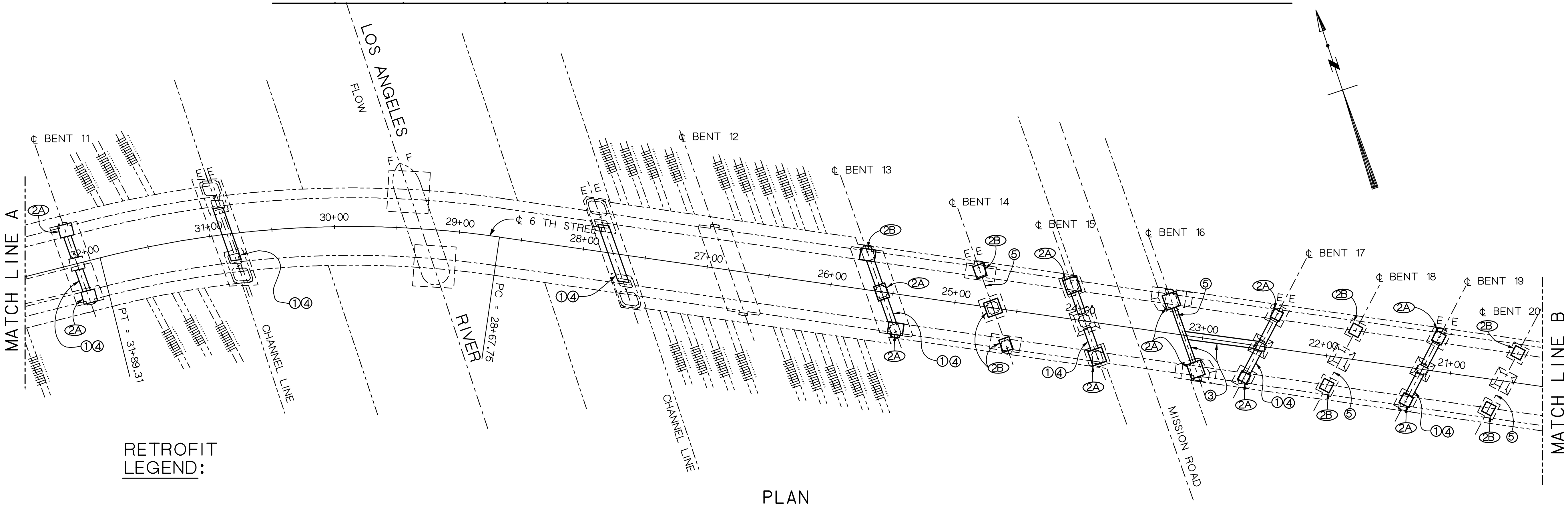
SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
GENERAL PLAN NO. 1

DESIGNED	
DRAWN	
CHECKED	
SUPERVISED	
PROJECT ENGR.	
R. E. NO.	
ASST.DIV./DIST.ENGR.	
R. E. NO.	

CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER
DATE ________ 20 ____
DIV./DIST.ENGR. R. E. NO.

SCALES HORZ. = VERT. = SHEET OF SHEETS INDEX NUMBER

SEISMIC RETROFIT STRATEGY ALTERNATIVE 5 - SHEAR WALL, STEEL CASING AND ASR PROTECTION



PLAN

Scale 1" = 40'

RETROFIT LEGEND:

① SHEAR WALL

2A STEEL CASING (7/8" PLATE)

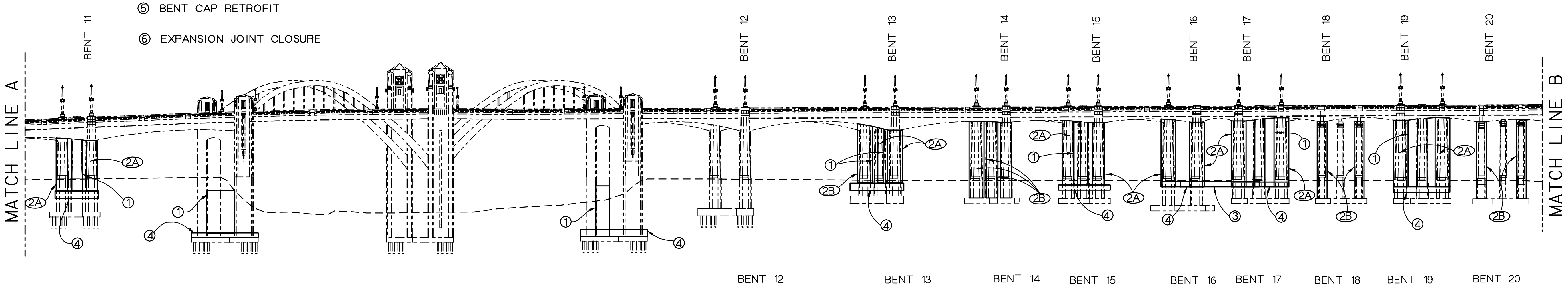
2B STEEL CASING (5/8" PLATE)

③ GRADE BEAM

④ NEW PILE FOUNDATION

⑤ BENT CAP RETROFIT

⑥ EXPANSION JOINT CLOSURE



ELEVATION

Scale 1" = 40'

SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
GENERAL PLAN NO. 2

DATE
DESIGNED
DRAWN
CHECKED
SUPERVISED
PROJECT ENGR.
R. E. NO.
ASST.DIV./DIST.ENGR.
R. E. NO.

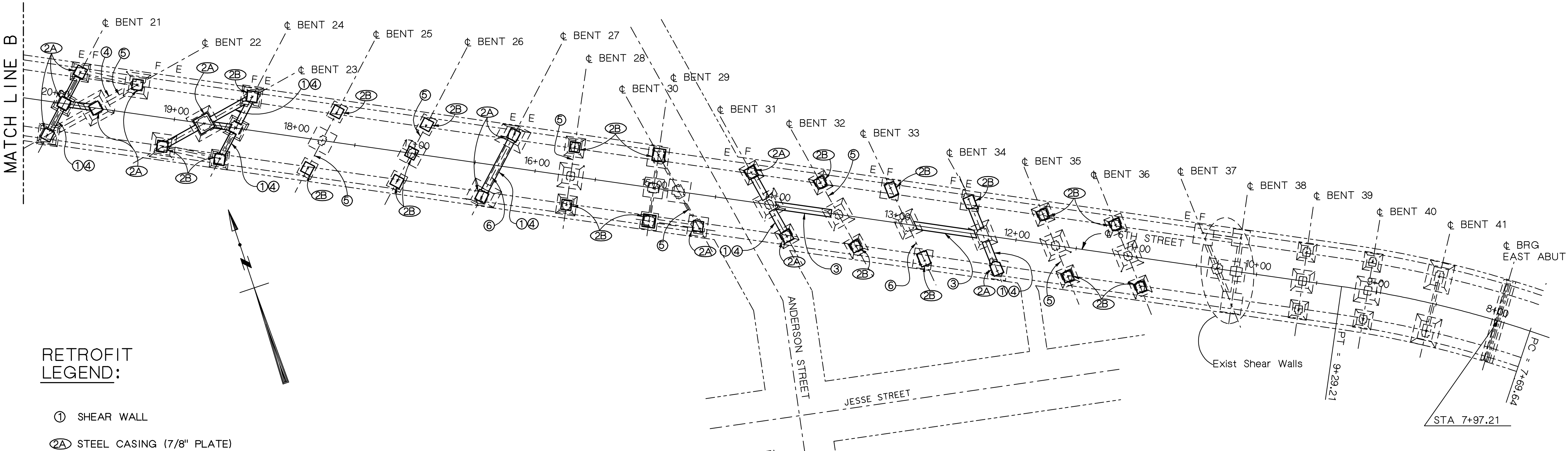
DATE
DIV./DIST. ENGR.
REVISION DESCRIPTION
NO.

CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER
DATE ________ 20 ____
DIV./DIST.ENGR. R. E. NO.

$DATE$ $TIME$ $FILE$

SCALES HORZ. = VERT. = SHEET OF SHEETS INDEX NUMBER

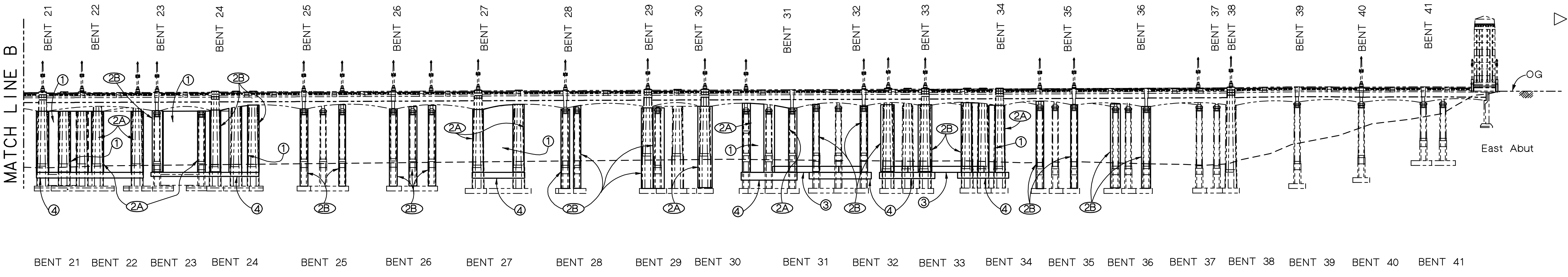
SEISMIC RETROFIT STRATEGY ALTERNATIVE 5 - SHEAR WALL, STEEL CASING AND ASR PROTECTION



PLAN
Scale 1" = 40'

RETROFIT LEGEND:

- ① SHEAR WALL
- (2A) STEEL CASING (7/8" PLATE)
- (2B) STEEL CASING (5/8" PLATE)
- ③ GRADE BEAM
- ④ NEW PILE FOUNDATION
- ⑤ BENT CAP RETROFIT
- ⑥ EXPANSION JOINT CLOSURE



ELEVATION
Scale 1" = 40'

SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
GENERAL PLAN NO. 3

DESIGNED
DRAWN
CHECKED
SUPERVISED
PROJECT ENGR.
R. E. NO.
ASST.DIV./DIST.ENGR.
R. E. NO.
DATE

DATE
DIV./DIST. ENGR.
REVISION DESCRIPTION
NO.

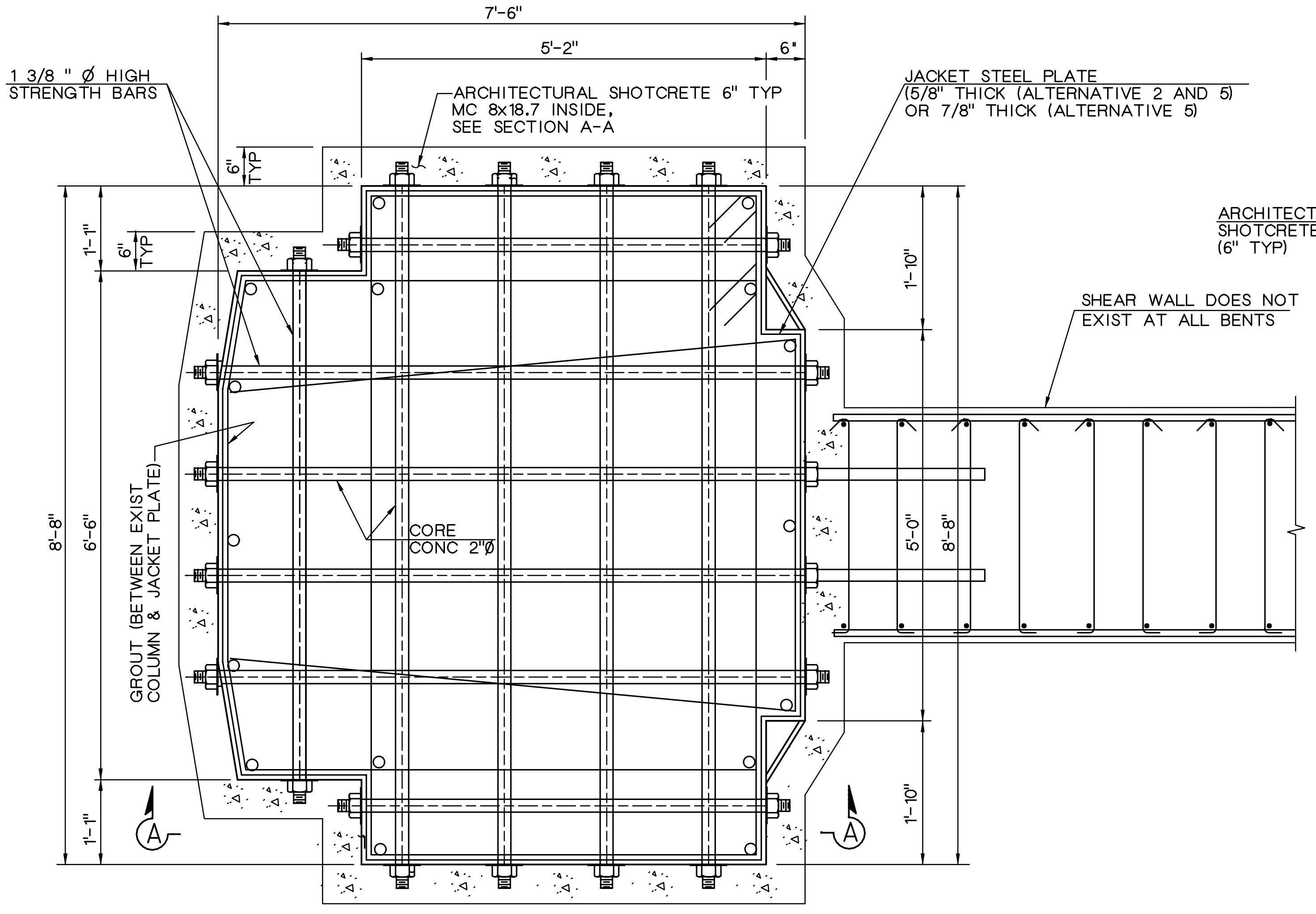
CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER
DATE ________ 20 ____
DIV./DIST.ENGR. R. E. NO.

SCALES HRZ. = VERT. = SHEET OF SHEETS INDEX NUMBER

$DATE$ $TIME$ $FILE$

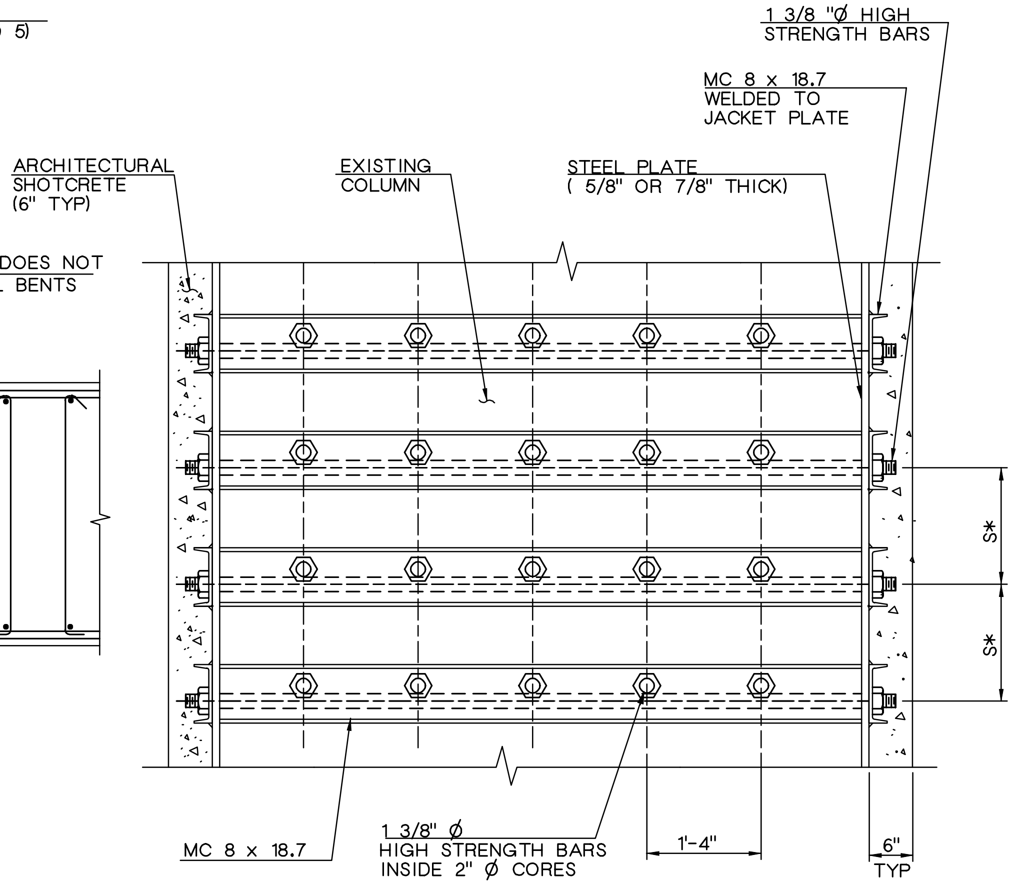
SEISMIC RETROFIT STRATEGY ALTERNATIVES 2 AND 5

TYPICAL DETAIL OF STEEL CASINGS



PLAN

NOTE:
EXTERIOR COLUMNS
AT BENT 3 SHOWN,
OTHERS SIMILAR



SECTION A-A

S* = 1'-4" IN TOP & BOTTOM ZONES OF THE COLUMN
S* = 2'-8" IN MIDDLE ZONE OF THE COLUMN

NO.	REVISION DESCRIPTION	DIV./DIST. ENGR.	DATE

SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
STEEL CASING DETAILS

DESIGNED
DRAWN
CHECKED
SUPERVISED
PROJECT ENGR.
R. E. NO.
ASST.DIV./DIST.ENGR.
R. E. NO.
DATE

CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER

DATE ________ 20 ____

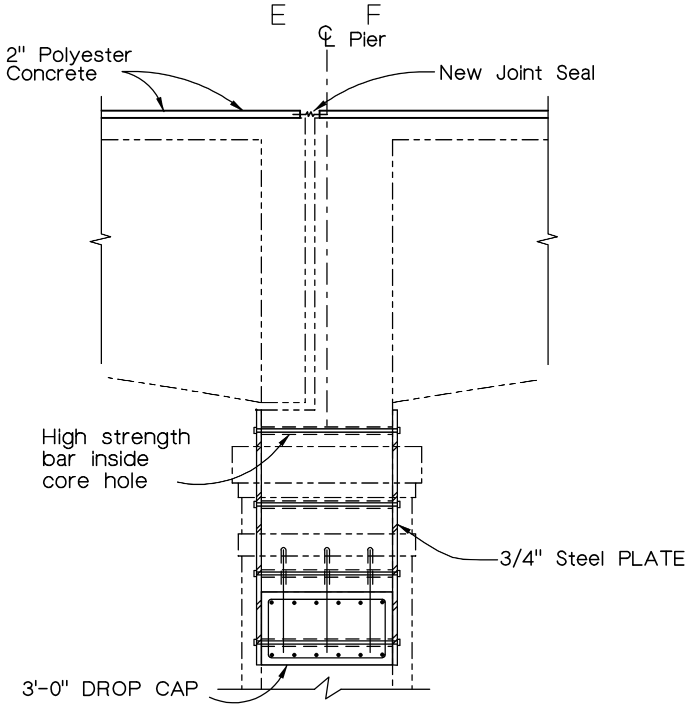
DIV./DIST.ENGR. R. E. NO.

SCALES HORIZ. =
VERT. =
SHEET OF SHEETS INDEX NUMBER

$DATE$ $TIME$ $FILE$

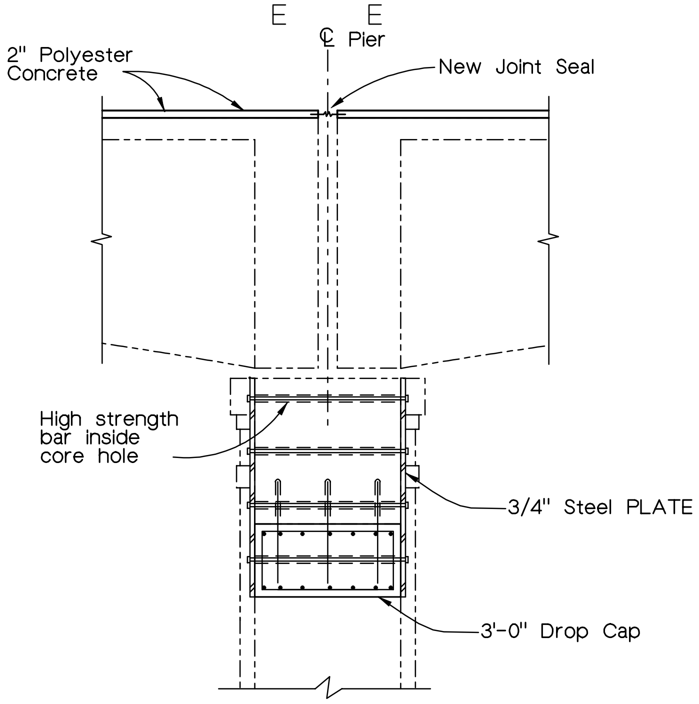
SEISMIC RETROFIT STRATEGY ALTERNATIVE 5 SHEAR WALL, STEEL CASING AND ASR PROTECTION

BENT CAP RETROFITS



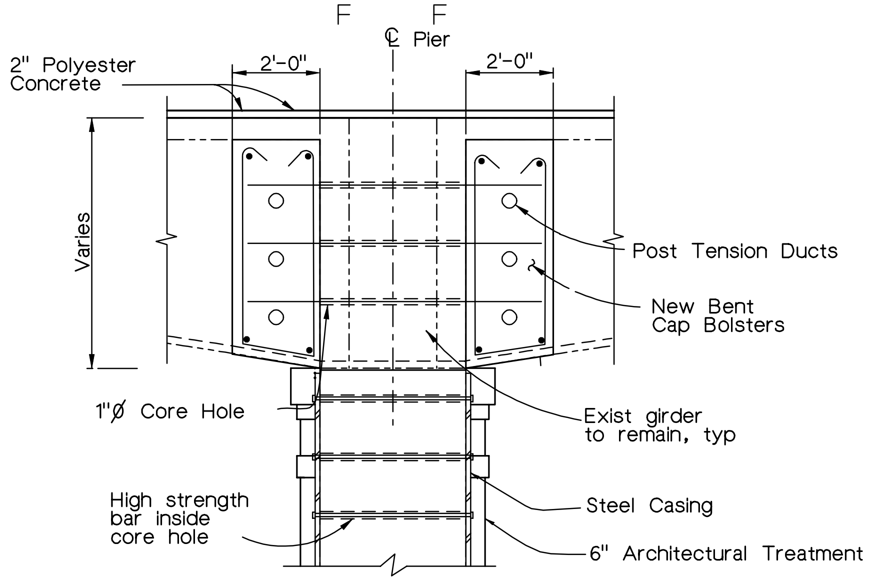
Bent Cap Retrofit
One Sided Expansion Joint
SCALE:5/32" = 1'-0"

BENT 22



Bent Cap Retrofit
Two Sided Expansion Joint
SCALE:5/32" = 1'-0"

BENTS 6 & 14



Bent Cap Bolsters
SCALE:5/32" = 1'-0"

BENTS 4, 5, 8, 10, 16, 18
20, 25, 26, 28, 30, 32 & 35

NOTE: EXPANSION JOINTS WILL BE CLOSED AT BENTS 27 AND 33 (NOT SHOWN)

SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
BENT CAP RETROFITS

DATE
DESIGNED
DRAWN
CHECKED
SUPERVISED
PROJECT ENGR.
R. E. NO.
ASST.DIV./DIST.ENGR.
R. E. NO.

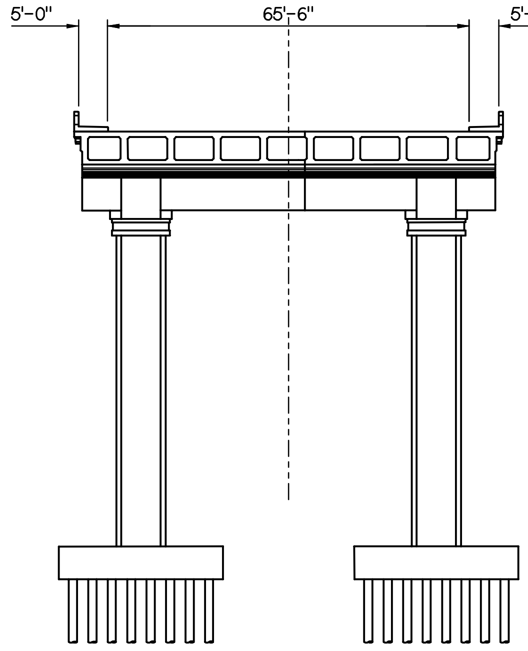
NO.
REVISION DESCRIPTION
DIV./DIST. ENGR.
DATE

CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER
DATE ______ 20 ___
DIV./DIST.ENGR.
R. E. NO.

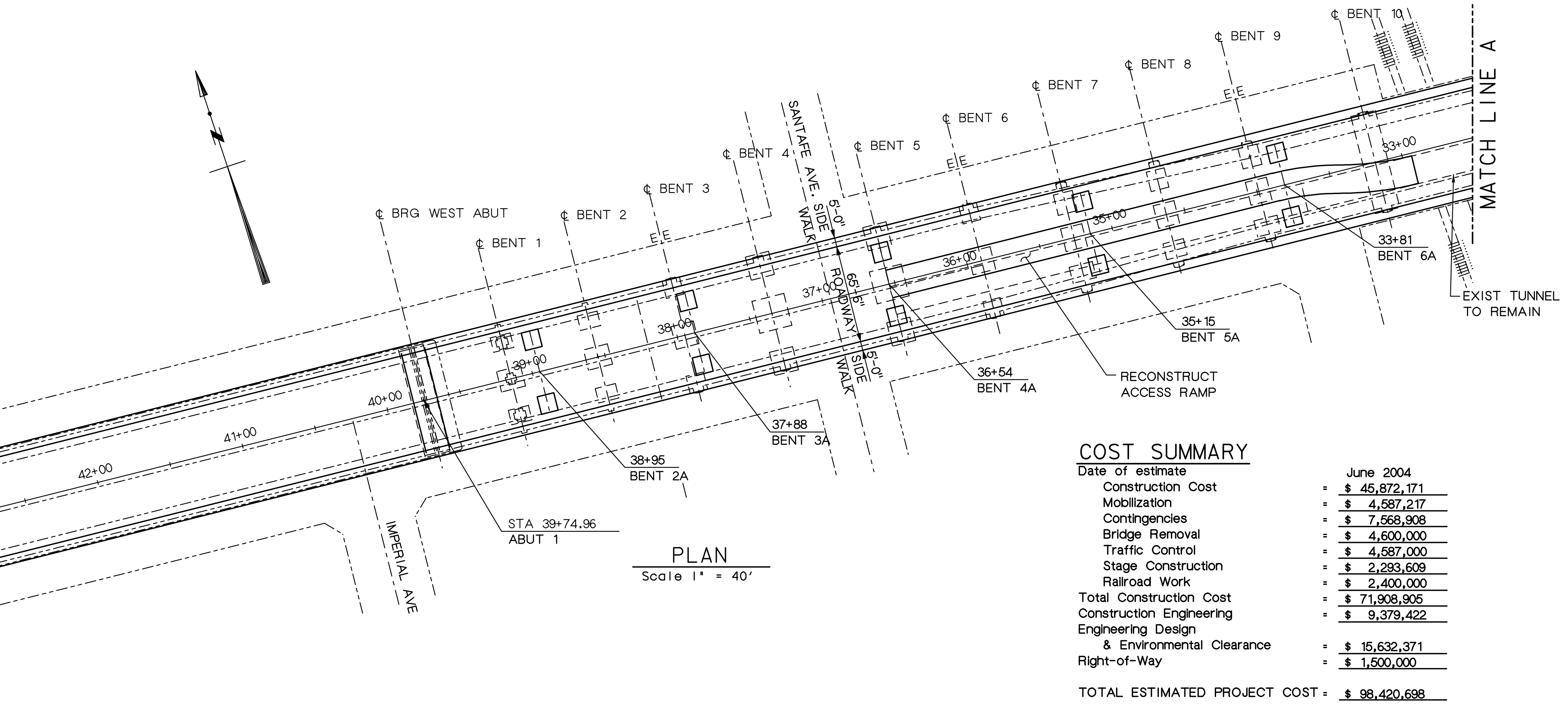
SCALES HORIZ. =
VERT. =
SHEET OF SHEETS
INDEX NUMBER

$DATE$ $TIME$ $FILE$

SEISMIC RETROFIT STRATEGY ALTERNATIVE 6A - REPLACE IN-KIND



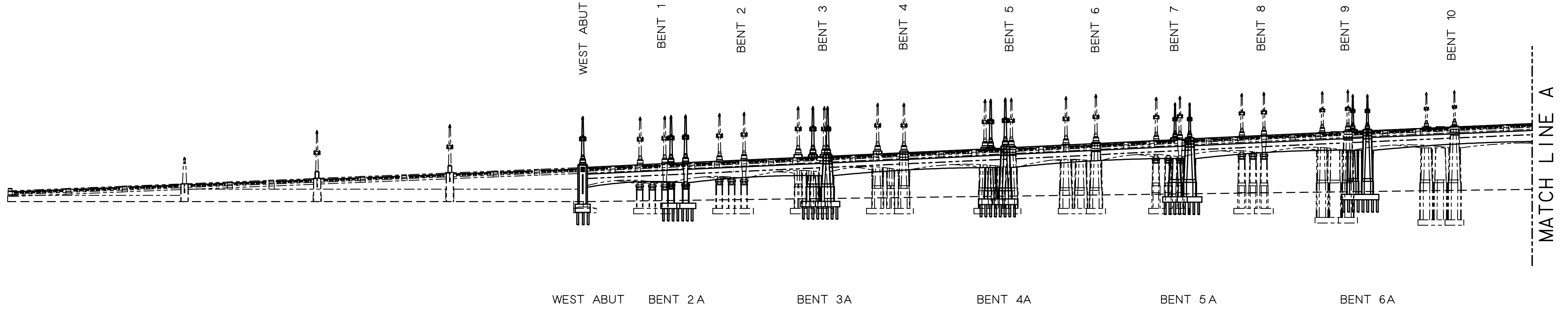
TYPICAL SECTION
Scale 1" = 20'



PLAN
Scale 1" = 40'

COST SUMMARY

Date of estimate		June 2004
Construction Cost	=	$ 45,872,171
Mobilization	=	$ 4,587,217
Contingencies	=	$ 7,568,908
Bridge Removal	=	$ 4,600,000
Traffic Control	=	$ 4,587,000
Stage Construction	=	$ 2,293,609
Railroad Work	=	$ 2,400,000
Total Construction Cost	=	$ 71,908,905
Construction Engineering	=	$ 9,379,422
Engineering Design & Environmental Clearance	=	$ 15,632,371
Right-of-Way	=	$ 1,500,000
TOTAL ESTIMATED PROJECT COST	=	$ 98,420,698



ELEVATION
Scale 1" = 40'

DESIGNED
DRAWN
CHECKED
SUPERVISED
PROJECT ENGR.
R. E. NO.
ASST.DIV./DIST.ENGR.
R. E. NO.
DATE

SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
GENERAL PLAN NO. 1

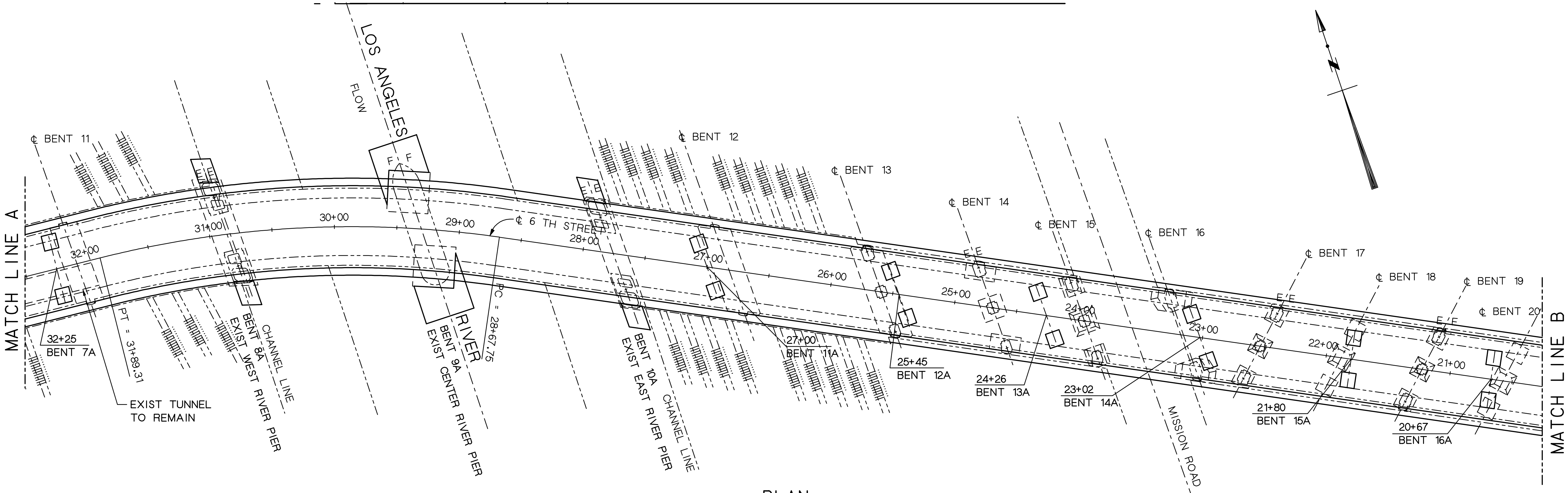
NO.	REVISION DESCRIPTION	DIV./DIST. ENGR.	DATE

CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER
DATE ________ 20 ____
DIV./DIST.ENGR. R. E. NO.

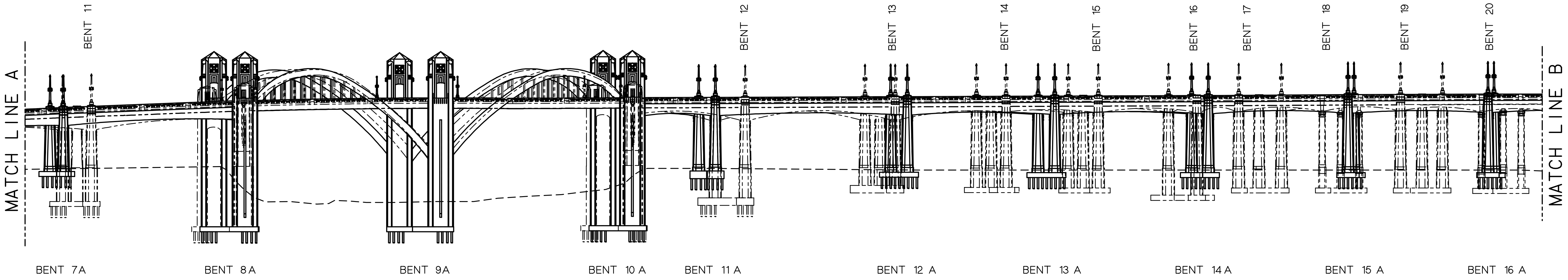
SCALES HORIZ. = VERT. = SHEET OF SHEETS INDEX NUMBER

$DATE$ $TIME$

SEISMIC RETROFIT STRATEGY ALTERNATIVE 6A - REPLACE IN-KIND



PLAN
Scale 1" = 40'



ELEVATION
Scale 1" = 40'

NO.	REVISION DESCRIPTION	DIV./DIST. ENGR.	DATE

SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
GENERAL PLAN NO. 2

	DATE
DESIGNED	
DRAWN	
CHECKED	
SUPERVISED	
PROJECT ENGR.	
R. E. NO.	
ASST.DIV./DIST.ENGR.	
R. E. NO.	

CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER

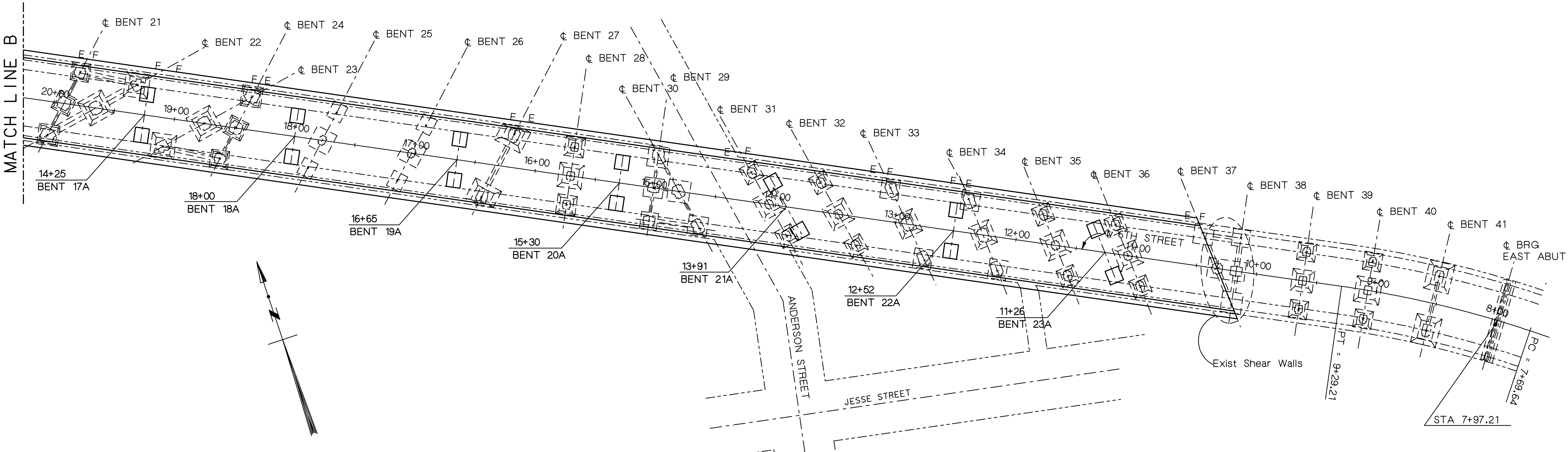
DATE ______ 20 ____

DIV./DIST.ENGR. R. E. NO.

SCALES HORZ. =
VERT. =
SHEET OF SHEETS INDEX NUMBER

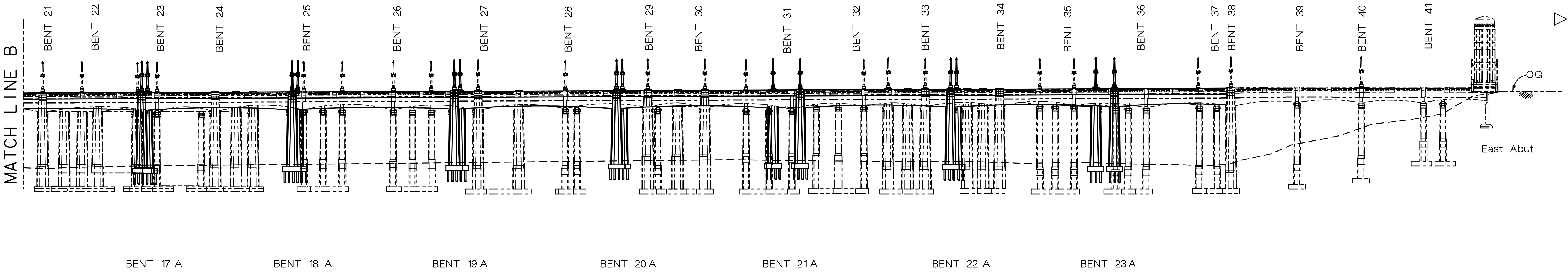
$DATE$ $TIME$
$FILE$

SEISMIC RETROFIT STRATEGY ALTERNATIVE 6A - REPLACE IN-KIND



PLAN

Scale 1" = 40'



ELEVATION

Scale 1" = 40'

DESIGNED	DATE
DRAWN	
CHECKED	
SUPERVISED	
PROJECT ENGR.	
R. E. NO.	
ASST.DIV./DIST.ENGR.	
R. E. NO.	

SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
GENERAL PLAN NO. 3

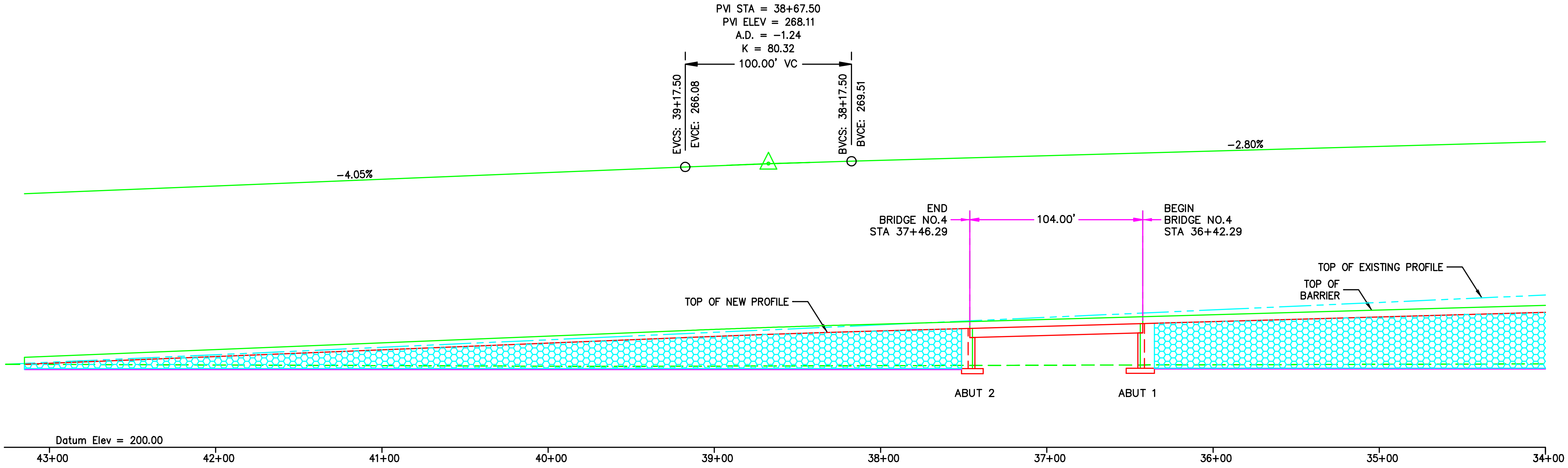
NO.	REVISION DESCRIPTION	DIV./DIST. ENGR.	DATE

CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER

DATE ________ 20 ____

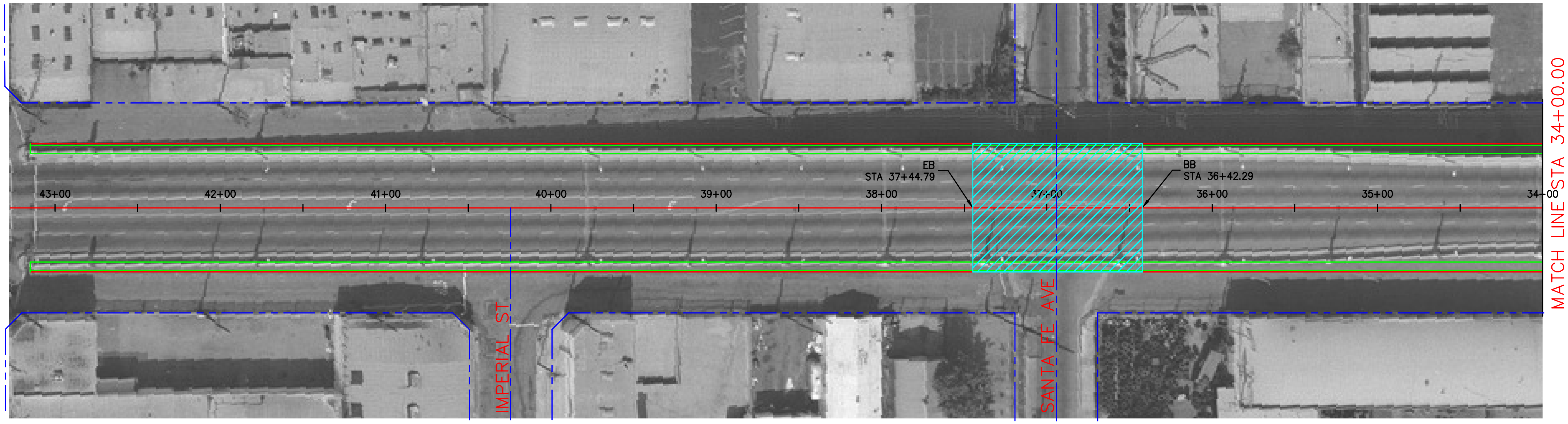
DIV./DIST.ENGR. R. E. NO.

SCALES HORIZ. = VERT. = SHEET OF SHEETS INDEX NUMBER



DEVELOPED ELEVATION

SCALE 1"=30'



PLAN

SCALE 1"=30'

BUREAU OF ENGINEERING ○ STRUCTURAL ENGINEERING DIVISION

ARY L. MOORE P.E. CITY ENGINEER • BRADLEY M. SMITH DEPUTY CITY ENGINEER • PETER SEE PROGRAM MANAGER • JOHN KOO DIVISION ENGINEER • 650 S. SPRING STREET, SUITE 400 TEL: (213) 847-8777

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NO.	REVISIONS:	DATE:	BY:

INDEX NO.

DESIGN GROUP

	DATE:
DESIGNED:	
DRAWN:	
CHECKED: LIC. NO.	
PROJECT MANAGER: LIC. NO.	
ENGINEER IN RESPONSIBLE CHARGE: LIC. NO.	

STRUCTURE NO.

SHEET TITLE:

PROJECT: SIXTH STREET VIADUCT OVER LOS ANGELES RIVER SEISMIC RETROFIT PRE-STRATEGY

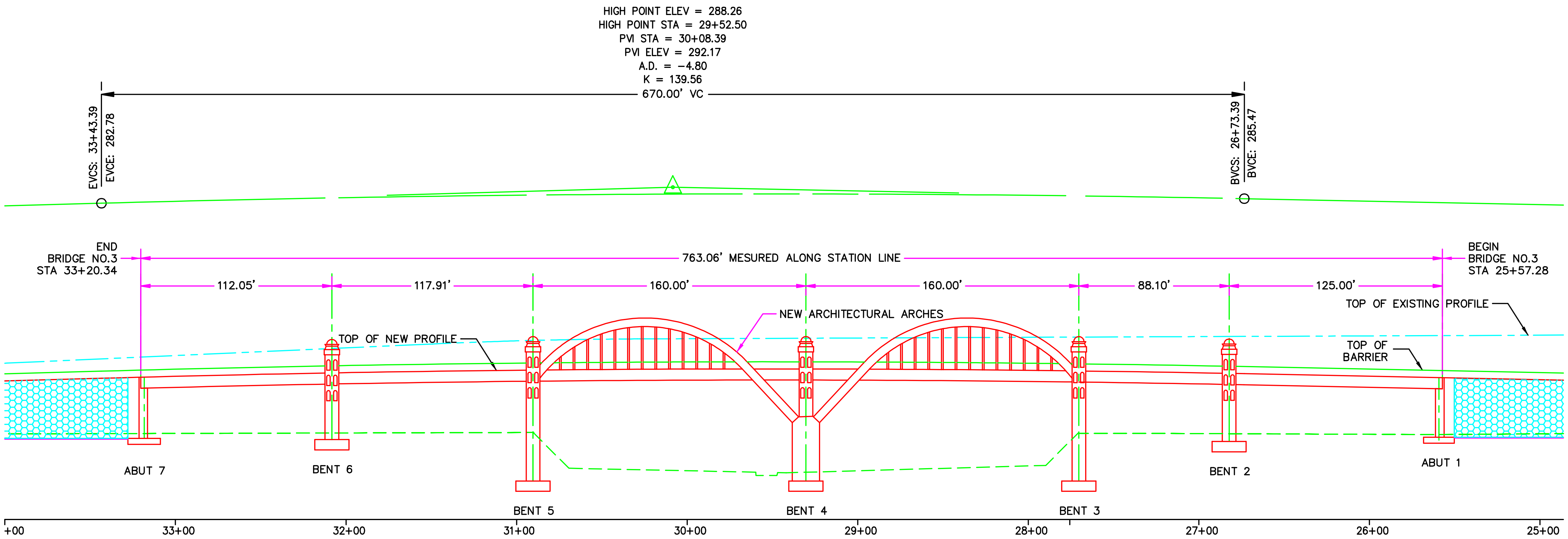
ADDRESS:

WORK ORDER NO.
E230D

DRAWING NO.

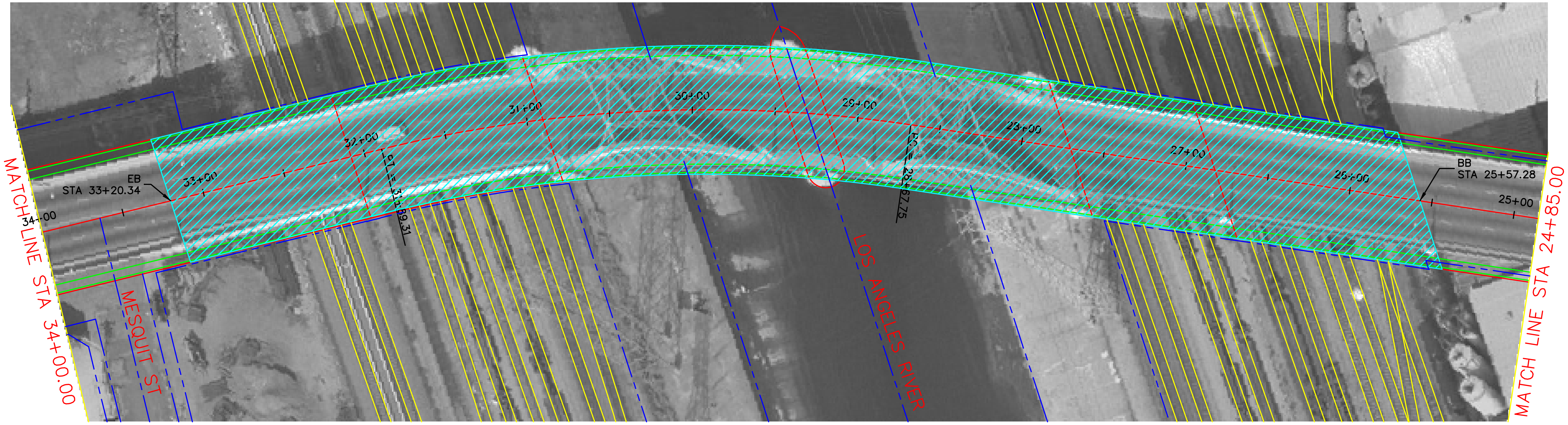
SHEET OF SHEETS

REVISION DATES (DESIGN STAGE ONLY)



DEVELOPED ELEVATION

SCALE 1"=30'



PLAN

SCALE 1"=30'

BUREAU OF ENGINEERING STRUCTURAL ENGINEERING DIVISION

GARY L. MOORE P.E. CITY ENGINEER
BRADLEY M. SMITH
PETER SEE
JOHN KOO

650 S. SPRING STREET, SUITE 400 TEL (213) 847-8777

INDEX NO.

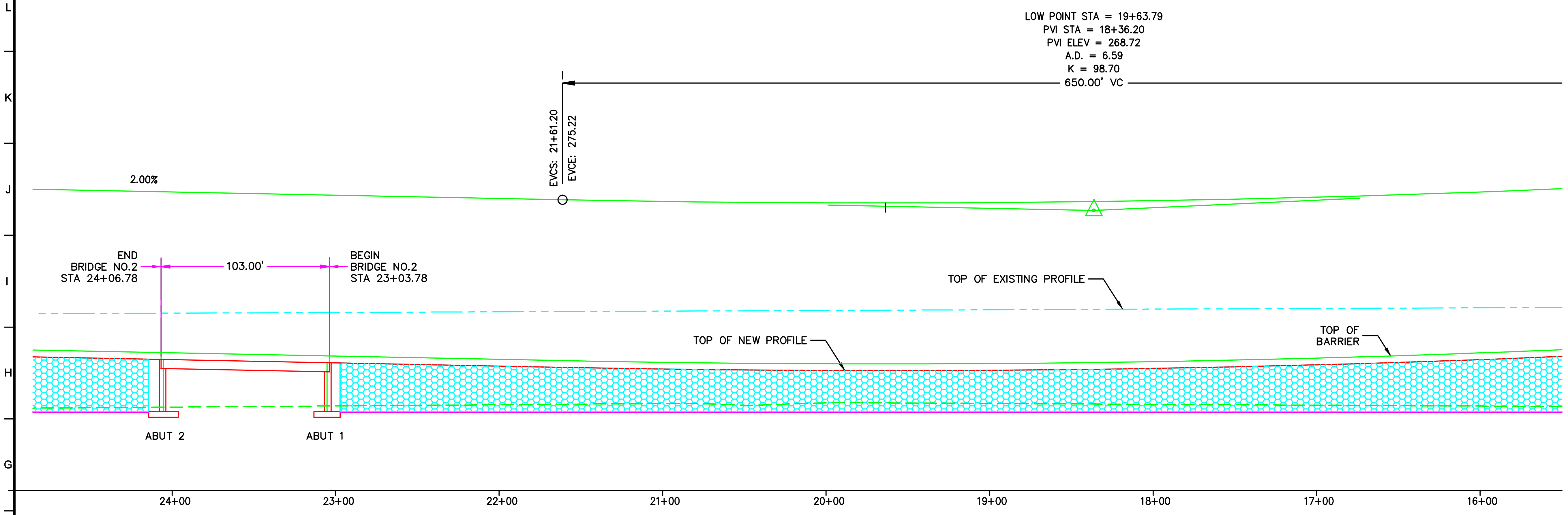
STRUCTURE NO.

SIXTH STREET VIADUCT OVER LOS ANGELES RIVER SEISMIC RETROFIT PRE-STRATEGY

WORK ORDER NO. E230D

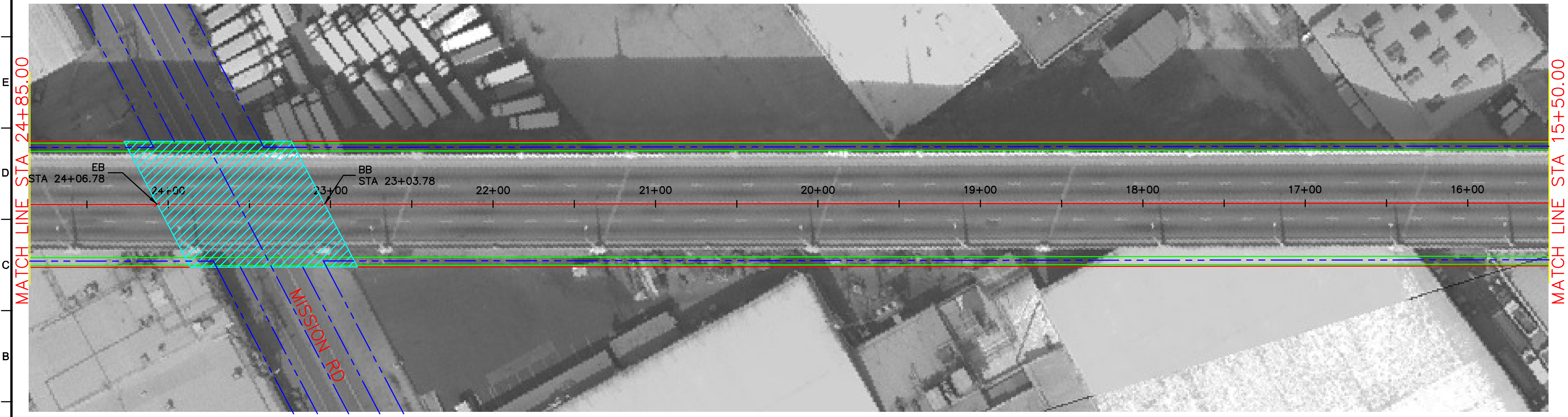
DRAWING NO.

SHEET OF SHEETS



DEVELOPED ELEVATION

SCALE 1"=30'



PLAN

SCALE 1"=30'

BUREAU OF ENGINEERING ○ STRUCTURAL ENGINEERING DIVISION

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INDEX NO.

DESIGN GROUP

		DATE:
DESIGNED:		
DRAWN:		
CHECKED:	LIC. NO.	
PROJECT MANAGER:	LIC. NO.	
ENGINEER IN RESPONSIBLE CHARGE:	LIC. NO.	

STRUCTURE NO.

SHEET TITLE:

PROJECT: SIXTH STREET VIADUCT OVER LOS ANGELES RIVER SEISMIC RETROFIT PRE-STRATEGY

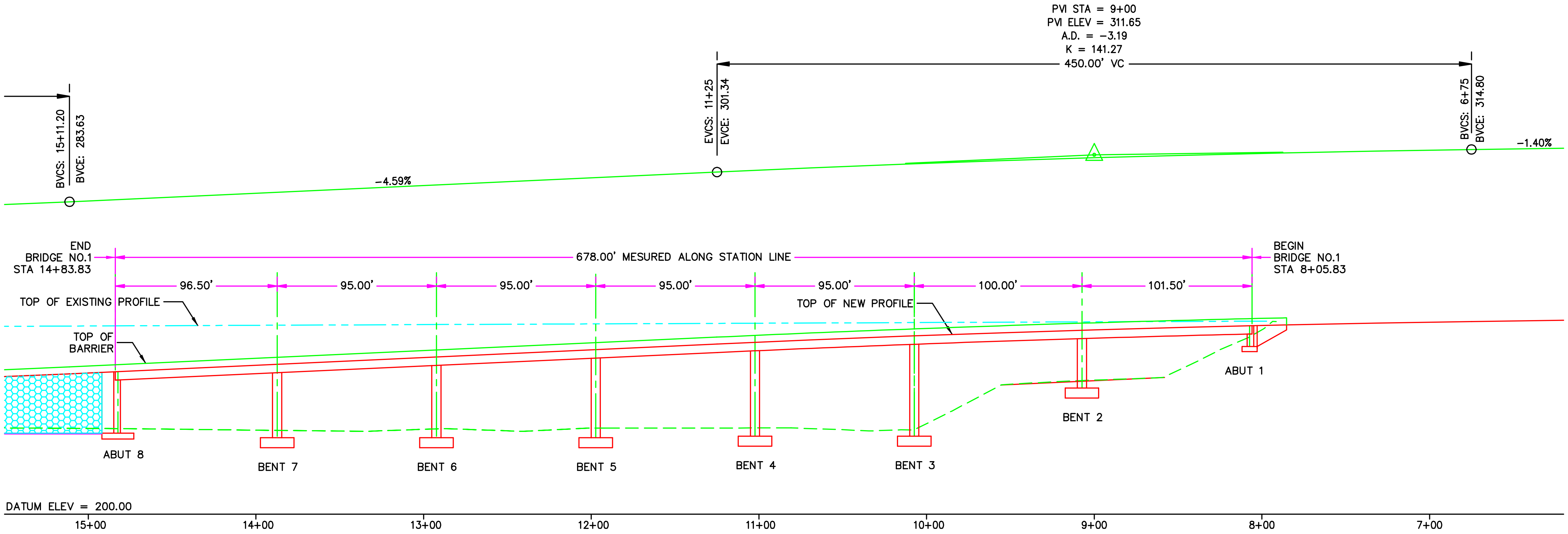
ADDRESS:

WORK ORDER NO. E230D

DRAWING NO.

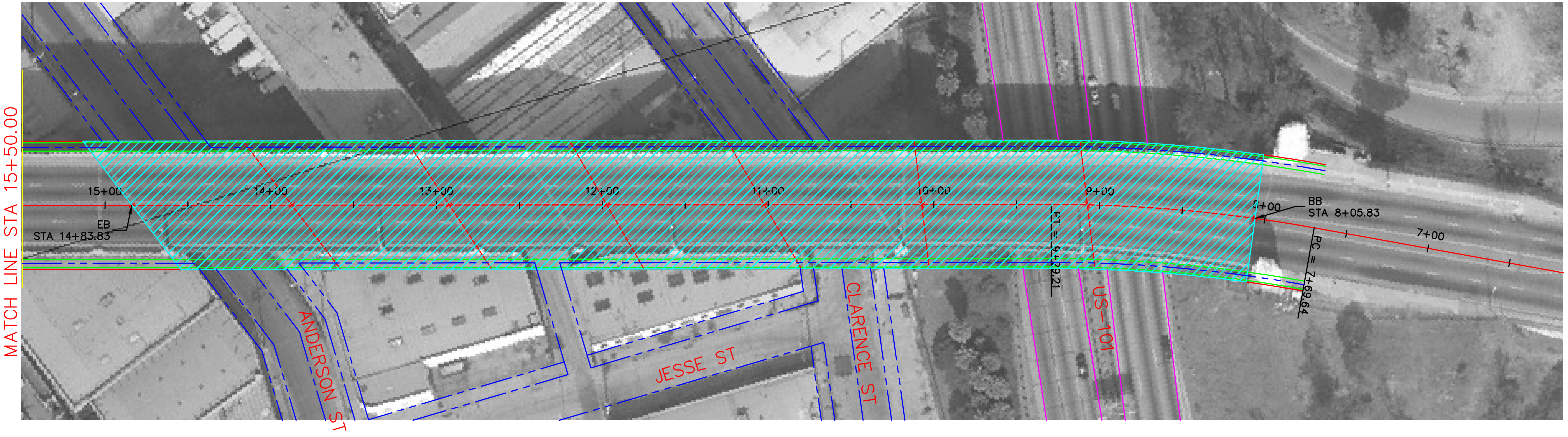
SHEET OF SHEETS

REVISION DATES (DESIGN STAGE ONLY)



DEVELOPED ELEVATION

SCALE 1"=30'



PLAN

SCALE 1"=30'

REVISION DATES (DESIGN STAGE ONLY)

BUREAU OF ENGINEERING ○ STRUCTURAL ENGINEERING DIVISION

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NO.	REVISIONS:	DATE:	BY:

INDEX NO.

DESIGN GROUP

		DATE:
DESIGNED:		
DRAWN:		
CHECKED:	LIC. NO.	
PROJECT MANAGER:	LIC. NO.	
ENGINEER IN RESPONSIBLE CHARGE:	LIC. NO.	

STRUCTURE NO.

SHEET TITLE:

PROJECT: SIXTH STREET VIADUCT OVER LOS ANGELES RIVER SEISMIC RETROFIT PRE-STRATEGY

ADDRESS:

WORK ORDER NO. E230D

DRAWING NO.

SHEET OF SHEETS

APPENDIX C

COST ESTIMATES

6/9/2004

Sixth Street Viaduct over the Los Angeles River

Summary of Seismic Retrofit Alternative Costs

Alternative Description	Infill Wall	Steel Casing + Infill Wall	Catcher Wall	Concrete Casing	Steel Casing + ASR Protection	Replacement In-Kind	Replacement with CIP Box Girder Bridges & Retaining Walls (Revised Alignment)	
Alternative No.	1	2	3	4	5	6A	6B (City of LA)	6B (Caltrans)^{xx}
Construction Cost	$ 15,526,912	$ 27,196,651	$ 29,967,437	$ 45,338,945	$ 44,012,322	$ 45,872,171	$ 33,262,518	$1,482,306
Mobilization Cost (10%)	$ 1,040,000	$ 2,719,665	$ 2,996,744	$ 4,533,894	$ 4,401,232	$ 4,587,217	$ 3,326,252	$ 148,231
Contingencies (15%)*	$ 1,552,691	$ 4,487,447	$ 4,944,627	$ 7,480,926	$ 7,262,033	$ 7,568,908	$ 5,488,315	$ 244,580
Bridge Total	*$ 18,119,603*	*$ 34,403,763*	*$ 37,908,807*	*$ 57,353,765*	*$ 55,675,587*	*$ 58,028,297*	*$ 42,077,085*	*$ 1,875,117*
Bridge Removal	$ 33,088	$ -	$ -	$ -	$ -	$ 4,600,000	$ 4,303,778	$296,222
Traffic Control (5%)**	$ 100,000	$ 1,360,000	$ 1,498,000	$ 2,267,000	$ 2,201,000	$ 4,587,000	$ 3,326,000	$ 148,000
Stage Construction (2%)***	$ 300,000	$ 543,933	$ -	$ 906,779	$ 880,246	$ 2,293,609	$ 1,663,126	$ 74,115
Railroad Work	$ 1,200,000	$ 1,200,000	$ 2,400,000	$ 1,200,000	$ 1,200,000	$ 2,400,000	$ 2,400,000	$ -
Total Construction Costs	*$ 19,752,691*	*$ 37,507,696*	*$ 41,806,807*	*$ 61,727,544*	*$ 59,956,834*	*$ 71,908,905*	*$ 53,769,989*	*$ 2,393,454*
Construction Engineering (15%)****	$ 2,707,731	$ 4,892,308	$ 5,453,062	$ 8,051,419	$ 7,820,457	$ 9,379,422	$ 7,013,477	$ 312,190
Engineering Design and Environmental Clearance (25%)****	$ -	$ 8,153,847	$ 9,088,436	$ 13,419,031	$ 13,034,094	$ 15,632,371	$ 11,689,128	$ 520,316
Right-of-Way	$ -	$ -	$ -	$ -	$ -	$ 1,500,000	$ 1,500,000	$ -
TOTAL PROJECT COST	$ 22,460,422	$ 50,553,851	$ 56,348,306	$ 83,197,994	$ 80,811,385	$ 98,420,698	$ 73,972,594	$ 3,225,960

Notes:

* 10% for Alternative 1.

** Values for Alternative 1 were estimated by the Los Angeles City BOE. Use 10% for Alternatives 6A & 6B.

*** 5% for Alternatives 6A & 6B.

**** Percent of "Total Construction Cost" excluding contingency.

^{xx} State Owened Portion of Bridge 1 Over the US101 Freeway

Alternative 1 - Infill Shear Wall
(From City Estimate - September 6, 2000)

Item Number	Const Cost	Incidental Cost	Incidental Item
1		970,000	mobilization
2		33,088	Bridge removal
3	504,000		
4	330,000		
5	7,672,500		
6	1,058,000		
7	4,000		
8	2,340		
9	1,020		
10	3,380		
11	2,210		
12	2,400		
13	7,900		
14	8,242		
15	600		
16	20,500		
17	4,800		
18	8,450		
19	1,080		
20	3,050,000		
21	18,490		
22	82,000		
23		300,000	Stage Construction
24	170,000		
25		1,200,000	Railroad
26	750,000		
27	550,000		
28		100,000	Traffic Control
29		20,000	mobilization
30	1,000,000		
31		50,000	mobilization
32	125,000		
33	100,000		
34	50,000		
	15,526,912	2,673,088	

SB — JK ✓

CITY ENGINEER'S ESTIMATE
FIRST REVISED ESTIMATE

SIXTH STREET VIADUCT OVER LOS ANGELES RIVER
SEISMIC RETROFIT
W.O. E6000230

BID ITEM NO.	DESCRIPTION	*UNIT	ESTIMATED QUANTITY	UNIT PRICE (in figures) Dollars	Cents	ITEM TOTAL (in figures) Dollars	Cents
1.	Mobilization. Mobilization	LS	▨	▨	▨	970,000	00
2.	Demolition, in part, of ex. bridge and clearing all debris. Bridg Removal	LS	▨	▨	▨	33,088	00
3.	Structural excavation.	cu.m	3,360	150	00	504,000	00
4.	Structural backfill.	cu.m	2,200	150	00	330,000	00
5.	Reinforced conc. incl. reinforcing steel and all necessary form work.	cu.m	5,115	1,500	00	7,672,500	00
6.	Steel rod dowels of varying sizes, lengths and embedments both vertical and horizontal.	LS	▨	▨	▨	1,058,000	00
7.	Railroad ballast removal.	cu.m	40	100	00	4,000	00
8.	Asphalt pvmt. removal.	cu.m	18	130	00	2,340	00
9.	Conc. pvmt. removal:	cu.m	6	170	00	1,020	00
10.	Railroad ballast.	cu.m	26	130	00	3,380	00
11.	Asphalt pvmt.	cu.m	13	170	00	2,210	00
12.	Conc. pvmt.	cu.m	4	600	00	2,400	00
13.	High strength steel rods, 32mm dia. x 6.5m long.	EA	79	100	00	7,900	00
14.	Miscellaneous steel.	kg	3,170	2	60	8,242	00
15.	Neoprene pads - 12mm x 381mm. x 2260 - 6mm.	EA	3	200	00	600	00
16.	Neoprene pads - 25mm x 305mm x 305mm.	EA	41	500	00	20,500	00

CONTINUED ON NEXT PAGE

nit Abbreviations: EA (Each), m (meter), cu.m (cubic meter), sq.m (square meter)
LS (Lump Sum), kg (kilogram)

September 6, 2000

CITY ENGINEER'S ESTIMATE
FIRST REVISED ESTIMATE

SIXTH STREET VIADUCT OVER LOS ANGELES RIVER
SEISMIC RETROFIT
W.O. E6000230

BID ITEM NO.	DESCRIPTION	*UNIT	ESTIMATED QUANTITY	UNIT PRICE (in figures)		ITEM TOTAL (in figures)	
				Dollars	Cents	Dollars	Cents
17.	Neoprene pads - 13mm x 150mm x 150mm.	EA	16	300	00	4,800	00
18.	Roughen ex concrete surface.	sq.m	1,690	5	00	8,450	00
19.	Conc. curb, Type "C".	m	15	72	00	1,080	00
20.	Crack repair.	m	30,500	100	00	3,050,000	00
21.	Remove and replace chain link fence.	m	215	86	00	18,490	00
22.	Remove and reconstruct bridge guard rail.	LS				82,000	00
23.	Street services yard office coordination at Bent No. 1 to be relocated between Bents 3 and 4 incl. all facilities and lay down area.	LS	Stage Construction			300,000	00
24.	Clean surface of conc. columns, girders, wing walls, pylons, piers and railing.	sq.m	17,000	10	00	170,000	00
25.	Allowance for Right-of-Entry Agreement related reimbursements to the various railroad agencies per DWP, US Corp of Engineers & specifications.	LS				Railroad 200,000	00
26.	Street lighting system refurbishment - Plan SL-5580.	LS				750,000	00
27.	Street lighting system refurbishment on 1st St. viaduct over LA River - Plan SL-5581.	LS				550,000	00

CONTINUED ON NEXT PAGE

*Unit Abbreviations: EA (Each), m (meter), cu.m (cubic meter), sq.m (square meter)
LS (Lump Sum), kg (kilogram)

September 6, 2000

CITY ENGINEER'S ESTIMATE
FIRST REVISED ESTIMATE

SIXTH STREET VIADUCT OVER LOS ANGELES RIVER
SEISMIC RETROFIT
W.O. E6000230

BID ITEM NO.	DESCRIPTION	*UNIT	ESTIMATED QUANTITY	UNIT PRICE (in figures)		ITEM TOTAL (in figures)	
				Dollars	Cents	Dollars	Cents
28.	Traffic control requirements incl. signs. [illegible]	LS	▨	▨	▨	100,000	00
29.	Field office Class "B" with air conditioning. Mobilization	LS	▨	▨	▨	20,000	00
30.	Spall repair.	sq.m	1000	1,000	00	1,000,000	00
31.	Internet based project management system. Mobilization	LS	▨	▨	▨	50,000	00
32.	Excavation - hazardous contaminated soil, handling, removal and disposal.	cu.m	500	250	00	125,000	00
33.	Excavation - hazardous non-contaminated soil, handling and removal.	cu.m	500	200	00	100,000	00
34.	Bronze plaques - clean and recondition (4-EA) and new fabrication (1-EA).	LS	▨	▨	▨	50,000	00

TOTAL ESTIMATED CONSTRUCTION COST... $18,200,000.00

1. Item No. 25 is considered a Fixed Cost Item at the time of bid. Contractor shall not change the pre-printed dollar amount on the Schedule of Work and Prices. Any additions, deletions, or provisions by the bidder may render the proposal irregular and may cause its rejection. The actual payments for this item will be adjusted at the end of the project to reflect actual costs expended. No overhead or profit will be paid for this item.
2. **MOBILIZATION shall not exceed $970,000.00**

*Unit Abbreviations: EA (Each), m (meter), cu.m (cubic meter), sq.m (square meter)
LS (Lump Sum), kg (kilogram)

September 6, 2000

Plan No.: D-31273 (Revised)
Liq. Damages: $2,000.00 per calendar day
Completion Time: 550 working days
C.D. 14

Min work to be performed by Contractors own organization:
50% of the contract price after deduction of the
designated "Specialty Items".

Item Nos. 7 thru 12, 19, 21, 22, 26, 27 and 31 are "Specialty Items for Specifications.

DBE = 15%

TG (1992) p. 634, grid H6

Prepared by A/E Consulting Services, Construction Management
Estimating Section: CLC/FEO/ETG/kb

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

BRIDGE GENERAL PLAN ESTIMATE ___X___ OR PLANNING ESTIMATE __________

DPD-DSD-DIS (Rev 8/92)

STRUCTURE: Sixth Street Viaduct Seismic Retrofit
BR. NO.: 53C-1880
RCVD BY:
ESTIMATING GROUP
IN:
OUT:
TYPE: ALTERNATIVE 2
Steel Casing Option
DISTRICT: 7
CO: LA
RTE:
KP:
LENGTH 2,843 x WIDTH Variable = AREA 163,413 FT2

DESIGN SECTION: WKA
1 STRUCTURES
AND $ ________ ROADWORK
QUANTITIES BY: Fahim Hakemi
DATE: June 25, 2003
QUANTITIES CHK BY: Sami Megally
DATE: June 3, 2003
CHG UNIT AND EA:
ESTIMATE NO
PRICED BY
COST INDEX

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT	
Columns	Structure Excavation (Columns)	CY	438	$100.00	$43,793	
	Structure Backfill (Columns)	CY	438	$80.00	$35,035	
	Confinement Plate 5/8"	LBS	1501208	$3.00	$4,503,625	
	Thread Bar 1 3/8"	LBS	330475	$2.00	$660,951	
	Channel MC 8x18.7	LBS	533579	$2.25	$1,200,553	
	Core Concrete (2" dia)	FT	59,273	$35.00	$2,074,548	
	Architectural Treatment	SF	55,515	$30.00	$1,665,456	$10,183,961
Footing	Structural Concrete, Bridge Footing	CY	5,827	$350.00	$2,039,544	
	14" Precast Concrete, Bridge Footing	FT	120,600	$15.00	$1,809,000	
	Drive 14" Piles	EA	2,412	$2,000.00	$4,824,000	
	Structure Excavation (Footing)	CY	9500	$100.00	$950,017	
	Structure Backfill(Footing)	CY	3673	$80.00	$293,832	$9,916,393
Wall	Structural Concrete (Shear Wall)	CY	2905	$800.00	$2,324,369	
	Bar Reinforcing Steel (Shear Wall)	LBS	399271	$0.75	$299,454	$2,623,822
BentCap	Confinement Plate 3/4"	LBS	4427	$3.50	$15,495	
	Thread Bar 1"	LBS	21275	$2.00	$42,550	
	Core Concrete (1 1/2" dia)	FT	7,589	$35.00	$265,608	
	Core Concrete (3" dia)	FT	318	$50.00	$15,903	
	Structural Concrete (Bentcap)	CY	433	$600.00	$259,532	
	Post Tensioning	LS	1	$360,000.00	$360,000	
	Architectural Treatment	SF	2027	$30.00	$60,808	$1,019,896
	Main Span Retrofit	LS	1	$3,452,578.00	$3,452,578	

SUB TOTAL	$27,196,651
MOBILIZATION (10%)	$2,719,665
SUB TOTAL BRIDGE ITEMS	$29,916,316
CONTINGENCIES (15%)	$4,487,447
BRIDGE TOTAL ($ 211 / ft2)	$34,403,763
BRIDGE REMOVAL (CONTINGENCY INCLUDED)	$0
TRAFFIC CONTROL (5%)	$1,360,000
STAGE CONSTRUCTION (2%)	$543,933
WORK BY RAILROAD OR UTILITY FORCES	$1,200,000
GRAND TOTAL	$37,507,696
FOR BUDGET PURPOSES ONLY - SAY	$37,508,000

COMMENTS $ 211 / ft2
BR SF = 163413 SF

FM 91 1416

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

BRIDGE GENERAL PLAN ESTIMATE ___X____ OR PLANNING ESTIMATE __________

DPD-DSD-DIS (Rev 8/92)

STRUCTURE: Sixth Street Viaduct Seismic Retrofit
BR. NO.: 53C-1880
RCVD BY:
ESTIMATING GROUP
IN:
OUT:
TYPE: ALTERNATIVE 3
Catcher Wall System
DISTRICT: 7
CO: LA
RTE:
KP:
LENGTH 2843 x WIDTH Variable = AREA 163413 FT2

DESIGN SECTION: WKA
1 STRUCTURES
AND $ ________ ROADWORK
QUANTITIES BY: D. Weddell
DATE: June 25, 2003
QUANTITIES CHK B: Sami Megally
DATE: June 3, 2003
CHG UNIT AND EA:
ESTIMATE NO
PRICED BY
COST INDEX

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT	
Catcher Walls	Structure Excavation	CY	11829	$100.00	$1,182,870	
	Structure Backfill	CY	4642	$80.00	$371,381	
	Structural Concrete (Shear Wall)	CY	15269	$800.00	$12,215,416	
	Structural Concrete (Footing)	CY	5791	$400.00	$2,316,533	
	14" Precast Concrete, Bridge Footing	FT	120450	$15.00	$1,806,750	
	Drive 14" Piles	EA	2409	$2,000.00	$4,818,000	
	Architectural Treatment	SF	190195	$20.00	$3,803,908	$26,514,859
	Main Span Retrofit	LS	1	$3,452,578.00	$3,452,578	

SUB TOTAL	$29,967,437
MOBILIZATION (10%)	$2,996,744
SUB TOTAL BRIDGE ITEMS	$32,964,180
CONTINGENCIES (15%)	$4,944,627
BRIDGE TOTAL ($ 232 / ft2)	$37,908,807
BRIDGE REMOVAL (CONTINGENCY INCLUDED)	$0
TRAFFIC CONTROL (5%)	$1,498,000
STAGE CONSTRUCTION (0%)	$0
WORK BY RAILROAD OR UTILITY FORCES	$2,400,000
GRAND TOTAL	$41,806,807
FOR BUDGET PURPOSES ONLY - SAY	$41,807,000

COMMENTS $ 232 / ft2
BR SF = 163413 SF

FM 91 1416

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

BRIDGE GENERAL PLAN ESTIMATE __X__ OR PLANNING ESTIMATE ______

DPD-DSD-DIS (Rev 8/92)

STRUCTURE Sixth Street Viaduct Seismic Retrofit
BR. NO. 53C-1880
RCVD BY
ESTIMATING GROUP
IN
OUT
TYPE ALTERNATIVE 4
Concrete Column Casing
DISTRICT 7
CO LA
RTE
KP
LENGTH 2843 x WIDTH Variable = AREA 163413 FT2

DESIGN SECTION WKA
1 STRUCTURES
AND $ ______ ROADWORK
QUANTITIES BY D. Weddell
DATE June 25, 2003
QUANTITIES CHK BY Sami Megally
DATE June 3, 2003
CHG UNIT AND EA
ESTIMATE NO
PRICED BY
COST INDEX

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT	
Columns & Foundations	Structure Excavation	CY	65653	$100.00	$6,565,319	
	Structure Backfill	CY	46923	$80.00	$3,753,847	
	Structural Concrete (Column)	CY	8160	$800.00	$6,527,771	
	Reinforcement (Column)	LBS	4318121	$0.75	$3,238,590	
	Structural Concrete (Footing)	CY	18730	$400.00	$7,492,045	
	Reinforcement (Footing)	LBS	2477994	$0.75	$1,858,495	
	14" Precast Concrete, Bridge Footing	FT	159600	$15.00	$2,394,000	
	Drive 14" Piles	EA	3192	$2,000.00	$6,384,000	$38,214,068
BentCap	Core Concrete (1" dia)	FT	18,060	$35.00	$632,100	
	Core Concrete (3" dia)	FT	3,067	$50.00	$153,351	
	Structural Concrete (Bentcap)	CY	3614	$600.00	$2,168,270	
	Reinforcement	LBS	478104	$0.75	$358,578	
	Post Tension	LS	1	$360,000.00	$360,000	$3,672,299
	Main Span Retrofit	LS	1	$3,452,578.00	$3,452,578	

SUB TOTAL	$45,338,945
MOBILIZATION (10%)	$4,533,894
SUB TOTAL BRIDGE ITEMS	$49,872,839
CONTINGENCIES (15%)	$7,480,926
BRIDGE TOTAL ($ 351 / ft2)	$57,353,765
BRIDGE REMOVAL (CONTINGENCY INCLUDED)	$0
TRAFFIC CONTROL (5%)	$2,267,000
STAGE CONSTRUCTION (2%)	$906,779
WORK BY RAILROAD OR UTILITY FORCES	$1,200,000
GRAND TOTAL	$61,727,544
FOR BUDGET PURPOSES ONLY - SAY	$61,728,000

COMMENTS $ 351 / ft2
BR SF = 163413 SF

FM 91 1416

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

BRIDGE GENERAL PLAN ESTIMATE ___X____ OR PLANNING ESTIMATE __________

DPD-DSD-DIS (Rev 8/92)

STRUCTURE: Sixth Street Viaduct Seismic Retrofit
BR. NO.: 53C-1880
RCVD BY:
ESTIMATING GROUP
IN:
OUT:
TYPE: ALTERNATIVE 5
Steel Casing Option
DISTRICT: 7
CO: LA
RTE:
KP:
LENGTH 2843 x WIDTH 57.4790714 = AREA 163413 FT2
Average Width
DESIGN SECTION: WKA
1 STRUCTURES
AND $ ________ ROADWORK
QUANTITIES BY: Fahim Hakemi DATE: June 25, 2003
QUANTITIES CHK BY: Sami Megally DATE: June 3, 2004
CHG UNIT AND EA:
ESTIMATE NO
PRICED BY
COST INDEX

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT	
Columns	Structure Excavation (Columns)	CY	4518	$100.00	$451,801	
	Structure Backfill (Columns)	CY	4518	$80.00	$361,441	
	Confinement Plate 7/8"	LBS	1939514	$3.00	$5,818,542	
	Confinement Plate 5/8"	LBS	2119525	$3.00	$6,358,574	
	Thread Bar 1 3/8"	LBS	767379	$2.00	$1,534,759	
	Channel MC 8x18.7	LBS	1222700	$2.25	$2,751,075	
	Core Concrete (2" dia)	FT	137,634	$35.00	$4,817,197	
	Architectural Treatment	SF	111,687	$30.00	$3,350,623	$25,444,013
Footing	Structural Concrete, Bridge Footing	CY	5,827	$350.00	$2,039,544	
	14" Precast Concrete, Bridge Footing	FT	120,600	$15.00	$1,809,000	
	Drive 14" Piles	EA	2,412	$2,000.00	$4,824,000	
	Structure Excavation (Footing)	CY	9474	$100.00	$947,381	
	Structure Backfill(Footing)	CY	3647	$80.00	$291,723	$9,911,649
Wall	Structural Concrete (Shear Wall)	CY	2905	$800.00	$2,324,369	
	Bar Reinforcing Steel (Shear Wall)	LBS	399271	$0.75	$299,454	$2,623,822
BentCap	Confinement Plate 3/4"	LBS	10369	$3.50	$36,293	
	Thread Bar 1"	LBS	53431	$2.00	$106,863	
	Core Concrete (1 1/2" dia)	FT	19059	$35.00	$667,058	
	Core Concrete (3" dia)	FT	1407	$50.00	$70,329	
	Structural Concrete (Bentcap)	CY	1800	$600.00	$1,079,726	
	Post Tensioning	LS	1	$360,000.00	$360,000	
	Architectural Treatment	SF	7969	$30.00	$239,070	$2,559,338
	Main Span Retrofit	LS	1	$3,452,578.00	$3,452,578	$3,452,578
519081	JOINT SEAL (MR 1/2")	LF	805	$26.00	$20,922	$20,922

SUB TOTAL	$44,012,322
MOBILIZATION (10%)	$4,401,232
SUB TOTAL BRIDGE ITEMS	$48,413,554
CONTINGENCIES (15%)	$7,262,033
BRIDGE TOTAL ($ 341 / ft2)	$55,675,587
BRIDGE REMOVAL (CONTINGENCY INCLUDED)	$0
TRAFFIC CONTROL (5%)	$2,201,000
STAGE CONSTRUCTION (2%)	$880,246
WORK BY RAILROAD OR UTILITY FORCES	$1,200,000
GRAND TOTAL	$59,956,834
FOR BUDGET PURPOSES ONLY - SAY	$59,957,000

COMMENTS $ 341 / ft2

BR SF = 163413 SF

FM 91 1416

☐ **GENERAL PLAN ESTIMATE** ☒ **PLANNING ESTIMATE**

BRIDGE **6TH STREET VIADUCT - Arch Spans**		BR NO **1**		REC'D BY
TYPE		DIST	CO RS	PROJECT
LENGTH **304**	X WIDTH **67.75**	= AREA **20,596**		

DESIGN SECTION	**DH ENG.**	QUANTITIES BY	**RKD**	DATE	**7/1**	ESTIMATE NO 1
PROJECT INCLUDES	CULVERT	QUANT. CHECKED BY		DATE		PRICE BY
		CHG UNIT AND EA				COST INDEX

NO.	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT
1					
2					
3	STRUCTURAL CONCRETE, BRIDGE	CY	544	$500.00	$272,000
4	STRUCTURAL CONCRETE, BRIDGE FOOTING	CY	2443	$350.00	$855,050
5	BAR REINFORCING STEEL (BRIDGE)	LB	596600	$0.50	$298,300
6	DRILL AND BOND	LF	8000	$75.00	$600,000
7	STRUCTURE EXCAVATION (BRIDGE)	CY	8333	$100.00	$833,300
8	STRUCTURE BACKFILL (BRIDGE)	CY	5865	$60.00	$351,900
9	16" CIDH CONCRETE PILING	LF	6480	$37.35	$242,028
10					
11					
12					
13					
14					
15					
16					
17					
18					

SUB TOTAL		$3,452,578
MOBILIZATION	**10%**	$345,258
SUB TOTAL - BRIDGE ITEMS		$3,797,836
DOLLAR PER SQ. FOOT	**184**	
CONTINGENCIES @	**15%**	$569,675
SOFT COSTS	**20%**	$873,502
TOTAL		$5,241,013
FOR BUDGET PURPOSES - SAY		**$5,245,000**

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

BRIDGE GENERAL PLAN ESTIMATE ___X____ OR PLANNING ESTIMATE __________

DPD-DSD-DIS (Rev 8/92)

STRUCTURE Sixth Street Viaduct Seismic Retrofit
BR. NO. 53C-1880
RCVD BY
ESTIMATING GROUP IN OUT
TYPE ALTERNATIVE 6A
Total In-Kind Replacement
DISTRICT 7 CO LA RTE KP
LENGTH 2938 x WIDTH 77.5 = AREA 227657 FT2
Average Width
DESIGN SECTION WKA
QUANTITIES BY Wei Koo DATE June 25, 2003
ESTIMATE NO
1 STRUCTURES
QUANTITIES CHK BY Sami Megally DATE November 25, 2003
PRICED BY
AND $ ______ ROADWORK
CHG UNIT AND EA ______
COST INDEX

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT	
Superstructure	Girders	CY	8866	$900.00	$7,979,108	
	Bent Caps	CY	3911	$600.00	$2,346,528	
	Concrete Hinges	CY	504	$600.00	$302,400	
	JOINT SEAL (MR 1/2")	FT	465	$26.00	$12,090	
	Deck Slab	CY	5846	$800.00	$4,676,619	
	Sidewalks & Barriers	FT	5900	$1,500.00	$8,850,000	
						$24,166,745
Substructure	Abutments	CY	523	$800.00	$418,400	
	Columns	CY	4,193	$600.00	$2,515,903	
	Precast Architectural Column Casing	CY	2,098	$700.00	$1,468,652	
						$4,402,954
Foundation	Structure Excavation	CY	11,560	$100.00	$1,156,000	
	Structure Fill	CY	5,005	$80.00	$400,383	
	Pile Caps	CY	6,111	$350.00	$2,138,889	
	14" Precast Concrete Piles	FT	167680	$15.00	$2,515,200	
	Drive 14" Piles	EA	2096	$2,000.00	$4,192,000	
	Shoring Allowance	LS			$1,500,000	$11,902,472
Incidentals	Street Lights	EA	80	$10,000.00	$800,000	
	Utilities	LS			$500,000	
	Approach Works	LS			$600,000	
	Site Improvement Allowance	LS			$1,500,000	
	LA River Improvement	LS			$500,000	
	Reset Existing Arch Ribs and Obelisks	LS			$1,500,000	
						$5,400,000

SUB TOTAL	$45,872,171
MOBILIZATION (10%)	$4,587,217
SUB TOTAL BRIDGE ITEMS	$50,459,388
CONTINGENCIES (15%)	$7,568,908
BRIDGE TOTAL ($ 255 / ft2)	$58,028,297
BRIDGE REMOVAL (CONTINGENCY INCLUDED)	$4,600,000
TRAFFIC CONTROL (10%)	$4,587,000
STAGE CONSTRUCTION (5%)	$2,293,609
RIGHT OF WAY	$1,500,000
WORK BY RAILROAD OR UTILITY FORCES	$2,400,000
GRAND TOTAL	$73,408,905
FOR BUDGET PURPOSES ONLY - SAY	$73,409,000

COMMENTS $ 255 / ft2

BR SF = 227657 SF

FM 91 1416

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

BRIDGE GENERAL PLAN ESTIMATE ___X____ OR PLANNING ESTIMATE __________

DPD-DSD-DIS (Rev 8/92)

STRUCTURE: Sixth Street Viaduct Seismic Retrofit

BR. NO.: 53C-1880

RCVD BY:

ESTIMATING GROUP

IN:

OUT:

TYPE: ALTERNATIVE 6B
Replacement with CIP Bridges & Retaining Walls

DISTRICT: 7 CO: LA RTE: KP:

LENGTH 3494 x WIDTH 77.5 = AREA 270785 FT2
Average Width

DESIGN SECTION: WKA
1 STRUCTURES
AND $ ______________ ROADWORK

QUANTITIES BY: A. Moubayed DATE: January 28, 2004
QUANTITIES CHK B: Sami Megally DATE: June 4, 2004
CHG UNIT AND EA: ________

ESTIMATE NO
PRICED BY
COST INDEX

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT	
CIP Bridges	Bridge No. 1	LS			$4,466,682	
	Bridge No. 2	LS			$1,074,981	
	Bridge No. 3	LS			$9,926,692	
	Bridge No. 4	LS			$993,393	
						$16,461,748
Retained Backfill	Retaining Walls with Pile Foundations	LS			$10,865,965	
	Soil Backfill	CY	87,237	$30.00	$2,617,111	
						$13,483,076
Incidentals	Street Lights	EA	80	$10,000.00	$800,000	
	Utilities	LS			$500,000	
	Site Improvement Allowance	LS			$1,500,000	
	LA River Improvement	LS			$500,000	
	Reset Existing Arch Ribs and Obelisks	LS			$1,500,000	$4,800,000

SUB TOTAL	$34,744,824
MOBILIZATION (10%)	$3,474,482
SUB TOTAL BRIDGE ITEMS	$38,219,306
CONTINGENCIES (15%)	$5,732,896
BRIDGE TOTAL ($ 162 / ft2)	$43,952,202
BRIDGE REMOVAL (CONTINGENCY INCLUDED)	$4,600,000
TRAFFIC CONTROL (10%)	$3,474,000
STAGE CONSTRUCTION (5%)	$1,737,241
RIGHT OF WAY	$1,500,000
WORK BY RAILROAD OR UTILITY FORCES	$2,400,000
GRAND TOTAL	$57,663,444
FOR BUDGET PURPOSES ONLY - SAY	$57,663,000

COMMENTS $ 162 / ft2

BR SF = 270785 SF

FM 91 1416

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

BRIDGE GENERAL PLAN ESTIMATE ____ OR PLANNING ESTIMATE X

DPD-DSD-DIS (Rev 8/92)

STRUCTURE Sixth Street Viaduct Seismic Retrofit

BR. NO.

RCVD BY

ESTIMATING GROUP

IN

OUT

TYPE ALTERNATIVE 6B (Bridge No. 1)
Over Fwy

DISTRICT CO RTE KP

LENGTH 678.00 x WIDTH 77.5 = AREA 52545.00 FT2

DESIGN SECTION WKA

1 STRUCTURES

AND $ ______ ROADWORK

QUANTITIES BY A. Moubayec DATE 2/2/2004

QUANTITIES CHK BY ______ DATE ______

CHG UNIT AND EA ______

ESTIMATE NO

PRICED BY

COST INDEX

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT	
192003	STRUCTURE EXCAVATION (BRIDGE)	CY	2640	$100.00	$264,000	X
192037	STRUCTURE EXCAVATION (RETAINING WALL)	CY	0	$20.00	$0	X
193003	STRUCTURE BACKFILL (BRIDGE)	CY	2138	$30.00	$64,134	X
193013	STRUCTURE BACKFILL (RETAINING WALL)	CY	0	$30.00	$0	X
490505	FURNISH STEEL PILING	FT	44960	$10.00	$449,600	X
490506	DRIVE STEEL PILING	EA	562	$960.00	$539,520	X
490656	450 MM CAST-IN-DRILLED HOLE CONCRETE PILING	FT	0	$110.00	$0	
500001	PRESRESSING CAST-IN-PLACE CONCRETE	LB	105090	$2.00	$210,180	X
510053	STRUCTURAL CONCRETE (BRIDGE)	CY	5018	$410.00	$2,057,331	X
510060	STRUCTURAL CONCRETE (RETAINING WALL)	CY	0	$270.00	$0	
510086	STRUCTURAL CONCRETE (APPROACH SLAB)	CY	172	$315.00	$54,250	X
519117	JOINT SEAL ASSEMBLY (MR = 30 MM)	FT	155	$35.00	$5,425	X
520102	BAR REINFORCING STEEL (BRIDGE)	LB	542914	$0.60	$325,748	X
520103	BAR REINFORCING STEEL (RETAINING WALL)	LB	0	$0.60	$0	X
512004	FURNISH PRECAST PRESTRESS CONCRETE GIRDER	EA	0	$0.00	$0	
512500	ERECT PRECAST PRESTRESS CONCRETE GIRDER	EA	0	$0.00	$0	
560203	FURNISH SIGN STRUCTURE	LB	0	$0.00	$0	
560204	INSTALL SIGN STRUCTURE	LB	0	$0.00	$0	
590115	CLEAN AND PAINT STRUCTURAL STEEL	LS	0	$0.00	$0	
721810	SLOPE PAVING	CY	202	$375.00	$75,833	
511035	ARCHITECTURAL TREATMENT	FT2	4,428	$35.00	$154,980	
833032	CHAIN LINK RAILING (TYPE 7) (Mod)	FT	1,476	$60.00	$88,560	X
833088	TUBULAR HAND RAILING	FT	0	$40.00	$0	
833125	CONCRETE BARRIER (TYPE 26) (MOD)	FT	1,476	$120.00	$177,120	X
839709	CONCRETE BARRIER (TYPE 60GE)	FT	0	$205.00	$0	
839704	CONCRETE BARRIER (TYPE 60D)	FT	0	$40.00	$0	

SUB TOTAL		$4,466,682
MOBILIZATION (10%)		$446,668
SUB TOTAL BRIDGE ITEMS		$4,913,350
CONTINGENCIES (15%)		$737,003
BRIDGE TOTAL	= $107.53 /SF	$5,650,353
BRIDGE REMOVAL (CONTINGENCY INCLUDED)		$892,616
TRAFFIC CONTROL (10%)		$447,000
STAGE CONSTRUCTION (5%)		$223,334.00
WORK BY RAILROAD OR UTILITY FORCES		$0.00
GRAND TOTAL		$7,213,303
FOR BUDGET PURPOSES ONLY - SAY		$7,213,000
COMMENTS	= $108 /SF	

BR SF = 52545 SF

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

BRIDGE GENERAL PLAN ESTIMATE ____ OR **PLANNING ESTIMATE** X

DPD-DSD-DIS (Rev 8/92)

STRUCTURE Sixth Street Viaduct Seismic Retrofit

BR. NO.

RCVD BY

ESTIMATING GROUP IN OUT

TYPE ALTERNATIVE 6B (Bridge No. 2) Over Mission Rd.

DISTRICT CO RTE KP

LENGTH 103.00 x WIDTH 77.5 = AREA 7982.50 FT2

DESIGN SECTION WKA

1 STRUCTURES

AND $ ROADWORK

QUANTITIES BY A. Moubayed DATE 2/2/2004

QUANTITIES CHK BY DATE

CHG UNIT AND EA

ESTIMATE NO

PRICED BY

COST INDEX

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT	
192003	STRUCTURE EXCAVATION (BRIDGE)	CY	783	$100.00	$78,322	X
192037	STRUCTURE EXCAVATION (RETAINING WALL)	CY	0	$20.00	$0	X
193003	STRUCTURE BACKFILL (BRIDGE)	CY	1402	$30.00	$42,050	X
193013	STRUCTURE BACKFILL (RETAINING WALL)	CY	0	$30.00	$0	X
490505	FURNISH STEEL PILING	FT	9600	$10.00	$96,000	X
490506	DRIVE STEEL PILING	EA	120	$960.00	$115,200	X
490656	450 MM CAST-IN-DRILLED HOLE CONCRETE PILING	FT	0	$110.00	$0	
500001	PRESRESSING CAST-IN-PLACE CONCRETE	LB	23948	$2.00	$47,895	X
510053	STRUCTURAL CONCRETE (BRIDGE)	CY	1129	$410.00	$462,742	X
510060	STRUCTURAL CONCRETE (RETAINING WALL)	CY	0	$270.00	$0	
510086	STRUCTURAL CONCRETE (APPROACH SLAB)	CY	172	$315.00	$54,250	X
519117	JOINT SEAL ASSEMBLY (MR = 30 MM)	FT	155	$35.00	$5,425	X
520102	BAR REINFORCING STEEL (BRIDGE)	LB	133644	$0.60	$80,187	X
520103	BAR REINFORCING STEEL (RETAINING WALL)	LB	0	$0.60	$0	X
512004	FURNISH PRECAST PRESTRESS CONCRETE GIRDER	EA	0	$0.00	$0	
512500	ERECT PRECAST PRESTRESS CONCRETE GIRDER	EA	0	$0.00	$0	
560203	FURNISH SIGN STRUCTURE	LB	0	$0.00	$0	
560204	INSTALL SIGN STRUCTURE	LB	0	$0.00	$0	
590115	CLEAN AND PAINT STRUCTURAL STEEL	LS	0	$0.00	$0	
721810	SLOPE PAVING	CY	0	$375.00	$0	
511035	ARCHITECTURAL TREATMENT	FT2	978	$35.00	$34,230	
833032	CHAIN LINK RAILING (TYPE 7) (Mod)	FT	326	$60.00	$19,560	X
833088	TUBULAR HAND RAILING	FT	0	$40.00	$0	
833125	CONCRETE BARRIER (TYPE 26) (MOD)	FT	326	$120.00	$39,120	X
839709	CONCRETE BARRIER (TYPE 60GE)	FT	0	$205.00	$0	
839704	CONCRETE BARRIER (TYPE 60D)	FT	0	$40.00	$0	

SUB TOTAL	$1,074,981
MOBILIZATION (10%)	$107,498
SUB TOTAL BRIDGE ITEMS	$1,182,479
CONTINGENCIES (15%)	$177,372
BRIDGE TOTAL = $170.35 /SF	$1,359,851
BRIDGE REMOVAL (CONTINGENCY INCLUDED)	$135,604
TRAFFIC CONTROL (10%)	$107,000
STAGE CONSTRUCTION (5%)	$53,749.00
WORK BY RAILROAD OR UTILITY FORCES	$0.00
GRAND TOTAL	$1,656,204
FOR BUDGET PURPOSES ONLY - SAY	$1,656,000
COMMENTS = $170 /SF	

BR SF = 7983 SF

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

BRIDGE GENERAL PLAN ESTIMATE ____ OR PLANNING ESTIMATE X

DPD-DSD-DIS (Rev 8/92)

STRUCTURE: Sixth Street Viaduct Seismic Retrofit

BR. NO.

RCVD BY

ESTIMATING GROUP

IN

OUT

TYPE: ALTERNATIVE 6B (Bridge No. 3)
Over LA River

DISTRICT CO RTE KP

LENGTH 763.06 x WIDTH 77.5 = AREA 59137.15 FT2

DESIGN SECTION WKA

1 STRUCTURES

AND $ ____ ROADWORK

QUANTITIES BY A. Moubayec

DATE 6/9/2004

QUANTITIES CHK BY ____

DATE ____

CHG UNIT AND EA ____

ESTIMATE NO

PRICED BY

COST INDEX

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT
192003	STRUCTURE EXCAVATION (BRIDGE)	CY	4974	$100.00	$497,393
192037	STRUCTURE EXCAVATION (RETAINING WALL)	CY	0	$20.00	$0
193003	STRUCTURE BACKFILL (BRIDGE)	CY	3345	$30.00	$100,363
193013	STRUCTURE BACKFILL (RETAINING WALL)	CY	0	$30.00	$0
490505	FURNISH STEEL PILING	FT	37920	$10.00	$379,200
490506	DRIVE STEEL PILING	EA	474	$960.00	$455,040
490656	450 MM CAST-IN-DRILLED HOLE CONCRETE PILING	FT	0	$110.00	$0
500001	PRESRESSING CAST-IN-PLACE CONCRETE	LB	177411	$2.00	$354,823
510053	STRUCTURAL CONCRETE (BRIDGE)	CY	10951	$600.00	$6,570,772
510060	STRUCTURAL CONCRETE (RETAINING WALL)	CY	0	$270.00	$0
510086	STRUCTURAL CONCRETE (APPROACH SLAB)	CY	172	$315.00	$54,250
519117	JOINT SEAL ASSEMBLY (MR = 30 MM)	FT	155	$35.00	$5,425
520102	BAR REINFORCING STEEL (BRIDGE)	LB	1733803	$0.60	$1,040,282
520103	BAR REINFORCING STEEL (RETAINING WALL)	LB	0	$0.60	$0
512004	FURNISH PRECAST PRESTRESS CONCRETE GIRDER	EA	0	$0.00	$0
512500	ERECT PRECAST PRESTRESS CONCRETE GIRDER	EA	0	$0.00	$0
560203	FURNISH SIGN STRUCTURE	LB	0	$0.00	$0
560204	INSTALL SIGN STRUCTURE	LB	0	$0.00	$0
590115	CLEAN AND PAINT STRUCTURAL STEEL	LS	0	$0.00	$0
721810	SLOPE PAVING	CY	0	$375.00	$0
511035	ARCHITECTURAL TREATMENT	FT2	4,938	$35.00	$172,843
833032	CHAIN LINK RAILING (TYPE 7) (Mod)	FT	1,646	$60.00	$98,767
833088	TUBULAR HAND RAILING	FT	0	$40.00	$0
833125	CONCRETE BARRIER (TYPE 26) (MOD)	FT	1,646	$120.00	$197,534
839709	CONCRETE BARRIER (TYPE 60GE)	FT	0	$205.00	$0

SUB TOTAL		$9,926,692
MOBILIZATION (10%)		$992,669
SUB TOTAL BRIDGE ITEMS		$10,919,361
CONTINGENCIES (15%)		$1,637,904
BRIDGE TOTAL	= $212.34 /SF	$12,557,265
BRIDGE REMOVAL (CONTINGENCY INCLUDED)		$1,004,522
TRAFFIC CONTROL (10%)		$993,000
STAGE CONSTRUCTION (5%)		$496,335.00
WORK BY RAILROAD OR UTILITY FORCES		$2,400,000.00
GRAND TOTAL		$17,451,122
FOR BUDGET PURPOSES ONLY - SAY		$17,451,000
COMMENTS	= $212 /SF	

BR SF = 59137 SF

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

BRIDGE GENERAL PLAN ESTIMATE ____ OR PLANNING ESTIMATE X

DPD-DSD-DIS (Rev 8/92)

STRUCTURE: Sixth Street Viaduct Seismic Retrofit
BR. NO.
RCVD BY
ESTIMATING GROUP IN
OUT
TYPE: ALTERNATIVE 6B (Bridge No. 4)
Over SanataFe Ave.
DISTRICT
CO
RTE
KP
LENGTH 104.00 x WIDTH 77.5 = AREA 8060.00 FT2

DESIGN SECTION WKA
1 STRUCTURES
AND $ ______ ROADWORK
QUANTITIES BY A. Moubayec DATE 2/2/2004
QUANTITIES CHK BY ______ DATE ______
CHG UNIT AND EA ______
ESTIMATE NO
PRICED BY
COST INDEX

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT	
192003	STRUCTURE EXCAVATION (BRIDGE)	CY	701	$100.00	$70,078	X
192037	STRUCTURE EXCAVATION (RETAINING WALL)	CY	0	$20.00	$0	X
193003	STRUCTURE BACKFILL (BRIDGE)	CY	1023	$30.00	$30,678	X
193013	STRUCTURE BACKFILL (RETAINING WALL)	CY	0	$30.00	$0	X
490505	FURNISH STEEL PILING	FT	9600	$10.00	$96,000	X
490506	DRIVE STEEL PILING	EA	120	$960.00	$115,200	X
490656	450 MM CAST-IN-DRILLED HOLE CONCRETE PILING	FT	0	$110.00	$0	
500001	PRESRESSING CAST-IN-PLACE CONCRETE	LB	24180	$2.00	$48,360	X
510053	STRUCTURAL CONCRETE (BRIDGE)	CY	1007	$410.00	$412,962	X
510060	STRUCTURAL CONCRETE (RETAINING WALL)	CY	0	$270.00	$0	
510086	STRUCTURAL CONCRETE (APPROACH SLAB)	CY	172	$315.00	$54,250	X
519117	JOINT SEAL ASSEMBLY (MR = 30 MM)	FT	155	$35.00	$5,425	X
520102	BAR REINFORCING STEEL (BRIDGE)	LB	111600	$0.60	$66,960	X
520103	BAR REINFORCING STEEL (RETAINING WALL)	LB	0	$0.60	$0	X
512004	FURNISH PRECAST PRESTRESS CONCRETE GIRDER	EA	0	$0.00	$0	
512500	ERECT PRECAST PRESTRESS CONCRETE GIRDER	EA	0	$0.00	$0	
560203	FURNISH SIGN STRUCTURE	LB	0	$0.00	$0	
560204	INSTALL SIGN STRUCTURE	LB	0	$0.00	$0	
590115	CLEAN AND PAINT STRUCTURAL STEEL	LS	0	$0.00	$0	
721810	SLOPE PAVING	CY	0	$375.00	$0	
511035	ARCHITECTURAL TREATMENT	FT2	984	$35.00	$34,440	
833032	CHAIN LINK RAILING (TYPE 7) (Mod)	FT	328	$60.00	$19,680	X
833088	TUBULAR HAND RAILING	FT	0	$40.00	$0	
833125	CONCRETE BARRIER (TYPE 26) (MOD)	FT	328	$120.00	$39,360	X
839709	CONCRETE BARRIER (TYPE 60GE)	FT	0	$205.00	$0	
839704	CONCRETE BARRIER (TYPE 60D)	FT	0	$40.00	$0	

SUB TOTAL		$993,393
MOBILIZATION (10%)		$99,339
SUB TOTAL BRIDGE ITEMS		$1,092,732
CONTINGENCIES (15%)		$163,910
BRIDGE TOTAL	= $155.91 /SF	$1,256,642
BRIDGE REMOVAL (CONTINGENCY INCLUDED)		$136,920
TRAFFIC CONTROL (10%)		$99,000
STAGE CONSTRUCTION (5%)		$49,670.00
WORK BY RAILROAD OR UTILITY FORCES		$0.00
GRAND TOTAL		$1,542,232
FOR BUDGET PURPOSES ONLY - SAY		$1,542,000
COMMENTS	= $156 /SF	

BR SF = 8060 SF

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

BRIDGE GENERAL PLAN ESTIMATE ____ OR PLANNING ESTIMATE X

DPD-DSD-DIS (Rev 8/92)

STRUCTURE Sixth Street Viaduct Seismic Retrofit
BR. NO.
RCVD BY
ESTIMATING GROUP IN OUT
TYPE ALTERNATIVE 6B (Retaining Walls & Soil Backfill)
DISTRICT CO RTE KP
LENGTH 1846.00 x WIDTH 77.5 = AREA 143065.00 FT2

DESIGN SECTION WKA
1 STRUCTURES
AND $ ROADWORK
QUANTITIES BY A. Moubaye
DATE 6/9/2004
QUANTITIES CHK BY
DATE
CHG UNIT AND EA
ESTIMATE NO
PRICED BY
COST INDEX

CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT
Structure Excavation	CY	11,115	$100.00	$1,111,461
Structure Backfill	CY	34,719	$30.00	$1,041,561
Structral Concrete	CY	11,708	$270.00	$3,161,232
Bar Reinforcing Steel	LB	1,823,352	$0.60	$1,094,011
Furnish Piles (Class 400C)	FT	114,300	$15.00	$1,714,500
Drive Piles	EA	2,286	$1,200.00	$2,743,200
Soil Backfill	CY	87237	$30.00	$2,617,111

SUB TOTAL	$13,483,076
MOBILIZATION (10%)	$1,348,308
SUB TOTAL BRIDGE ITEMS	$14,831,384
CONTINGENCIES (15%)	$2,224,708
BRIDGE TOTAL = $119.22 /SF	$17,056,091
BRIDGE REMOVAL (CONTINGENCY INCLUDED)	$2,430,338
TRAFFIC CONTROL (10%)	$1,348,000
STAGE CONSTRUCTION (5%)	$674,154.00
WORK BY RAILROAD OR UTILITY FORCES	$0.00
GRAND TOTAL	$21,508,583
FOR BUDGET PURPOSES ONLY - SAY	$21,509,000
COMMENTS = $119 /SF	

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

BRIDGE GENERAL PLAN ESTIMATE ___X____ OR PLANNING ESTIMATE __________

DPD-DSD-DIS (Rev 8/92)

STRUCTURE Sixth Street Viaduct Seismic Retrofit	BR. NO. 53C-1880	RCVD BY	ESTIMATING GROUP
TYPE ALTERNATIVE 5 (Rehabilitation Only) Steel Casing Option	DISTRICT 7 CO LA RTE KP		IN OUT

LENGTH 2843 x WIDTH 57.4790714 = AREA 163413 FT2

Average Width

DESIGN SECTION WKA

1 STRUCTURES

AND $ ________ ROADWORK

QUANTITIES BY Sami Megally DATE June 2, 2003

QUANTITIES CHK BY ________ DATE ________

CHG UNIT AND EA ________

ESTIMATE NO

PRICED BY

COST INDEX

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT	
Crack Repair	Epoxy Repair (Columns)	LF	9,094	$25.00	$227,345	
	Epoxy Repair (Bent Caps)	LF	19,040	$25.00	$475,991	
	Epoxy Repair (Superstructure Girders)	LF	45,130	$25.00	$1,128,243	
	Epoxy Repair (Deck Slab)	LF	28,724	$25.00	$718,103	$2,549,681
	Barrier Repair	LF	569	$950.00	$540,170	$540,170
153101	Plane Asphault Concrete	Sq.Yd	18,157	$7.82	$141,988	
515041	FURNISH POLYESTER CONCRETE OVERLAY	CY	1,009	$1,400.00	$1,412,211	
515042	PLACE POLYESTER CONCRETE OVERLAY	SF	163,413	$3.00	$490,239	$2,044,438

SUB TOTAL	$5,134,289
MOBILIZATION (10%)	$513,429
SUB TOTAL BRIDGE ITEMS	$5,647,718
CONTINGENCIES (15%)	$847,158
BRIDGE TOTAL ($ 40 / ft2)	$6,494,875
BRIDGE REMOVAL (CONTINGENCY INCLUDED)	$0
TRAFFIC CONTROL (5%)	$257,000
STAGE CONSTRUCTION (2%)	$102,686
WORK BY RAILROAD OR UTILITY FORCES	$0
GRAND TOTAL	$6,854,561
FOR BUDGET PURPOSES ONLY - SAY	$6,855,000

COMMENTS $ 40 / ft2

BR SF = 163413 SF

FM 91 1416

Sixth Street Viaduct over the Los Angeles River

Summary of Rehabilitation Costs (Alternative 5)

Alternative Description	Crack Repair	Barrier Rail Repair	Deck Rehabilitation	Total Rehabilitation Cost
Construction Cost	$ 2,549,681	$ 540,170	$ 2,044,438	$ 5,134,289
Mobilization Cost (10%)	$ 254,968	$ 54,017	$ 204,444	$ 513,429
Contingencies (15%)	$ 420,697	$ 89,128	$ 337,332	$ 847,158
Bridge Total	*$ 3,225,346*	*$ 683,315*	*$ 2,586,214*	*$ 6,494,875*
Bridge Removal	$ -	$ -	$ -	$ -
Traffic Control (5%)	$ 127,484	$ 27,009	$ 102,222	$ 256,715
Stage Construction (2%)	$ 50,994	$ 10,803	$ 40,889	$ 102,686
Railroad Work	$ -	$ -	$ -	$ -
Total Construction Costs	*$ 3,404,000*	*$ 721,000*	*$ 2,730,000*	*$ 6,855,000*
Construction Engineering (15%)*	$ 443,977	$ 94,060	$ 355,999	$ 894,036
Engineering Design and Environmental Clearance (25%)*	$ 739,962	$ 156,767	$ 593,331	$ 1,490,060
TOTAL PROJECT COST	$ 4,588,000	$ 972,000	$ 3,679,000	$ 9,239,000

Notes:
* Percent of "Total Construction Cost" excluding contingency.
Rehabilitation costs based on one time repair of cracks, barrier and deck rehabilitation.

50 YEAR LIFE CYCLE COST ANALYSIS OF SIXTH STREET VIADUCT @ 2.01% Discount Rate

All numbers are present value in Millions $

Initial Construction Cost

	Project cost	frequency	occurrences	Cost per occurance				Present Value
				1	2	3	4	
	$	yrs	#					Project Total
Alt 2 - Retrofit min Steel	57.411	0	1	57.4110	-	-	-	57.411
Alt 3 - Catcher Wall	64.547	0	1	64.5470	-	-	-	64.547
Alt 4 - Concrete Casing	91.577	0	1	91.5770	-	-	-	91.577
Alt 5 - Retrofit full Steel	90.050	0	1	90.0500	-	-	-	90.050
Alt 6A - Replacement In-Kind	98.421	0	1	98.4210	-	-	-	98.421
Alt 6B - Replacement New	77.199	0	1	77.1990	-	-	-	77.199

Epoxy Crack Repair

	Project cost	frequency	occurrences	Occurrences 1	2	3	4	Present Value
		yrs	#					Project Total
Alt 2 - Retrofit min Steel	5.885	10	0	-	-	-	-	0.000
Alt 3 - Catcher Wall	7.227	10	0	-	-	-	-	0.000
Alt 4 - Concrete Casing	3.728	10	1	3.0553	-	-	-	3.055
Alt 5 - Retrofit full Steel	4.588	10	2	3.7601	3.0815	-	-	6.842
Alt 6A - Replacement In-Kind	0.000	0	0	-	-	-	-	0.000
Alt 6B - Replacement New	0.000	0	0	-	-	-	-	0.000

Deck Rehab

	Project cost	frequency	occurrences	Occurrences 1	2	3	4	Present Value
		yrs	#					Project Total
Alt 2 - Retrofit min Steel	0.000	50	0	-	-	-	-	0.000
Alt 3 - Catcher Wall	0.000	50	0	-	-	-	-	0.000
Alt 4 - Concrete Casing	3.679	50	0	-	-	-	-	0.000
Alt 5 - Retrofit full Steel	3.679	50	0	-	-	-	-	0.000
Alt 6A - Replacement In-Kind	3.679	50	1	1.3602	-	-	-	1.360
Alt 6B - Replacement New	3.679	50	1	1.3602	-	-	-	1.360

Barrier Rehab

	Project cost	frequency	occurrences	Occurrences 1	2	3	4	Present Value
		yrs	#					Project Total
Alt 2 - Retrofit min Steel	0.972	15	0	-	-	-	-	0.000
Alt 3 - Catcher Wall	0.972	15	0	-	-	-	-	0.000
Alt 4 - Concrete Casing	0.972	15	1	0.7211	-	-	-	0.721
Alt 5 - Retrofit full Steel	0.972	15	1	0.7211	-	-	-	0.721
Alt 6A - Replacement In-Kind	0.972	30	1	0.5350	-	-	-	0.535
Alt 6B - Replacement New	0.972	30	1	0.5350	-	-	-	0.535

Structure Replacement

	Project cost	frequency	occurrences	Occurrences 1	2	3	4	Present Value
		yrs	#					Project Total
Alt 2 - Retrofit min Steel	98.42	10	1	80.6604	-	-	-	80.660
Alt 3 - Catcher Wall	98.42	10	1	80.6604	-	-	-	80.660
Alt 4 - Concrete Casing	98.42	20	1	66.1048	-	-	-	66.105
Alt 5 - Retrofit full Steel	98.42	30	1	54.1758	-	-	-	54.176
Alt 6A - Replacement In-Kind	98.42	75	0	-	-	-	-	0.000
Alt 6B - Replacement New	77.20	75	0	-	-	-	-	0.000

Residual Value of Bridges

	Future Value of Remaining life	Remaining life expectancy	Present Value
Alt 2 - Retrofit min Steel	45.93	35	-16.981
Alt 3 - Catcher Wall	45.93	35	-16.981
Alt 4 - Concrete Casing	59.05	45	-21.832
Alt 5 - Retrofit full Steel	72.18	55	-26.684
Alt 6A - Replacement In-Kind	32.81	25	-12.129
Alt 6B - Replacement New	25.73	25	-9.514

Totals

	Present Value Cost/sf		Project Total
Alt 2 - Retrofit min Steel	$	741	121.091
Alt 3 - Catcher Wall	$	785	128.227
Alt 4 - Concrete Casing	$	854	139.626
Alt 5 - Retrofit full Steel	$	766	125.105
Alt 6A - Replacement In-Kind	$	387	88.187
Alt 6B - Replacement New	$	257	69.580

COMPARISONS

Alternative 5 versus Alternative 6A

Initial Retrofit w/ Rehab Cost vs Replacement	**91%**	Retrofit has a lower INITIAL cost.
Life Cycle Retrofit vs Replacement	**142%**	Replacement has a lower total LIFETIME cost over the next 50 years.
Lifetime Savings of Alternative 6A over Alternative 5	**41.9%**	Savings

Alternative 5 versus Alternative 6B

Initial Retrofit w/ Rehab Cost vs Replacement	**117%**	Replacement has a lower INITIAL cost.
Life Cycle Retrofit vs Replacement	**180%**	Replacement has a lower total LIFETIME cost over the next 50 years.
Lifetime Savings of Alternative 6B over Alternative 5	**79.8%**	Savings

Assumptions

1) Annual Discount Rate **2.01%** (Cost of Bonds minus inflation)
2) Initial Construction Cost Includes initial Rehabilitation items for Alternative 5
3) All costs are project costs which include 10% mobilization, 15% contingencies, 5% traffic control, 2% stage construction, 15% construction engineering and 25% engineering/adinstration.
4) All replacement bridges have estimated lifespan of 75 years
5) Residual value based on remaining life expectancy times 1/75th of construciotn cost
6) Present Value = ($cost today)/[(1+discount rate)^(years until money is needed)]

7) AVERAGE CPI 3.50% <-- Bureau of Labor Statistics CPI from 1980 to 2004 ($100=$228.16)

8) Cost of Bonds 5.51% <-- Current 24+ year constant maturity treasury index, forward looking, from Wall Street Journal
"Key Interest Rates" June 1, 2004

APPENDIX D

SEISMIC DEMANDS AND CAPACITIES

Main Spans over the Los Angeles River

(Steel Arch Spans)

D/C Ratios for Main Spans over the Los Angeles River (Steel Arch Spans)

Columns of River Piers

Dec-03

As-Built Analysis Pier Columns								
Rotation Demands (radians)			**Rotation Capacity (radians)**			**Rotation D/C Ratios**		
East River Pier			**East River Pier**			**East River Pier**		
Location	Transverse	Longitudinal	Location	Transverse	Longitudinal	Location	Transverse	Longitudinal
N-Bottom	Pin	Pin	N-Bottom	Pin	Pin	N-Bottom	Pin	Pin
N-Top	0.00975	0	N-Top	0.00535	0.0387	N-Top	**1.82**	**0.00**
S-Bottom	Pin	Pin	S-Bottom	Pin	Pin	S-Bottom	Pin	Pin
S-Top	0.0128	0	S-Top	0.00535	0.0387	S-Top	**2.39**	**0.00**
West River Pier			**West River Pier**			**West River Pier**		
Location	Transverse	Longitudinal	Location	Transverse	Longitudinal	Location	Transverse	Longitudinal
N-Bottom	Pin	Pin	N-Bottom	Pin	Pin	N-Bottom	Pin	Pin
N-Top	0.00872	0	N-Top	0.00535	0.0387	N-Top	**1.63**	**0.00**
S-Bottom	Pin	Pin	S-Bottom	Pin	Pin	S-Bottom	Pin	Pin
S-Top	0.0132	0	S-Top	0.00535	0.0387	S-Top	**2.47**	**0.00**
Center River Pier			**Center River Pier**			**Center River Pier**		
Location	Transverse	Longitudinal	Location	Transverse	Longitudinal	Location	Transverse	Longitudinal
N-Bottom	0	0.0113	N-Bottom	0.00288	0.0146	N-Bottom	**0.00**	**0.77**
N-Middle	0.00286	0	N-Middle	0.00067	0.00144	N-Middle	**4.27**	**0.00**
N-Top	4.40E-05	0	N-Top	0.00075	0.00154	N-Top	**0.06**	**0.00**
S-Bottom	0	0.0102	S-Bottom	0.00126	0.0149	S-Bottom	**0.00**	**0.68**
S-Middle	0.00276	0	S-Middle	0.00052	0.00122	S-Middle	**5.31**	**0.00**
S-Top	0.000921	0	S-Top	0.00065	0.00094	S-Top	**1.42**	**0.00**

D/C Ratios for Main Spans over the Los Angeles River (Steel Arch Spans)

Columns of River Piers

Dec-03

Retrofit Analysis **Pier Columns**								
Rotation Demands (radians)			**Rotation Capacity (radians)**			**Rotation D/C Ratios**		
East River Pier			**East River Pier**			**East River Pier**		
Location	Transverse	Longitudinal	Location	Transverse	Longitudinal	Location	Transverse	Longitudinal
N-Bottom	Fixed	0.0103	N-Bottom	Fixed	0.0387	N-Bottom	Fixed	**0.27**
N-Top	0.00016	0	N-Top	0.00535	0.0387	N-Top	**0.03**	**0.00**
S-Bottom	Fixed	0.0103	S-Bottom	Fixed	0.0387	S-Bottom	Fixed	**0.27**
S-Top	0.000248	0	S-Top	0.00535	0.0387	S-Top	**0.05**	**0.00**
West River Pier			**West River Pier**			**West River Pier**		
Location	Transverse	Longitudinal	Location	Transverse	Longitudinal	Location	Transverse	Longitudinal
N-Bottom	Fixed	0.0098	N-Bottom	Fixed	0.0387	N-Bottom	Fixed	**0.25**
N-Top	0.000144	0	N-Top	0.00535	0.0387	N-Top	**0.03**	**0.00**
S-Bottom	Fixed	0.00973	S-Bottom	Fixed	0.0387	S-Bottom	Fixed	**0.25**
S-Top	0.000316	0	S-Top	0.00535	0.0387	S-Top	**0.06**	**0.00**
Center River Pier			**Center River Pier**			**Center River Pier**		
Location	Transverse	Longitudinal	Location	Transverse	Longitudinal	Location	Transverse	Longitudinal
N-Bottom	0	0.00962	N-Bottom	0.00288	0.0146	N-Bottom	**0.00**	**0.66**
N-Middle	0.00106	0	N-Middle	0.00144	0.00144	N-Middle	**0.74**	**0.00**
N-Top	3.88E-05	0	N-Top	0.00075	0.00154	N-Top	**0.05**	**0.00**
S-Bottom	0	0.00848	S-Bottom	0.00126	0.0149	S-Bottom	**0.00**	**0.57**
S-Middle	0.00058	0	S-Middle	0.00066	0.00122	S-Middle	**0.88**	**0.00**
S-Top	2.79E-04	0	S-Top	0.00065	0.00094	S-Top	**0.43**	**0.00**

D/C Ratios for Main Spans over the Los Angeles River (Steel Arch Spans)

Bent Caps

Dec-03

	Bent Cap Rotations (radians)								Rotation D/C Ratios			
As-Built Bent Caps	Positive Demand North Column	Positive Capacity North Column	Negative Demand North Column	Negative Capacity North Column	Positive Demand South Column	Positive Capacity South Column	Negative Demand South Column	Negative Capacity South Column	**D/C Ratio Positive North Column**	**D/C Ratio Negative North Column**	**D/C Ratio Positive South Column**	**D/C Ratio Negative South Column**
West Pier	0	0.00451	0.0114	0.0201	0.000535	0.00451	0.0065	0.0201	**0.00**	**0.57**	**0.12**	**0.32**
East Pier	0.00162	0.00451	0.00818	0.0201	0	0.00451	0.00742	0.0201	**0.36**	**0.41**	**0.00**	**0.37**
Center Pier	0	0.003	0.000611	0.0034	0	0.003	0.00226	0.0034	**0.00**	**0.18**	**0.00**	**0.66**
Retrofit Bent Caps	Positive Demand North Column	Positive Capacity North Column	Negative Demand North Column	Negative Capacity North Column	Positive Demand South Column	Positive Capacity South Column	Negative Demand South Column	Negative Capacity South Column	**D/C Ratio Positive North Column**	**D/C Ratio Negative North Column**	**D/C Ratio Positive South Column**	**D/C Ratio Negative South Column**
West Pier	0	0.00451	0.000339	0.0201	0	0.00451	0	0.0201	**0.00**	**0.02**	**0.00**	**0.00**
East Pier	0	0.00451	0	0.0201	0	0.00451	0	0.0201	**0.00**	**0.00**	**0.00**	**0.00**
Center Pier	0	0.003	0.00056	0.0034	0	0.003	0.000802	0.0034	**0.00**	**0.16**	**0.00**	**0.24**

	Bent Cap Shear (kips)				Shear D/C Ratios	
Bent Caps	**As-Built** Demand	**As-Built** Capacity	**Retrofit** Demand	**Retrofit** Capacity	**As-Built D/C Ratio**	**Retrofit D/C Ratio**
West Pier	2104	4270	1765	4270	**0.49**	**0.41**
East Pier	2173	4270	1325	4270	**0.51**	**0.31**
Center Pier	8278	7400	7015	7400	**1.12**	**0.95**

D/C Ratios for Main Spans over the Los Angeles River (Steel Arch Spans)

Steel Arch Ribs

Dec-03

Arch Rib Rotations at Critical Arch-Deck Interface

As-Built

Arch Rib Location	Member Length (ft)	Yield Rot. (rad.)	Rot. Cap. (rad.)	As-Built Demand (rad.)	Ductility Demand	**D/C Ratio**
B-South	3.91	0.00164	0.00328	0.00466	2.84	**1.42**
B-North	3.91	0.00164	0.00328	0.0034	2.07	**1.04**
A-South	3.36	0.00141	0.00282	0.0057	4.04	**2.02**
A-North	3.36	0.00141	0.00282	0.0056	3.97	**1.99**

Retrofit

Arch Rib Location	Member Length (ft)	Yield Rot. (rad)	Rot. Cap. (rad.)	Retrofit Demand (rad.)	Ductility Demand	**D/C Ratio**
B-South	6.35	0.00266	0.00532	0.00215	0.81	**0.40**
B-North	6.35	0.00266	0.00532	0.00245	0.92	**0.46**
A-South	3.36	0.00141	0.00282	0.00208	1.48	**0.74**
A-North	3.36	0.00141	0.00282	0.00233	1.65	**0.83**

Refer to Figure 2 for Arch Rib Locations (Points A and B in Figure 2 for South and North Arches).

Approach Spans (As-Built)

Summary of seismic displacement D/C ratios in the approach spans

Frame #	**Displacement D/C Ratio**			
	Transverse Direction		**Longitudinal Direction**	
	D/C	**@ Bent #**	**D/C**	**@ Bent #**
1	**1.33** **1.21**	8 9	**1.17**	4
3	**2.22** **11.90** (Shear failure)	19 18	0.38 **4.40** (Shear failure**)**	16 18
4	**1.95**	34	**2.44**	26

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		Concrete Cross Section					Shear Check			
Bent	Column	Eff. width (ft)	Eff. Height (ft)	A_g (ft^2)	f'_c (psi)	A_{eff} (ft^2)	V_c (kips)	$V_{push-Long}$ (kips)	$V_{push-Trans}$ (kips)	
1	S	2.33	2	20.65	3634	4.660	80.9			
	M	2.33	2	20.65	3634	4.660	80.9			
	N	2.33	2	20.65	3634	4.660	80.9			
2	S	2.33	2	20.65	3634	4.660	80.9			
	M	2.33	2	20.65	3634	4.660	80.9			
	N	2.33	2	20.65	3634	4.660	80.9			
3	S	4.5	5.5	59.083	3634	24.750	429.7	54	78	N.G
	M	2.833	3.000	32.983	3634	8.500	147.6		168	
	N	4.5	5.5	59.083	3634	24.750	429.7		154	
4	S	4.000	5.500	40.427	3634	22.000	382.0	326	154	N.G
	M	2.833	4.875	53.75	3634	13.811	239.8		207	
	N	4.000	5.500	40.427	3634	22.000	382.0		269	
5	S	4.000	5.500	40.427	3634	22.000	382.0	334	134	N.G
	M	2.833	4.875	53.75	3634	13.811	239.8		198	
	N	4.000	5.500	40.427	3634	22.000	382.0		232	
6	S	4.5	5.5	59.083	3634	24.750	429.7	56.2	45	
	M	2.833	3.000	32.983	3634	8.500	147.6		107	
	N	4.5	5.5	59.083	3634	24.750	429.7		101	
7	S	5.833	5.833	24.5	3634	34.026	590.7	221	177	
	M	5.833	5.833	24.5	3634	34.026	590.7		202	
	N	5.833	5.833	24.5	3634	34.026	590.7		207	
8	S	5.833	5.833	24.5	3634	34.026	590.7	212	175	
	M	5.833	5.833	24.5	3634	34.026	590.7		192	
	N	5.833	5.833	24.5	3634	34.026	590.7		174	
9	S	5.833	5.833	70.851	3634	34.024	590.7	39	127	N.G
	M	2.833	2.813	32.366	3634	7.968	138.3		181	
	N	5.833	5.833	70.851	3634	34.024	590.7		172	
10	S	5.042	6.583	63.202	3634	33.191	576.2			
	M	3.500	6.583	62.292	3634	23.042	400.0			
	N	5.042	6.583	63.202	3634	33.191	576.2			
11	S	5.521	7.823	66.278	3634	43.189	749.8			
	M	5.167	7.333	76.6443	3634	37.890	657.8			
	N	5.521	7.823	66.278	3634	43.189	749.8			

	Transverse Displacement				Longitudinal Disp.	
	Pier #8		Pier #9		Pier #4	
72 yr	0.34970'	4.20 in	0.46970'	5.64 in	0.24953'	2.99 in
475 yr	0.86139'	10.34 in	1.15714'	13.89 in	0.59572'	7.15 in
950 yr	1.10708'	13.28 in	1.48723'	17.85 in	0.76195'	9.14 in
MCE	1.32609'	15.91 in	1.78046'	21.37 in	0.91780'	11.01 in
collapse	11.96 in		17.73 in		9.39 in	
D/C Ratios	1.33		1.21		1.17	

W. Koo & Associates, Inc.
Structural Engineers

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Pier #	15				16			
	Logitudinal		Transverse		Logitudinal		Transverse	
	U2	U3	U2	U3	U2	U3	U2	U3
72yr	0.77997'	1.11368'	0.15515'	0.76631'	0.32367'	0.07609'	0.65735'	0.07217'
	9.36 in	13.36 in	1.86 in	9.20 in	3.88 in	0.91 in	7.89 in	0.87 in
475yr	1.89929'	2.78281'	0.30662'	1.82551'	0.79360'	0.08766'	1.58431'	0.07743'
	22.79 in	33.39 in	3.68 in	21.91 in	9.52 in	1.05 in	19.01 in	0.93 in
950yr	2.39443'	3.53583'	0.36305'	2.32512'	1.01302'	0.09287'	2.03009'	0.07986'
	28.73 in	42.43 in	4.36 in	27.90 in	12.16 in	1.11 in	24.36 in	0.96 in
MCE	1.58558'	2.60693'	0.10025'	2.03866'	0.83536'	0.08652'	1.81120'	0.07791'
	19.03 in	31.28 in	1.20 in	24.46 in	10.02 in	1.04 in	21.73 in	0.93 in
collapse					26.60 in			
D/C Ratios					0.38			

Pier #	17				18			
	Logitudinal		Transverse		Logitudinal		Transverse	
	U2	U3	U2	U3	U2	U3	U2	U3
72yr	1.23873'	0.11337'	0.71692'	0.62936'	1.33016'	0.18759'	0.85064'	0.96992'
	14.86 in	1.36 in	8.60 in	7.55 in	15.96 in	2.25 in	10.21 in	11.64 in
475yr	3.05295'	0.27422'	1.67171'	1.45364'	3.27776'	0.40816'	1.99074'	2.31932'
	36.64 in	3.29 in	20.06 in	17.44 in	39.33 in	4.90 in	23.89 in	27.83 in
950yr	3.86724'	0.03326'	2.11285'	1.85000'	4.15187'	0.52085'	2.51960'	2.97031'
	46.41 in	0.40 in	25.35 in	22.20 in	49.82 in	6.25 in	30.24 in	35.64 in
MCE	2.74612'	0.06602'	1.66997'	1.69527'	2.92959'	0.49826'	1.96347'	2.48689'
	32.95 in	0.79 in	20.04 in	20.34 in	35.16 in	5.98 in	23.56 in	29.84 in
collapse								
D/C Ratios								

Pier #	19				20			
	Logitudinal		Transverse		Logitudinal		Transverse	
	U2	U3	U2	U3	U2	U3	U2	U3
72yr	1.45625'	0.50994'	1.04257'	1.47585'	1.37689'	0.29295'	0.81929'	0.84796'
	17.48 in	6.12 in	12.51 in	17.71 in	16.52 in	3.52 in	9.83 in	10.18 in
475yr	3.58537'	1.24929'	2.44927'	3.58049'	3.39180'	0.71978'	1.90524'	2.04979'
	43.02 in	14.99 in	29.39 in	42.97 in	40.70 in	8.64 in	22.86 in	24.60 in
950yr	4.54294'	1.60043'	3.10559'	4.59247'	4.29467'	0.92090'	2.40797'	2.62935'
	54.52 in	19.21 in	37.27 in	55.11 in	51.54 in	11.05 in	28.90 in	31.55 in
MCE	3.22587'	1.30584'	2.45188'	3.83071'	3.02125'	0.74285'	1.84860'	2.20501'
	38.71 in	15.67 in	29.42 in	45.97 in	36.26 in	8.91 in	22.18 in	26.46 in
collapse				20.69 in				
D/C Ratios				2.22				

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Bent	Column	Concrete Cross Section					Shear Check			
		Eff. width (ft)	Eff. Height (ft)	A_g (ft^2)	f'_c (psi)	A_{eff} (ft^2)	V_c	$V_{push\text{-}Trans}$ (kips)	$V_{push\text{-}Trans}$ (kips)	
12	S	4.67	5.80	62.2	2100	26.80	353.7			
	M	5.00	6.14	77.168	2100	30.70	405.2			
	N	4.67	5.80	62.2	2100	26.80	353.7			
13	S	4.67	5.80	62.2	2100	26.80	353.7			
	M	5.00	6.14	77.168	2100	30.70	405.2			
	N	4.67	5.80	62.2	2100	26.80	353.7			
14	S	5.25	7.92	81.348	2100	35.31	466.0			
	M	5.25	8.67	69.78	2100	26.50	349.7			
	N	5.25	7.92	81.348	2100	35.31	466.0			
15	S	4.00	7.52	62.844	2100	25.66	338.7	259.0	176	
	M	4.00	4.00	43	2100	16.00	211.2	259.0	176	N.G
	N	4.00	7.52	62.844	2100	25.66	338.7	259.0	176	
16	S	5.25	6.06	79.979	2100	37.83	499.3			
	N	5.25	6.06	79.979	2100	37.83	499.3			
17	S	4.00	7.52	62.844	2100	25.66	338.7			
	M	4.00	4.00	43.002	2100	16.00	211.2			
	N	4.00	7.52	62.844	2100	25.66	338.7			
18	S	2.00	4.05	29.407	2100	7.34	96.9	178.0	157	N.G
	M	2.00	2.50	24.502	2100	5.00	66.0	178.0	157	N.G
	N	2.00	4.05	29.407	2100	7.34	96.9	178.0	157	N.G
19	S	4.00	7.52	62.844	2100	25.66	338.7			
	M	4.00	4.00	43.002	2100	16.00	211.2			
	N	4.00	7.52	62.844	2100	25.66	338.7			
20	S	2.00	4.05	29.407	2100	7.34	96.9			
	M	2.00	2.50	24.502	2100	5.00	66.0			
	N	2.00	4.05	29.407	2100	7.34	96.9			
21	S	3.83	6.40	41	2100	26.01	343.3			
	M	4.79	4.50	52.44	2100	21.56	284.5			
22	S	3.67	4.63	47.024	2100	11.18	147.6			
	M	3.00	4.00	66.299	2100	12.00	158.4			
21+22	N	5.08	6.06	62.938	2100	24.07	317.7			

W. Koo & Associates, Inc.
Structural Engineers

BY: F.H DATE: 8/30/02 CLIENT: The City of Los Angeles SHEET NO.: OF:
CHECKED: DATE: JOB: Six Street Bridge SUBJECT: JOB NO.: 22002

Pier #	26				34			
	Logitudinal		Transverse		Logitudinal		Transverse	
	U2	U3	U2	U3	U2	U3	U2	U3
72yr	0.65083'	0.75750'	0.65250'	0.75917'	0.02500'	0.67917'	0.02917'	0.69167'
	7.81 in	9.09 in	7.83 in	9.11 in	0.30 in	8.15 in	0.35 in	8.30 in
475yr	2.53833'	0.48333'	1.58667'	1.82333'	1.05917'	2.66750'	0.07417'	1.66667'
	30.46 in	5.80 in	19.04 in	21.88 in	12.71 in	32.01 in	0.89 in	20.00 in
950yr	3.25083'	0.62000'	2.03417'	2.34167'	1.35417'	3.41417'	0.09167'	2.13417'
	39.01 in	7.44 in	24.41 in	28.10 in	16.25 in	40.97 in	1.10 in	25.61 in
MCE	2.52250'	0.62000'	1.81750'	2.45250'	0.92667'	2.74917'	0.10833'	1.97833'
	30.27 in	7.44 in	21.81 in	29.43 in	11.12 in	32.99 in	1.30 in	23.74 in
collapse	12.41 in							12.17 in
D/C Ratios	2.44							1.95

6th Street Bridge Steel Casing Alternative Column Shear D/C Table (Only cased for EQ Requirement not all ASR columns)

Transverse Direction 70% Concrete area is used for bad concrete columns

Pier #	COLUMN	Disp-eql	Disp-eqt	[Disp] y-ft	(D/Y)d	phro-s	Fac-1	P-DL	Fac-2	Ag (in^2)	d (in)	fc'	[Vc]	As (in^2)	s (in)	Vs	[V] tot	Veq-T	(D/C)v
1	Exterior	0.0059	0.0156	0.0332	0.47	0.000212	3.26	-667	1.11	2976.00	54.55	3500	510	0.2500	12.0000	50	560	124	0.22
	Interior	0.0073	0.0161	0.0332	0.48	0.000212	3.24	-689	1.15	2976.00	54.55	3500	525	0.2500	12.0000	50	575	124	0.22
2	Exterior	0.0069	0.0129	0.0506	0.25	0.000212	3.47	-629	1.11	2976.00	54.55	3500	541	0.2500	12.0000	50	591	100	0.17
	Interior	0.0071	0.0160	0.0506	0.32	0.000212	3.41	-839	1.14	2976.00	54.55	3500	548	0.2500	12.0000	50	598	100	0.17
3	Exterior	0.0045	0.0073	0.0162	0.45	0.001076	3.50	-565	1.01	33530.00	638.00	2100	4344	0.8800	4.0000	6156	10501	9833	0.94
	Interior	0.0052	0.0090	0.0162	0.55	0.001076	3.40	-754	1.01	33530.00	638.00	2100	4232	0.8800	4.0000	6156	10388	9833	0.95
4	Exterior	0.0224	0.0468	0.0702	0.67	0.000000	3.00	-952	1.09	5418.00	73.61	3500	838	0.2500	12.0000	67	905	334	0.37
	Interior	0.0233	0.0462	0.0962	0.48	0.000000	3.19	-1269	1.11	5822.00	76.30	3500	975	0.2500	12.0000	70	1045	193	0.18
5	Exterior	0.0482	0.0943	0.0803	1.17	0.000000	2.50	-945	1.09	5418.00	73.61	3500	696	0.2500	12.0000	67	783	312	0.41
	Interior	0.0471	0.0930	0.1100	0.85	0.000000	2.82	-1260	1.11	5822.00	76.30	3500	863	0.2500	12.0000	70	933	180	0.19
6	Exterior	0.0542	0.1076	0.1010	1.07	0.000000	2.60	-667	1.06	5762.40	75.91	3500	751	0.3910	12.0000	109	860	353	0.41
	Interior	0.0517	0.1016	0.1544	0.66	0.000000	3.01	-889	1.09	4778.00	69.12	3500	744	0.3910	12.0000	99	844	149	0.18
7	Exterior	0.0296	0.0577	0.0435	1.32	0.001076	2.63	-681	1.01	25390.00	576.00	3500	3206	0.8800	12.0000	1859	5065	3991	0.79
	Interior	0.0317	0.0618	0.0435	1.42	0.001076	2.54	-908	1.02	25390.00	576.00	3500	3106	0.8800	12.0000	1859	4964	3991	0.80
8	Exterior	0.0211	0.0389	0.0946	0.41	0.000000	3.26	-731	1.09	4248.00	65.18	3500	712	0.3910	12.0000	93	805	83	0.10
	Interior	0.0214	0.0395	0.1048	0.38	0.000000	3.29	-974	1.09	5400.00	73.48	3500	918	0.3910	12.0000	105	1023	90	0.09
9	Exterior	0.0317	0.0589	0.0566	1.04	0.001076	2.92	-596	1.01	37870.00	576.00	3500	5271	0.8800	6.0000	3717	8988	7629	0.85
	Interior	0.0331	0.0615	0.0566	1.09	0.001076	2.87	-795	1.01	37870.00	576.00	3500	5200	0.8800	6.0000	3717	8917	7629	0.86
10	Exterior	0.0514	0.0884	0.3001	0.29	0.000000	3.38	-1537	1.13	5723.90	75.66	2100	803	0.7030	12.0000	195	998	129	0.13
	Interior	0.0511	0.0843	0.3531	0.24	0.000000	3.43	-2049	1.14	7490.00	86.54	2100	1071	0.5625	12.0000	178	1250	94	0.08
11	Exterior	0.0787	0.0814	0.0617	1.32	0.001076	2.64	-796	1.01	43270.00	672.00	3500	5451	0.8800	6.0000	4337	9788	9336	0.95
	Interior	0.0789	0.0839	0.0617	1.36	0.001076	2.60	-1061	1.01	43270.00	672.00	3500	5385	0.8800	6.0000	4337	9722	9336	0.96
12	Exterior	0.5972	0.6640	0.2216	3.00	0.000000	0.67	-2033	1.11	9625.00	98.11	2100	263	0.7500	12.0000	270	532	396	0.74
	Interior	0.5915	0.6579	0.2390	2.75	0.000000	0.92	-2711	1.16	8638.00	92.94	2100	336	0.5000	12.0000	170	507	390	0.77
13	Exterior	0.0497	0.0597	0.0596	1.00	0.001076	2.96	-1895	1.02	45060.00	756.00	3500	6421	0.8800	4.0000	7318	13739	11756	0.86
	Interior	0.0477	0.0580	0.0596	0.97	0.001076	2.98	-2260	1.03	45060.00	758.00	3500	6524	0.8800	4.0000	7318	13842	11756	0.85
14	Exterior	0.1986	0.2440	0.2518	0.97	0.000000	2.70	-1067	1.08	8970.60	83.49	2100	743	1.2500	12.0000	383	1126	166	0.15
	Interior	0.1698	0.2001	0.4136	0.48	0.000000	3.19	-1422	1.15	4872.00	69.80	2100	852	1.0000	12.0000	256	908	74	0.08
15	Exterior	0.0572	0.0924	0.0729	1.27	0.001076	2.69	-382	1.00	41190.00	720.00	3500	5269	0.8800	6.0000	4648	9916	7682	0.77
	Interior	0.0577	0.0929	0.0729	1.27	0.001076	2.68	-509	1.01	41190.00	720.00	3500	5263	0.8800	6.0000	4646	9909	7682	0.78
16	Exterior	0.2932	0.2336	0.1340	2.19	0.001076	1.77	-1453	1.06	11530.00	107.38	3500	1027	0.3100	3.0000	488	1515	693	0.46
	Interior	0.3081	0.2391	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
17	Exterior	0.0278	0.0552	0.0697	0.79	0.001076	3.17	-569	1.01	35852.00	672.00	3500	5414	0.8800	12.0000	2188	7582	6485	0.86
	Interior	0.0277	0.0557	0.0697	0.80	0.001076	3.16	-759	1.01	35852.00	672.00	3500	5417	0.8800	12.0000	2168	7586	6485	0.85
18	Exterior	0.0305	0.0696	0.4671	0.15	0.000000	3.52	-436	1.07	2935.10	54.18	3500	525	0.7031	12.0000	140	665	94	0.14
	Interior	0.0275	0.0643	0.5897	0.11	0.000000	3.56	-581	1.08	3528.00	59.40	3500	644	0.5625	12.0000	123	766	72	0.09
19	Exterior	0.0286	0.0516	0.0718	0.72	0.001076	3.24	-561	1.01	35852.00	672.00	3500	5539	0.8800	12.0000	2168	7708	6390	0.83
	Interior	0.0290	0.0521	0.0718	0.73	0.001076	3.23	-749	1.01	35852.00	672.00	3500	5541	0.8800	12.0000	2168	7709	6390	0.83
20	Exterior	0.1026	0.1596	0.4265	0.37	0.000000	3.30	-946	1.16	2935.10	54.18	2100	412	0.7031	12.0000	140	551	99	0.18
	Interior	0.0955	0.1516	0.5384	0.28	0.000000	3.39	-1261	1.18	3528.00	59.40	2100	517	0.5625	12.0000	123	639	76	0.12
21	Exterior	0.0331	0.0553	0.0594	0.93	0.001076	3.03	-975	1.01	34224.00	696.00	3500	4973	0.8800	12.0000	2246	7219	4916	0.68
	Interior	0.0308	0.0533	0.0594	0.90	0.001076	3.06	-1300	1.02	34224.00	696.00	3500	5050	0.8800	12.0000	2246	7296	4916	0.67
22	Exterior	0.2869	0.2442	0.3995	0.72	0.001076	3.24	-1023	1.08	6772.00	840.00	3500	1117	0.6200	3.0000	7638	8755	250	0.03
	Interior	0.2231	0.2538	0.4160	0.61	0.001076	3.35	-1364	1.10	6820.00	840.00	3500	1188	0.6200	3.0000	7638	8827	303	0.03
23	Exterior	0.0183	0.0265	0.0971	0.27	0.001076	3.68	-407	1.00	48238.00	696.00	2100	6543	0.8800	4.0000	6737	13280	11312	0.85
	Interior	0.0245	0.0257	0.0971	0.27	0.001076	3.69	-407	1.00	48238.00	696.00	3500	8465	0.8800	4.0000	6737	15202	11312	0.74
24	Exterior	0.0507	0.0637	0.0978	0.65	0.001076	3.31	-424	1.01	34224.00	696.00	2100	4173	0.8800	6.0000	4492	8665	4705	0.54
	Interior	0.0544	0.0658	0.0978	0.67	0.001076	3.26	-565	1.01	34224.00	696.00	2100	4155	0.8800	6.0000	4492	8646	4705	0.54
25	Exterior	0.0945	0.3032	0.4581	0.66	0.000000	3.01	-989	1.18	2699.20	51.95	3500	455	1.0000	18.0000	127	582	90	0.15
	Interior	0.0926	0.2966	0.4591	0.65	0.000000	3.02	-1319	1.16	4176.00	64.62	3500	692	1.0000	18.0000	158	850	71	0.08
26	Exterior	0.1062	0.3027	0.4604	0.66	0.000000	3.01	-825	1.15	2699.20	51.95	3500	444	1.0000	18.0000	127	571	90	0.16
	Interior	0.0992	0.2805	0.4614	0.61	0.000000	3.06	-1100	1.19	2923.20	54.07	3500	503	1.0000	18.0000	132	636	70	0.11
27	Exterior	0.0280	0.0639	0.0720	0.89	0.001076	3.07	-1080	1.01	37011.00	696.00	3500	5454	0.8800	12.0000	2248	7700	5831	0.76
	Interior	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
28	Exterior	0.2120	0.1770	0.7517	0.28	0.000000	3.39	-778	1.14	2845.00	53.34	3500	519	0.5625	12.0000	110	629	55	0.09
	Interior	0.2119	0.1771	0.8275	0.26	0.000000	3.41	-1038	1.17	3136.00	56.00	3500	591	0.5625	12.0000	116	706	63	0.09
29	Exterior	0.5486	0.4705	0.1482	3.70	0.000000	0.30	-818	1.03	11809.00	108.67	3500	173	0.5625	18.0000	149	323	226	0.70
	Interior	0.5785	0.4955	0.2089	2.77	0.000000	0.90	-1091	1.08	7200.00	84.85	3500	330	0.5625	18.0000	117	447	203	0.45
30	Exterior	0.3224	0.2427	0.1711	1.88	0.000000	1.79	-524	1.03	7639.80	87.41	3500	668	0.5625	18.0000	120	788	303	0.38
	Interior	0.3420	0.2645	0.1774	1.93	0.000000	1.74	-698	1.05	7015.40	83.76	3500	607	0.5625	18.0000	115	722	278	0.38
31	Exterior	0.0465	0.0571	0.1096	0.52	0.001076	3.44	-834	1.01	30255.00	696.00	3500	4989	0.8800	12.0000	2246	7234	4444	0.61
	Interior	0.0461	0.0599	0.1096	0.55	0.001076	3.41	-1112	1.02	30255.00	696.00	3500	4974	0.8800	12.0000	2246	7220	4444	0.62
32	Exterior	0.1554	0.2017	0.1874	1.08	0.000000	2.59	-668	1.11	2950.50	54.32	3500	403	0.5625	18.0000	75	478	57	0.12
	Interior	0.1569	0.2054	0.3833	0.54	0.000000	3.13	-891	1.14	3096.00	55.64	3500	525	0.5625	18.0000	77	602	78	0.13
33	Exterior	0.1713	0.2267	0.1612	1.41	0.000000	2.26	-1117	1.05	10210.00	101.04	3500	1154	0.5625	18.0000	139	1293	217	0.17
	Interior	0.1682	0.2250	0.1654	1.36	0.000000	2.31	-1490	1.08	9508.00	97.51	3500	1121	0.5625	18.0000	134	1255	131	0.10
34	Exterior	0.0436	0.0437	0.1158	0.38	0.001076	3.58	-519	1.01	31652.00	756.00	3500	5406	0.8800	12.0000	2439	7846	4324	0.55
	Interior	0.0438	0.0394	0.1158	0.38	0.001076	3.58	-693	1.01	31652.00	756.00	3500	5420	0.8800	12.0000	2439	7859	4324	0.55
35	Exterior	0.3850	0.2757	0.3891	0.99	0.000000	2.68	-756	1.13	2950.50	54.32	3500	422	0.5625	18.0000	75	497	56	0.11
	Interior	0.3916	0.2807	0.7960	0.49	0.000000	3.18	-1008	1.16	3096.00	55.64	3500	542	0.5625	18.0000	77	618	76	0.12
36	Exterior	0.4546	0.3259	0.3735	1.22	0.000000	2.45	-1073	1.13	4215.00	64.92	3500	552	0.5625	18.0000	89	641	57	0.09
	Interior	0.4395	0.3153	0.7640	0.58	0.000000	3.09	-1431	1.23	3096.00	55.64	3500	558	0.5625	18.0000	77	635	78	0.12
37	Exterior	0.0464	0.0312	0.1138	0.41	0.001076	3.55	-340	1.00	34392.00	756.00	3500	5806	0.8800	12.0000	2439	8245	5693	0.69
	Interior	0.0462	0.0310	[illegible]	[illegible]	0.001076	3.55	[illegible]	1.01	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

6th Street Bridge Steel Casing Alternative Column Shear D/C Table (Only cased for EQ Requirement not all ASR columns)

Longitudinal Direction 70% Concrete area is used for bad concrete columns fac= 0.7

Pier #	COLUMN	Disp-eqt	Disp-eqt	(Disp-L) y	(D/Y)d	phro-s	Fac-1	P-DL	Fac-2	Ag (in^2)	d (in)	fc'	[Vc]	As (in^2)	s (in)	Vs	[V] tot	Veq-L	(D/C)v
1	Exterior	0.1874	0.0977	0.0366	5.11	0.000212	0.30	-667	1.11	2976.00	54.55	3500	47	0.2500	12.0000	50	97	115	1.18
	Interior	0.2531	0.1276	0.0366	8.91	0.000212	0.30	-889	1.15	2876.00	54.55	3500	49	0.2500	12.0000	50	99	115	1.17
2	Exterior	0.2019	0.1054	0.0559	3.62	0.000212	0.30	-629	1.11	2976.00	54.55	3500	47	0.2500	12.0000	50	97	93	0.97
	Interior	0.1825	0.0950	0.0559	3.27	0.000212	0.46	-839	1.14	2976.00	54.55	3500	74	0.2500	12.0000	50	124	93	0.75
3	Exterior	0.3955	0.2068	0.0688	5.75	0.001076	0.30	-565	1.01	33530.00	33.00	2100	372	10.0000	12.0000	1210	1582	1500	0.95
	Interior	0.3878	0.2026	0.0688	5.64	0.001076	0.30	-754	1.01	33530.00	33.00	2100	373	10.0000	12.0000	1210	1583	1500	0.95
4	Exterior	0.3565	0.1874	0.0577	6.18	0.000000	0.30	-952	1.09	5418.00	73.61	3500	84	0.2500	12.0000	67	151	376	2.48
	Interior	0.2864	0.1496	0.0613	4.67	0.000000	0.30	-1269	1.11	5822.00	76.30	3500	92	0.2500	12.0000	70	162	271	1.68
5	Exterior	0.3931	0.2056	0.0660	5.96	0.000000	0.30	-945	1.09	5418.00	73.61	3500	84	0.2500	12.0000	67	151	351	2.32
	Interior	0.3143	0.1839	0.0701	4.48	0.000000	0.30	-1260	1.11	5822.00	76.30	3500	92	0.2500	12.0000	70	182	253	1.57
6	Exterior	0.4720	0.2478	0.0985	4.79	0.000000	0.30	-667	1.06	5782.40	75.91	3500	87	0.3910	12.0000	109	185	325	1.67
	Interior	0.4582	0.2394	0.1207	3.80	0.000000	0.30	-889	1.09	4778.00	89.12	3500	74	0.3810	12.0000	99	173	161	0.93
7	Exterior	0.4818	0.2579	0.3041	1.62	0.001076	2.34	-681	1.01	25390.00	33.00	3500	2850	9.6000	12.0000	1162	4011	355	0.09
	Interior	0.4939	0.2581	0.3041	1.62	0.001078	2.33	-908	1.02	25390.00	33.00	3500	2853	9.6000	12.0000	1162	4015	355	0.09
8	Exterior	0.5908	0.3087	0.2227	2.85	0.000000	1.02	-731	1.09	4248.00	65.16	3500	222	0.3910	12.0000	93	315	60	0.19
	Interior	0.6579	0.3438	0.2016	3.26	0.000000	0.41	-974	1.09	5400.00	73.48	3500	113	0.3910	12.0000	105	219	78	0.36
9	Exterior	0.6486	0.3390	0.3432	1.89	0.001076	2.07	-596	1.01	37870.00	33.00	3500	3735	9.6000	12.0000	1162	4897	1044	0.21
	Interior	0.6658	0.3478	0.3432	1.94	0.001076	2.02	-795	1.01	37870.00	33.00	3500	3654	9.6000	12.0000	1162	4816	1044	0.22
10	Exterior	0.4989	0.2604	0.2207	2.28	0.000000	1.41	-1537	1.13	5723.90	75.66	2100	335	0.7030	12.0000	195	530	167	0.31
	Interior	0.4270	0.2231	0.2064	2.07	0.000000	1.60	-2049	1.14	7490.00	86.54	2100	500	0.5825	12.0000	176	676	153	0.23
11	Exterior	0.6633	0.3472	0.3701	1.79	0.001076	2.17	-796	1.01	43270.00	33.00	3500	4475	11.2000	12.0000	1355	5830	1279	0.22
	Interior	0.6586	0.3432	0.3701	1.77	0.001076	2.18	-1061	1.01	43270.00	33.00	3500	4525	11.2000	12.0000	1355	5880	1279	0.22
12	Exterior	0.9792	0.5214	0.2839	3.45	0.000000	0.30	-2033	1.11	9625.00	98.11	2100	117	0.7500	12.0000	270	387	267	0.69
	Interior	0.9503	0.5328	0.2965	3.21	0.000000	0.46	-2711	1.16	8638.00	92.94	2100	170	0.5000	12.0000	170	341	329	0.96
13	Exterior	0.8378	0.3531	0.3976	2.11	0.001076	1.85	-1695	1.02	45060.00	33.00	3500	4020	12.0000	12.0000	1452	5472	1330	0.24
	Interior	0.8333	0.3381	0.3976	2.10	0.001076	1.88	-2280	1.03	45060.00	33.00	3500	4070	12.0000	12.0000	1452	5522	1330	0.24
14	Exterior	0.6887	0.3573	0.3328	2.67	0.000000	1.00	-1067	1.08	6970.60	83.49	2100	275	1.2500	12.0000	383	658	128	0.19
	Interior	0.6837	0.3520	0.3201	2.76	0.000000	0.91	-1422	1.15	4872.00	69.80	2100	186	1.0000	12.0000	258	442	98	0.22
15	Exterior	0.8234	0.3451	0.6036	1.36	0.001076	2.59	-382	1.00	41190.00	33.00	3500	5076	12.0000	12.0000	1452	6530	654	0.10
	Interior	0.8932	0.3535	0.6036	1.48	0.001076	2.48	-509	1.01	41190.00	33.00	3500	4860	12.0000	12.0000	1452	6312	654	0.10
16	Exterior	0.5933	0.2647	0.2000	2.97	0.001076	0.99	-1453	1.06	11530.00	107.38	3500	575	1.8600	12.0000	732	1307	538	0.41
	Interior																		
17	Exterior	0.7094	0.2627	0.5736	1.24	0.001076	2.72	-569	1.01	35852.00	33.00	3500	4653	11.2000	12.0000	1355	6008	864	0.14
	Interior	0.7205	0.2878	0.5736	1.26	0.001076	2.70	-759	1.01	35852.00	33.00	3500	4632	11.2000	12.0000	1355	5987	864	0.14
18	Exterior	0.6729	0.2767	0.5851	1.15	0.000000	2.52	-436	1.07	2935.10	54.16	3500	376	0.7031	12.0000	140	516	73	0.14
	Interior	0.6757	0.2776	0.6030	1.12	0.000000	2.55	-581	1.08	3528.00	59.40	3500	461	0.5625	12.0000	123	583	60	0.10
19	Exterior	0.6339	0.2723	0.5933	1.07	0.001076	2.89	-561	1.01	35852.00	33.00	3500	4940	11.2000	12.0000	1355	6295	849	0.13
	Interior	0.6318	0.2704	0.5933	1.06	0.001076	2.89	-749	1.01	35852.00	33.00	3500	4960	11.2000	12.0000	1355	6315	849	0.13
20	Exterior	0.5478	0.2582	0.5342	1.03	0.000000	2.64	-946	1.16	2935.10	54.18	2100	330	0.7031	12.0000	140	470	76	0.16
	Interior	0.5406	0.2462	0.5506	0.98	0.000000	2.69	-1261	1.18	3528.00	59.40	2100	410	0.5625	12.0000	123	532	63	0.12
21	Exterior	0.4076	0.4601	0.2999	1.53	0.001076	2.42	-975	1.01	34224.00	33.00	3500	3981	11.8000	12.0000	1404	5364	607	0.11
	Interior	0.4869	0.2217	0.2999	1.62	0.001076	2.33	-1300	1.02	34224.00	33.00	3500	3852	11.6000	12.0000	1404	5255	607	0.12
22	Exterior	1.0445	1.0359	0.4644	2.25	0.001076	1.71	-1023	1.08	6772.00	82.29	3500	589	1.2400	12.0000	374	963	89	0.09
	Interior	0.5941	0.4331	0.3013	1.97	0.001076	1.99	-1364	1.10	6820.00	82.58	3500	705	1.2400	12.0000	375	1080	151	0.14
23	Exterior	0.4832	0.3369	0.5835	0.83	0.001076	3.13	-407	1.00	48238.00	33.00	2100	5557	11.6000	12.0000	1404	6961	1128	0.16
	Interior	0.3404	0.2124	0.5835	0.58	0.001076	3.37	-407	1.00	48238.00	33.00	3500	7735	11.6000	12.0000	1404	9139	1128	0.12
24	Exterior	0.3708	0.2500	0.3158	1.17	0.001076	2.78	-424	1.01	34224.00	33.00	2100	3514	11.6000	12.0000	1404	4917	577	0.12
	Interior	0.2644	0.1685	0.4433	0.60	0.001076	3.36	-565	1.01	34224.00	33.00	2100	4252	11.6000	12.0000	1404	5655	577	0.10
25	Exterior	0.2721	0.1825	0.7049	0.39	0.000000	3.28	-989	1.18	2699.20	51.95	3500	496	1.0000	18.0000	127	623	67	0.11
	Interior	0.2767	0.2162	0.6132	0.45	0.000000	3.22	-1319	1.16	4176.00	64.62	3500	737	1.0000	18.0000	158	895	49	0.06
26	Exterior	0.3204	0.3024	0.7085	0.45	0.000000	3.22	-825	1.15	2699.20	51.95	3500	474	1.0000	18.0000	127	601	67	0.11
	Interior	0.3246	0.2414	0.6163	0.53	0.000000	3.14	-1100	1.19	2923.20	54.07	3500	517	1.0000	18.0000	132	649	49	0.08
27	Exterior	0.4122	0.2893	0.4567	0.90	0.001076	3.05	-1080	1.01	37011.00	33.00	3500	5429	11.6000	12.0000	1404	6832	765	0.11
	Interior	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
28	Exterior	0.4867	0.3333	0.8188	0.59	0.000000	3.08	-778	1.14	2845.00	53.34	3500	471	0.5625	12.0000	110	581	49	0.08
	Interior	0.3776	0.2326	0.8275	0.46	0.000000	3.21	-1038	1.17	3136.00	56.00	3500	556	0.5625	12.0000	116	671	58	0.09
29	Exterior	0.4893	0.3571	0.4501	1.09	0.000000	2.58	-818	1.03	11809.00	108.67	3500	1494	0.5625	18.0000	149	1643	212	0.13
	Interior	0.4065	0.2500	0.5812	0.70	0.000000	2.97	-1091	1.08	7200.00	84.85	3500	1089	0.5625	18.0000	117	1206	154	0.13
30	Exterior	0.7547	0.6387	0.4090	1.85	0.000000	1.82	-524	1.03	7639.80	87.41	3500	682	0.5625	18.0000	120	803	320	0.40
	Interior	0.6623	0.5325	0.4345	1.52	0.000000	2.15	-698	1.05	7015.40	83.76	3500	748	0.5625	18.0000	115	863	247	0.29
31	Exterior	0.6298	0.3651	0.6472	0.97	0.001076	2.98	-834	1.01	30255.00	33.00	3500	4332	12.4000	12.0000	1500	5832	541	0.09
	Interior	0.6294	0.3965	0.7917	0.79	0.001076	3.16	-1112	1.02	30255.00	33.00	3500	4611	12.5000	12.0000	1513	6124	541	0.09
32	Exterior	0.6527	0.4174	0.4512	1.45	0.000000	2.22	-668	1.11	2950.50	54.32	3500	346	0.5625	18.0000	75	420	104	0.25
	Interior	0.6655	0.3968	0.8110	0.82	0.000000	2.85	-891	1.14	3096.00	55.64	3500	478	0.5625	18.0000	77	554	135	0.24
33	Exterior	0.6087	0.3715	0.4187	1.45	0.000000	2.22	-1117	1.05	10210.00	101.04	3500	1129	0.5625	18.0000	139	1268	280	0.22
	Interior	0.6591	0.3974	0.4319	1.53	0.000000	2.14	-1490	1.08	9508.00	97.51	3500	1040	0.5625	18.0000	134	1175	211	0.18
34	Exterior	0.5965	0.3836	0.6884	0.87	0.001076	3.09	-519	1.01	31652.00	33.00	3500	4668	11.2000	12.0000	1355	6023	525	0.09
	Interior	0.6242	0.3800	0.8422	0.74	0.001076	3.22	-693	1.01	31652.00	33.00	3500	4871	11.2000	12.0000	1355	6226	525	0.08
35	Exterior	0.8905	0.5675	0.4684	1.90	0.000000	1.77	-756	1.13	2950.50	54.32	3500	279	0.5625	18.0000	75	353	51	0.14
	Interior	0.7536	0.4738	0.8420	0.90	0.000000	2.77	-1008	1.16	3096.00	55.64	3500	473	0.5625	18.0000	77	549	66	0.12
36	Exterior	0.7406	0.4602	0.4496	1.65	0.000000	2.02	-1073	1.13	4215.00	64.92	3500	455	0.5625	18.0000	89	544	52	0.10
	Interior	0.7831	0.4959	0.8081	0.97	0.000000	2.70	-1431	1.23	3096.00	55.64	3500	487	0.5625	18.0000	77	564	68	0.12
37	Exterior	0.6104	0.3747	[illegible]	[illegible]	0.001076	1.91	[illegible]	1.00	[illegible]	33.00	3500	[illegible]	11.2000	12.0000	[illegible]	[illegible]	[illegible]	[illegible]
	Interior	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.01	[illegible]	33.00	3500	[illegible]	11.2000	12.0000	1355	5334	45	0.14

6th Street Bridge Steel Casing Alternative Column Shear D/C Table (Only cased for EQ Requirement not all ASR columns)
Transverse Direction

Pier #	COLUMN	Disp-eql	Disp-eqt	[Disp] y-ft	(D/Y)d	phro-s	Fac-1	P-DL	Fac-2	Ag (in^2)	d (in)	fc'	[Vc]	As (in^2)	s (in)	Vs	[V] tot	Veq-T	(D/C)v
1	Exterior	0.0059	0.0156	0.0332	0.47	0.000212	3.26	-667	1.11	2976.00	54.55	3500	510	0.2500	12.0000	50	560	124	0.22
	Interior	0.0073	0.0161	0.0332	0.48	0.000212	3.24	-889	1.15	2976.00	54.55	3500	525	0.2500	12.0000	50	575	124	0.22
2	Exterior	0.0069	0.0129	0.0506	0.25	0.000212	3.47	-629	1.11	2976.00	54.55	3500	541	0.2500	12.0000	50	591	100	0.17
	Interior	0.0071	0.0160	0.0506	0.32	0.000212	3.41	-839	1.14	2976.00	54.55	3500	548	0.2500	12.0000	50	598	100	0.17
3	Exterior	0.0045	0.0073	0.0162	0.45	0.001076	3.50	-565	1.01	33530.00	636.00	2100	4344	0.8800	4.0000	6156	10501	9833	0.94
	Interior	0.0052	0.0090	0.0162	0.55	0.001076	3.40	-754	1.01	33530.00	636.00	2100	4232	0.8800	4.0000	6156	10388	9833	0.95
4	Exterior	0.0224	0.0468	0.0702	0.67	0.000000	3.00	-952	1.06	7740.00	87.98	3500	1168	0.2500	12.0000	81	1249	334	0.27
	Interior	0.0233	0.0462	0.0962	0.48	0.000000	3.19	-1269	1.11	5822.00	76.30	3500	975	0.2500	12.0000	70	1045	193	0.18
5	Exterior	0.0482	0.0943	0.0803	1.17	0.000000	2.50	-945	1.06	7740.00	87.98	3500	970	0.2500	12.0000	81	1051	312	0.30
	Interior	0.0471	0.0930	0.1100	0.85	0.000000	2.82	-1260	1.11	5822.00	76.30	3500	863	0.2500	12.0000	70	933	180	0.19
6	Exterior	0.0542	0.1076	0.1010	1.07	0.000000	2.60	-667	1.04	8232.00	90.73	3500	1056	0.3910	12.0000	130	1186	353	0.30
	Interior	0.0517	0.1016	0.1544	0.66	0.000000	3.01	-889	1.09	4778.00	69.12	3500	744	0.3910	12.0000	99	844	149	0.18
7	Exterior	0.0296	0.0577	0.0435	1.32	0.001076	2.63	-681	1.01	25390.00	576.00	3500	3206	0.8800	12.0000	1859	5065	3991	0.79
	Interior	0.0317	0.0618	0.0435	1.42	0.001076	2.54	-908	1.02	25390.00	576.00	3500	3106	0.8800	12.0000	1859	4964	3991	0.80
8	Exterior	0.0211	0.0389	0.0946	0.41	0.000000	3.26	-731	1.09	4248.00	65.18	3500	712	0.3910	12.0000	93	805	83	0.10
	Interior	0.0214	0.0395	0.1048	0.38	0.000000	3.29	-974	1.09	5400.00	73.48	3500	918	0.3910	12.0000	105	1023	90	0.09
9	Exterior	0.0317	0.0589	0.0566	1.04	0.001076	2.92	-596	1.01	37870.00	576.00	3500	5271	0.8800	6.0000	3717	8988	7629	0.85
	Interior	0.0331	0.0615	0.0566	1.09	0.001076	2.87	-795	1.01	37870.00	576.00	3500	5200	0.8800	6.0000	3717	8917	7629	0.86
10	Exterior	0.0514	0.0884	0.3001	0.29	0.000000	3.38	-1537	1.09	8177.00	90.43	2100	1107	0.7030	12.0000	233	1340	129	0.10
	Interior	0.0511	0.0843	0.3531	0.24	0.000000	3.43	-2049	1.14	7490.00	86.54	2100	1071	0.5625	12.0000	178	1250	94	0.08
11	Exterior	0.0787	0.0814	0.0617	1.32	0.001076	2.64	-796	1.01	43270.00	672.00	3500	5451	0.8800	6.0000	4337	9788	9336	0.95
	Interior	0.0789	0.0839	0.0617	1.36	0.001076	2.60	-1061	1.01	43270.00	672.00	3500	5385	0.8800	6.0000	4337	9722	9336	0.96
12	Exterior	0.5972	0.6640	0.2216	3.00	0.000000	0.67	-2033	1.07	13750.00	117.26	2100	364	0.7500	12.0000	322	687	396	0.58
	Interior	0.5915	0.6579	0.2390	2.75	0.000000	0.92	-2711	1.11	12340.00	111.09	2100	461	0.5000	12.0000	204	664	390	0.59
13	Exterior	0.0497	0.0597	0.0596	1.00	0.001076	2.96	-1695	1.02	45060.00	756.00	3500	6421	0.8800	4.0000	7318	13739	11756	0.86
	Interior	0.0477	0.0580	0.0596	0.97	0.001076	2.98	-2260	1.03	45060.00	756.00	3500	6524	0.8800	4.0000	7318	13842	11756	0.85
14	Exterior	0.1986	0.2440	0.2518	0.97	0.000000	2.70	-1067	1.05	9958.00	99.79	2100	1039	1.2500	12.0000	457	1496	166	0.11
	Interior	0.1698	0.2001	0.4136	0.48	0.000000	3.19	-1422	1.10	6960.00	83.43	2100	896	1.0000	12.0000	306	1202	74	0.06
15	Exterior	0.0572	0.0924	0.0729	1.27	0.001076	2.69	-382	1.00	41190.00	720.00	3500	5269	0.8800	6.0000	4646	9916	7682	0.77
	Interior	0.0577	0.0929	0.0729	1.27	0.001076	2.68	-509	1.01	41190.00	720.00	3500	5263	0.8800	6.0000	4646	9909	7682	0.78
16	Exterior	0.2932	0.2336	0.1340	2.19	0.001076	1.77	-1453	1.06	11530.00	107.38	3500	1027	0.3100	3.0000	488	1515	693	0.46
	Interior	0.3081	0.2391	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
17	Exterior	0.0278	0.0552	0.0697	0.79	0.001076	3.17	-569	1.01	35852.00	672.00	3500	5414	0.8800	12.0000	2168	7582	6485	0.86
	Interior	0.0277	0.0557	0.0697	0.80	0.001076	3.16	-759	1.01	35852.00	672.00	3500	5417	0.8800	12.0000	2168	7586	6485	0.85
18	Exterior	0.0305	0.0696	0.4671	0.15	0.000000	3.52	-436	1.05	4193.00	64.75	3500	735	0.7031	12.0000	167	902	94	0.10
	Interior	0.0275	0.0643	0.5897	0.11	0.000000	3.56	-581	1.08	3528.00	59.40	3500	644	0.5625	12.0000	123	766	72	0.09
19	Exterior	0.0286	0.0516	0.0718	0.72	0.001076	3.24	-561	1.01	35852.00	672.00	3500	5539	0.8800	12.0000	2168	7708	6390	0.83
	Interior	0.0290	0.0521	0.0718	0.73	0.001076	3.23	-749	1.01	35852.00	672.00	3500	5541	0.8800	12.0000	2168	7709	6390	0.83
20	Exterior	0.1026	0.1596	0.4265	0.37	0.000000	3.30	-946	1.11	4193.00	64.75	2100	564	0.7031	12.0000	167	731	99	0.14
	Interior	0.0955	0.1516	0.5384	0.28	0.000000	3.39	-1261	1.18	3528.00	59.40	2100	517	0.5625	12.0000	123	639	76	0.12
21	Exterior	0.0331	0.0553	0.0594	0.93	0.001076	3.03	-975	1.01	34224.00	696.00	3500	4973	0.8800	12.0000	2246	7219	4916	0.68
	Interior	0.0308	0.0533	0.0594	0.90	0.001076	3.06	-1300	1.02	34224.00	696.00	3500	5050	0.8800	12.0000	2246	7296	4916	0.67
22	Exterior	0.2869	0.2442	0.3995	0.72	0.001076	3.24	-1023	1.08	6772.00	840.00	3500	1117	0.6200	3.0000	7638	8755	250	0.03
	Interior	0.2231	0.2538	0.4160	0.61	0.001076	3.35	-1364	1.10	6820.00	840.00	3500	1188	0.6200	3.0000	7638	8827	303	0.03
23	Exterior	0.0183	0.0265	0.0971	0.27	0.001076	3.68	-407	1.00	48238.00	696.00	2100	6543	0.8800	4.0000	6737	13280	11312	0.85
	Interior	0.0245	0.0257	0.0971	0.27	0.001076	3.69	-407	1.00	48238.00	696.00	3500	8465	0.8800	4.0000	6737	15202	11312	0.74
24	Exterior	0.0507	0.0637	0.0978	0.65	0.001076	3.31	-424	1.01	34224.00	696.00	2100	4173	0.8800	6.0000	4492	8665	4705	0.54
	Interior	0.0544	0.0658	0.0978	0.67	0.001076	3.28	-565	1.01	34224.00	696.00	2100	4155	0.8800	6.0000	4492	8646	4705	0.54
25	Exterior	0.0945	0.3032	0.4581	0.66	0.000000	3.01	-989	1.13	3856.00	62.10	3500	619	1.0000	18.0000	152	771	90	0.12
	Interior	0.0926	0.2966	0.4591	0.65	0.000000	3.02	-1319	1.16	4176.00	64.62	3500	692	1.0000	18.0000	158	850	71	0.08
26	Exterior	0.1062	0.3027	0.4604	0.66	0.000000	3.01	-825	1.11	3856.00	62.10	3500	609	1.0000	18.0000	152	760	90	0.12
	Interior	0.0992	0.2805	0.4614	0.61	0.000000	3.06	-1100	1.13	4176.00	64.62	3500	685	1.0000	18.0000	158	843	70	0.08
27	Exterior	0.0280	0.0639	0.0720	0.89	0.001076	3.07	-1080	1.01	37011.00	696.00	3500	5454	0.8800	12.0000	2246	7700	5831	0.76
	Interior	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	-	-
28	Exterior	0.2120	0.1770	0.7517	0.28	0.000000	3.39	-778	1.14	2845.00	53.34	3500	519	0.5625	12.0000	110	629	55	0.09
	Interior	0.2119	0.1771	0.8275	0.26	0.000000	3.41	-1038	1.17	3136.00	56.00	3500	591	0.5625	12.0000	116	706	63	0.09
29	Exterior	0.5486	0.4705	0.1482	3.70	0.000000	0.30	-818	1.03	11809.00	108.67	3500	173	0.5625	18.0000	149	323	226	0.70
	Interior	0.5785	0.4955	0.2089	2.77	0.000000	0.90	-1091	1.08	7200.00	84.85	3500	330	0.5625	18.0000	117	447	203	0.45
30	Exterior	0.3224	0.2427	0.1711	1.88	0.000000	1.79	-524	1.02	10914.00	104.47	3500	945	0.5625	18.0000	144	1089	303	0.28
	Interior	0.3420	0.2645	0.1774	1.93	0.000000	1.74	-698	1.03	10022.00	100.11	3500	855	0.5625	18.0000	138	993	278	0.28
31	Exterior	0.0465	0.0571	0.1096	0.52	0.001076	3.44	-834	1.01	30255.00	696.00	3500	4989	0.8800	12.0000	2246	7234	4444	0.61
	Interior	0.0461	0.0599	0.1096	0.55	0.001076	3.41	-1112	1.02	30255.00	696.00	3500	4974	0.8800	12.0000	2246	7220	4444	0.62
32	Exterior	0.1554	0.2017	0.1874	1.08	0.000000	2.59	-668	1.08	4215.00	64.92	3500	558	0.5625	18.0000	89	648	57	0.09
	Interior	0.1569	0.2054	0.3833	0.54	0.000000	3.13	-891	1.14	3096.00	55.64	3500	525	0.5625	18.0000	77	602	78	0.13
33	Exterior	0.1713	0.2267	0.1612	1.41	0.000000	2.26	-1117	1.05	10210.00	101.04	3500	1154	0.5025	18.0000	139	1293	217	0.17
	Interior	0.1682	0.2250	0.1654	1.36	0.000000	2.31	-1490	1.08	9508.00	97.51	3500	1121	0.5625	18.0000	134	1255	131	0.10
34	Exterior	0.0436	0.0437	0.1158	0.38	0.001076	3.58	-519	1.01	31652.00	756.00	3500	5406	0.8800	12.0000	2439	7846	4324	0.55
	Interior	0.0438	0.0394	0.1158	0.38	0.001076	3.58	-693	1.01	31652.00	756.00	3500	5420	0.8800	12.0000	2439	7859	4324	0.55
35	Exterior	0.3850	0.2757	0.3891	0.99	0.000000	2.68	-756	1.09	4215.00	64.92	3500	583	0.5625	18.0000	89	672	56	0.08
	Interior	0.3916	0.2807	0.7960	0.49	0.000000	3.18	-1008	1.16	3096.00	55.64	3500	542	0.5625	18.0000	77	618	76	0.12
36	Exterior	0.4546	0.3259	0.3735	1.22	0.000000	2.45	-1073	1.13	4215.00	64.92	3500	552	0.5625	18.0000	89	641	57	0.09
	Interior	0.4395	0.3153	0.7640	0.58	0.000000	3.09	-1431	1.23	3096.00	55.64	3500	558	0.5625	18.0000	77	635	78	0.12
37	Exterior	0.0464	0.0312	0.1138	0.41	0.001076	3.55	-340	1.00	34392.00	756.00	3500	5806	0.8800	12.0000	2439	8245	5693	0.69
	Interior	0.0462	0.0310	0.1138	0.41	0.001076	3.55	[illegible]	1.01	34392.00	756.00	3500	5719	0.8800	12.0000	[illegible]	8252	5693	0.59

6th Street Bridge Steel Casing Alternative Column Shear D/C Table (Only cased for EQ Requirement not all ASR columns)
Longitudinal Direction

Pier #	COLUMN	Disp-eql	Disp-eqt	(Disp-L) y	(D/Y)d	phro-s	Fac-1	P-DL	Fac-2	Ag (in^2)	d (in)	fc'	[Vc]	As (in^2)	s (in)	Vs	[V] tot	Veq-L	(D/C)v
1	Exterior	0.1874	0.0977	0.0366	5.11	0.000212	0.30	-667	1.11	2976.00	54.55	3500	47	0.2500	12.0000	50	97	115	1.19
	Interior	0.2531	0.1276	0.0366	6.91	0.000212	0.30	-889	1.15	2976.00	54.55	3500	49	0.2500	12.0000	50	99	115	1.17
2	Exterior	0.2019	0.1054	0.0559	3.62	0.000212	0.30	-629	1.11	2976.00	54.55	3500	47	0.2500	12.0000	50	97	93	0.97
	Interior	0.1825	0.0950	0.0559	3.27	0.000212	0.46	-839	1.14	2976.00	54.55	3500	74	0.2500	12.0000	50	124	93	0.75
3	Exterior	0.3955	0.2068	0.0688	5.75	0.001076	0.30	-565	1.01	33530.00	33.00	2100	372	10.0000	12.0000	1210	1582	1500	0.95
	Interior	0.3878	0.2026	0.0688	5.64	0.001076	0.30	-754	1.01	33530.00	33.00	2100	373	10.0000	12.0000	1210	1583	1500	0.95
4	Exterior	0.3565	0.1874	0.0577	6.18	0.000000	0.30	-952	1.06	7740.00	87.98	3500	117	0.2500	12.0000	81	197	376	1.90
	Interior	0.2864	0.1496	0.0613	4.67	0.000000	0.30	-1269	1.11	5822.00	76.30	3500	92	0.2500	12.0000	70	162	271	1.68
5	Exterior	0.3931	0.2056	0.0660	5.96	0.000000	0.30	-945	1.06	7740.00	87.98	3500	117	0.2500	12.0000	81	197	351	1.78
	Interior	0.3143	0.1639	0.0701	4.48	0.000000	0.30	-1260	1.11	5822.00	76.30	3500	92	0.2500	12.0000	70	162	253	1.57
6	Exterior	0.4720	0.2478	0.0985	4.79	0.000000	0.30	-667	1.04	8232.00	90.73	3500	122	0.3910	12.0000	130	252	325	1.29
	Interior	0.4582	0.2394	0.1207	3.80	0.000000	0.30	-889	1.09	4778.00	69.12	3500	74	0.3910	12.0000	99	173	161	0.93
7	Exterior	0.4918	0.2579	0.3041	1.62	0.001076	2.34	-681	1.01	25390.00	33.00	3500	2850	9.6000	12.0000	1162	4011	355	0.09
	Interior	0.4939	0.2581	0.3041	1.62	0.001076	2.33	-908	1.02	25390.00	33.00	3500	2853	9.6000	12.0000	1162	4015	355	0.09
8	Exterior	0.5908	0.3087	0.2227	2.65	0.000000	1.02	-731	1.09	4248.00	65.18	3500	222	0.3910	12.0000	93	315	60	0.19
	Interior	0.6579	0.3438	0.2016	3.26	0.000000	0.41	-974	1.09	5400.00	73.48	3500	113	0.3910	12.0000	105	219	78	0.36
9	Exterior	0.6486	0.3390	0.3432	1.89	0.001076	2.07	-596	1.01	37870.00	33.00	3500	3735	9.6000	12.0000	1162	4897	1044	0.21
	Interior	0.6658	0.3479	0.3432	1.94	0.001076	2.02	-795	1.01	37870.00	33.00	3500	3654	9.6000	12.0000	1162	4816	1044	0.22
10	Exterior	0.4989	0.2604	0.2207	2.26	0.000000	1.41	-1537	1.09	8177.00	90.43	2100	482	0.7030	12.0000	233	695	167	0.24
	Interior	0.4270	0.2231	0.2064	2.07	0.000000	1.60	-2049	1.14	7490.00	86.54	2100	500	0.5625	12.0000	178	678	153	0.23
11	Exterior	0.6633	0.3472	0.3701	1.79	0.001076	2.17	-796	1.01	43270.00	33.00	3500	4475	11.2000	12.0000	1355	5830	1279	0.22
	Interior	0.6568	0.3432	0.3701	1.77	0.001076	2.18	-1061	1.01	43270.00	33.00	3500	4525	11.2000	12.0000	1355	5880	1279	0.22
12	Exterior	0.9792	0.5214	0.2839	3.45	0.000000	0.30	-2033	1.07	13750.00	117.26	2100	162	0.7500	12.0000	322	485	267	0.55
	Interior	0.9503	0.5328	0.2965	3.21	0.000000	0.46	-2711	1.11	12340.00	111.09	2100	233	0.5000	12.0000	204	437	329	0.75
13	Exterior	0.8378	0.3531	0.3976	2.11	0.001076	1.85	-1695	1.02	45060.00	33.00	3500	4020	12.0000	12.0000	1452	5472	1330	0.24
	Interior	0.8333	0.3381	0.3976	2.10	0.001076	1.86	-2260	1.03	45060.00	33.00	3500	4070	12.0000	12.0000	1452	5522	1330	0.24
14	Exterior	0.8887	0.3573	0.3328	2.67	0.000000	1.00	-1067	1.05	9958.00	99.79	2100	384	1.2500	12.0000	457	842	126	0.15
	Interior	0.8837	0.3520	0.3201	2.76	0.000000	0.91	-1422	1.10	6960.00	83.43	2100	256	1.0000	12.0000	306	562	98	0.17
15	Exterior	0.8234	0.3451	0.6036	1.36	0.001076	2.59	-382	1.00	41190.00	33.00	3500	5078	12.0000	12.0000	1452	6530	654	0.10
	Interior	0.8932	0.3535	0.6036	1.48	0.001076	2.48	-509	1.01	41190.00	33.00	3500	4860	12.0000	12.0000	1452	6312	654	0.10
16	Exterior	0.5933	0.2647	0.2000	2.97	0.001076	0.99	-1453	1.06	11530.00	107.38	3500	575	1.8600	12.0000	732	1307	538	0.41
	Interior																		
17	Exterior	0.7094	0.2827	0.5736	1.24	0.001076	2.72	-569	1.01	35852.00	33.00	3500	4653	11.2000	12.0000	1355	6008	864	0.14
	Interior	0.7205	0.2878	0.5736	1.26	0.001076	2.70	-759	1.01	35852.00	33.00	3500	4632	11.2000	12.0000	1355	5987	864	0.14
18	Exterior	0.6729	0.2767	0.5851	1.15	0.000000	2.52	-436	1.05	4193.00	64.75	3500	526	0.7031	12.0000	167	693	73	0.11
	Interior	0.6757	0.2776	0.6030	1.12	0.000000	2.55	-581	1.08	3528.00	59.40	3500	461	0.5625	12.0000	123	583	60	0.10
19	Exterior	0.6339	0.2723	0.5933	1.07	0.001076	2.89	-561	1.01	35852.00	33.00	3500	4940	11.2000	12.0000	1355	6295	849	0.13
	Interior	0.6316	0.2704	0.5933	1.06	0.001076	2.89	-749	1.01	35852.00	33.00	3500	4960	11.2000	12.0000	1355	6315	849	0.13
20	Exterior	0.5478	0.2562	0.5342	1.03	0.000000	2.64	-946	1.11	4193.00	64.75	2100	452	0.7031	12.0000	167	619	76	0.12
	Interior	0.5406	0.2462	0.5506	0.98	0.000000	2.69	-1261	1.18	3528.00	59.40	2100	410	0.5625	12.0000	123	532	63	0.12
21	Exterior	0.4076	0.4601	0.2999	1.53	0.001076	2.42	-975	1.01	34224.00	33.00	3500	3981	11.6000	12.0000	1404	5384	607	0.11
	Interior	0.4869	0.2217	0.2999	1.62	0.001076	2.33	-1300	1.02	34224.00	33.00	3500	3852	11.6000	12.0000	1404	5255	607	0.12
22	Exterior	1.0445	1.0359	0.4644	2.25	0.001076	1.71	-1023	1.08	6772.00	82.29	3500	589	1.2400	12.0000	374	963	89	0.09
	Interior	0.5941	0.4331	0.3013	1.97	0.001076	1.99	-1364	1.10	6820.00	82.58	3500	705	1.2400	12.0000	375	1080	151	0.14
23	Exterior	0.4832	0.3369	0.5835	0.83	0.001076	3.13	-407	1.00	48238.00	33.00	2100	5557	11.6000	12.0000	1404	6961	1128	0.16
	Interior	0.3404	0.2124	0.5835	0.58	0.001076	3.37	-407	1.00	48238.00	33.00	3500	7735	11.6000	12.0000	1404	9139	1128	0.12
24	Exterior	0.3708	0.2500	0.3158	1.17	0.001076	2.78	-424	1.01	34224.00	33.00	2100	3514	11.6000	12.0000	1404	4917	577	0.12
	Interior	0.2644	0.1685	0.4433	0.60	0.001076	3.36	-565	1.01	34224.00	33.00	2100	4252	11.6000	12.0000	1404	5655	577	0.10
25	Exterior	0.2721	0.1825	0.7049	0.39	0.000000	3.28	-989	1.13	3856.00	62.10	3500	676	1.0000	18.0000	152	828	67	0.08
	Interior	0.2767	0.2162	0.6132	0.45	0.000000	3.22	-1319	1.16	4176.00	64.62	3500	737	1.0000	18.0000	158	895	49	0.06
26	Exterior	0.3204	0.3024	0.7085	0.45	0.000000	3.22	-625	1.11	3856.00	62.10	3500	650	1.0000	18.0000	152	802	67	0.08
	Interior	0.3246	0.2414	0.6163	0.53	0.000000	3.14	-1100	1.13	4176.00	64.62	3500	703	1.0000	18.0000	158	861	49	0.06
27	Exterior	0.4122	0.2893	0.4567	0.90	0.001076	3.05	-1080	1.01	37011.00	33.00	3500	5429	11.6000	12.0000	1404	6832	765	0.11
	Interior																		
28	Exterior	0.4867	0.3333	0.8188	0.59	0.000000	3.08	-778	1.14	2845.00	53.34	3500	471	0.5625	12.0000	110	581	49	0.08
	Interior	0.3776	0.2326	0.8275	0.46	0.000000	3.21	-1038	1.17	3136.00	56.00	3500	556	0.5625	12.0000	116	671	58	0.09
29	Exterior	0.4893	0.3571	0.4501	1.09	0.000000	2.58	-818	1.03	11809.00	108.67	3500	1494	0.5625	18.0000	149	1643	212	0.13
	Interior	0.4065	0.2500	0.5812	0.70	0.000000	2.97	-1091	1.08	7200.00	84.85	3500	1089	0.5625	18.0000	117	1206	154	0.13
30	Exterior	0.7547	0.6387	0.4090	1.85	0.000000	1.82	-524	1.02	10914.00	104.47	3500	965	0.5625	18.0000	144	1109	320	0.29
	Interior	0.6623	0.5325	0.4345	1.52	0.000000	2.15	-698	1.03	10022.00	100.11	3500	1053	0.5625	18.0000	138	1191	247	0.21
31	Exterior	0.6298	0.3651	0.6472	0.97	0.001076	2.98	-834	1.01	30255.00	33.00	3500	4332	12.4000	12.0000	1500	5832	541	0.09
	Interior	0.6294	0.3965	0.7917	0.79	0.001076	3.16	-1112	1.02	30255.00	33.00	3500	4611	12.5000	12.0000	1513	6124	541	0.09
32	Exterior	0.6527	0.4174	0.4512	1.45	0.000000	2.22	-668	1.08	4215.00	64.92	3500	479	0.5625	18.0000	89	568	104	0.18
	Interior	0.6655	0.3968	0.8110	0.82	0.000000	2.85	-891	1.14	3096.00	55.64	3500	478	0.5625	18.0000	77	554	135	0.24
33	Exterior	0.6087	0.3715	0.4187	1.45	0.000000	2.22	-1117	1.05	10210.00	101.04	3500	1129	0.5625	18.0000	139	1268	280	0.22
	Interior	0.6591	0.3974	0.4319	1.53	0.000000	2.14	-1490	1.08	9508.00	97.51	3500	1040	0.5625	18.0000	134	1175	211	0.18
34	Exterior	0.5965	0.3836	0.6884	0.87	0.001076	3.09	-519	1.01	31652.00	33.00	3500	4668	11.2000	12.0000	1355	6023	525	0.09
	Interior	0.6242	0.3800	0.8422	0.74	0.001076	3.22	-693	1.01	31652.00	33.00	3500	4871	11.2000	12.0000	1355	6226	525	0.08
35	Exterior	0.8905	0.5675	0.4684	1.90	0.000000	1.77	-756	1.09	4215.00	64.92	3500	385	0.5625	18.0000	89	474	51	0.11
	Interior	0.7536	0.4738	0.8420	0.90	0.000000	2.77	-1008	1.16	3096.00	55.64	3500	473	0.5625	18.0000	77	549	66	0.12
36	Exterior	0.7406	0.4602	0.4496	1.65	0.000000	2.02	-1073	1.13	4215.00	64.92	3500	455	0.5625	18.0000	89	544	52	0.10
	Interior	0.7831	0.4959	0.8081	0.97	0.000000	2.70	-1431	1.23	3096.00	55.64	3500	487	0.5625	18.0000	77	564	68	0.12
37	Exterior	0.6104	0.3747	0.2987	2.04	0.001076	1.91	-340	1.00	34392.00	33.00	3500	3130	11.2000	12.0000	1355	4486	745	0.17
	Interior	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

6th Street Bridge Column Ultimate Displacement Capacity (Relative) for steel-casing option
Transverse Direction

Pier #	COLUMN	Phi-y (1/ft)	[Disp] y-ft	L-eff. (ft)	Phi-p (1/ft)	Lp (ft)	Theta-p	[Disp]-p (ft)	L (FT)	Du/Dy	Mp (k-')	Vp (kip)
1	Exterior	0.0005119	0.0332	6.975	0.010882	1.4040	0.015278	0.2131	21.95	6.42	1726	124
	Interior	0.0005119	0.0332	6.975	0.015160	1.4040	0.021285	0.2969		8.94	1726	124
2	Exterior	0.0005119	0.0506	8.612	0.010882	1.5350	0.016703	0.2877	25.22	5.68	1726	100
	Interior	0.0005119	0.0506	8.612	0.015160	1.5350	0.023270	0.4008		7.92	1726	100
3	Exterior	0.0000491	0.0162	31.444	0.001668	3.3615	0.005608	0.1763	30.21	10.89	309200	9833
	Interior	0.0000491	0.0162	31.444	0.001668	3.3615	0.005607	0.1763		10.89	309200	9833
4	Exterior	0.0003712	0.0702	11.911	0.005952	1.7989	0.010707	0.2551	31.82	3.63	7950	334
	Interior	0.0005087	0.0962	11.911	0.007241	1.7989	0.013026	0.3103		3.22	4587	193
5	Exterior	0.0003712	0.0803	12.736	0.005952	1.8649	0.011100	0.2827	33.47	3.52	7950	312
	Interior	0.0005087	0.1100	12.736	0.007241	1.8649	0.013503	0.3440		3.13	4587	180
6	Exterior	0.0003426	0.1010	14.873	0.005268	2.0358	0.010725	0.3190	37.75	3.16	10500	353
	Interior	0.0005234	0.1544	14.873	0.006571	2.0358	0.013377	0.3979		2.58	4427	149
7	Exterior	0.0000679	0.0435	43.850	0.000887	4.3540	0.003863	0.1694	41.38	3.89	175000	3991
	Interior	0.0000679	0.0435	43.850	0.000887	4.3540	0.003863	0.1694		3.89	175000	3991
8	Exterior	0.0004242	0.0946	18.292	0.012696	2.3094	0.029320	1.0726	44.58	11.34	3022	83
	Interior	0.0004696	0.1048	18.292	0.011897	2.3094	0.027474	1.0051		9.60	3295	90
9	Exterior	0.0000607	0.0566	52.900	0.001705	5.0780	0.008655	0.4579	52.90	8.09	403600	7629
	Interior	0.0000607	0.0566	52.900	0.001705	5.0780	0.008655	0.4579		8.09	403600	7629
10	Exterior	0.0003859	0.3001	24.151	0.010030	2.7781	0.027864	1.3459	56.30	4.48	6237	129
	Interior	0.0004540	0.3531	24.151	0.012030	2.7781	0.033420	1.6143		4.57	4534	94
11	Exterior	0.0000493	0.0617	61.280	0.001747	5.7484	0.010043	0.6154	61.28	9.97	572100	9336
	Interior	0.0000493	0.0617	61.280	0.001747	5.7484	0.010043	0.6154		9.97	572100	9336
12	Exterior	0.0002064	0.2216	28.373	0.003600	5.3857	0.019389	1.1002	64.75	4.97	22480	396
	Interior	0.0002227	0.2390	28.373	0.003096	5.3857	0.016674	0.9462		3.96	22150	390
13	Exterior	0.0000451	0.0596	63.000	0.001493	5.8860	0.008790	0.5538	56.75	9.29	740600	11756
	Interior	0.0000451	0.0596	63.000	0.001493	5.8860	0.008790	0.5538		9.29	740600	11756
14	Exterior	0.0002448	0.2518	27.776	0.003917	3.0681	0.012018	0.6676	63.55	2.65	9221	166
	Interior	0.0004021	0.4136	27.776	0.011340	3.0681	0.034792	1.9328		4.67	4108	74
15	Exterior	0.0000540	0.0729	63.640	0.001620	5.9372	0.009617	0.6120	63.64	8.39	488900	7682
	Interior	0.0000540	0.0729	63.640	0.001620	5.9372	0.009617	0.6120		8.39	488900	7682
16	Exterior	0.0002275	0.1340	29.729	0.003225	3.2243	0.010398	0.6183	67.46	4.61	10300	693
	Interior	-	-	29.729	-	3.2243	-	-		-		
17	Exterior	0.0000540	0.0697	62.240	0.001620	5.8252	0.009436	0.5873	62.24	8.42	403600	6485
	Interior	0.0000540	0.0697	62.240	0.001620	5.8252	0.009436	0.5873		8.42	403600	6485
18	Exterior	0.0004371	0.4671	28.311	0.006580	3.1108	0.020468	1.1589	64.62	2.48	5349	94
	Interior	0.0005518	0.5897	28.311	0.009058	3.1108	0.028177	1.5954		2.71	4087	72
19	Exterior	0.0000540	0.0718	63.160	0.001620	5.8988	0.009555	0.6035	63.16	8.40	403600	6390
	Interior	0.0000540	0.0718	63.160	0.001620	5.8988	0.009555	0.6035		8.40	403600	6390
20	Exterior	0.0004371	0.4265	27.052	0.006580	3.0102	0.019806	1.0716	62.10	2.51	5349	99
	Interior	0.0005518	0.5384	27.052	0.009058	3.0102	0.027265	1.4752		2.74	4087	76
21	Exterior	0.0000451	0.0594	62.900	0.001493	5.8780	0.008778	0.5521	62.90	9.29	309200	4916
	Interior	0.0000451	0.0594	62.900	0.001493	5.8780	0.008778	0.5521		9.29	309200	4916
22	Exterior	0.0003958	0.3995	27.513	0.020584	3.0470	0.062720	3.4512	63.03	8.64	6885	250
	Interior	0.0004122	0.4160	27.513	0.021078	3.0470	0.064225	3.5340		8.49	8323	303
23	Exterior	0.0000679	0.0971	65.470	0.000887	6.0836	0.005397	0.3533	65.47	3.64	740600	11312
	Interior	0.0000679	0.0971	65.470	0.000887	6.0836	0.005397	0.3533		3.64	740600	11312
24	Exterior	0.0000679	0.0978	65.720	0.000887	6.1036	0.005415	0.3559	65.72	3.64	309200	4705
	Interior	0.0000679	0.0978	65.720	0.000887	6.1036	0.005415	0.3559		3.64	309200	4705
25	Exterior	0.0004007	0.4581	29.284	0.004772	3.1887	0.015215	0.8911	66.57	1.95	5258	90
	Interior	0.0004015	0.4591	29.284	0.006980	3.1887	0.022256	1.3035		2.84	4130	71
26	Exterior	0.0004007	0.4604	29.358	0.004772	3.1947	0.015243	0.8950	66.72	1.94	5258	90
	Interior	0.0004015	0.4614	29.358	0.006980	3.1947	0.022297	1.3092		2.84	4130	70
27	Exterior	0.0000451	0.0720	69.220	0.001493	6.3836	0.009533	0.6599	69.22	9.17	403600	5831
	Interior	-	-	69.220	-	6.3836	-	-	-	-	-	-
28	Exterior	0.0005920	0.7517	30.859	0.005532	3.3147	0.018337	1.1317	69.72	1.51	3392	55
	Interior	0.0006517	0.8275	30.859	0.005895	3.3147	0.019541	1.2060		1.46	3869	63
29	Exterior	0.0002248	0.1482	31.443	0.006372	3.3614	0.021420	1.3470	70.89	9.09	7108	226
	Interior	0.0003169	0.2089	31.443	0.008424	3.3614	0.028317	1.7807		8.53	6371	203
30	Exterior	0.0002599	0.1711	31.429	0.004215	3.3603	0.014164	0.8903	70.86	5.20	9525	303
	Interior	0.0002694	0.1774	31.429	0.004170	3.3603	0.014011	0.8807		4.96	8739	278
31	Exterior	0.0000679	0.1096	69.570	0.000887	6.4116	0.005687	0.3957	69.57	3.61	309200	4444
	Interior	0.0000679	0.1096	69.570	0.000887	6.4116	0.005687	0.3957		3.61	309200	4444
32	Exterior	0.0002896	0.1874	31.155	0.004633	3.3384	0.015468	0.9638	70.31	5.14	3534	57
	Interior	0.0005924	0.3833	31.155	0.004208	3.3384	0.014047	0.8753		2.28	4848	78
33	Exterior	0.0002428	0.1612	31.557	0.003333	3.3706	0.011235	0.7091	71.11	4.40	13700	217
	Interior	0.0002492	0.1654	31.557	0.006488	3.3706	0.021868	1.3802		8.34	8260	131
34	Exterior	0.0000679	0.1158	71.500	0.000887	6.5660	0.005824	0.4164	71.50	3.60	309200	4324
	Interior	0.0000679	0.1158	71.500	0.000887	6.5660	0.005824	0.4164		3.60	309200	4324
35	Exterior	0.0002896	0.3891	31.746	0.004633	3.3857	0.015687	0.9960	71.49	2.56	3534	56
	Interior	0.0005924	0.7960	31.746	0.004208	3.3857	0.014245	0.9045		1.14	4848	76
36	Exterior	0.0002896	0.3735	31.100	0.004633	3.3340	0.015448	0.9608	70.20	2.57	3534	57
	Interior	0.0005924	0.7640	31.100	0.004208	3.3340	0.014028	0.8725		1.14	4848	78
37	Exterior	0.0000679	0.1136	70.890	0.000887	6.5172	0.005782	0.4099	70.89	3.60	403600	5693
	Interior	0.0000679	0.1136	70.890	0.000887	6.5172	0.005782	0.4099		3.60	403600	5693

6th Street Bridge Column Ultimate Displacement Capacity (Relative) for steel-casing opt blue = casing
Longitudinal Direction

Pier #	COLUMN	Phi-y (1/ft)	[Disp] y-ft	L-eff. (ft)	Phi-p (1/ft)	Lp (ft)	Theta-p	[Disp]-p (ft)	L (FT)	Du/Dy	Mp (k-')	Vp (kip)
1	Exterior	0.0005648	0.0366	13.950	0.008171	1.9620	0.016031	0.2236	21.95	6.10	1609	115
	Interior	0.0005648	0.0366	13.950	0.010882	1.9620	0.021350	0.2978		8.13	1609	115
2	Exterior	0.0005648	0.0559	17.224	0.008171	2.2239	0.018172	0.3130	25.22	5.60	1609	93
	Interior	0.0005648	0.0559	17.224	0.010882	2.2239	0.024201	0.4168		7.46	1609	93
3	Exterior	0.0004184	0.0688	22.210	0.010732	2.6228	0.028148	0.6252	30.21	9.09	33310	1500
	Interior	0.0004184	0.0688	22.210	0.010732	2.6228	0.028148	0.6252		9.09	33310	1500
4	Exterior	0.0003050	0.0577	23.823	0.006391	2.7518	0.017587	0.4190	31.82	7.26	8948	376
	Interior	0.0003241	0.0613	23.823	0.006522	2.7518	0.017947	0.4276		6.97	6451	271
5	Exterior	0.0003050	0.0660	25.472	0.006391	2.8838	0.018430	0.4695	33.47	7.12	8948	351
	Interior	0.0003241	0.0701	25.472	0.006522	2.8838	0.018808	0.4791		6.83	6451	253
6	Exterior	0.0003338	0.0985	29.746	0.007784	3.2257	0.025110	0.7469	37.75	7.59	9680	325
	Interior	0.0004093	0.1207	29.746	0.010620	3.2257	0.034257	1.0190		8.44	4794	161
7	Exterior	0.0008188	0.3041	33.377	0.005854	3.5161	0.020584	0.6870	41.38	2.26	11840	355
	Interior	0.0008188	0.3041	33.377	0.005854	3.5161	0.020584	0.6870		2.26	11840	355
8	Exterior	0.0004991	0.2227	36.584	0.014760	2.3094	0.034086	1.2470	44.58	5.60	2179	60
	Interior	0.0004519	0.2016	36.584	0.012912	2.3094	0.029819	1.0909		5.41	2866	78
9	Exterior	0.0005107	0.3432	44.903	0.009596	4.4383	0.042590	1.9124	52.90	5.57	46860	1044
	Interior	0.0005107	0.3432	44.903	0.008614	4.4383	0.038230	1.7166		5.00	46860	1044
10	Exterior	0.0002838	0.2207	48.302	0.007049	4.7101	0.033202	1.6037	56.30	7.27	8049	167
	Interior	0.0002654	0.2064	48.302	0.009460	4.7101	0.044558	2.1522		10.43	7413	153
11	Exterior	0.0003911	0.3701	53.283	0.007461	5.1087	0.038113	2.0308	61.28	5.49	68150	1279
	Interior	0.0003911	0.3701	53.283	0.004265	5.1087	0.021788	1.1610		3.14	68150	1279
12	Exterior	0.0002645	0.2839	56.747	0.006864	5.3857	0.036968	2.0978	64.75	7.39	15160	267
	Interior	0.0002762	0.2965	56.747	0.009042	5.3857	0.048698	2.7635		9.32	18650	329
13	Exterior	0.0005019	0.3976	48.753	0.009403	4.7462	0.044629	2.1758	56.75	5.47	64840	1330
	Interior	0.0005019	0.3976	48.753	0.009403	4.7462	0.044629	2.1758		5.47	64840	1330
14	Exterior	0.0003235	0.3328	55.553	0.005933	5.2902	0.031387	1.7436	63.55	5.24	6986	126
	Interior	0.0003112	0.3201	55.553	0.010750	5.2902	0.056870	3.1593		9.87	5432	98
15	Exterior	0.0005849	0.6036	55.640	0.006861	5.2972	0.036342	2.0220	63.64	3.35	36380	654
	Interior	0.0005849	0.6036	55.640	0.004201	5.2972	0.022254	1.2382		2.05	36380	654
16	Exterior	0.0003394	0.2000	59.458	0.004582	3.2243	0.014773	0.8784	67.46	4.39	16000	538
	Interior	[illegible]	[illegible]	59.458	[illegible]	5.6026	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
17	Exterior	0.0005849	0.5736	54.242	0.006861	5.1853	0.035575	1.9296	62.24	3.36	46860	864
	Interior	0.0005849	0.5736	54.242	0.006403	5.1853	0.033203	1.8010		3.14	46860	864
18	Exterior	0.0005475	0.5851	56.621	0.010489	5.3757	0.056387	3.1927	64.62	5.46	4123	73
	Interior	0.0005643	0.6030	56.621	0.010892	5.3757	0.058554	3.3154		5.50	3415	60
19	Exterior	0.0005849	0.5933	55.163	0.006861	5.2590	0.036080	1.9903	63.16	3.35	46860	849
	Interior	0.0005849	0.5933	55.163	0.006403	5.2590	0.033675	1.8576		3.13	46860	849
20	Exterior	0.0005475	0.5342	54.104	0.010489	5.1744	0.054275	2.9365	62.10	5.50	4123	76
	Interior	0.0005643	0.5506	54.104	0.010892	5.1744	0.056361	3.0494		5.54	3415	63
21	Exterior	0.0002985	0.2999	54.897	0.005803	5.2378	0.030395	1.6686	62.90	5.56	33310	607
	Interior	0.0002985	0.2999	54.897	0.005803	5.2378	0.030395	1.6686		5.56	33310	607
22	Exterior	0.0004601	0.4644	55.026	0.014583	5.2481	0.076533	4.2113	63.03	9.07	4901	89
	Interior	0.0002985	0.3013	55.026	0.005803	5.2481	0.030455	1.6758		5.56	8320	151
23	Exterior	0.0005301	0.5835	57.465	0.004591	5.4432	0.024990	1.4360	65.47	2.46	64840	1128
	Interior	0.0005301	0.5835	57.465	0.004591	5.4432	0.024990	1.4360		2.46	64840	1128
24	Exterior	0.0002844	0.3158	57.720	0.002724	5.4636	0.014883	0.8590	65.72	2.72	33310	577
	Interior	0.0003992	0.4433	57.720	0.006664	5.4636	0.036407	2.1014		4.74	33310	577
25	Exterior	0.0006165	0.7049	58.568	0.006334	5.5314	0.035033	2.0518	66.57	2.91	3942	67
	Interior	0.0005363	0.6132	58.568	0.011594	5.5314	0.064130	3.7560		6.13	2885	49
26	Exterior	0.0006165	0.7085	58.716	0.006334	5.5433	0.035109	2.0614	66.72	2.91	3942	67
	Interior	0.0005363	0.6163	58.716	0.011594	5.5433	0.064267	3.7736		6.12	2885	49
27	Exterior	0.0003655	0.4567	61.224	0.007215	5.7439	0.041444	2.5373	69.22	5.56	46860	765
	Interior		0.0000	61.224	-	5.7439	-	-	-	-	-	-
28	Exterior	0.0006449	0.8188	61.718	0.005907	5.7834	0.034163	2.1085	69.72	2.58	3008	49
	Interior	0.0006517	0.8275	61.718	0.005895	5.7834	0.034093	2.1041		2.54	3552	58
29	Exterior	0.0002687	0.4501	70.886	0.002535	3.6814	0.009334	0.6616	70.89	1.47	7526	212
	Interior	0.0003470	0.5812	70.886	0.008813	3.6814	0.032444	2.2999		3.96	5473	154
30	Exterior	0.0002444	0.4090	70.858	0.003486	3.6803	0.012828	0.9090	70.86	2.22	11330	320
	Interior	0.0002596	0.4345	70.858	0.003382	3.6803	0.012448	0.8821		2.03	8748	247
31	Exterior	0.0005122	0.6472	61.567	0.012374	5.7714	0.071415	4.3968	69.57	6.79	33310	541
	Interior	0.0006266	0.7917	61.567	0.004303	5.7714	0.024837	1.5291		1.93	33310	541
32	Exterior	0.0003486	0.4512	62.311	0.004083	5.8309	0.023810	1.4836	70.31	3.29	3244	104
	Interior	0.0006266	0.8110	62.311	0.004303	5.8309	0.025090	1.5634		1.93	4201	135
33	Exterior	0.0003153	0.4187	63.115	0.003990	5.8952	0.023520	1.4845	71.11	3.55	8847	280
	Interior	0.0003253	0.4319	63.115	0.009210	5.8952	0.054293	3.4267		7.93	6650	211
34	Exterior	0.0005122	0.6884	63.500	0.012374	5.9260	0.073329	4.6564	71.50	6.76	33310	525
	Interior	0.0006266	0.8422	63.500	0.004303	5.9260	0.025502	1.6194		1.92	33310	525
35	Exterior	0.0003486	0.4684	63.491	0.004083	5.9253	0.024195	1.5362	71.49	3.28	3244	51
	Interior	0.0006266	0.8420	63.491	0.004303	5.9253	0.025497	1.6188		1.92	4201	66
36	Exterior	0.0003486	0.4496	62.200	0.004083	5.8220	0.023774	1.4787	70.20	3.29	3244	52
	Interior	0.0006266	0.8081	62.200	0.004303	5.8220	0.025052	1.5582		1.93	4201	68
37	Exterior	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.021503	1.3523	70.89	[illegible]	46860	[illegible]
	Interior	[illegible]	0.3992	62.886	[illegible]	5.8769	0.049965	3.1421		7.87	46860	745

6th Street Bridge Retrofitted Model Column Ultimate Displacement D/C Ratio (Relative)
Transverse Direction

Pier #	COLUMN	Disp-eql	Disp-eqt	[Disp]-p (ft)	(D/C)-EQL	(D/C)-EQT
1	Exterior	0.0059	0.0156	0.213	0.03	0.07
	Interior	0.0073	0.0161	0.297	0.02	0.05
2	Exterior	0.0069	0.0129	0.288	0.02	0.04
	Interior	0.0071	0.0160	0.401	0.02	0.04
3	Exterior	0.0045	0.0073	0.176	0.03	0.04
	Interior	0.0052	0.0090	0.176	0.03	0.05
4	Exterior	0.0224	0.0468	0.255	0.09	0.18
	Interior	0.0233	0.0462	0.310	0.08	0.15
5	Exterior	0.0482	0.0943	0.283	0.17	0.33
	Interior	0.0471	0.0930	0.344	0.14	0.27
6	Exterior	0.0542	0.1076	0.319	0.17	0.34
	Interior	0.0517	0.1016	0.398	0.13	0.26
7	Exterior	0.0296	0.0577	0.169	0.17	0.34
	Interior	0.0317	0.0618	0.169	0.19	0.36
8	Exterior	0.0211	0.0389	1.073	0.02	0.04
	Interior	0.0214	0.0395	1.005	0.02	0.04
9	Exterior	0.0317	0.0589	0.458	0.07	0.13
	Interior	0.0331	0.0615	0.458	0.07	0.13
10	Exterior	0.0514	0.0884	1.346	0.04	0.07
	Interior	0.0511	0.0843	1.614	0.03	0.05
11	Exterior	0.0787	0.0814	0.615	0.13	0.13
	Interior	0.0789	0.0839	0.615	0.13	0.14
12	Exterior	0.5972	0.6640	1.100	0.54	0.60
	Interior	0.5915	0.6579	0.946	0.63	0.70
13	Exterior	0.0497	0.0597	0.554	0.09	0.11
	Interior	0.0477	0.0580	0.554	0.09	0.10
14	Exterior	0.1986	0.2440	0.668	0.30	0.37
	Interior	0.1698	0.2001	1.933	0.09	0.10
15	Exterior	0.0572	0.0924	0.612	0.09	0.15
	Interior	0.0577	0.0929	0.612	0.09	0.15
16	Exterior	0.2932	0.2336	0.618	0.47	0.38
	Interior	0.3081	0.2391		-	-
17	Exterior	0.0278	0.0552	0.587	0.05	0.09
	Interior	0.0277	0.0557	0.587	0.05	0.09
18	Exterior	0.0305	0.0696	1.159	0.03	0.06
	Interior	0.0275	0.0643	1.595	0.02	0.04
19	Exterior	0.0286	0.0516	0.603	0.05	0.09
	Interior	0.0290	0.0521	0.603	0.05	0.09
20	Exterior	0.1026	0.1596	1.072	0.10	0.15
	Interior	0.0955	0.1516	1.475	0.06	0.10
21	Exterior	0.0331	0.0553	0.552	0.06	0.10
	Interior	0.0308	0.0533	0.552	0.06	0.10
22	Exterior	0.2869	0.2442	3.451	0.08	0.07
	Interior	0.2231	0.2538	3.534	0.06	0.07
23	Exterior	0.0183	0.0265	0.353	0.05	0.07
	Interior	0.0245	0.0257	0.353	0.07	0.07
24	Exterior	0.0507	0.0637	0.356	0.14	0.18
	Interior	0.0544	0.0658	0.356	0.15	0.18
25	Exterior	0.0945	0.3032	0.891	0.11	0.34
	Interior	0.0926	0.2966	1.303	0.07	0.23
26	Exterior	0.1062	0.3027	0.895	0.12	0.34
	Interior	0.0992	0.2805	1.309	0.08	0.21
27	Exterior	0.0280	0.0639	0.660	0.04	0.10
	Interior	-	-	-	-	-
28	Exterior	0.2120	0.1770	1.132	0.19	0.16
	Interior	0.2119	0.1771	1.206	0.18	0.15
29	Exterior	0.5486	0.4705	1.347	0.41	0.35
	Interior	0.5785	0.4955	1.781	0.32	0.28
30	Exterior	0.3224	0.2427	0.890	0.36	0.27
	Interior	0.3420	0.2645	0.881	0.39	0.30
31	Exterior	0.0465	0.0571	0.396	0.12	0.14
	Interior	0.0461	0.0599	0.396	0.12	0.15
32	Exterior	0.1554	0.2017	0.964	0.16	0.21
	Interior	0.1569	0.2054	0.875	0.18	0.23
33	Exterior	0.1713	0.2267	0.709	0.24	0.32
	Interior	0.1682	0.2250	1.380	0.12	0.16
34	Exterior	0.0436	0.0437	0.416	0.10	0.11
	Interior	0.0438	0.0394	0.416	0.11	0.09
35	Exterior	0.3850	0.2757	0.996	0.39	0.28
	Interior	0.3916	0.2807	0.904	0.43	0.31
36	Exterior	0.4546	0.3259	0.961	0.47	0.34
	Interior	0.4395	0.3153	0.873	0.50	0.36
37	Exterior	0.0461	0.0312	0.410	0.11	0.09
	Interior	0.0452	0.0310	0.410	0.11	0.08

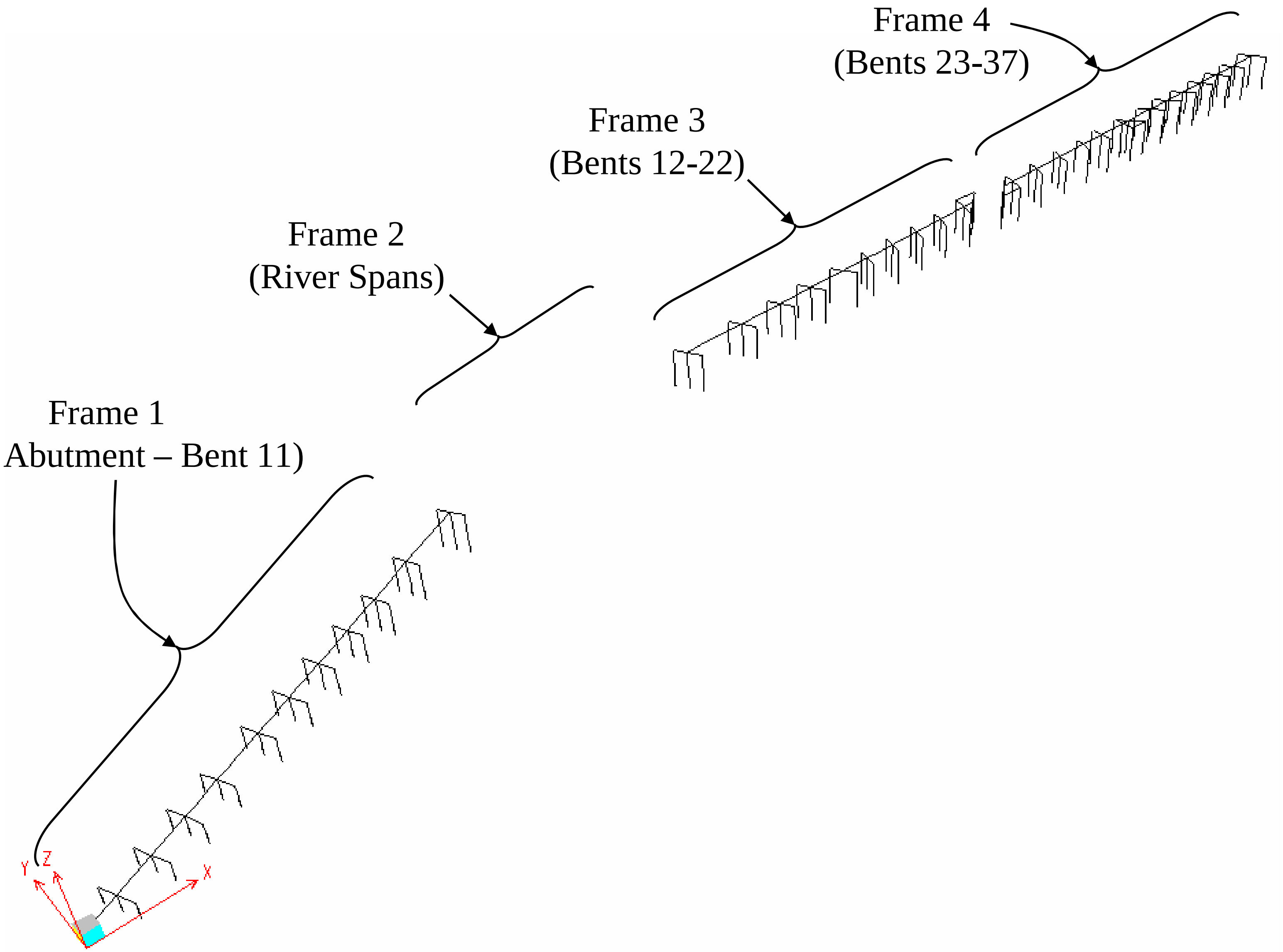
6th Street Bridge Retrofitted Model Column Ultimate Displacement D/C Ratio (Relative)
Longitudinal Direction

Pier #	COLUMN	U1 (EQ-L)	U1 (EQ-T)	[Disp]-p (ft)	(D/C)-EQL	(D/C)-EQT
1	Exterior	0.1874	0.0977	0.224	0.84	0.44
	Interior	0.2531	0.1276	0.298	0.85	0.43
2	Exterior	0.2019	0.1054	0.313	0.65	0.34
	Interior	0.1825	0.0950	0.417	0.44	0.23
3	Exterior	0.3955	0.2068	0.625	0.63	0.33
	Interior	0.3878	0.2026	0.625	0.62	0.32
4	Exterior	0.3565	0.1874	0.419	0.85	0.45
	Interior	0.2864	0.1496	0.428	0.67	0.35
5	Exterior	0.3931	0.2056	0.469	0.84	0.44
	Interior	0.3143	0.1639	0.479	0.66	0.34
6	Exterior	0.4720	0.2478	0.747	0.63	0.33
	Interior	0.4582	0.2394	1.019	0.45	0.23
7	Exterior	0.4918	0.2579	0.687	0.72	0.38
	Interior	0.4939	0.2581	0.687	0.72	0.38
8	Exterior	0.5908	0.3087	1.247	0.47	0.25
	Interior	0.6579	0.3438	1.091	0.60	0.32
9	Exterior	0.6486	0.3390	1.912	0.34	0.18
	Interior	0.6658	0.3479	1.717	0.39	0.20
10	Exterior	0.4989	0.2604	1.604	0.31	0.16
	Interior	0.4270	0.2231	2.152	0.20	0.10
11	Exterior	0.6633	0.3472	2.031	0.33	0.17
	Interior	0.6568	0.3432	1.161	0.57	0.30
12	Exterior	0.9792	0.5214	2.098	0.47	0.25
	Interior	0.9503	0.5328	2.763	0.34	0.19
13	Exterior	0.8378	0.3531	2.176	0.39	0.16
	Interior	0.8333	0.3381	2.176	0.38	0.16
14	Exterior	0.8887	0.3573	1.744	0.51	0.20
	Interior	0.8837	0.3520	3.159	0.28	0.11
15	Exterior	0.8234	0.3451	2.022	0.41	0.17
	Interior	0.8932	0.3535	1.238	0.72	0.29
16	Exterior	0.5933	0.2647	0.878	0.68	0.30
	Interior	-	-	-	-	-
17	Exterior	0.7094	0.2827	1.930	0.37	0.15
	Interior	0.7205	0.2878	1.801	0.40	0.16
18	Exterior	0.6729	0.2767	3.193	0.21	0.09
	Interior	0.6757	0.2776	3.315	0.20	0.08
19	Exterior	0.6339	0.2723	1.990	0.32	0.14
	Interior	0.6316	0.2704	1.858	0.34	0.15
20	Exterior	0.5478	0.2562	2.937	0.19	0.09
	Interior	0.5406	0.2462	3.049	0.18	0.08
21	Exterior	0.4076	0.4601	1.669	0.24	0.28
	Interior	0.4869	0.2217	1.669	0.29	0.13
22	Exterior	1.0445	1.0359	4.211	0.25	0.25
	Interior	0.5941	0.4331	1.676	0.35	0.26
23	Exterior	0.4832	0.3369	1.436	0.34	0.23
	Interior	0.3404	0.2124	1.436	0.24	0.15
24	Exterior	0.3708	0.2500	0.859	0.43	0.29
	Interior	0.2644	0.1685	2.101	0.13	0.08
25	Exterior	0.2721	0.1825	2.052	0.13	0.09
	Interior	0.2767	0.2162	3.756	0.07	0.06
26	Exterior	0.3204	0.3024	2.061	0.16	0.15
	Interior	0.3246	0.2414	3.774	0.09	0.06
27	Exterior	0.4122	0.2893	2.537	0.16	0.11
	Interior	-	-	-	-	-
28	Exterior	0.4867	0.3333	2.108	0.23	0.16
	Interior	0.3776	0.2326	2.104	0.18	0.11
29	Exterior	0.4893	0.3571	0.662	0.74	0.54
	Interior	0.4065	0.2500	2.300	0.18	0.11
30	Exterior	0.7547	0.6387	0.909	0.83	0.70
	Interior	0.6623	0.5325	0.882	0.75	0.60
31	Exterior	0.6298	0.3651	4.397	0.14	0.08
	Interior	0.6294	0.3965	1.529	0.41	0.26
32	Exterior	0.6527	0.4174	1.484	0.44	0.28
	Interior	0.6655	0.3968	1.563	0.43	0.25
33	Exterior	0.6087	0.3715	1.484	0.41	0.25
	Interior	0.6591	0.3974	3.427	0.19	0.12
34	Exterior	0.5965	0.3836	4.656	0.13	0.08
	Interior	0.6242	0.3800	1.619	0.39	0.23
35	Exterior	0.8905	0.5675	1.536	0.58	0.37
	Interior	0.7536	0.4738	1.619	0.47	0.29
36	Exterior	0.7406	0.4602	1.479	0.50	0.31
	Interior	0.7831	0.4959	1.558	0.50	0.32
37	Exterior	0.6101	0.3717	1.352	0.45	[illegible]
	Interior	[illegible]	0.3737	3.142	0.19	0.12

APPENDIX E

ANALYTICAL MODELS

Global Models of the Sixth Street Viaduct



Frame 1
(West Abutment – Bent 11)

Frame 2
(River Spans)

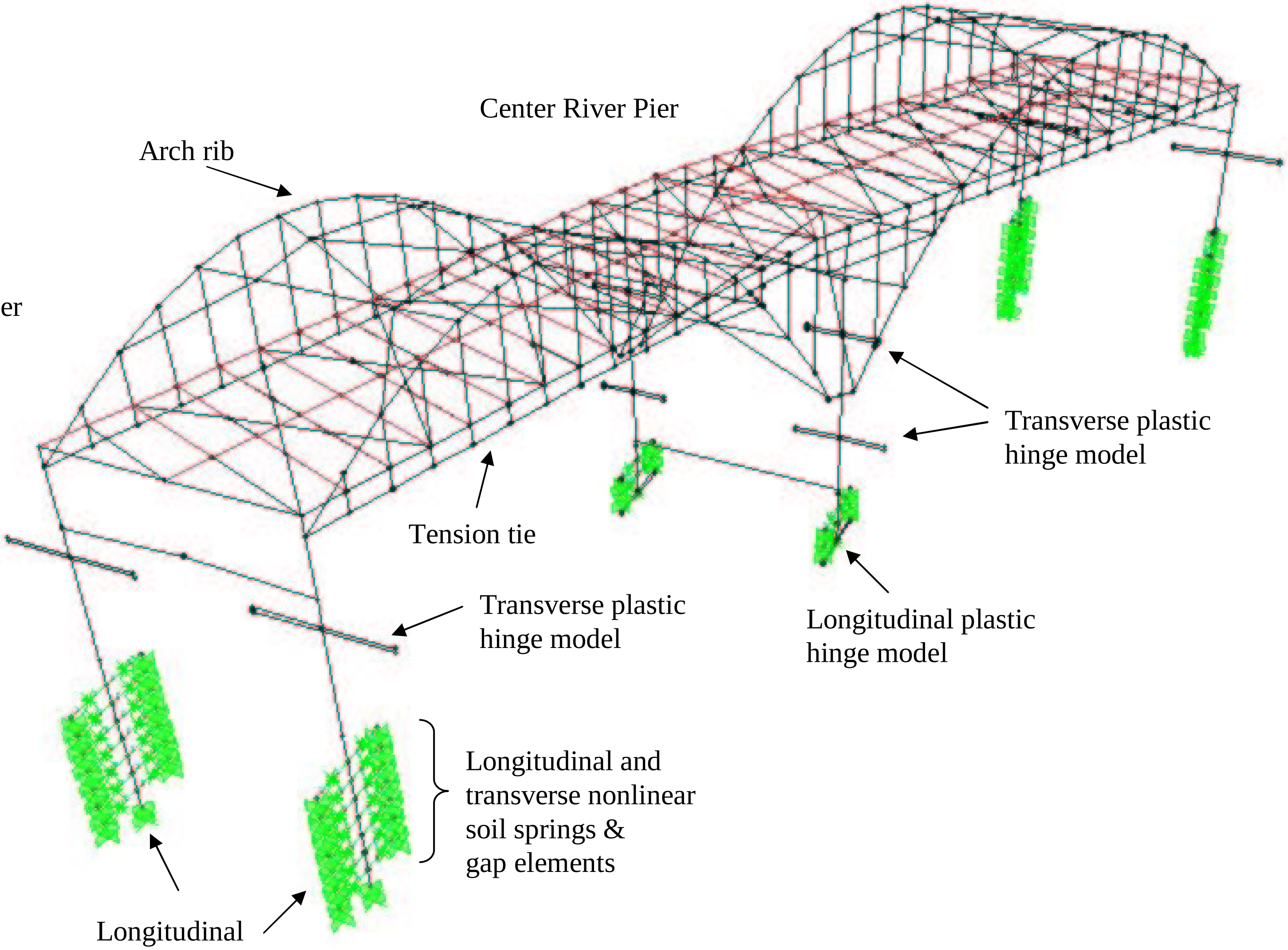
Frame 3
(Bents 12-22)

Frame 4
(Bents 23-37)

Main Spans over the Los Angeles River

(Steel Arch Spans)

As-Built Model of the Main Spans



East River Pier

Center River Pier

Arch rib

West River Pier

Transverse plastic hinge model

Tension tie

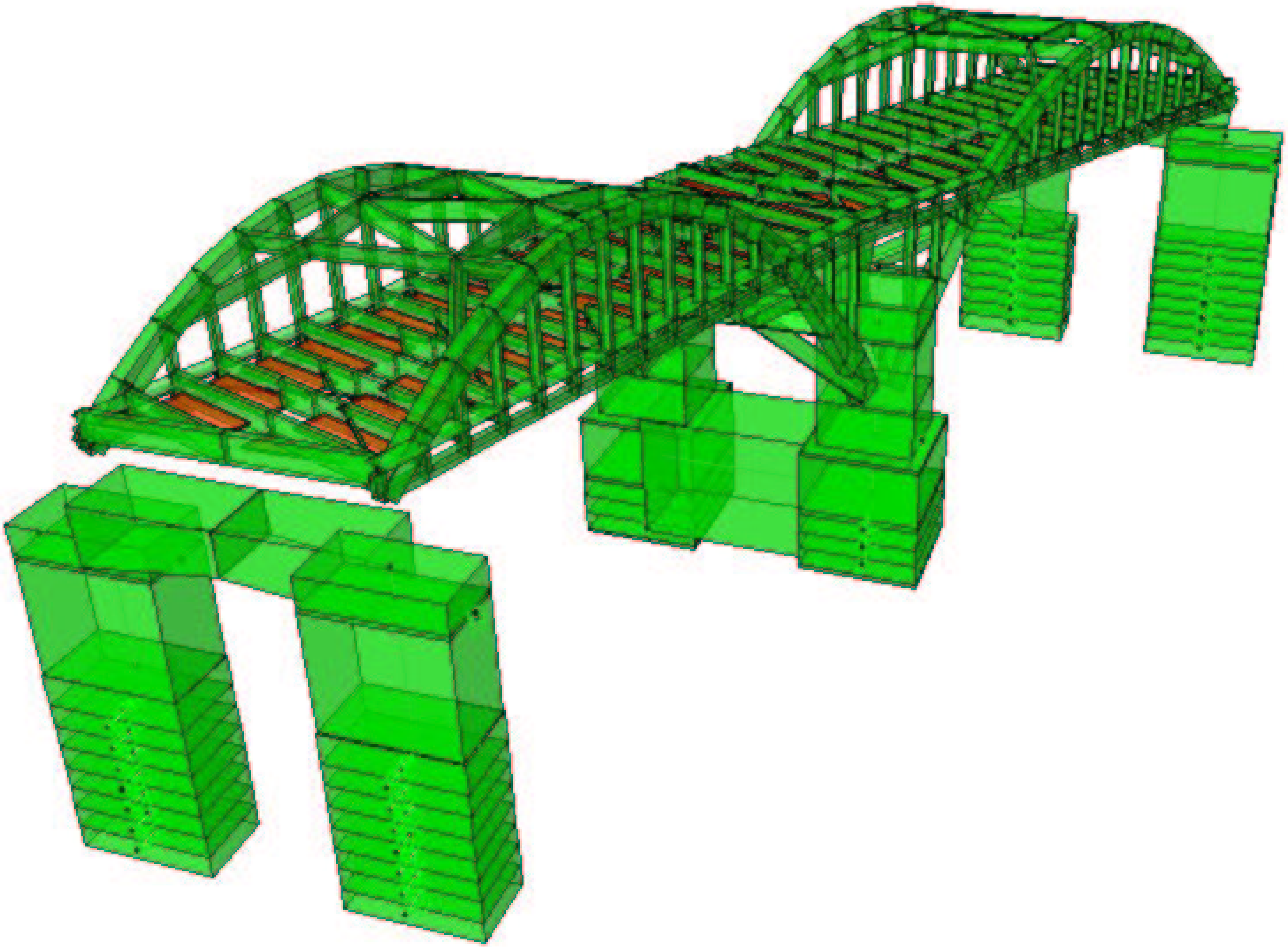
Transverse plastic hinge model

Longitudinal plastic hinge model

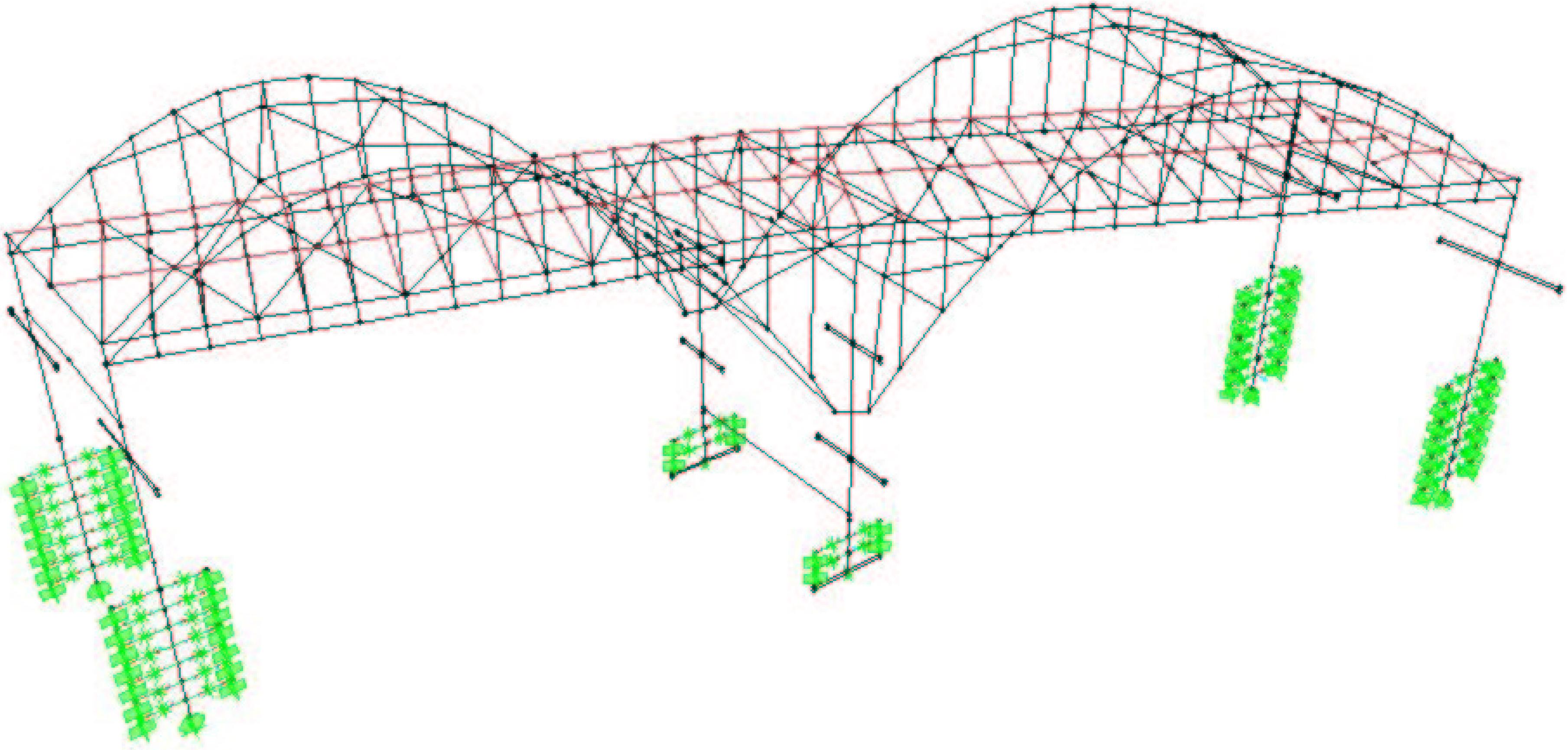
Longitudinal and transverse nonlinear soil springs & gap elements

Longitudinal and transverse pinned base

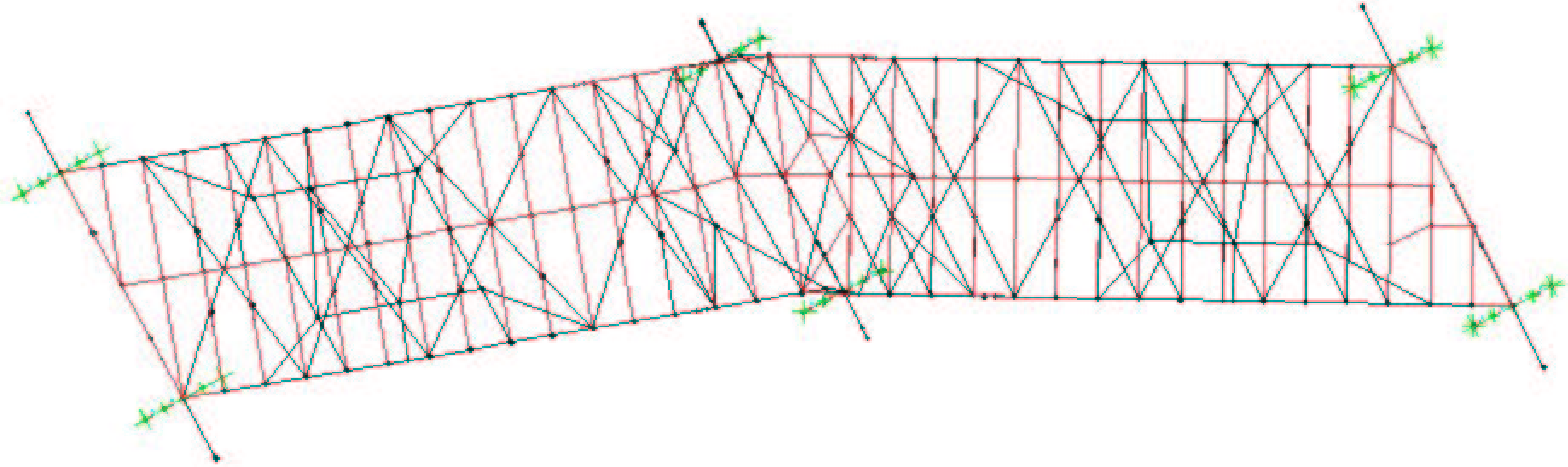
Extruded View of the As-Built Model



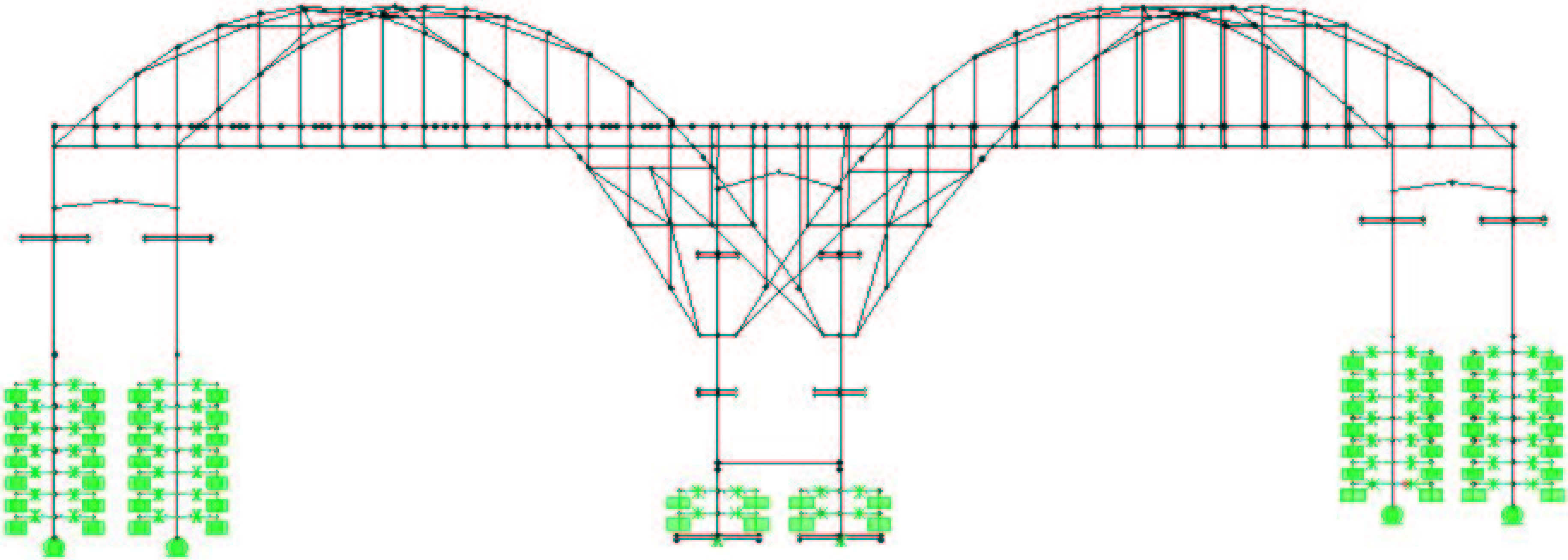
As-Built Model of the Main Spans



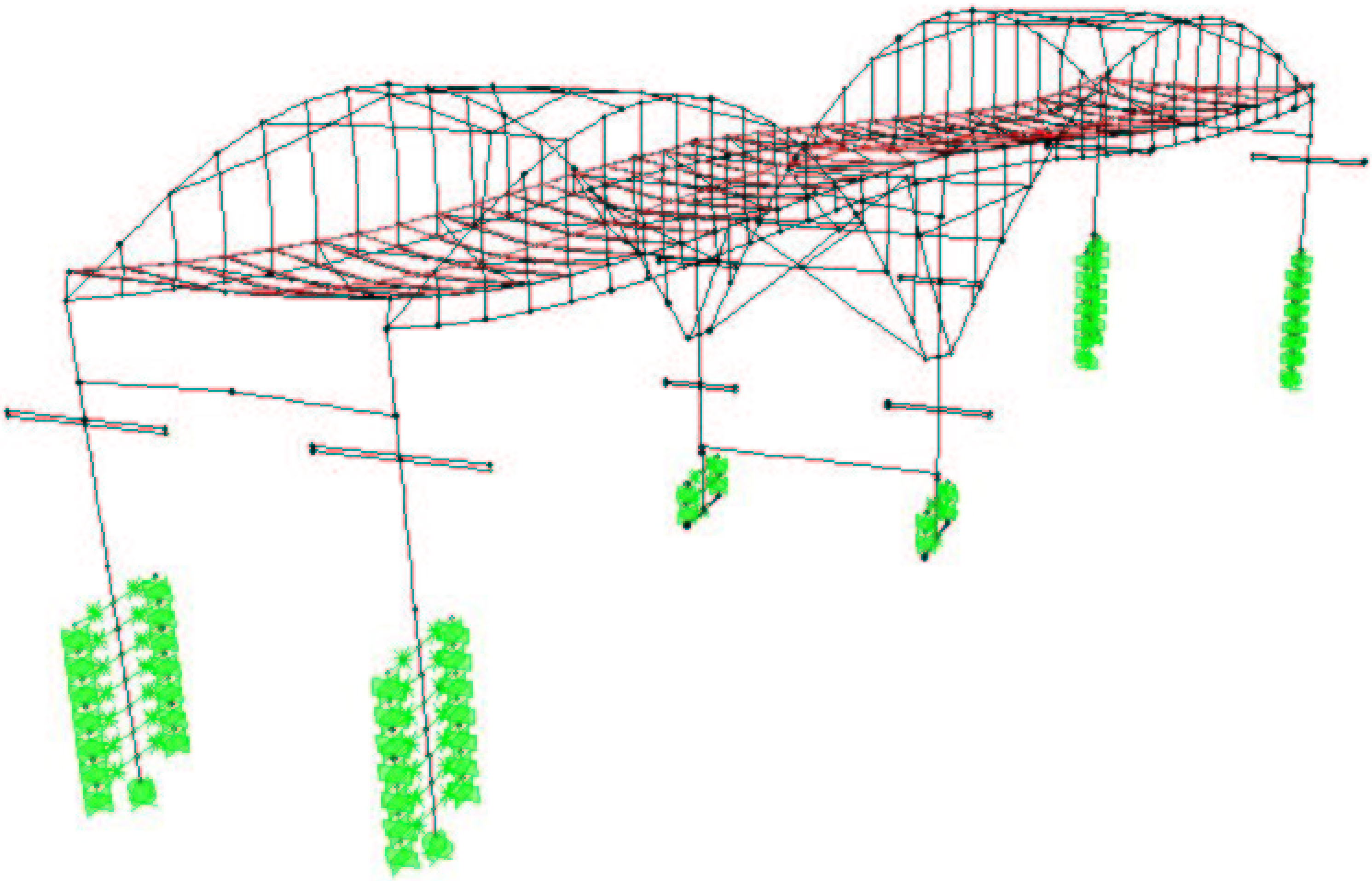
Plan View of the As-Built Model



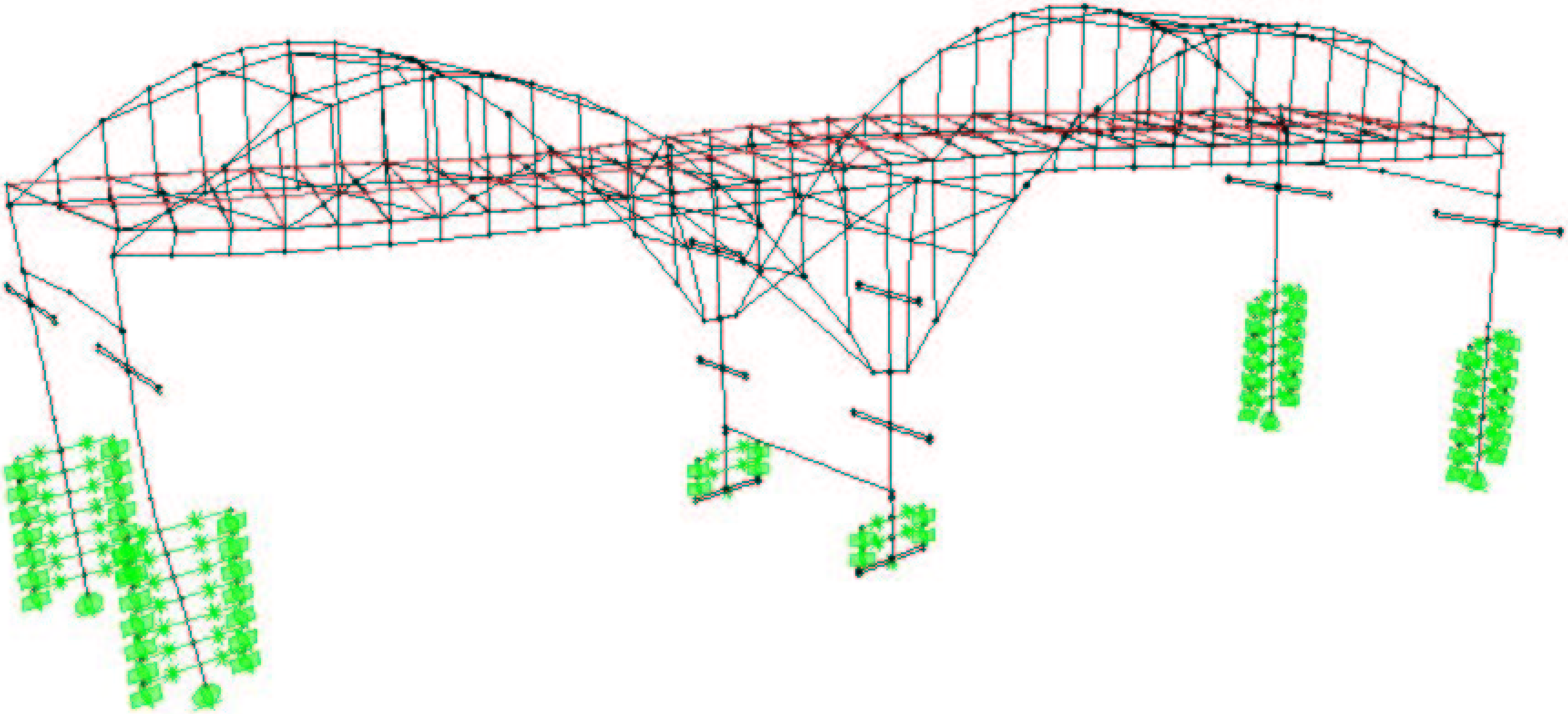
Elevation View of the As-Built Model



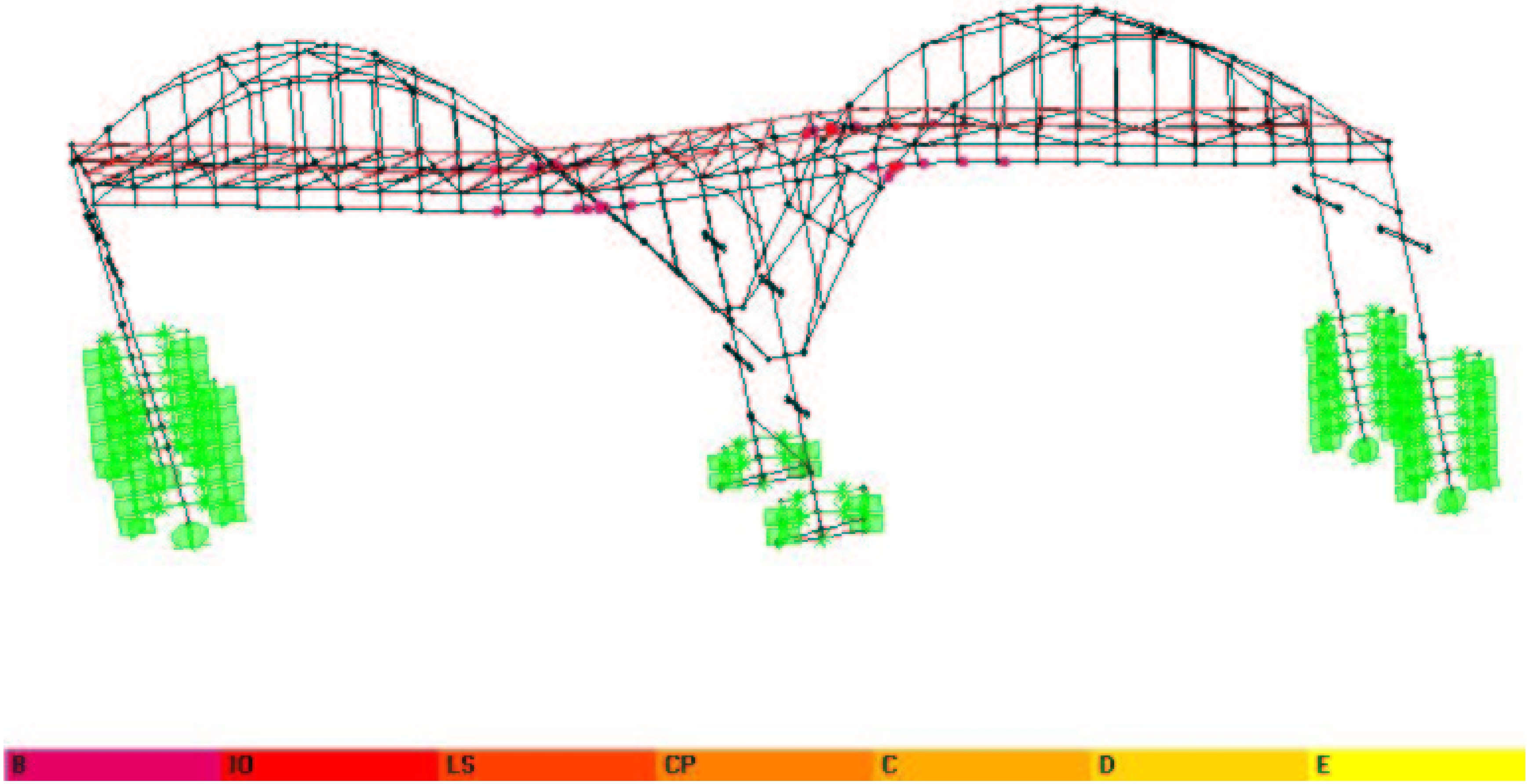
As-Built: Dead Load Deformations x 100



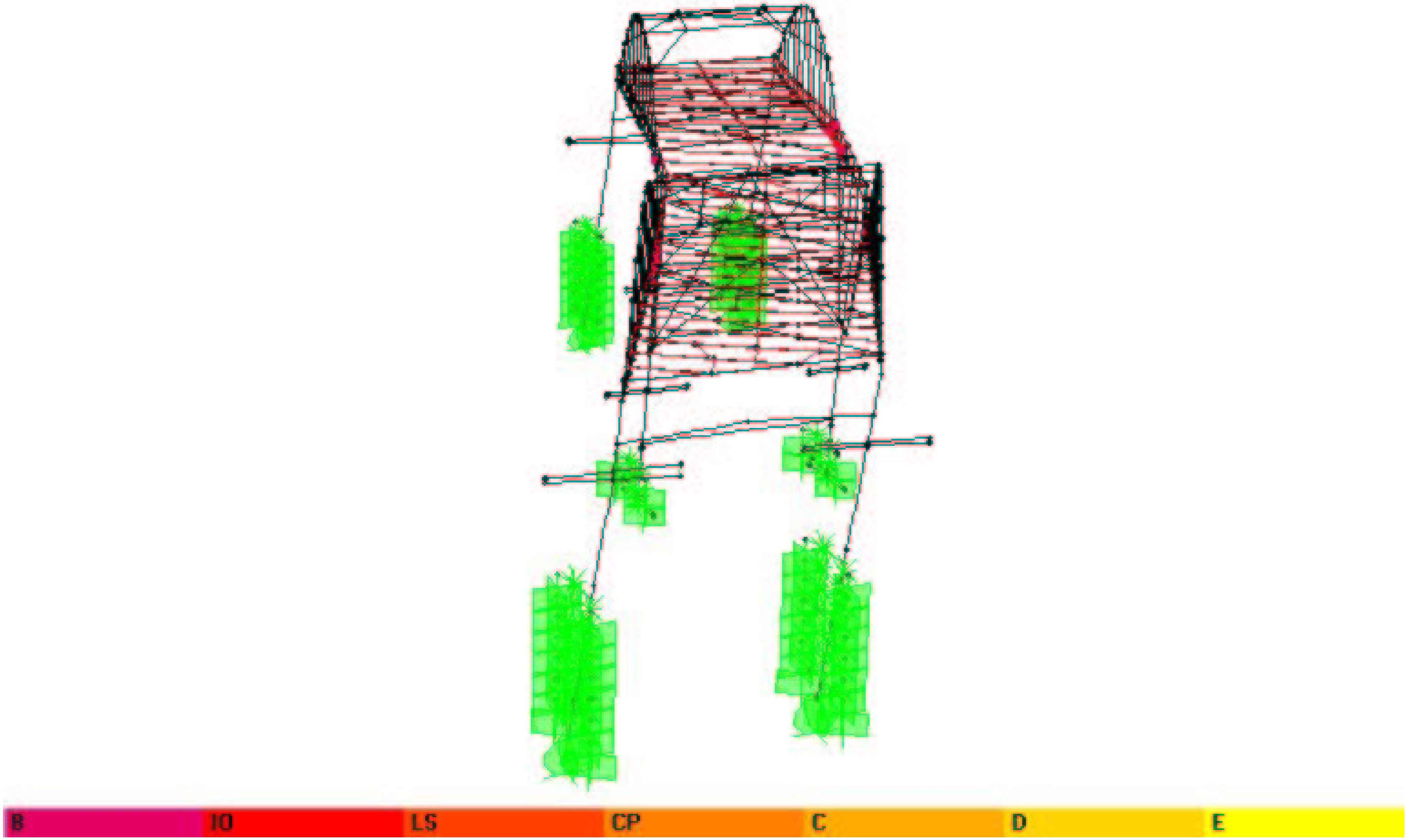
As-Built: Mode 1 Deformations (0.422 sec)



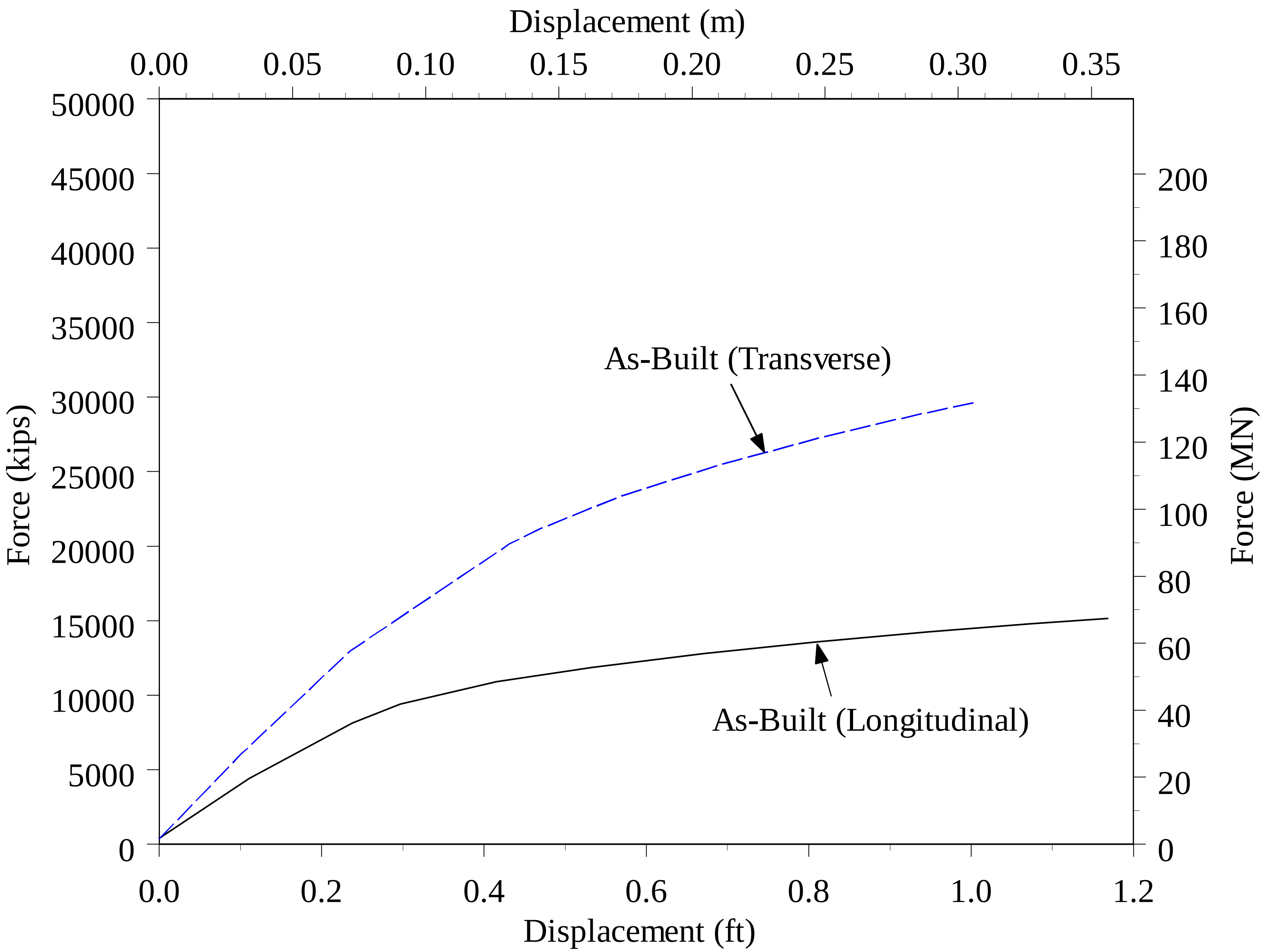
As-Built Longitudinal Pushover Analysis at 1.18 ft Displacement



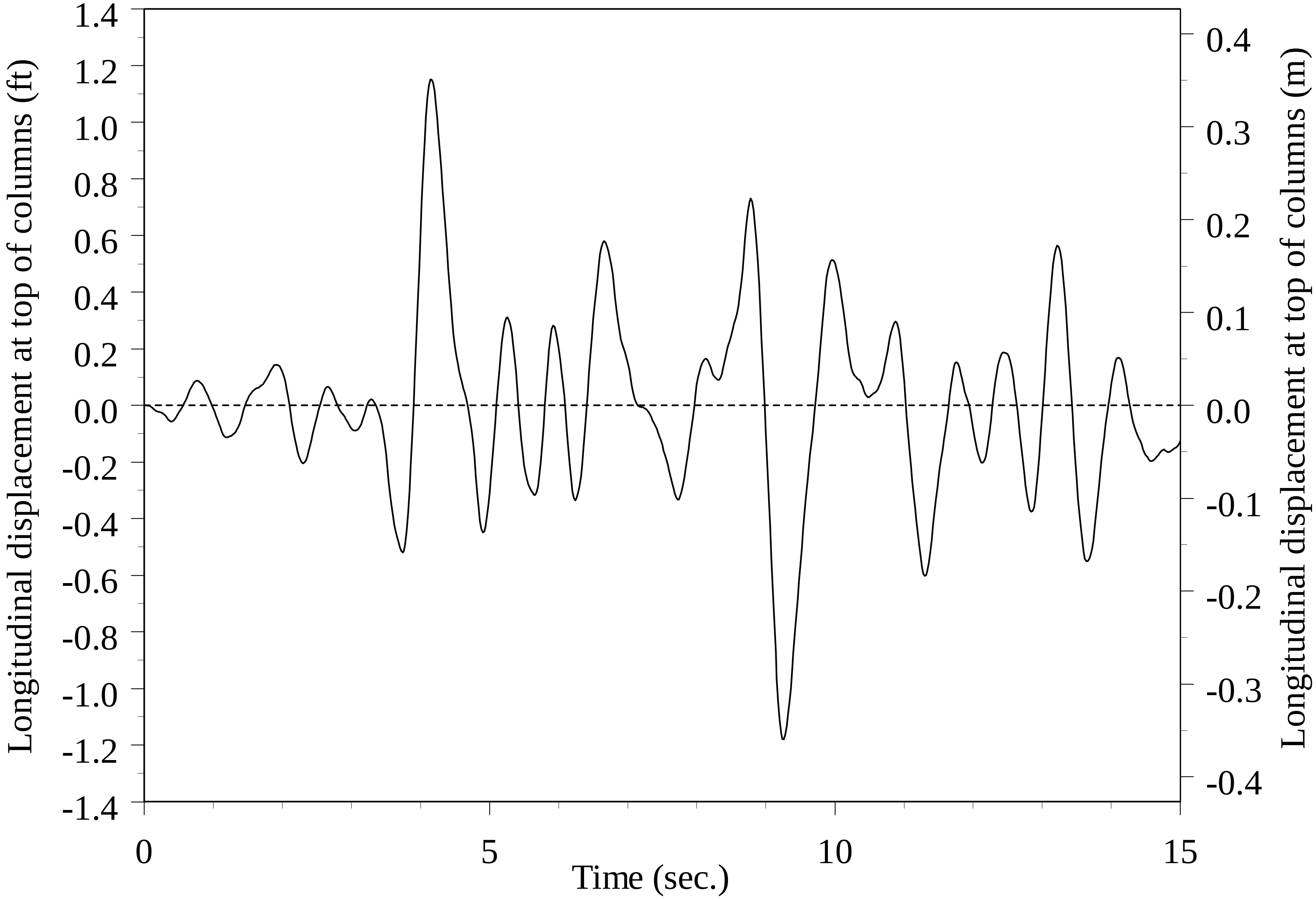
As-Built Transverse Pushover Analysis at 1.01 ft Displacement



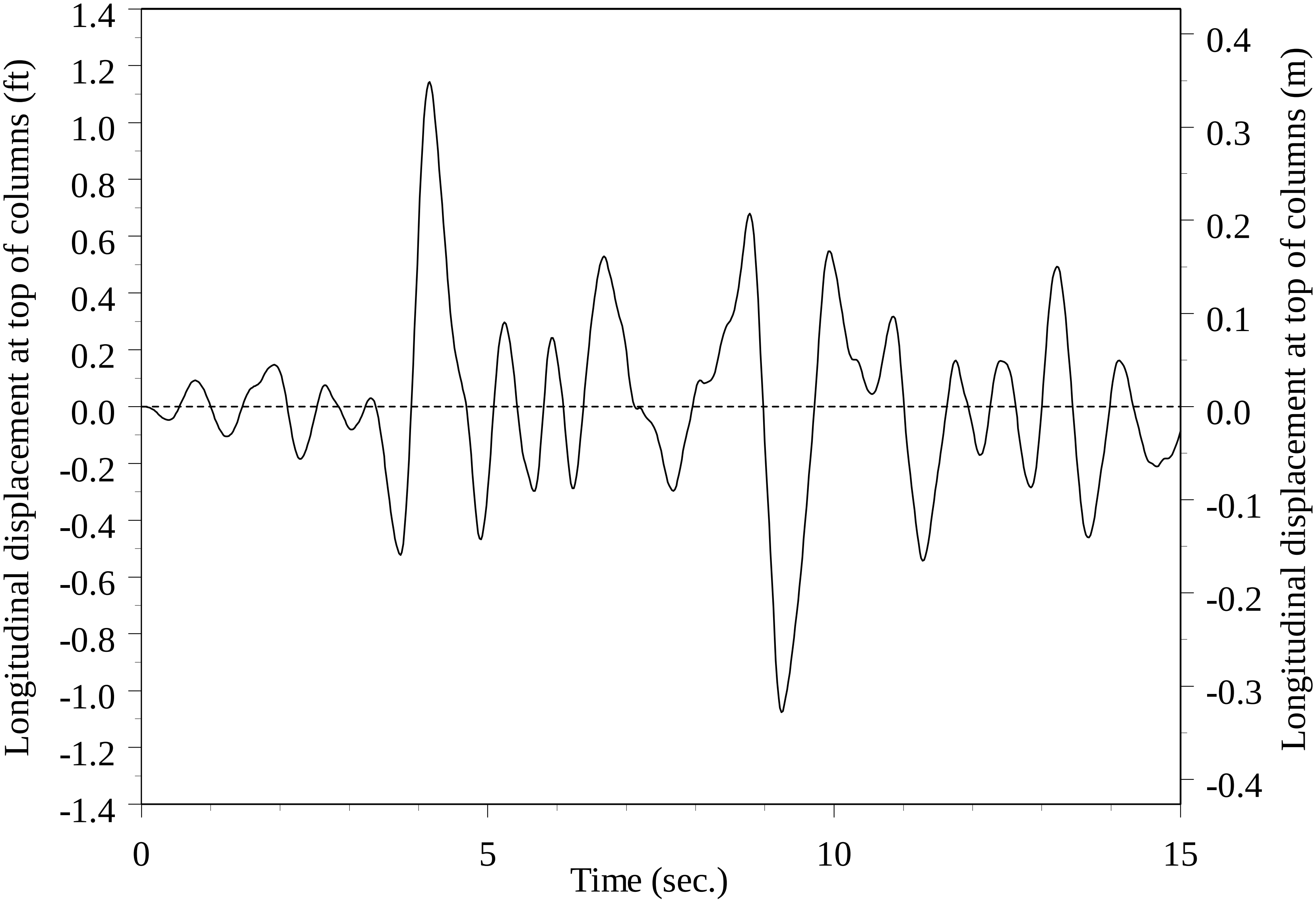
As-Built Load-Displacement Response (Pushover Analyses)



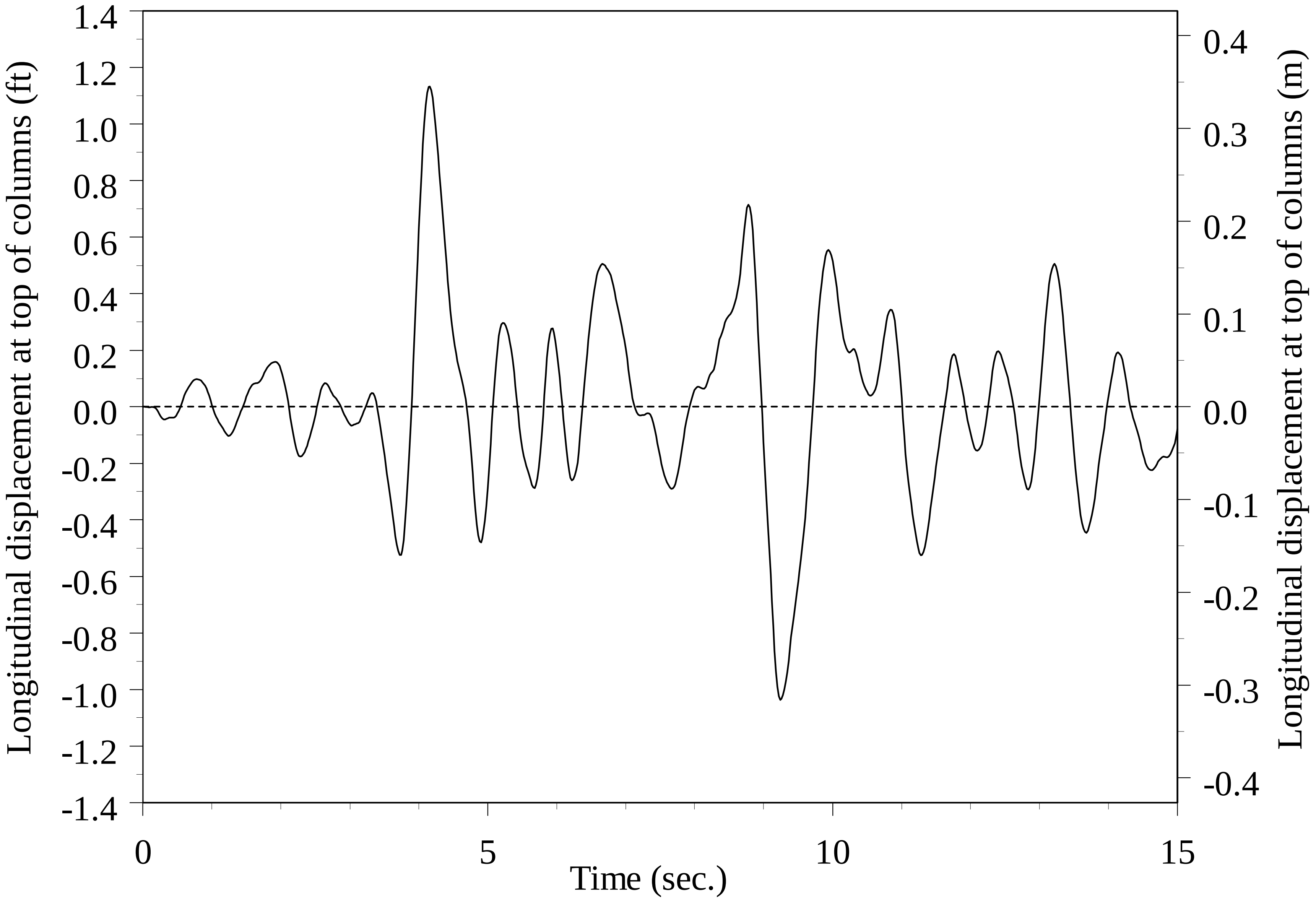
As-Built: Longitudinal Displacement at Top of West River Pier

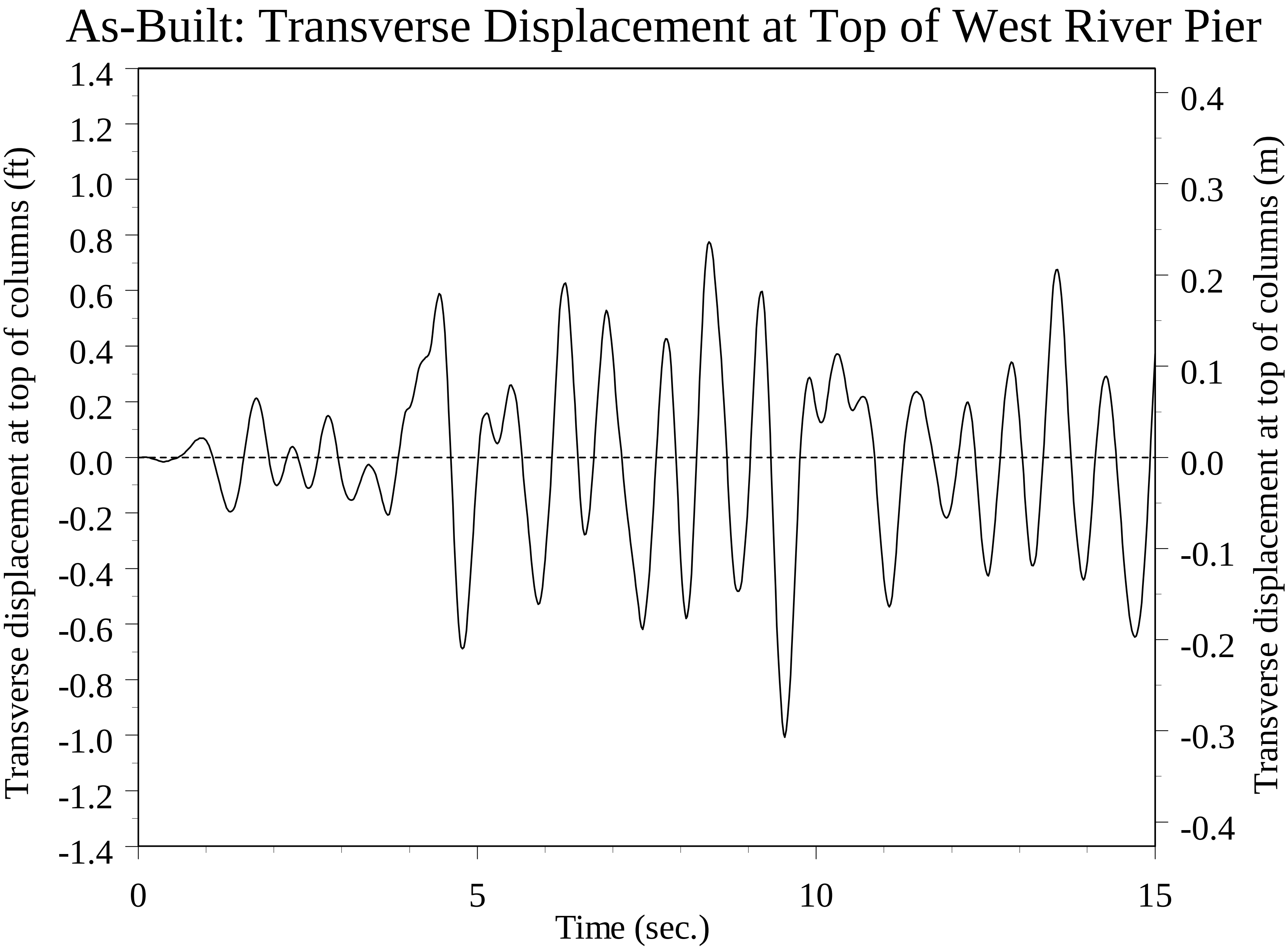


As-Built: Longitudinal Displacement at Top of Center River Pier

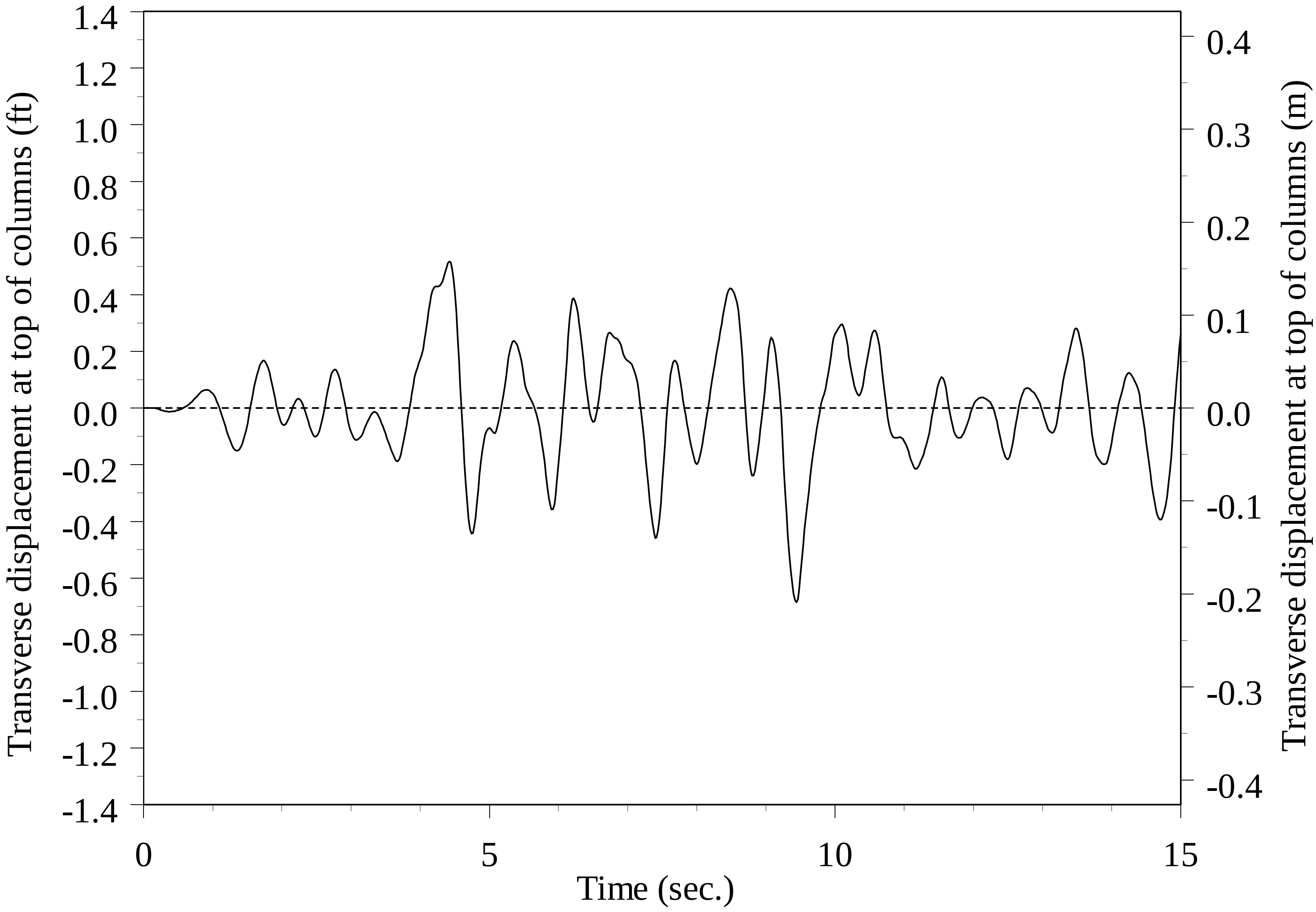


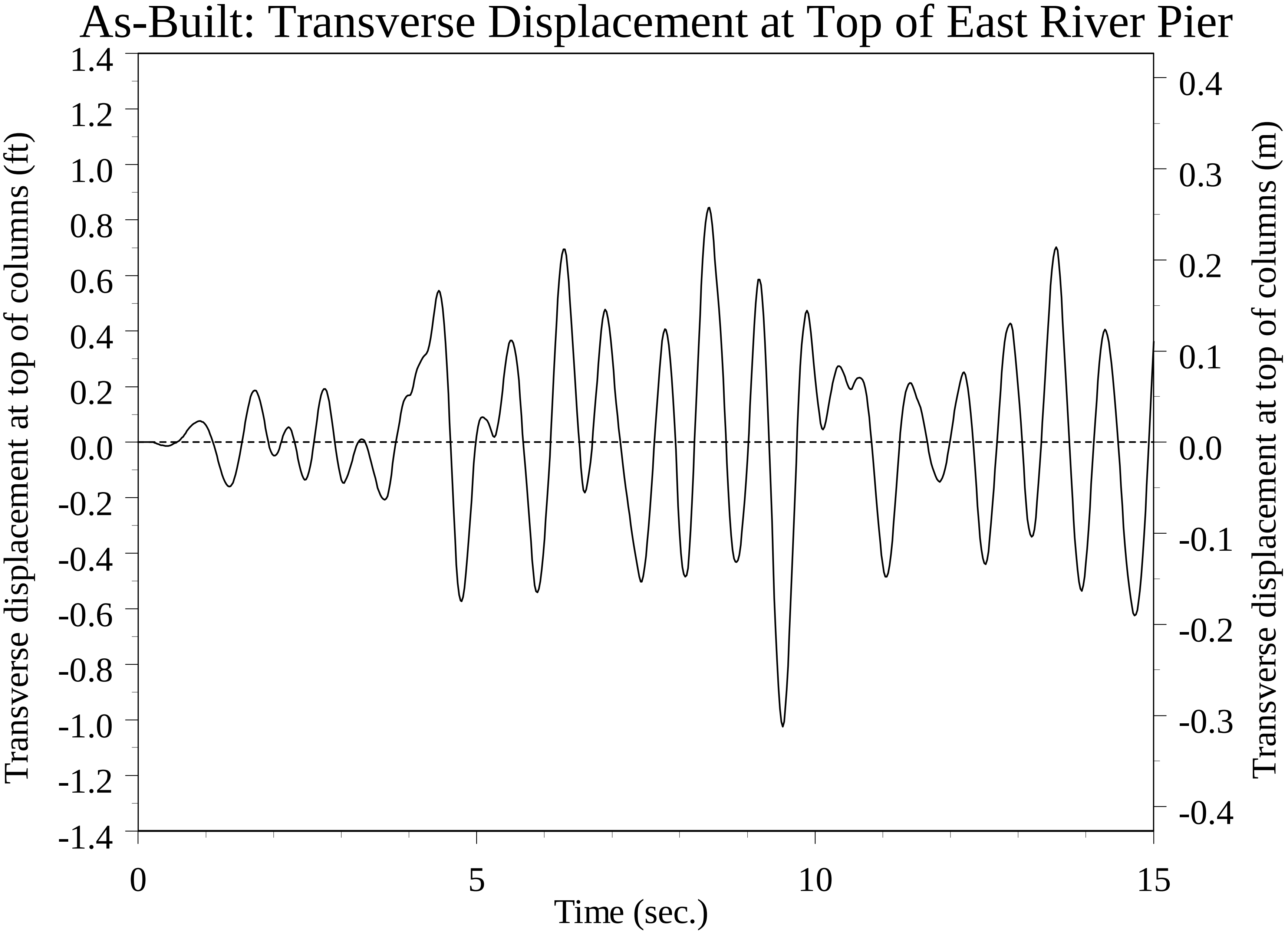
As-Built: Longitudinal Displacement at Top of East River Pier




As-Built: Transverse Displacement at Top of West River Pier
Transverse displacement at top of columns (ft)
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
-1.2
-1.4
Transverse displacement at top of columns (m)
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
-0.4
0
5
10
15
Time (sec.)

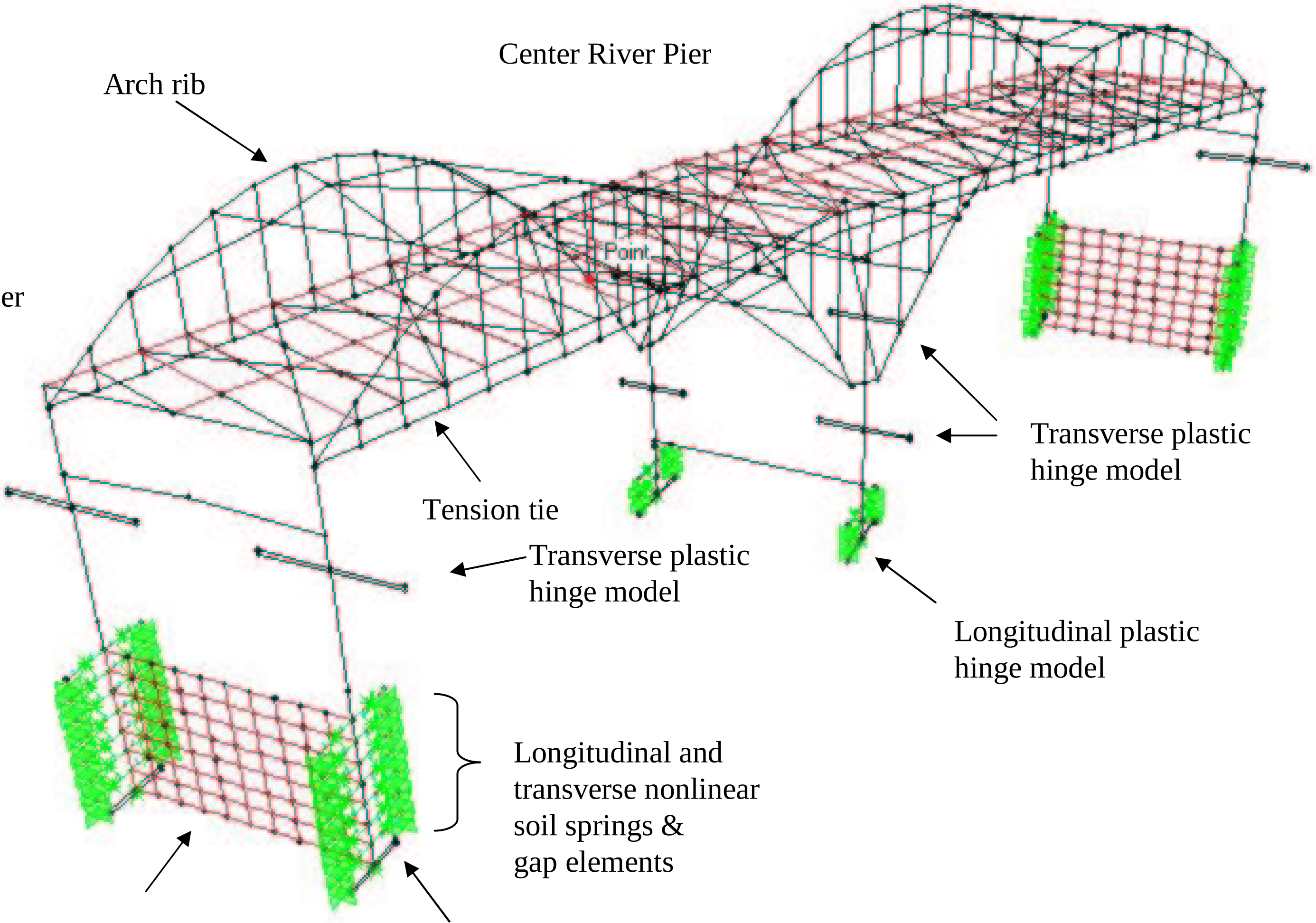
As-Built: Transverse Displacement at Top of Center River Pier





As-Built: Transverse Displacement at Top of East River Pier

Retrofit Model of the Main Spans



East River Pier

Center River Pier

Arch rib

West River Pier

Transverse plastic
hinge model

Tension tie

Transverse plastic
hinge model

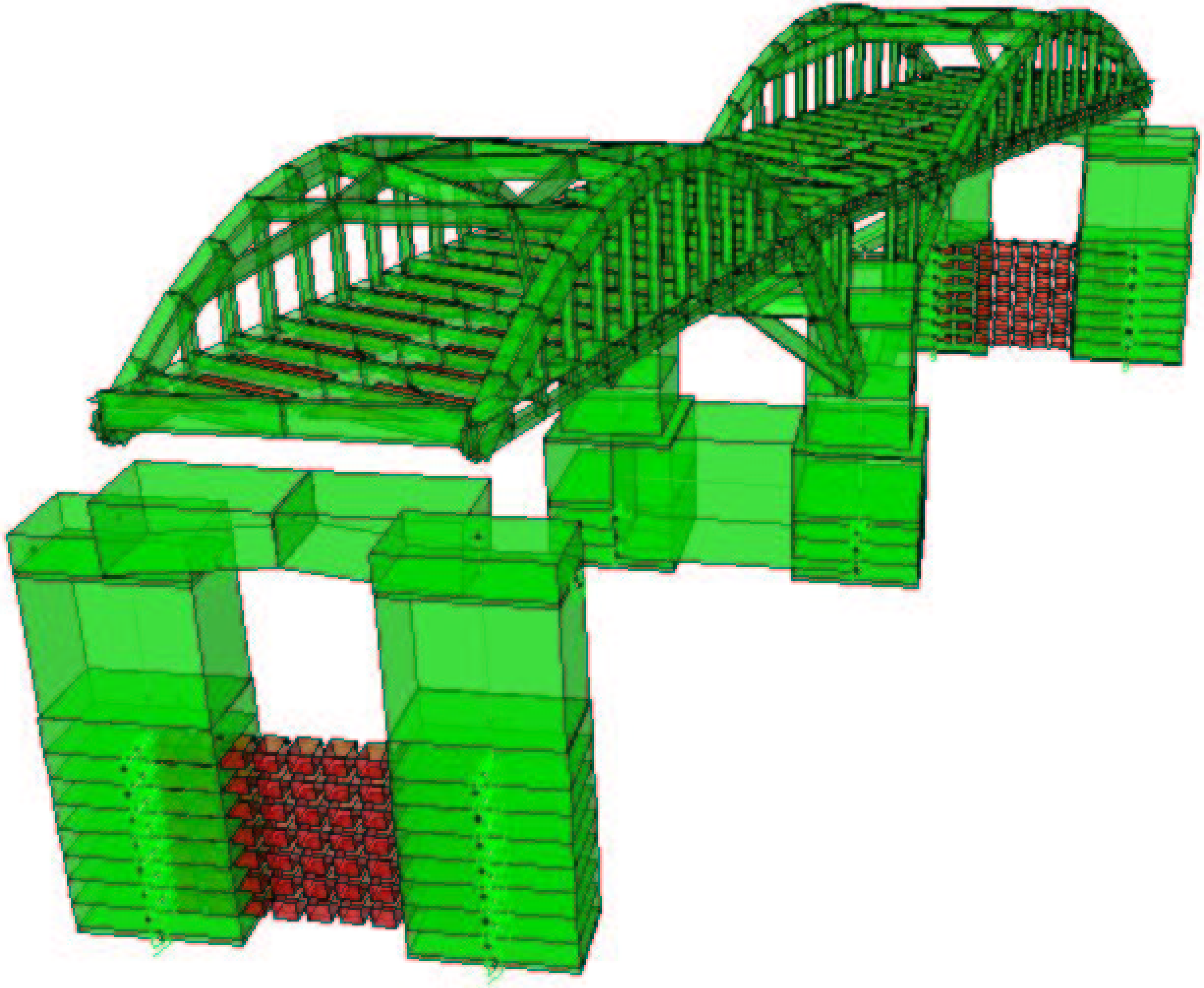
Longitudinal plastic
hinge model

Longitudinal and
transverse nonlinear
soil springs &
gap elements

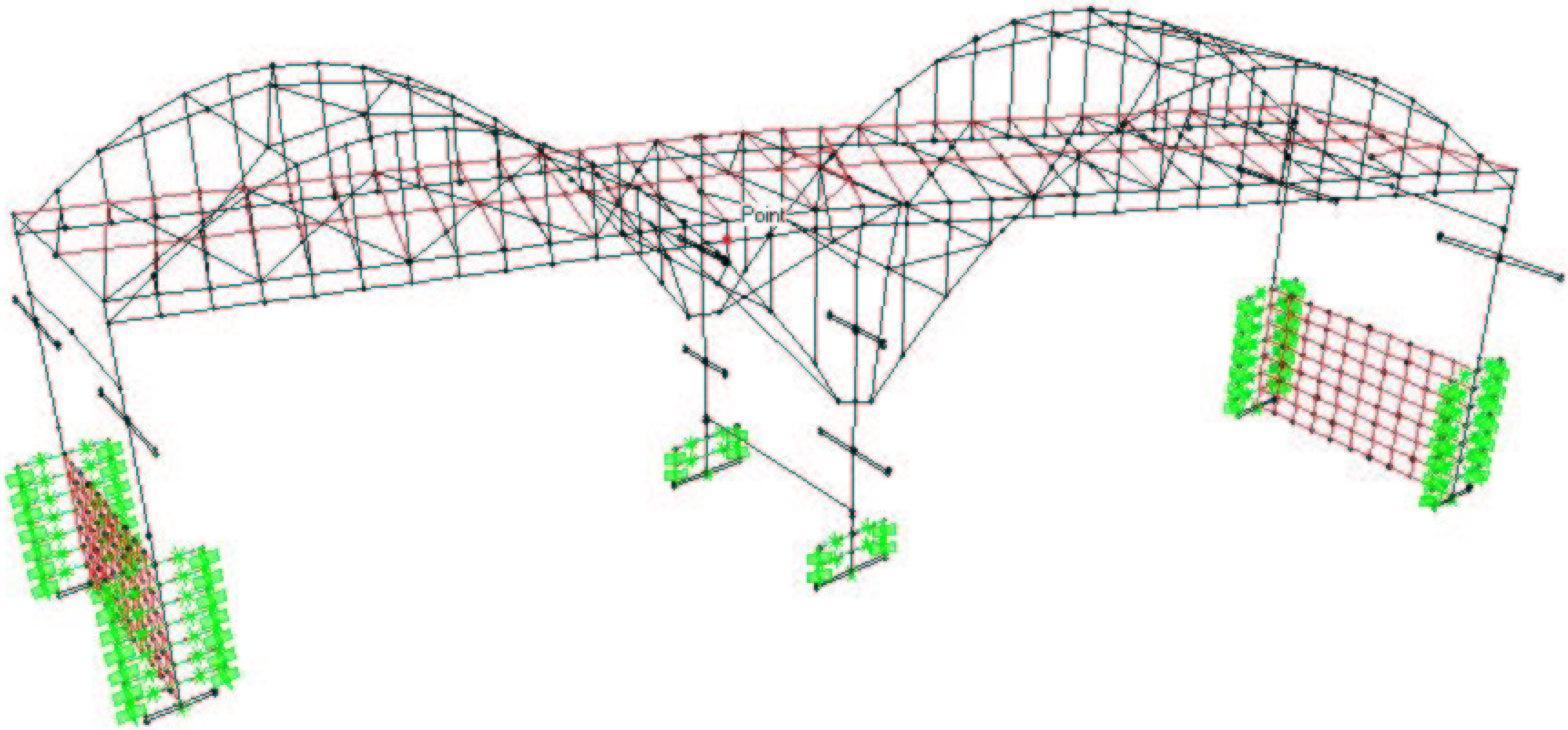
Infill wall

Longitudinal plastic
hinge model

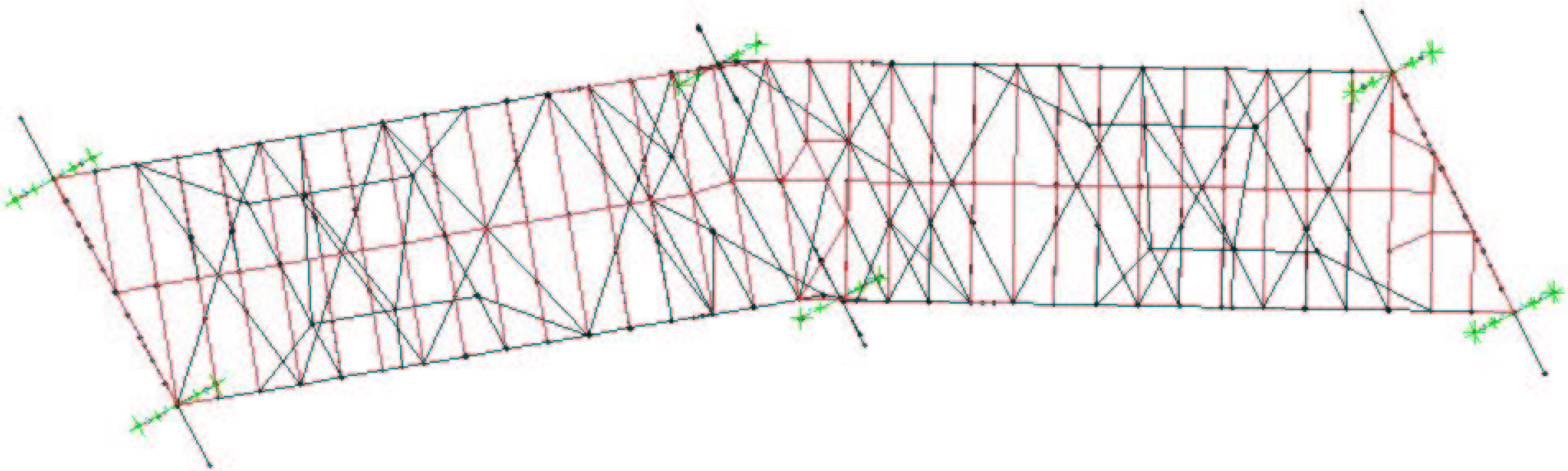
Extruded View of the Retrofit Model



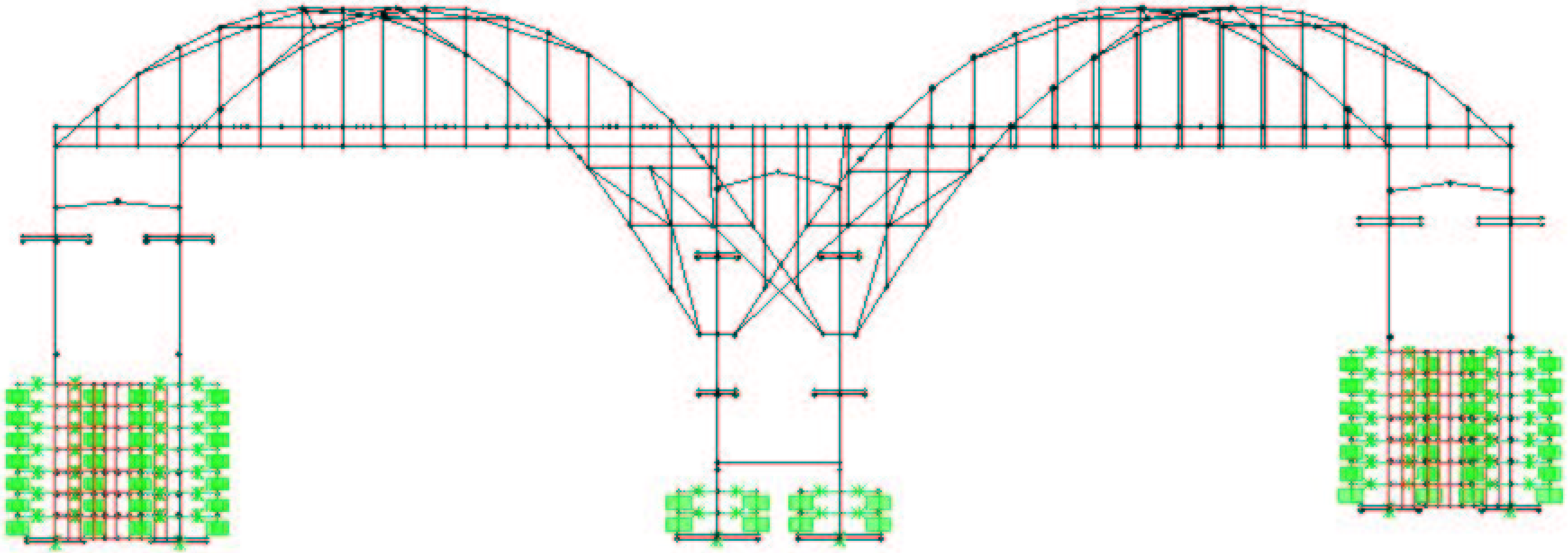
Retrofit Model for the Main Spans



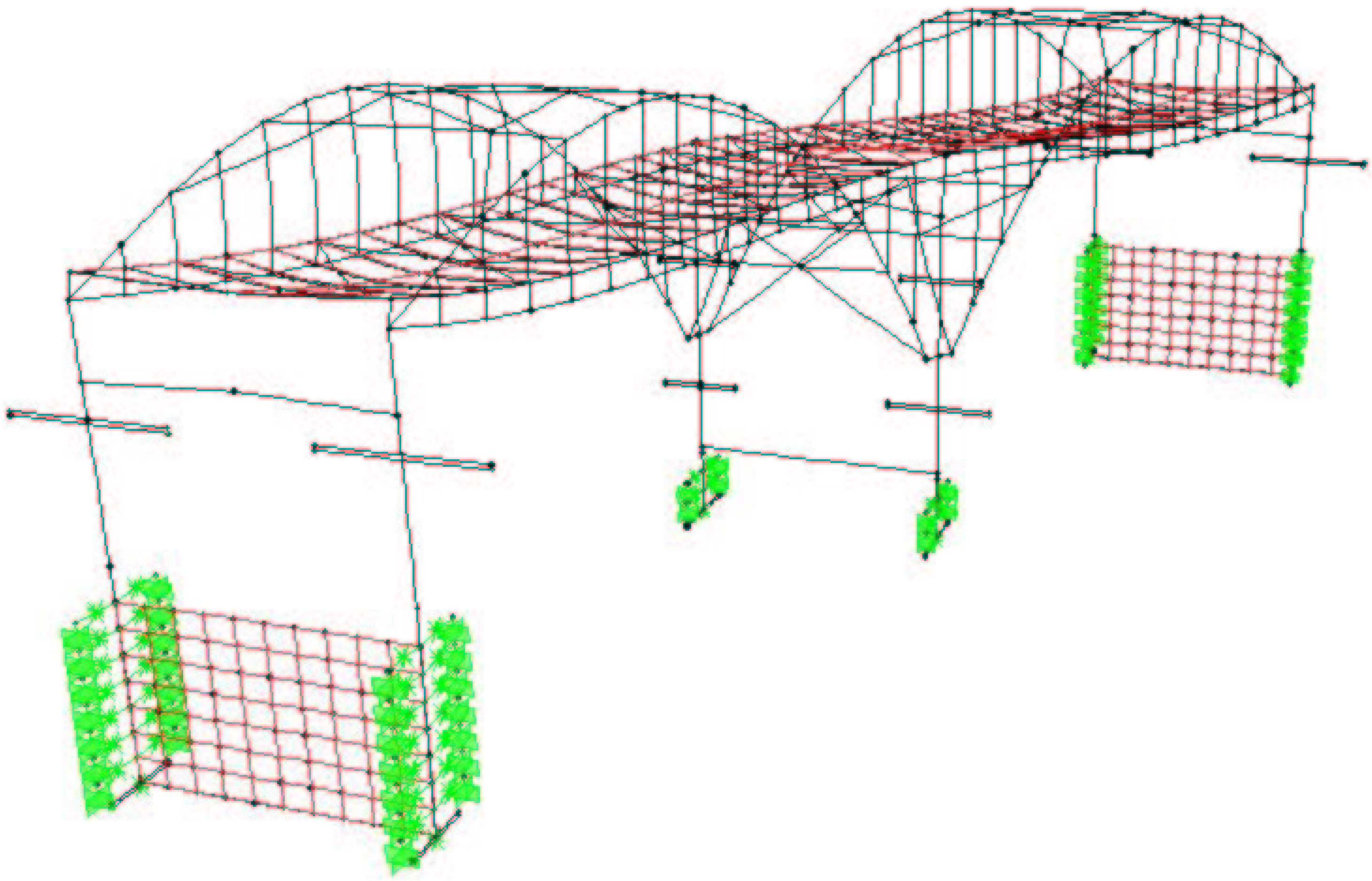
Plan View of the Retrofit Model



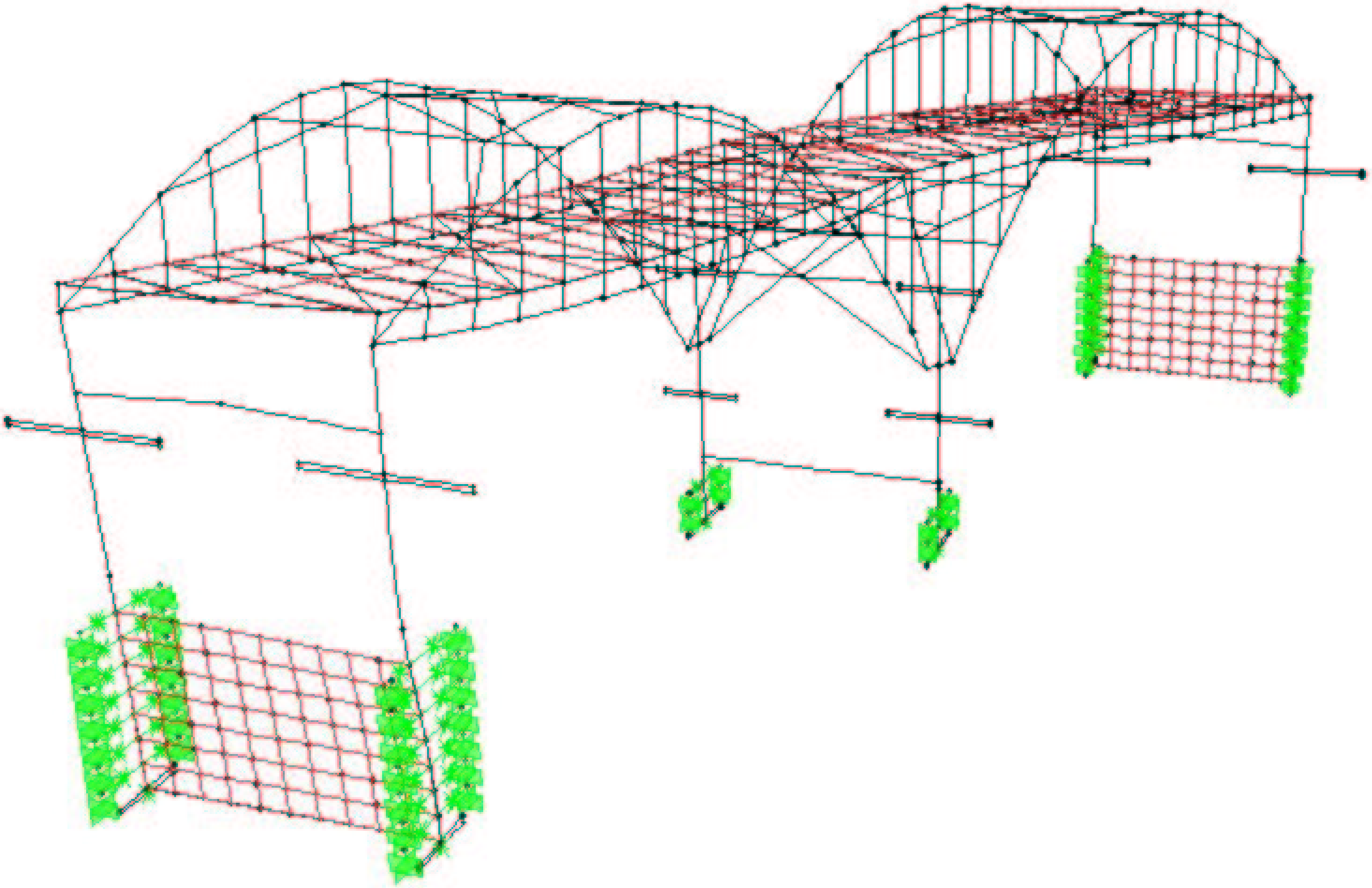
Elevation View of the Retrofit Model



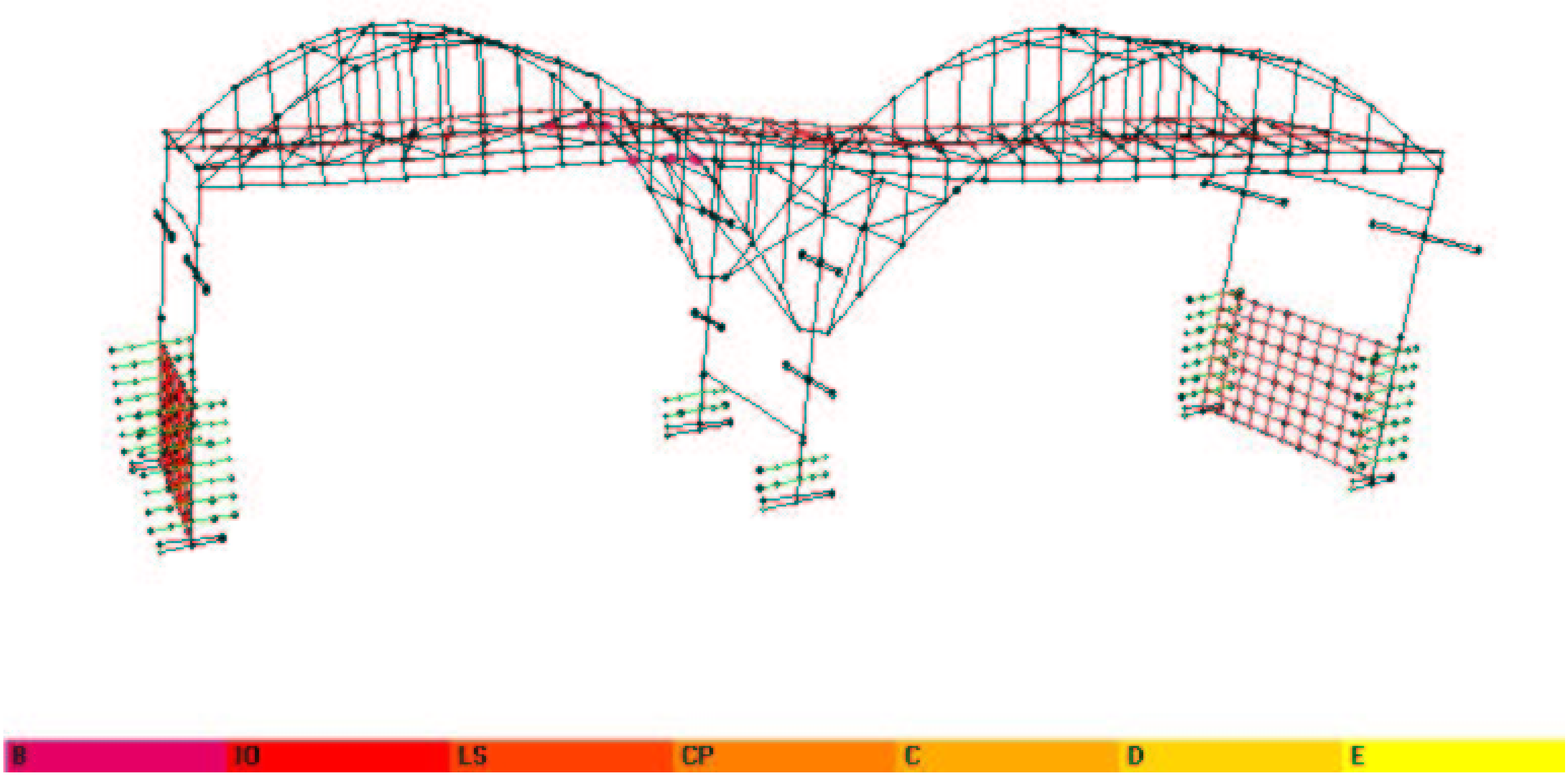
Retrofit: Dead Load Deformations x 100



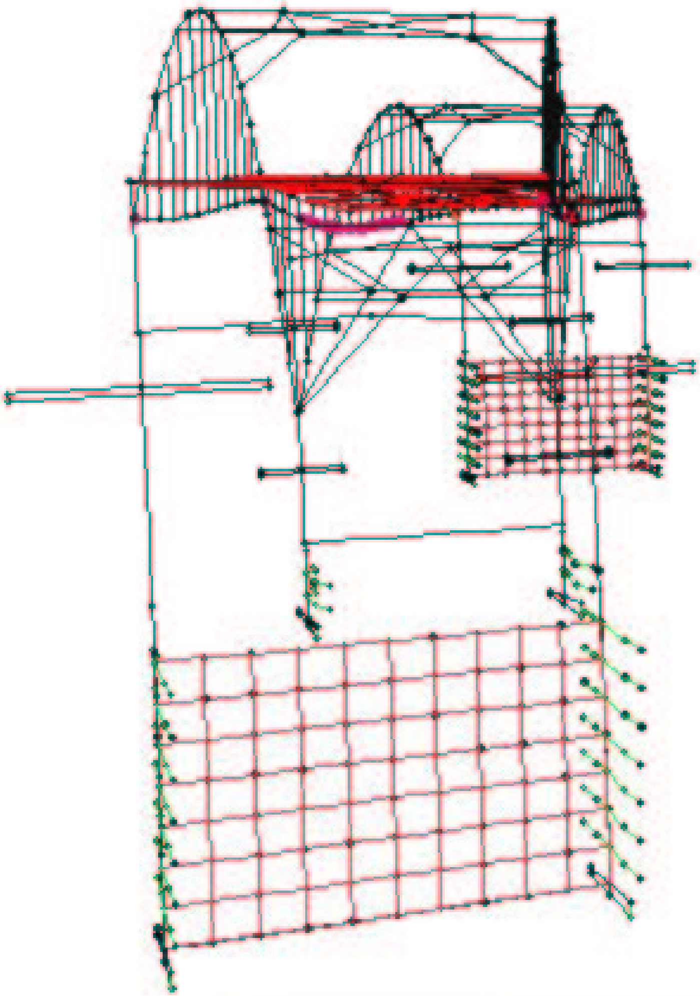
Retrofit: Mode 1 Deformations (0.276 sec)



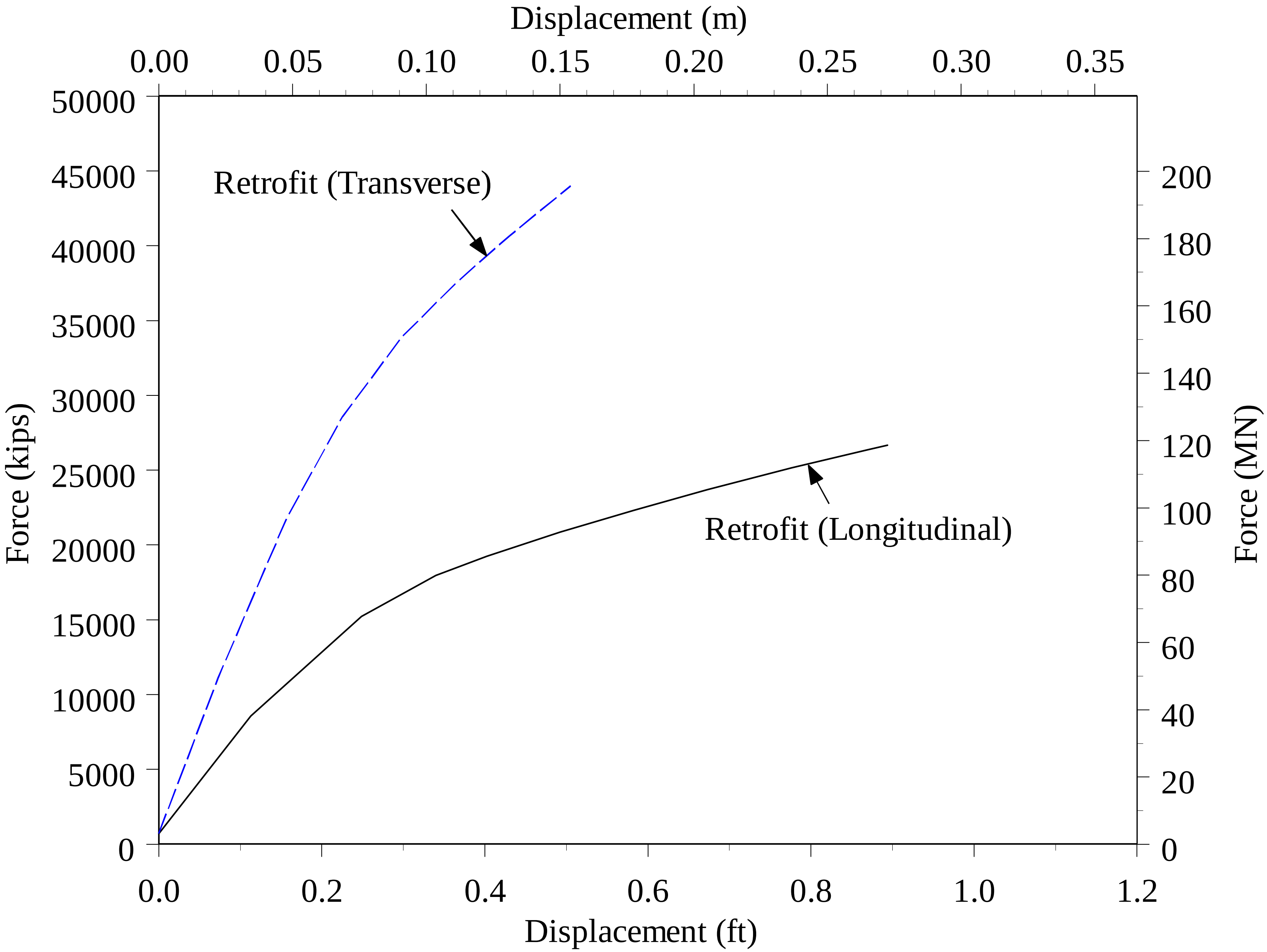
Retrofit Longitudinal Pushover Analysis at 0.891 ft Displacement



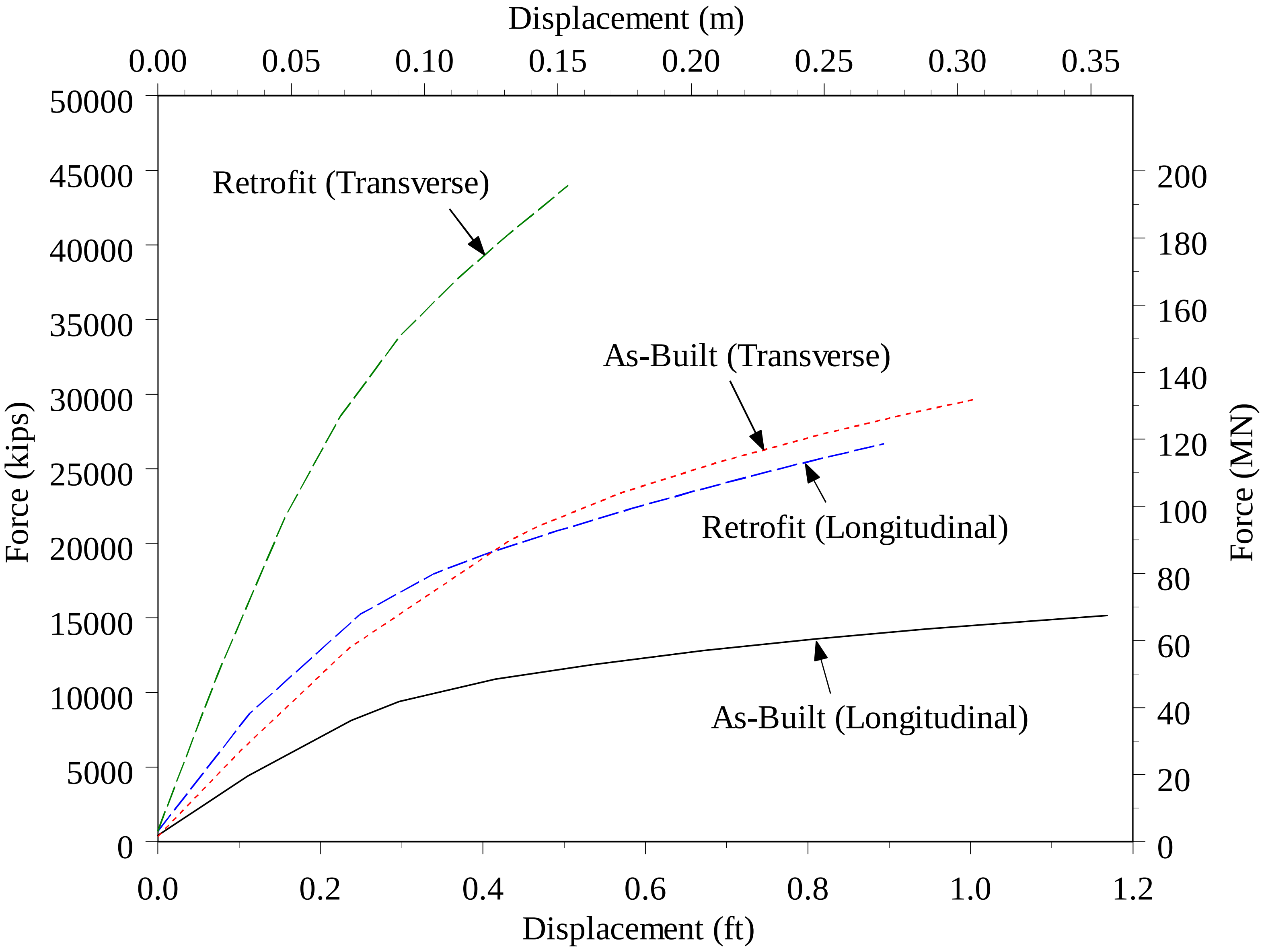
Retrofit Transverse Pushover Analysis at 0.459 ft Displacement



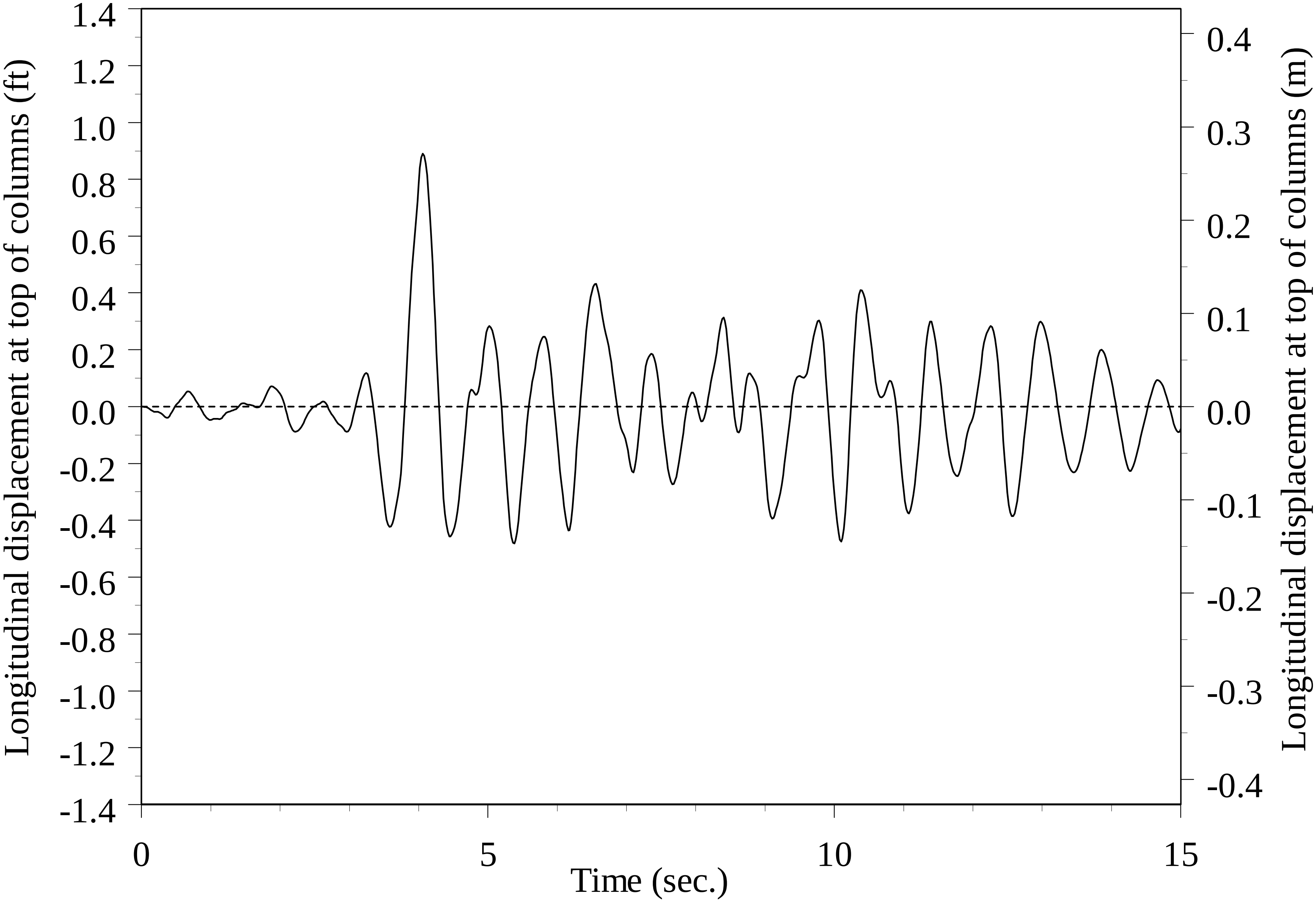
Retrofit Load-Displacement Response (Pushover Analyses)



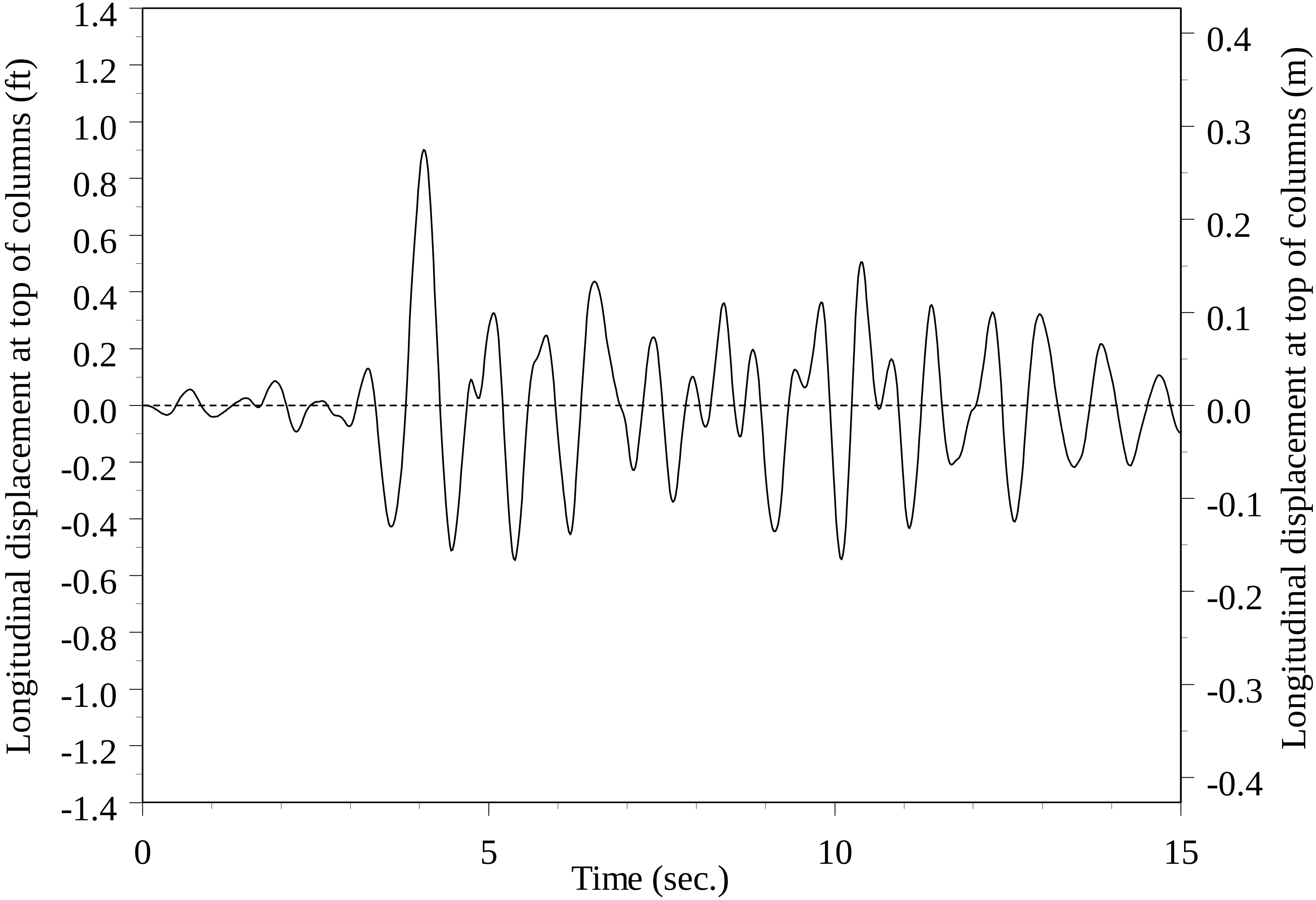
Load-Displacement Response from Pushover Analyses

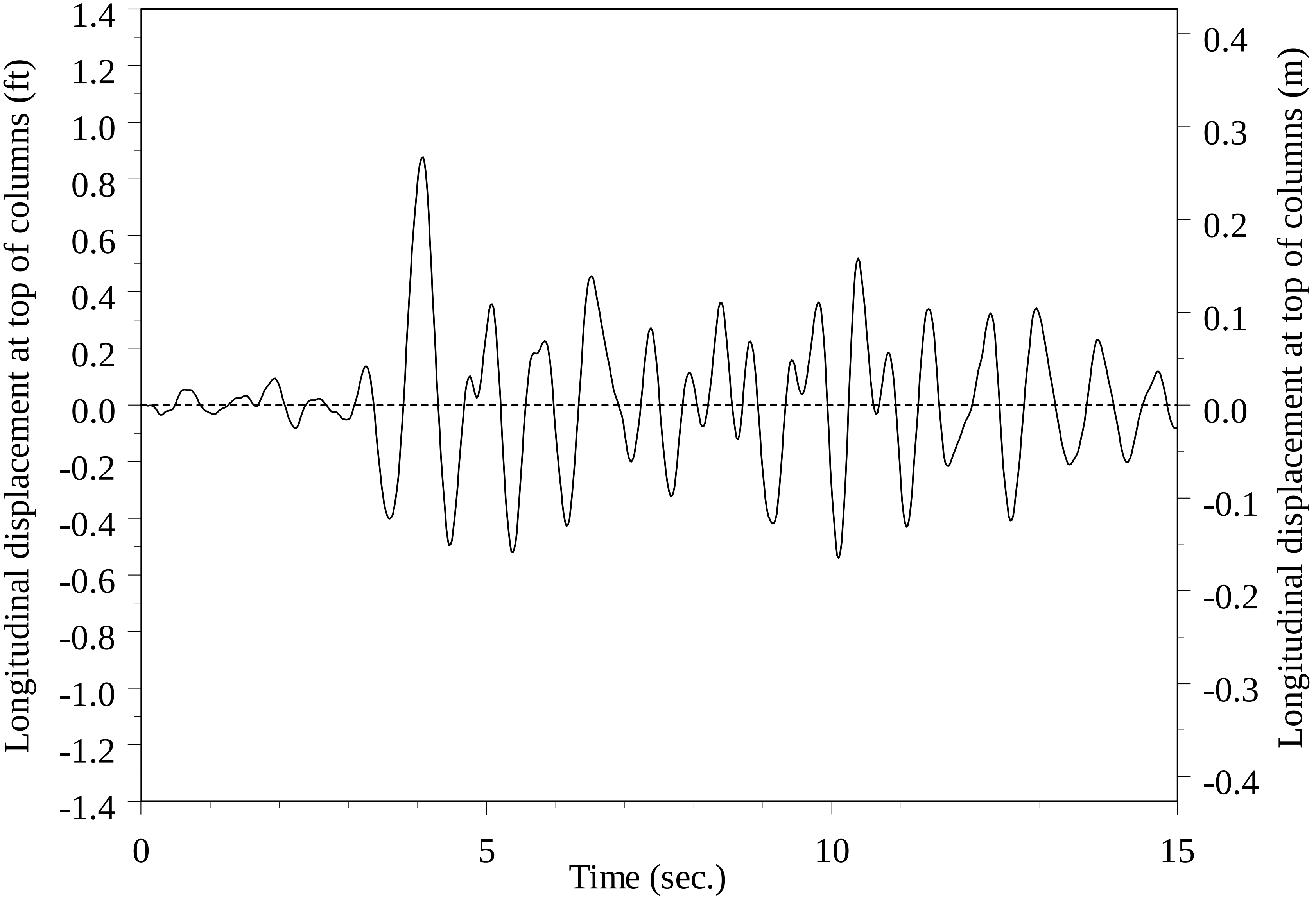


Retrofit: Longitudinal Displacement at Top of West River Pier

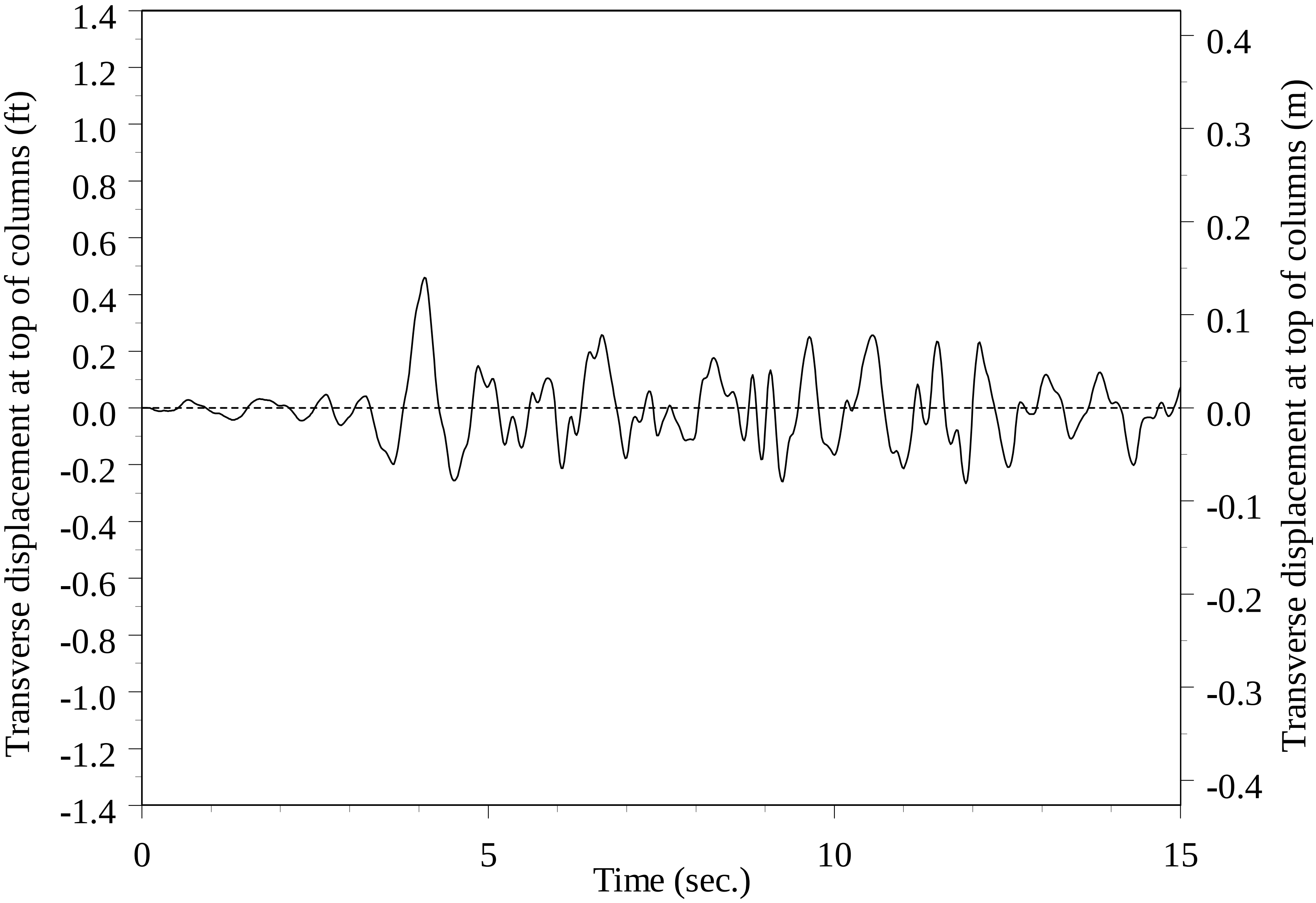


Retrofit: Longitudinal Displacement at Top of Center River Pier



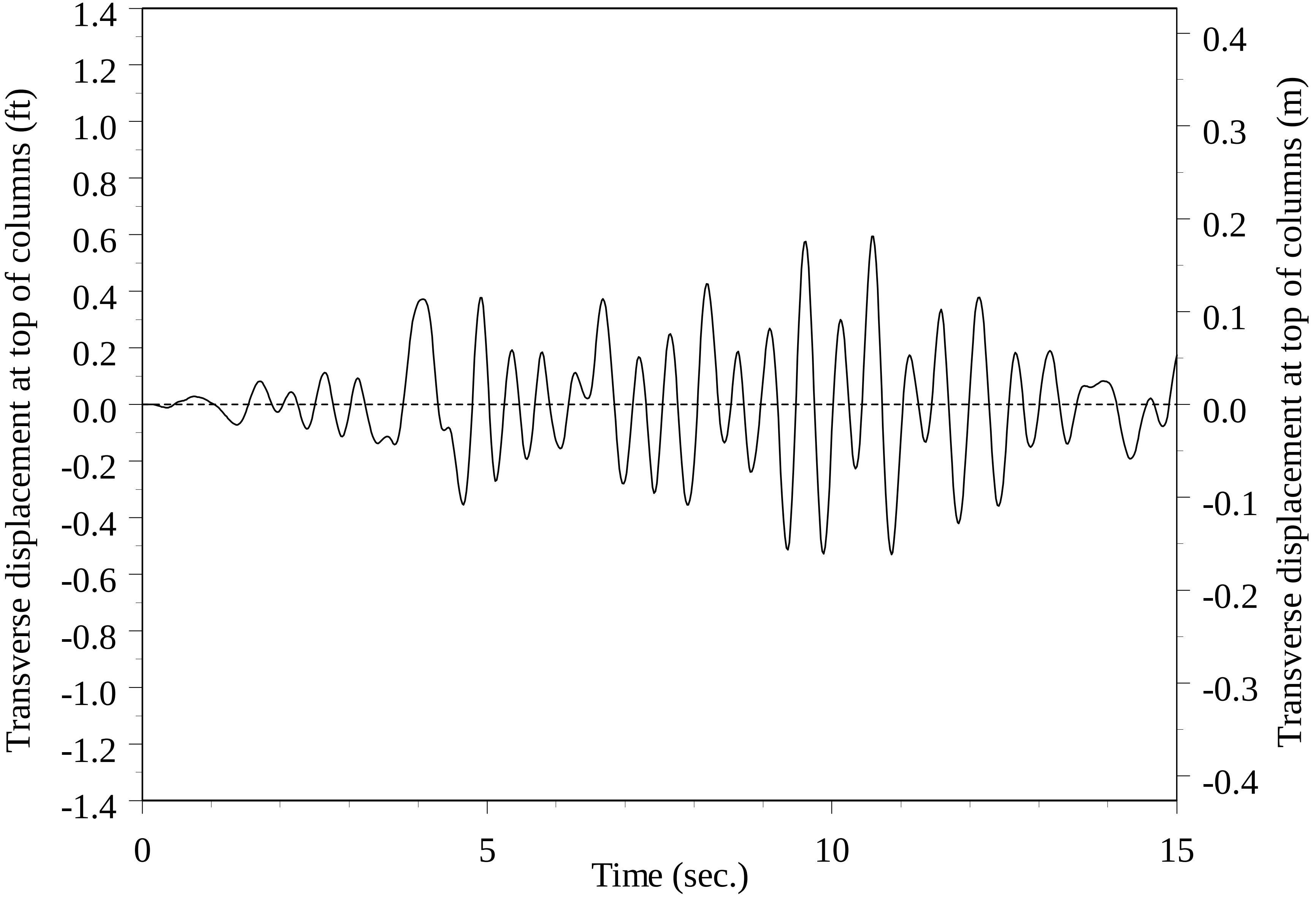


Retrofit: Longitudinal Displacement at Top of East River Pier

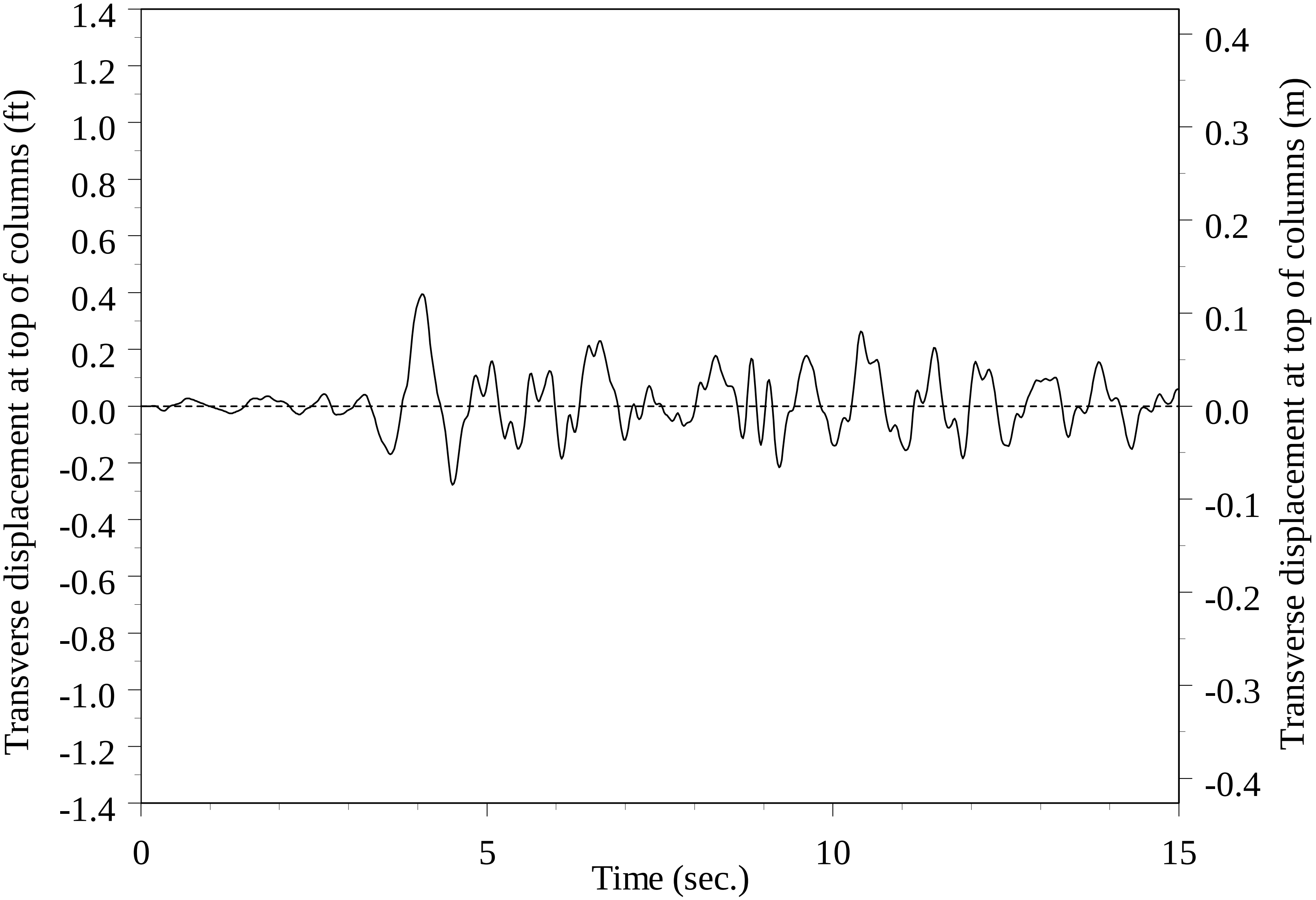


Retrofit: Transverse Displacement at Top of West River Pier

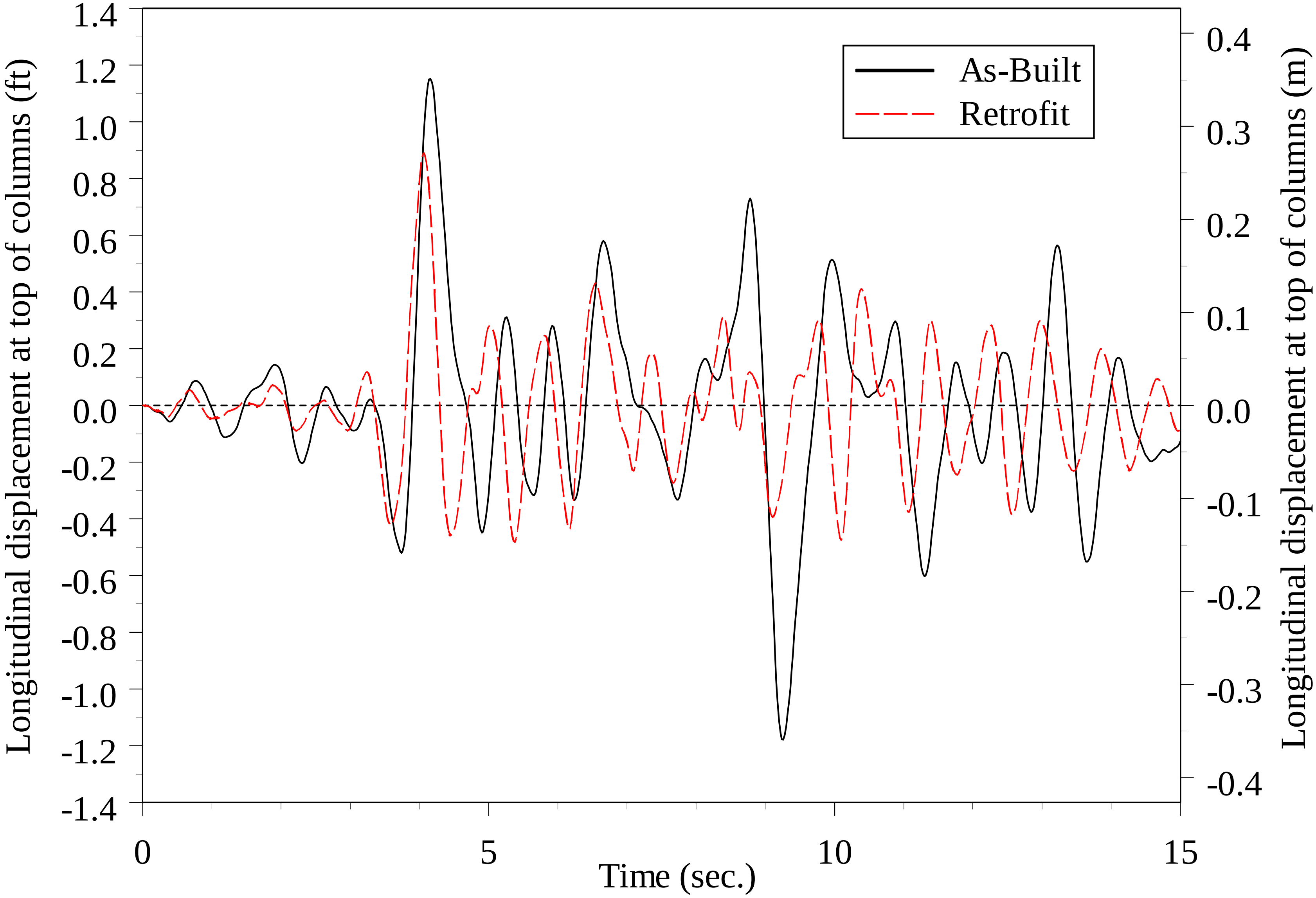
Retrofit: Transverse Displacement at Top of Center River Pier

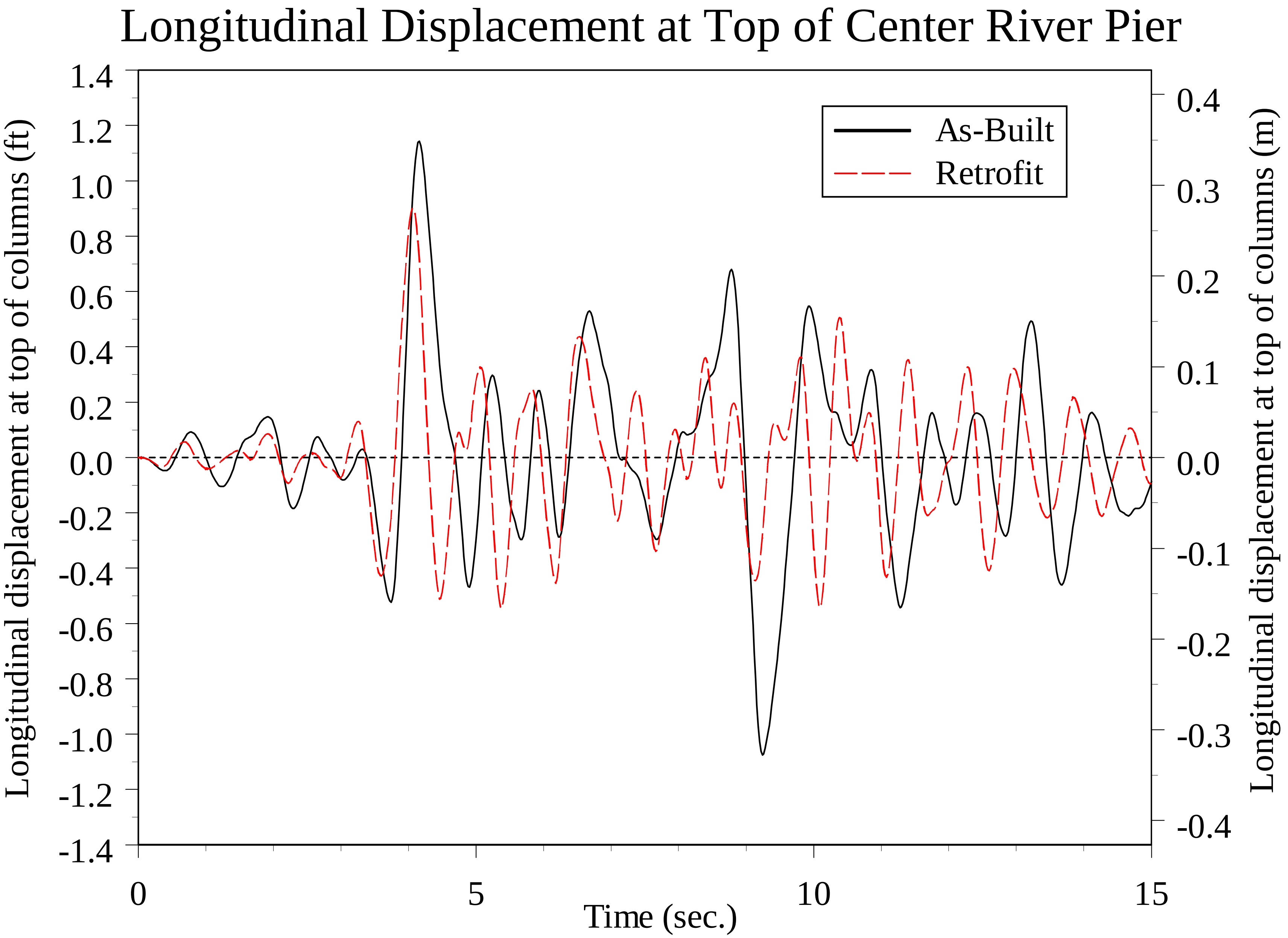


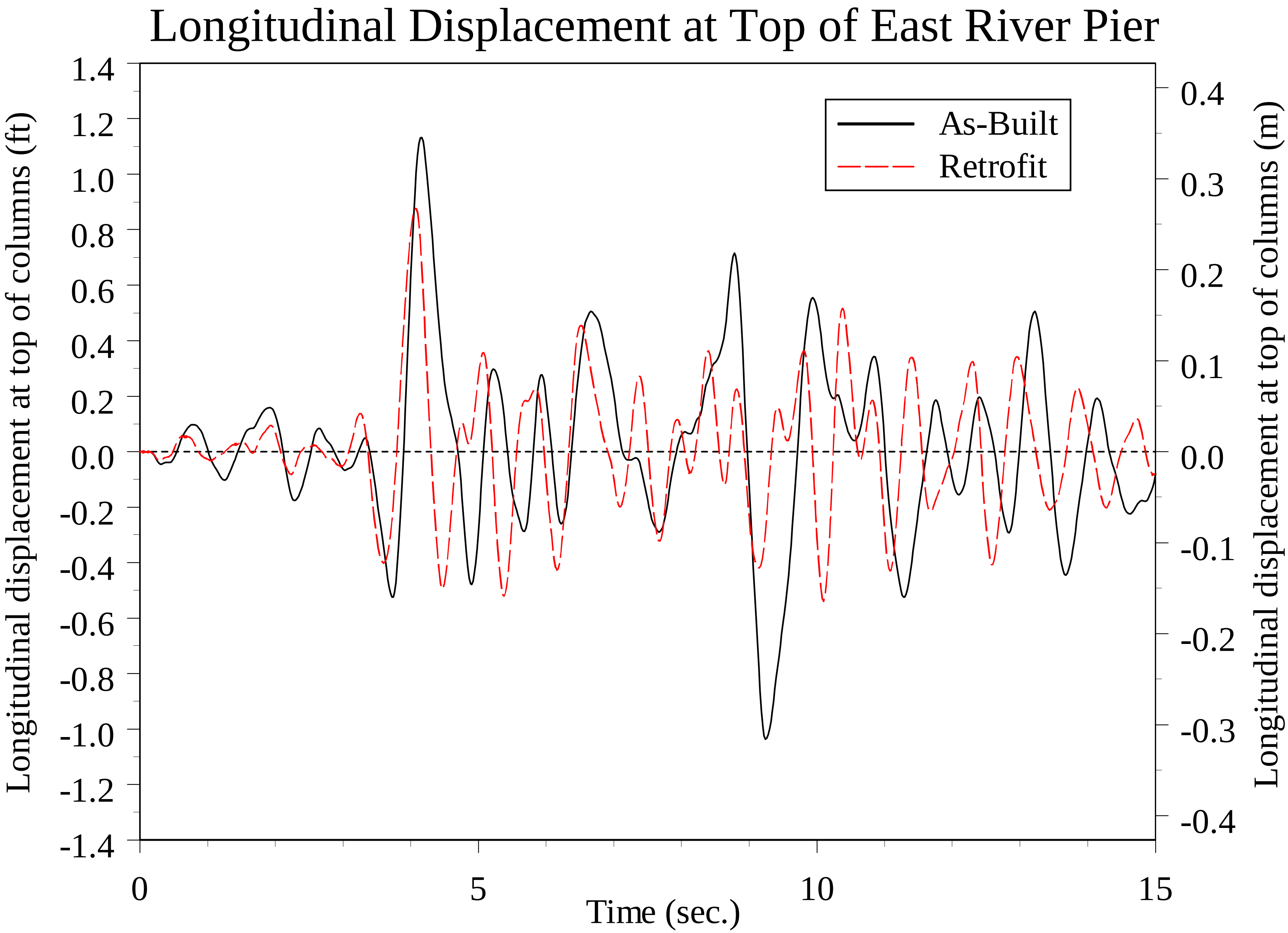
Retrofit: Transverse Displacement at Top of East River Pier

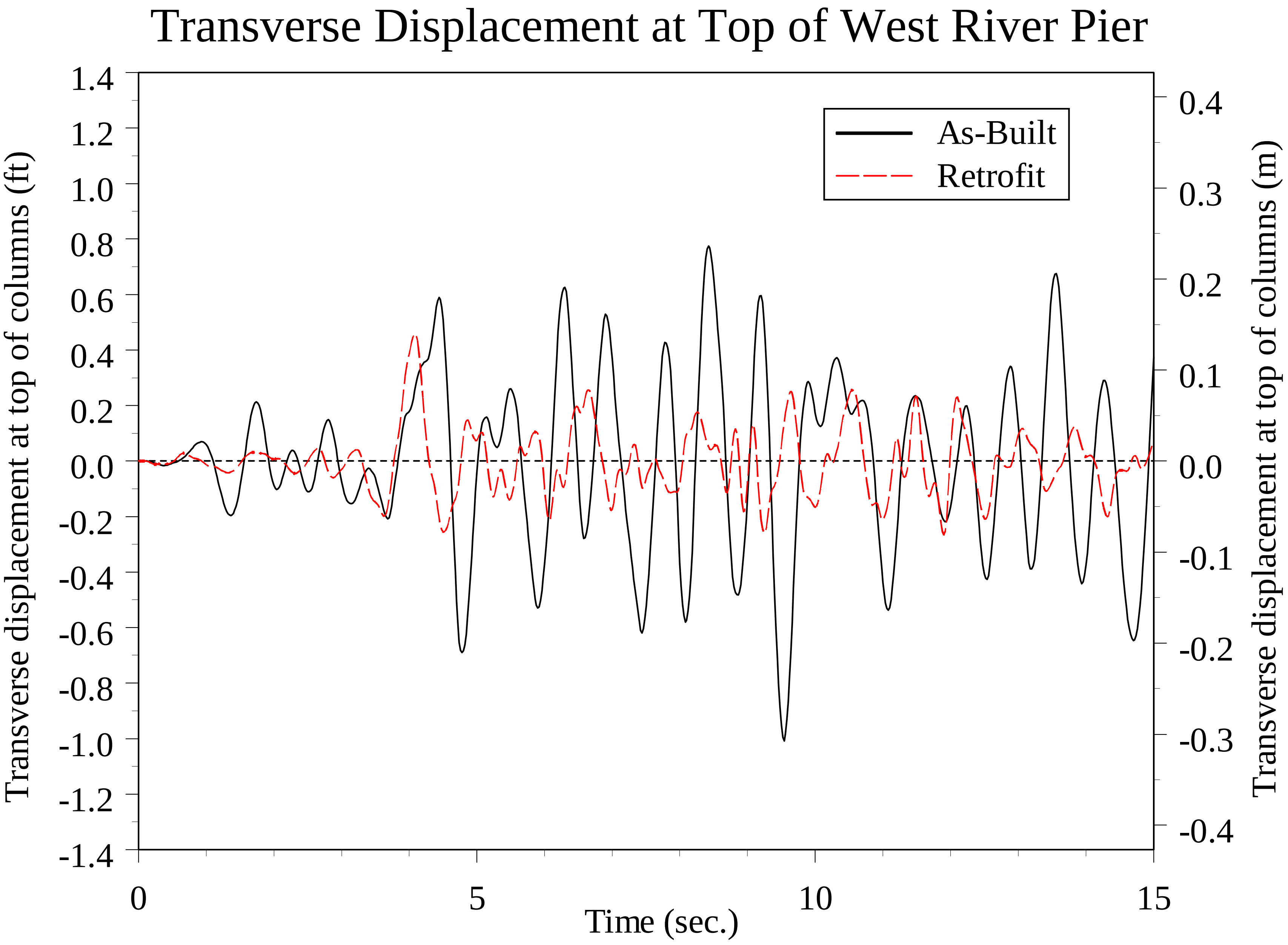


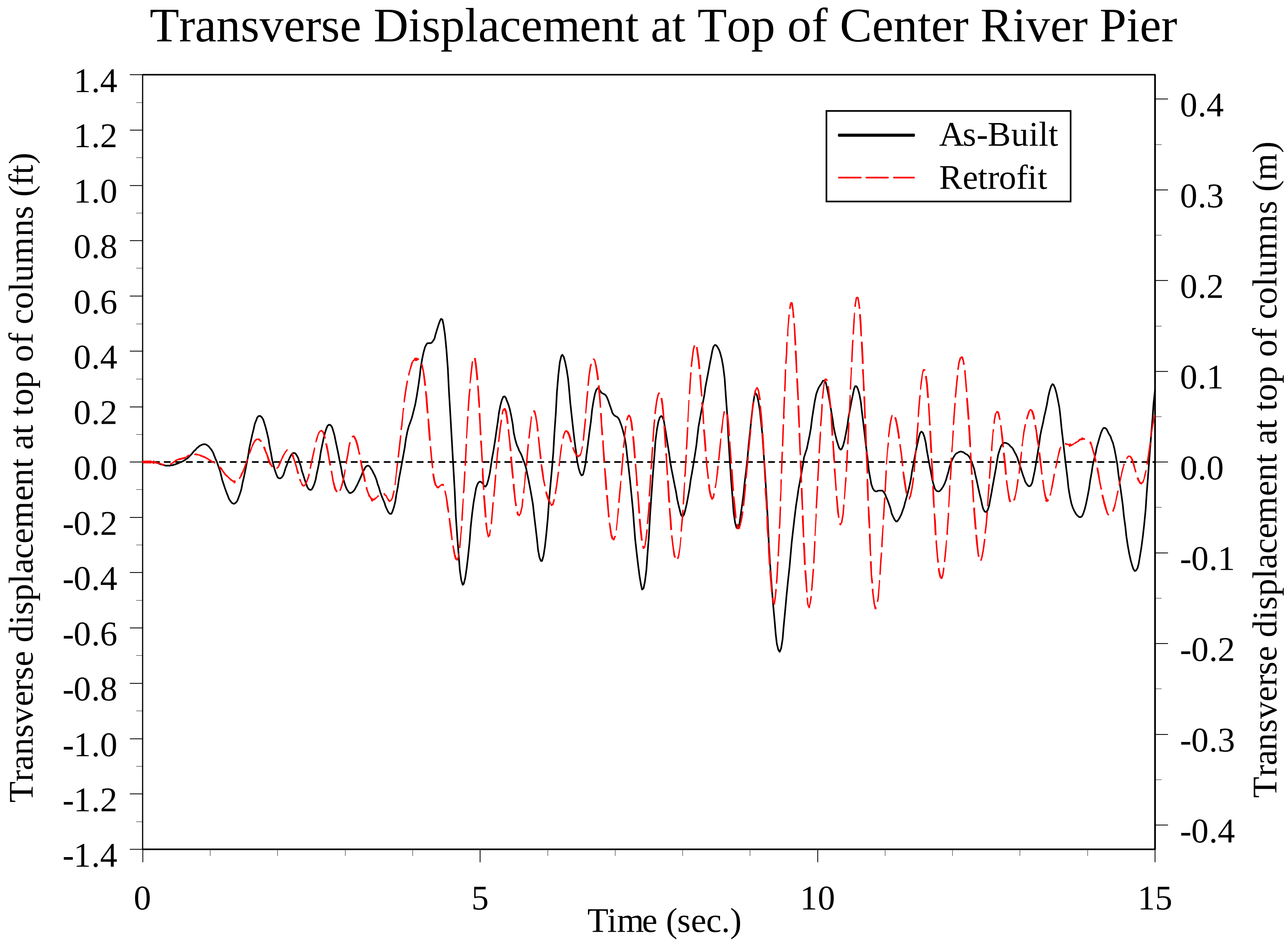
Longitudinal Displacement at Top of West River Pier

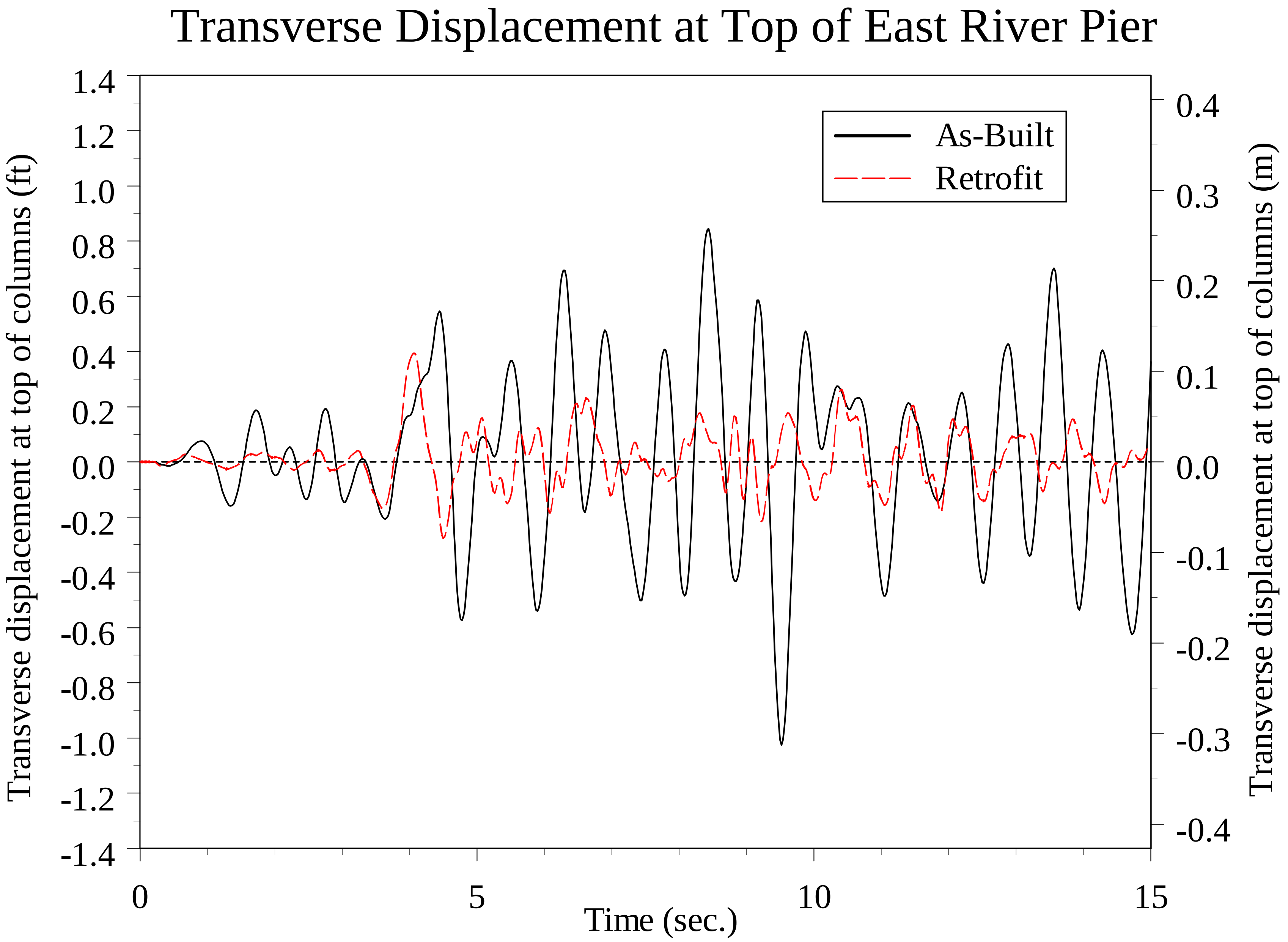



Longitudinal Displacement at Top of Center River Pier
As-Built
Retrofit
Longitudinal displacement at top of columns (ft)
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
-1.2
-1.4
Longitudinal displacement at top of columns (m)
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
-0.4
0
5
10
15
Time (sec.)

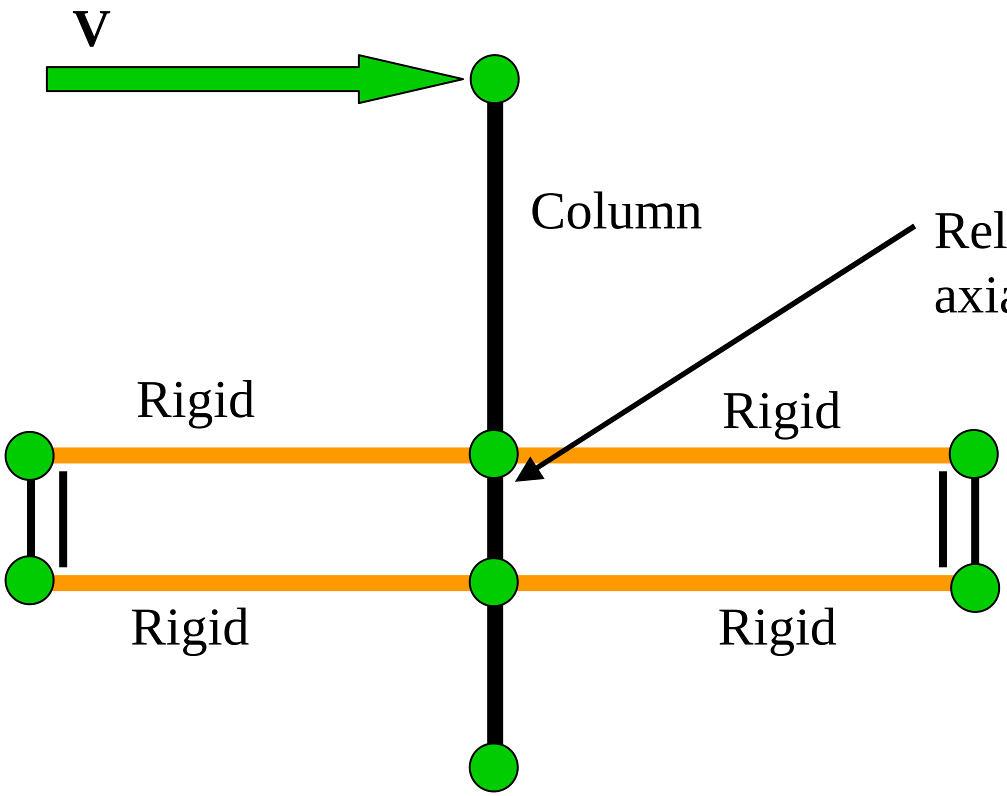

Longitudinal Displacement at Top of East River Pier
As-Built
Retrofit
Longitudinal displacement at top of columns (ft)
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
-1.2
-1.4
Longitudinal displacement at top of columns (m)
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
-0.4
0
5
10
15
Time (sec.)


Transverse Displacement at Top of West River Pier
As-Built
Retrofit
Transverse displacement at top of columns (ft)
Transverse displacement at top of columns (m)
Time (sec.)
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
-1.2
-1.4
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
-0.4
0
5
10
15


Transverse Displacement at Top of Center River Pier
As-Built
Retrofit
Transverse displacement at top of columns (ft)
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
-1.2
-1.4
Transverse displacement at top of columns (m)
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
-0.4
0
5
10
15
Time (sec.)


Transverse Displacement at Top of East River Pier
As-Built
Retrofit
Transverse displacement at top of columns (ft)
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
-1.2
-1.4
Transverse displacement at top of columns (m)
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
-0.4
0
5
10
15
Time (sec.)

Breakout Model for RC Column Plastic Hinges



V

Column

Release end moment and
axial load

Rigid

Rigid

Nonlinear axial
link elements

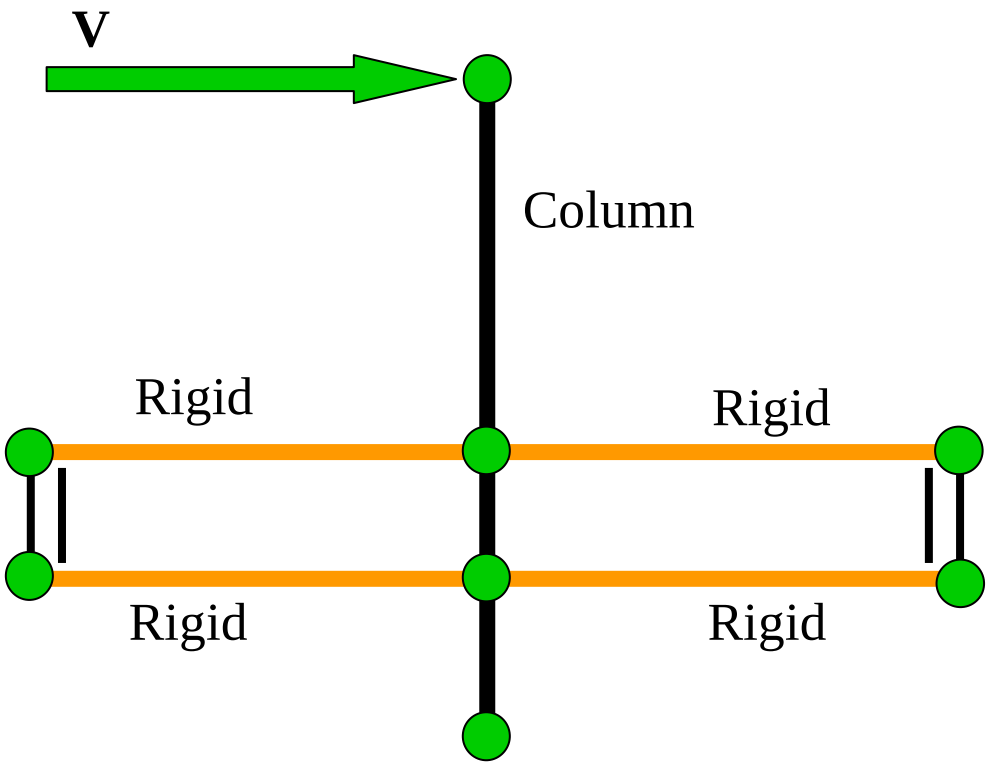
Rigid

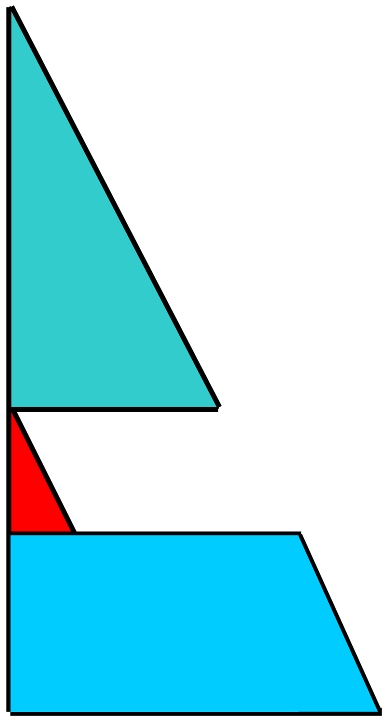
Rigid

Nonlinear axial link elements for concrete & reinforcement bars

Breakout Model for RC Column Plastic Hinges

Moment Transfer of RC Plastic Hinge Assembly





Column Bending
Moment Diagram

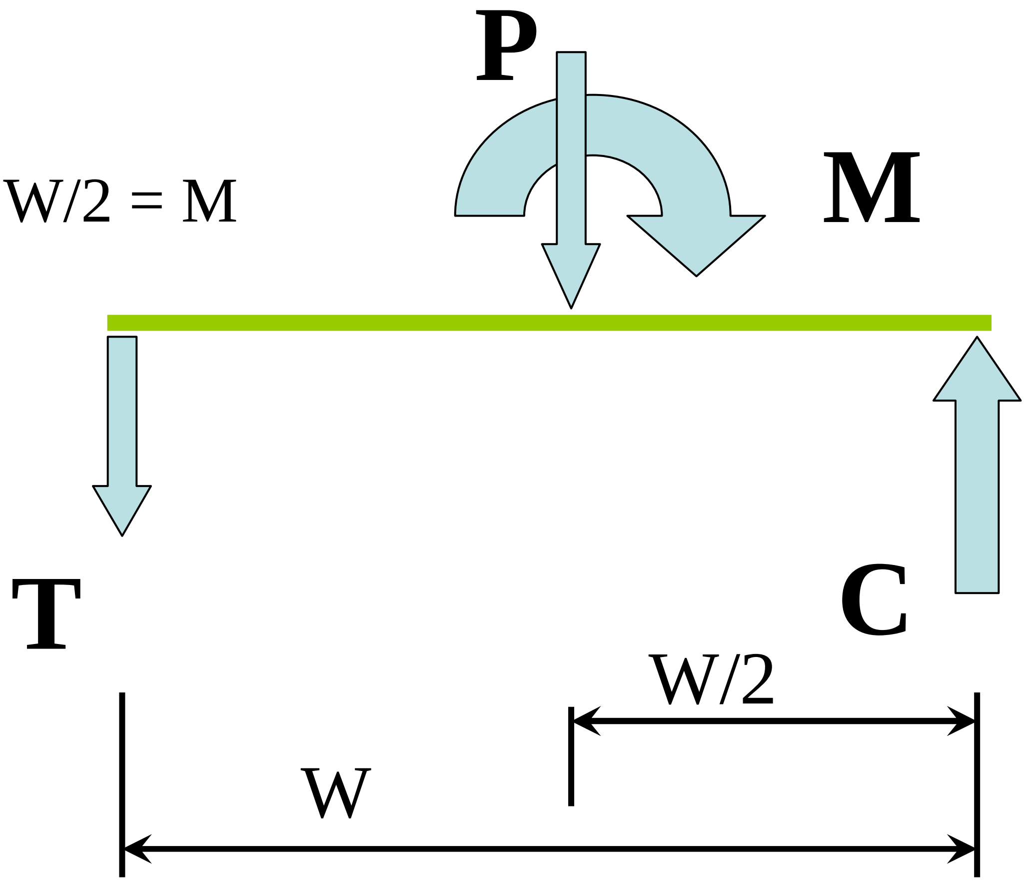
Breakout Model for RC Column Plastic Hinges

Force Balance of RC Plastic Hinges

$A_s = A_{st}/2$

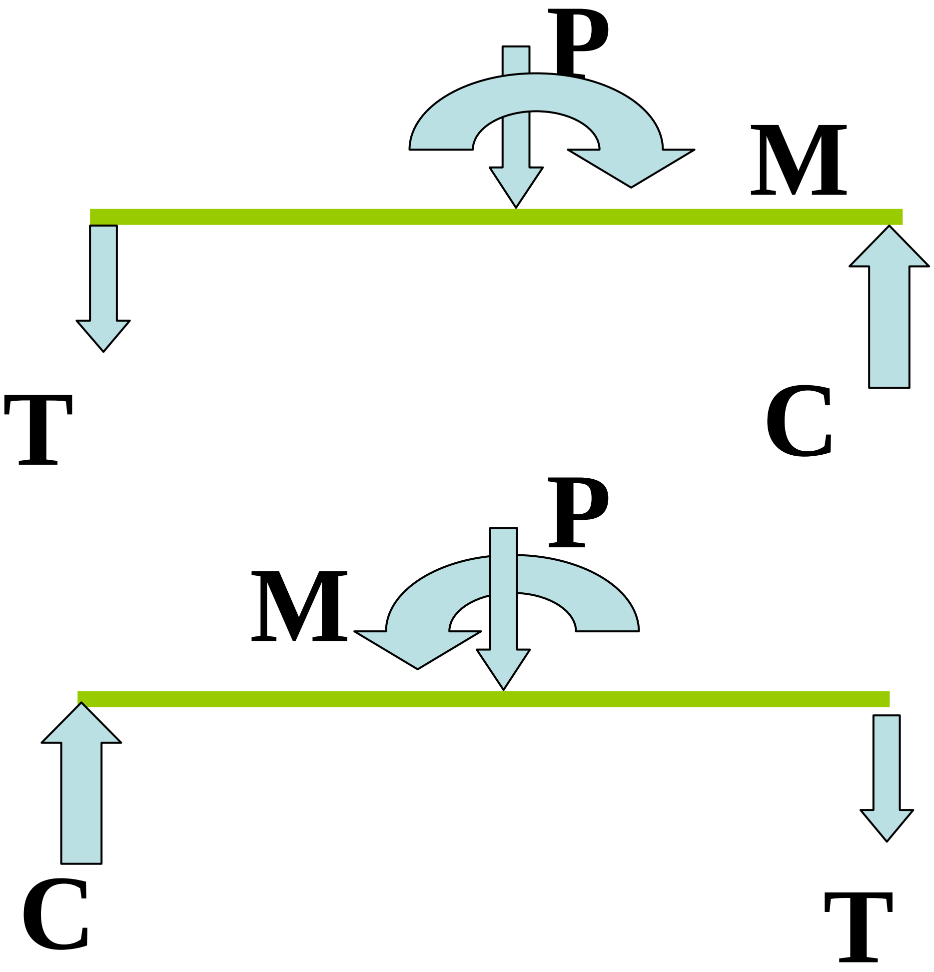
$T = A_{s*}f_y$

$T*W + P*W/2 = M$



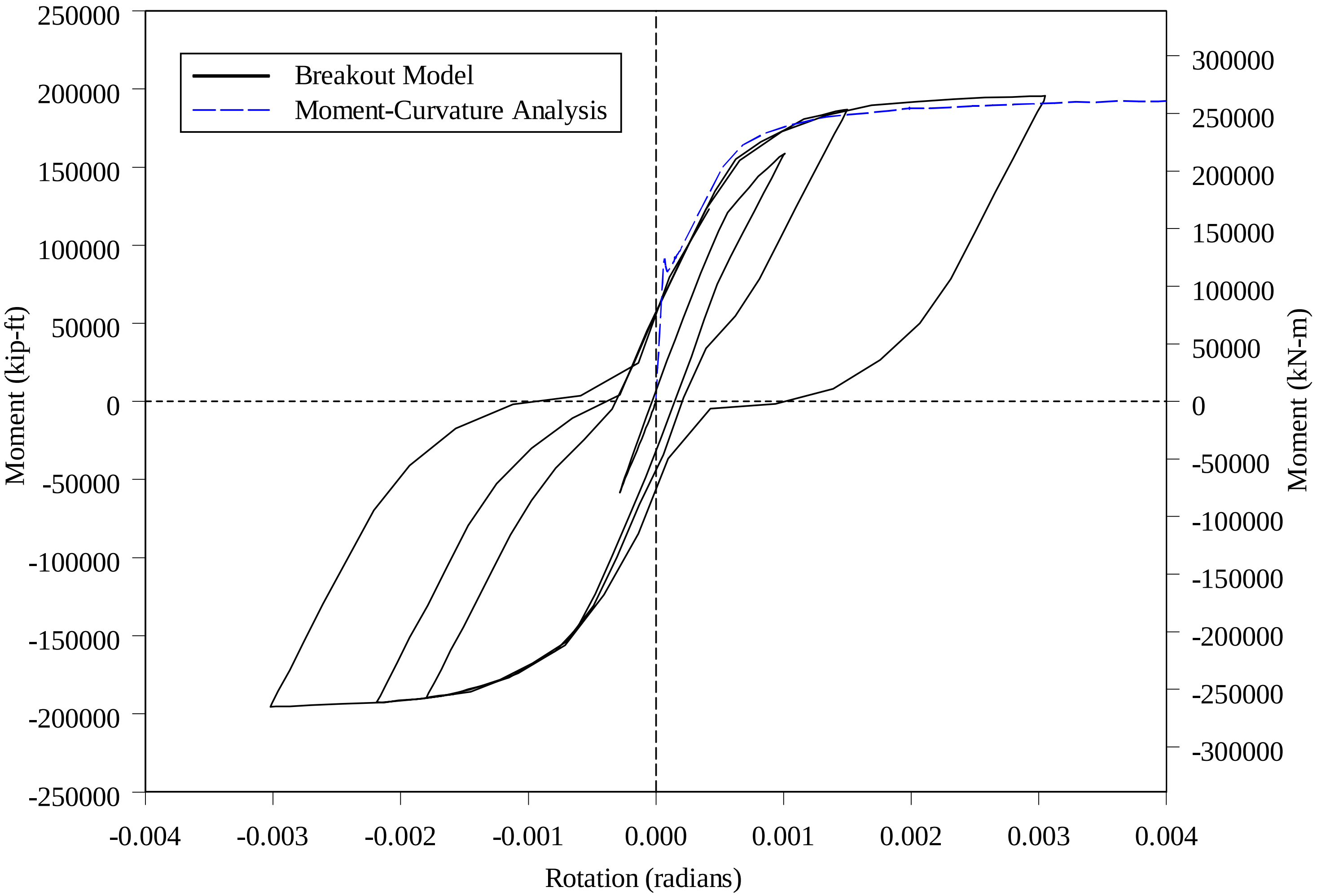
Breakout Model for RC Column Plastic Hinges

Forces from Cyclic Loading

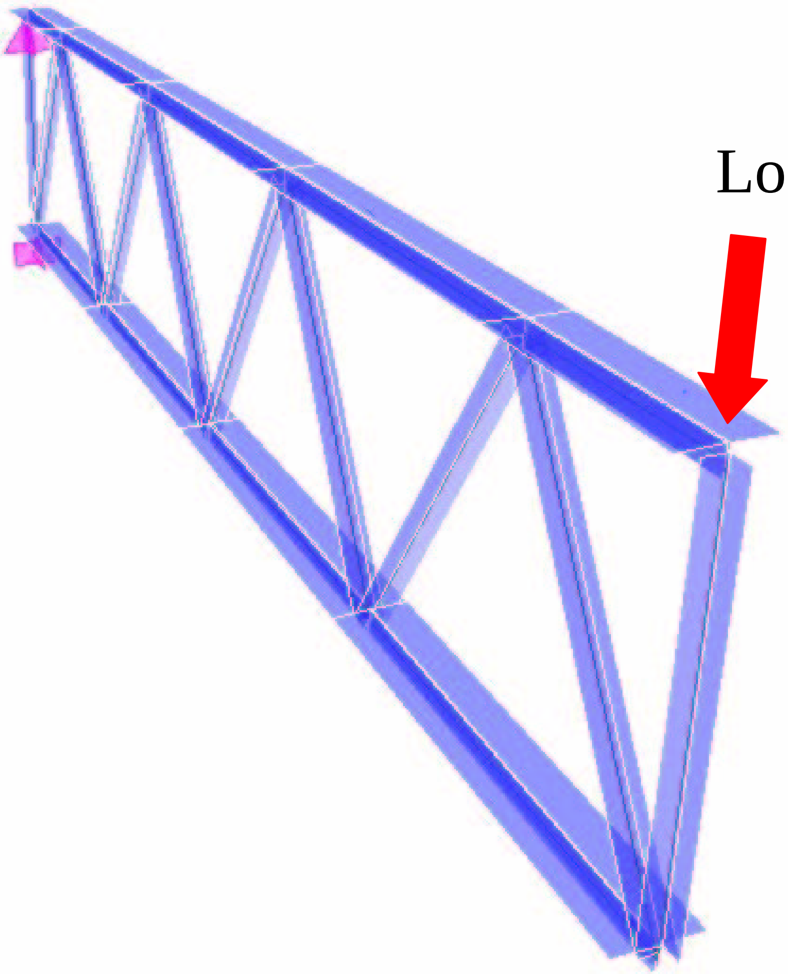


Breakout Model for RC Column Plastic Hinges

Validation of the Breakout Model

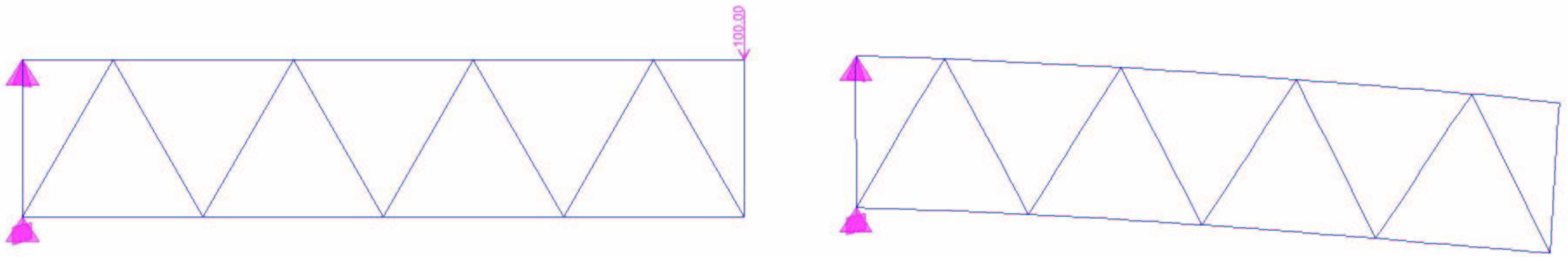


Breakout Model for a Cantilever Laced Member



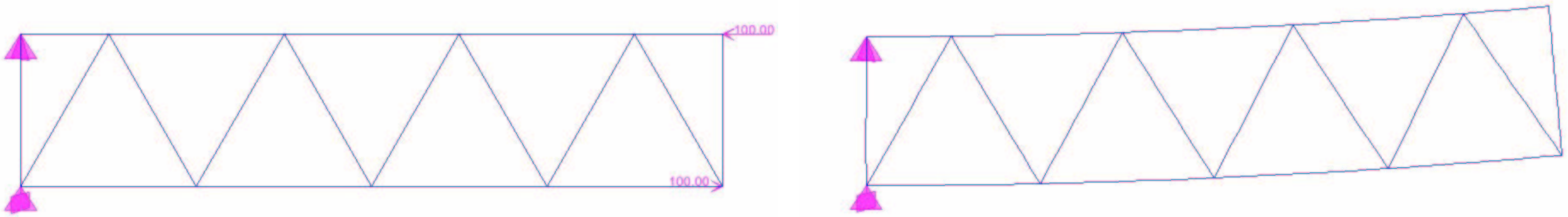
Breakout Model for a Cantilever Laced Member

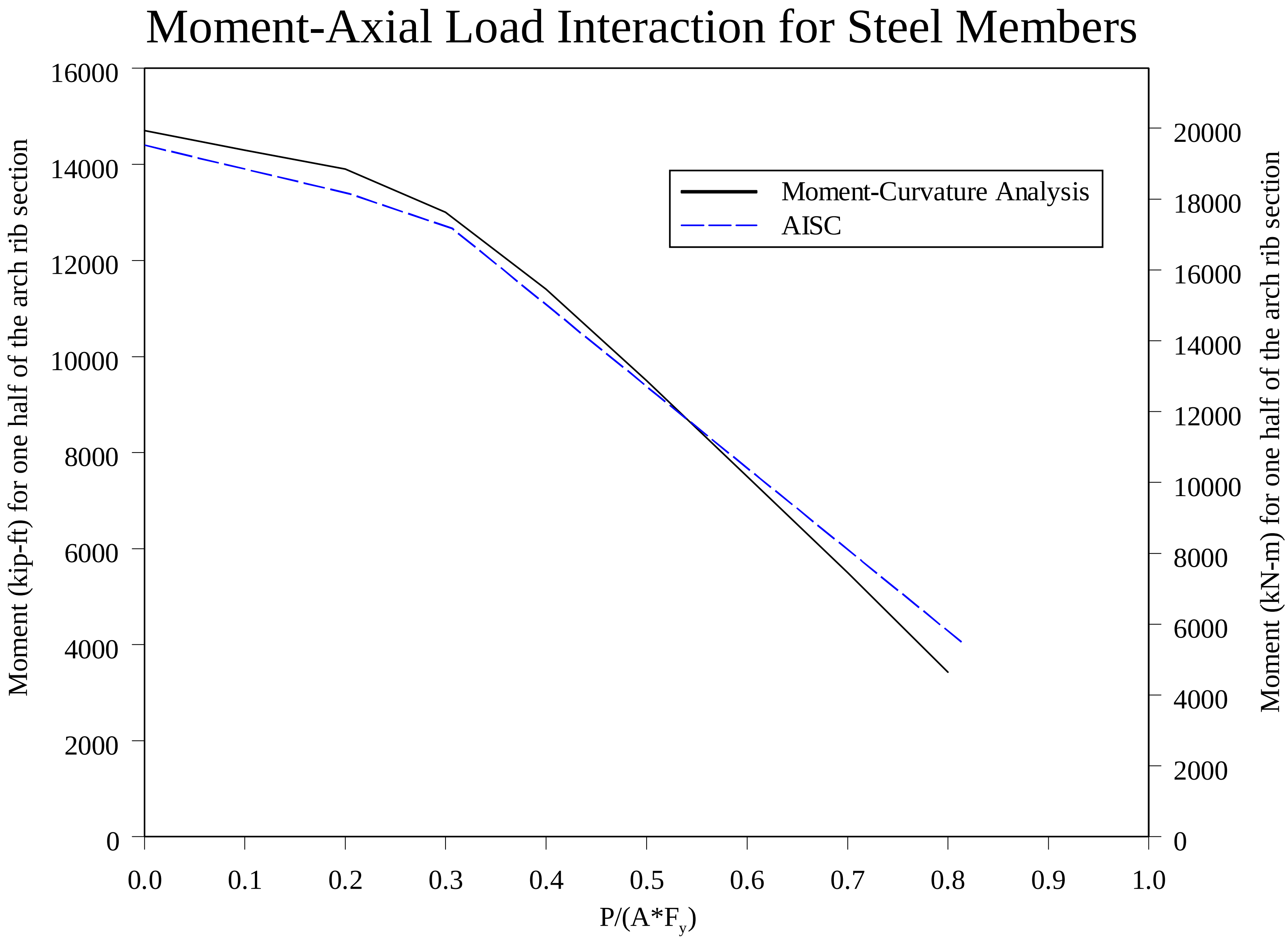
Laced Member Subjected to Shear and Linear Bending



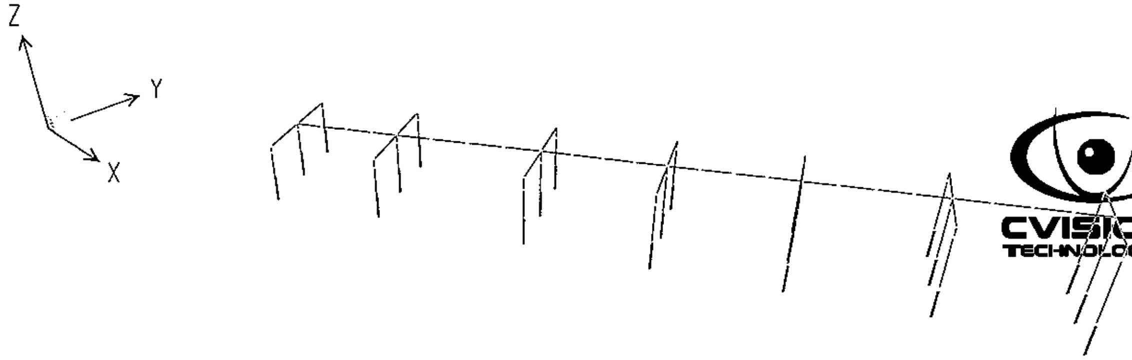
Breakout Model for a Cantilever Laced Member

Laced Member Subjected to Pure Bending

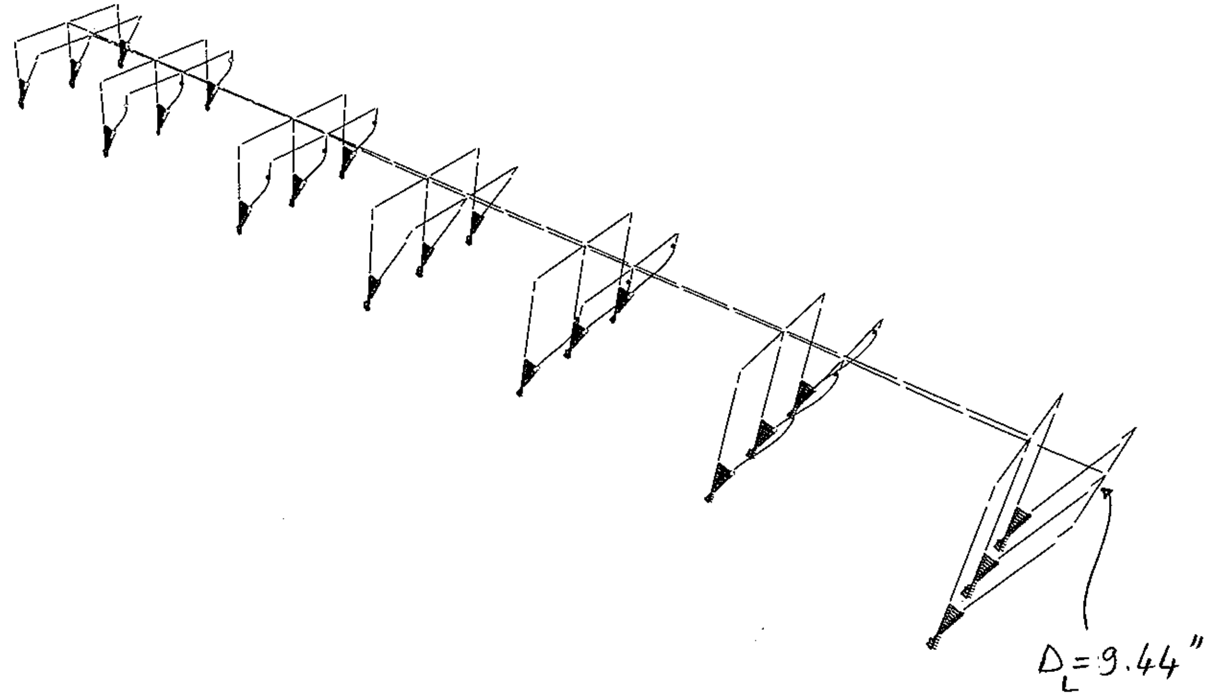



Moment-Axial Load Interaction for Steel Members
Moment (kip-ft) for one half of the arch rib section
0
2000
4000
6000
8000
10000
12000
14000
16000
Moment (kN-m) for one half of the arch rib section
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
P/(A*F$_y$)
Moment-Curvature Analysis
AISC

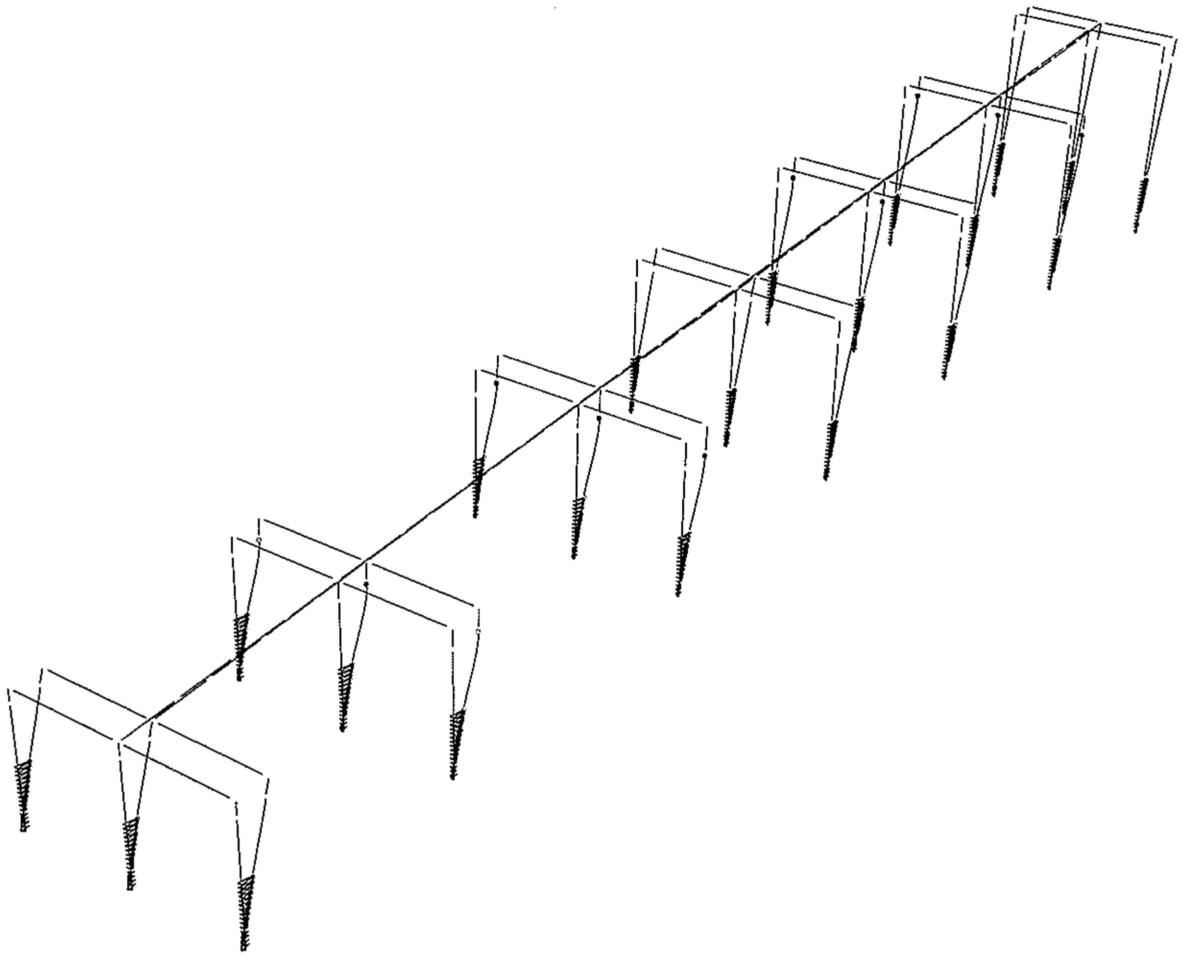
Approach Spans (As-Built)



SAP2000 v7.44 - File:b3-9l - 3-D View - Kip-ft Units

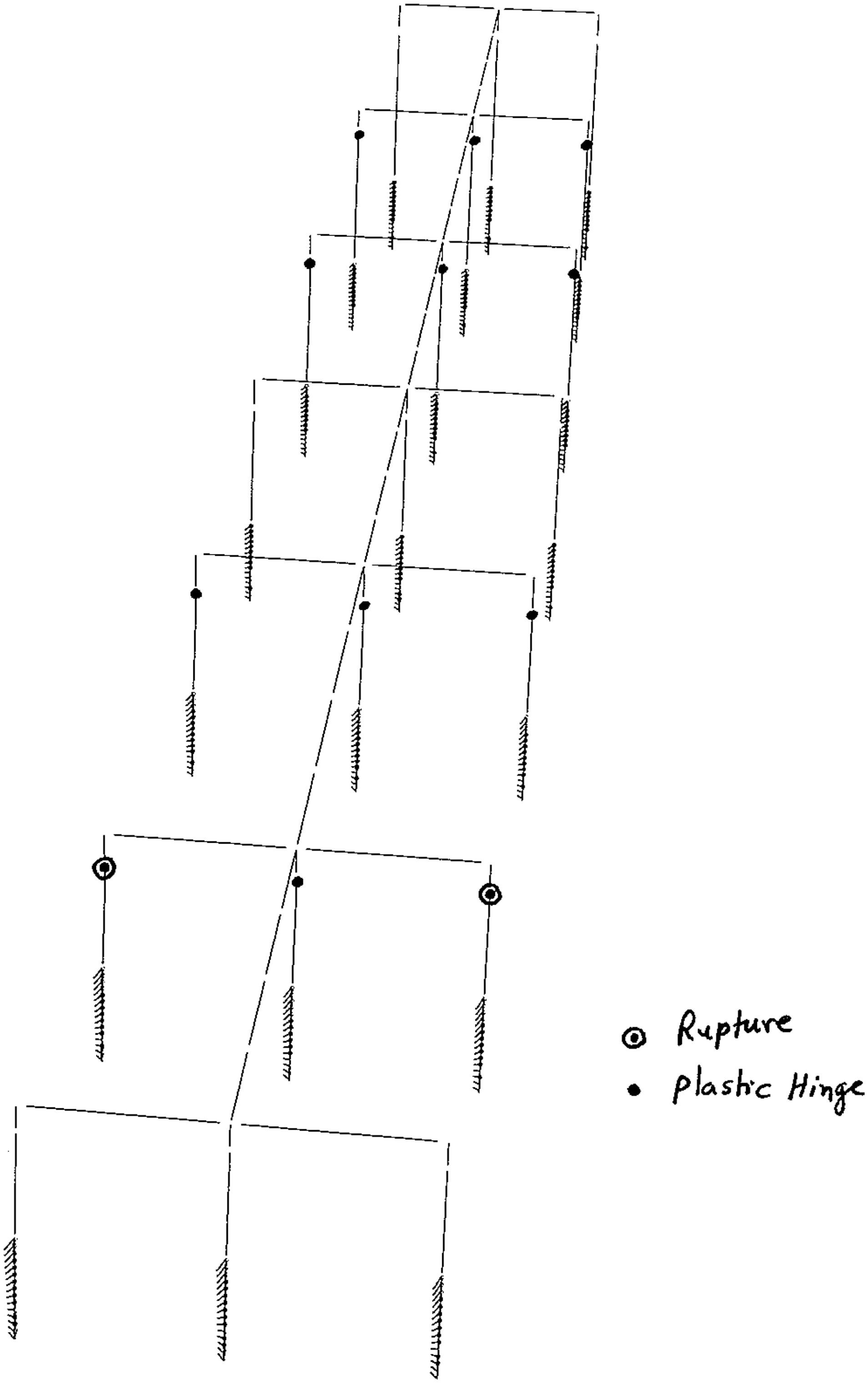


SAP2000 v7.44 - File:b3-9l - Deformed Shape (PUSHL - Step 17) - Kip-ft Units

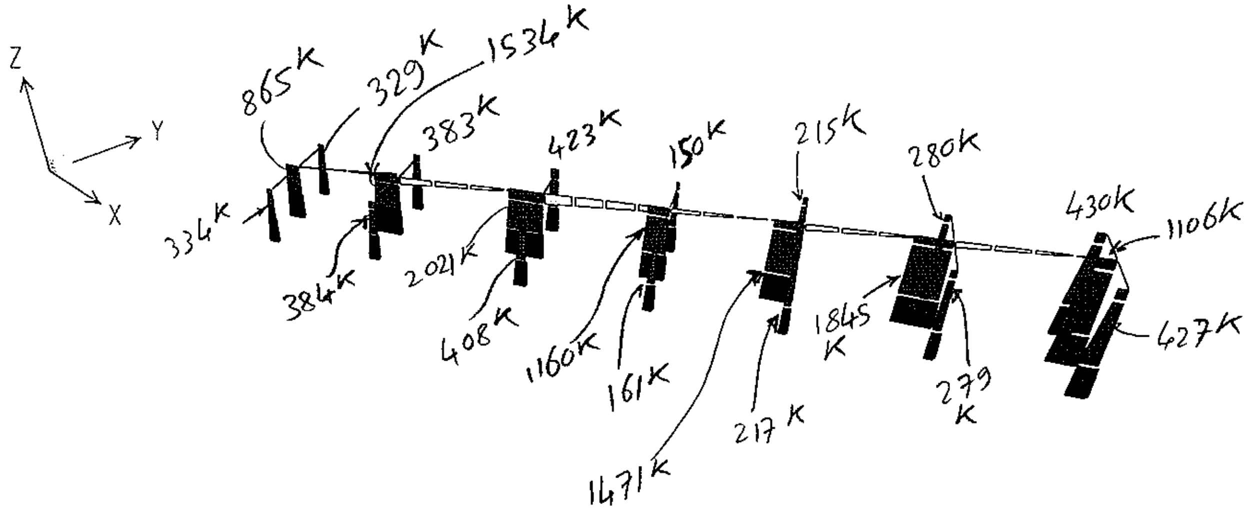


SAP2000 v7.44 - File:b3-9l - Deformed Shape (PUSHL - Step 17) - Kip-ft Units

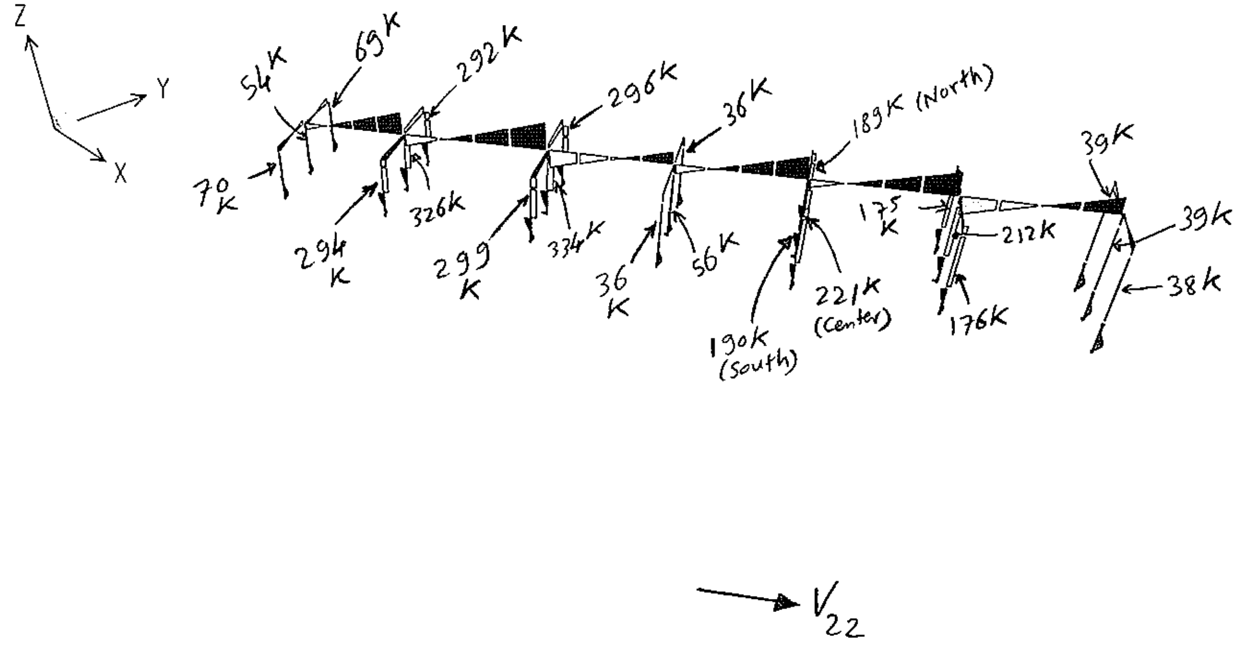
SAP2000 12/24/03 7:09:23



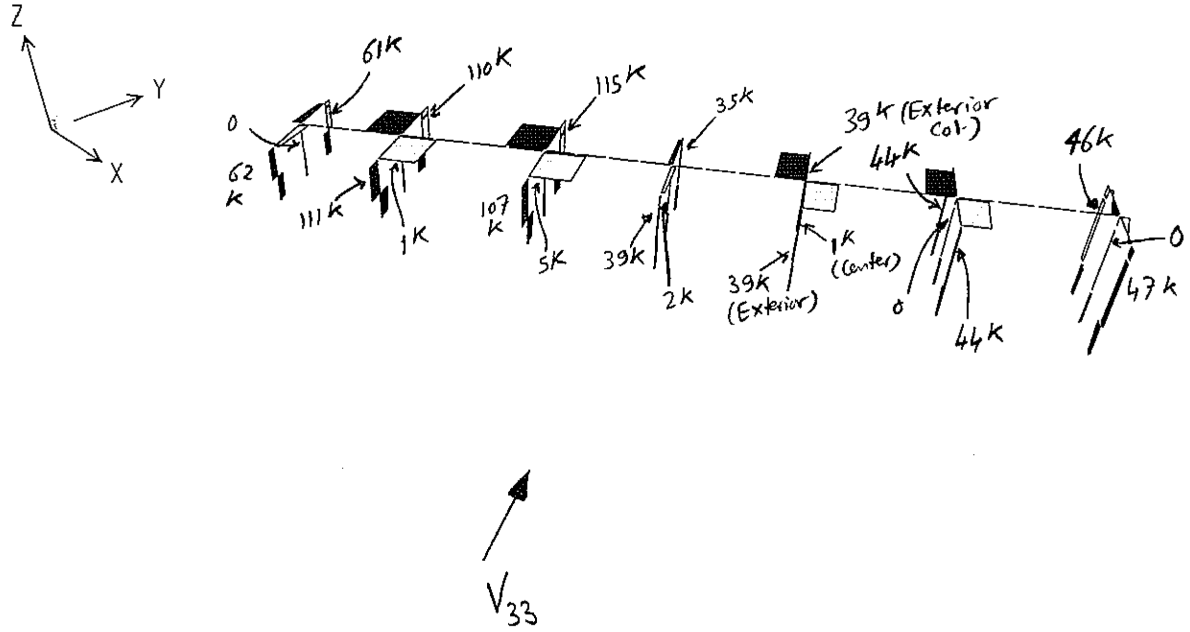
SAP2000 v7.44 - File:b3-9l - Deformed Shape (PUSHL - Step 17) - Kip-ft Units



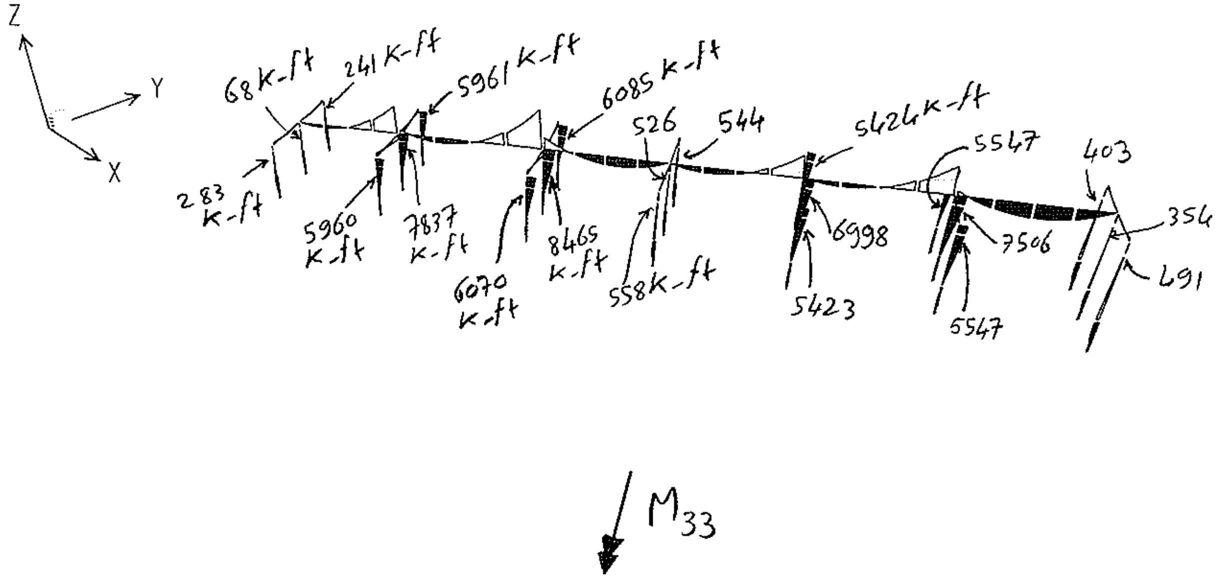
SAP2000 v7.44 - File:b3-9l - Axial Force Diagram (PUSHL - Step 17) - Kip-ft Units



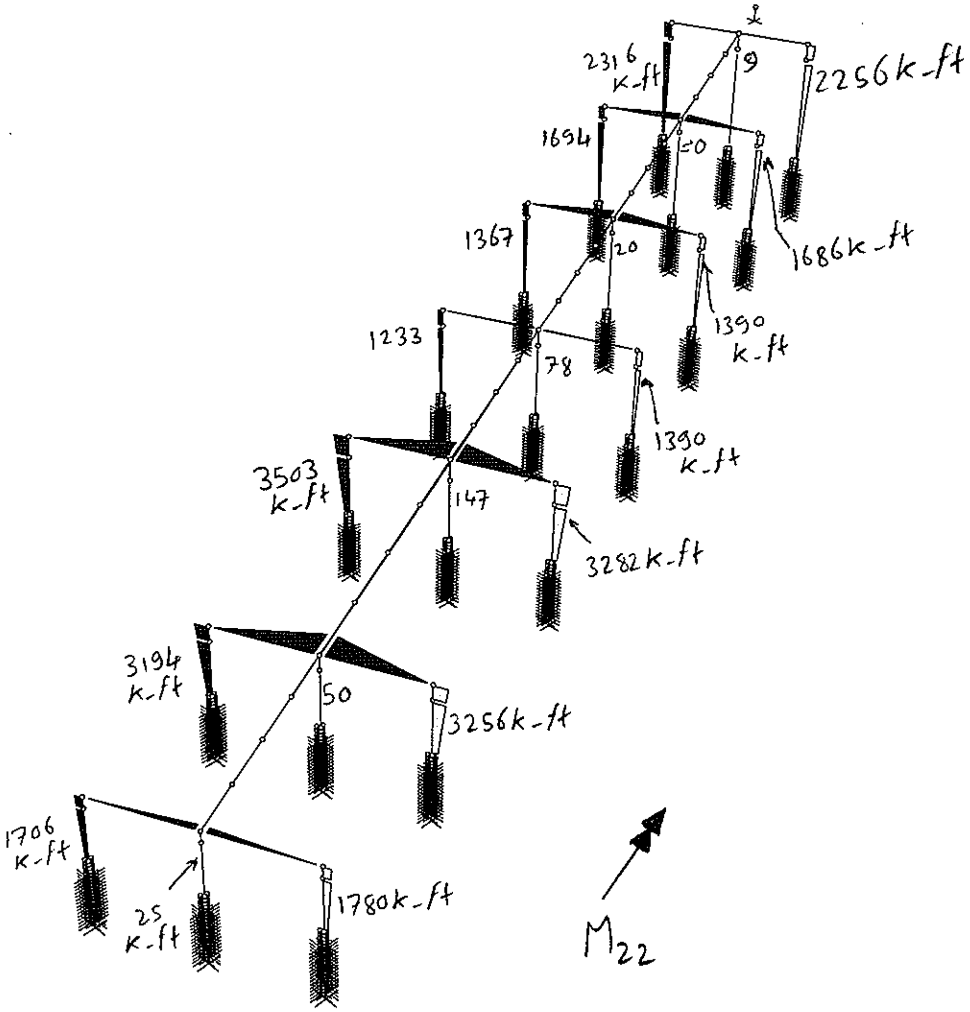
SAP2000 v7.44 - File:b3-9l - Shear Force 2-2 Diagram (PUSHL - Step 17) - Kip-ft Units



SAP2000 v7.44 - File:b3-9l - Shear Force 3-3 Diagram (PUSHL - Step 17) - Kip-ft Units



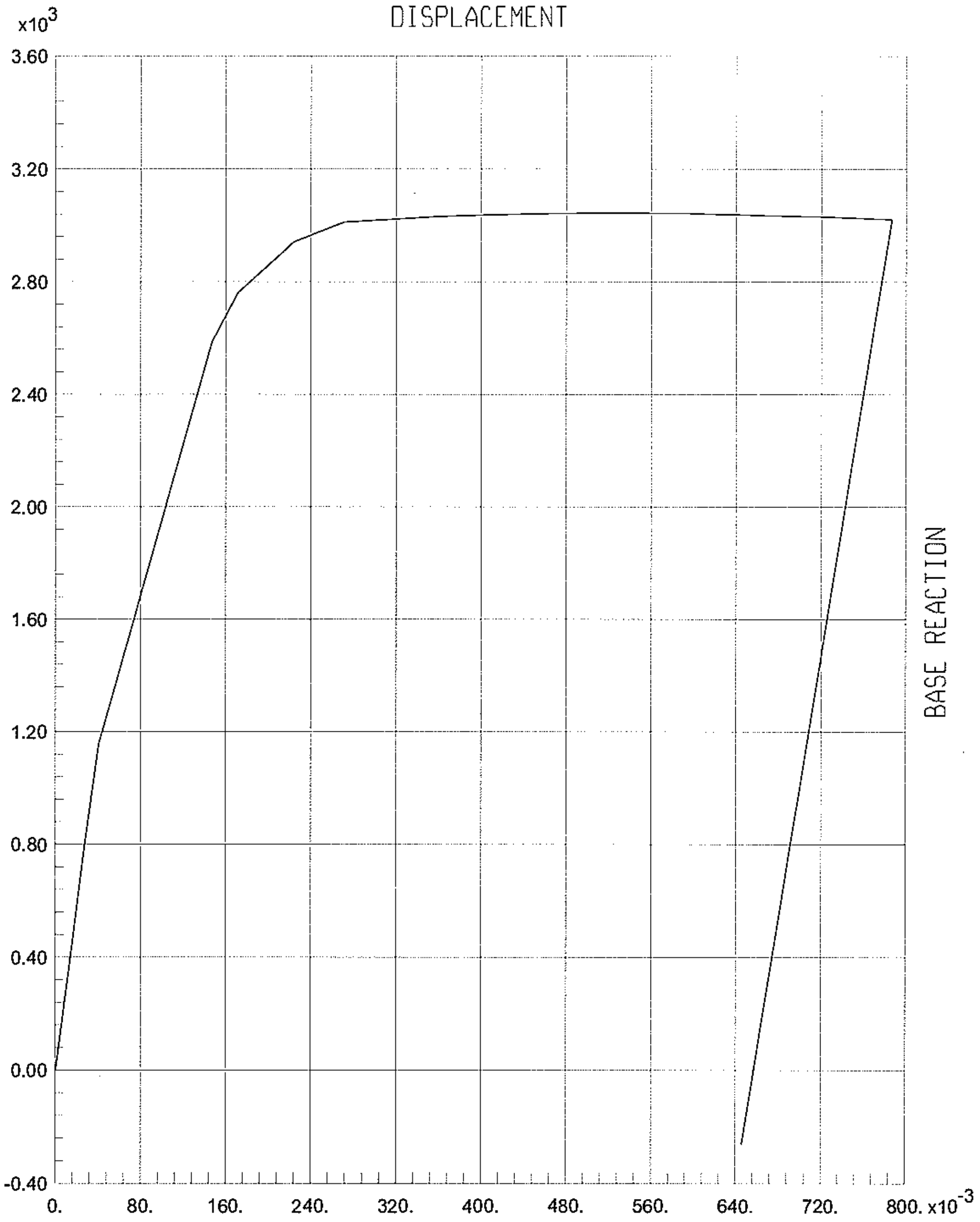
SAP2000 v7.44 - File:b3-9l - Moment 3-3 Diagram (PUSHL - Step 17) - Kip-ft Units



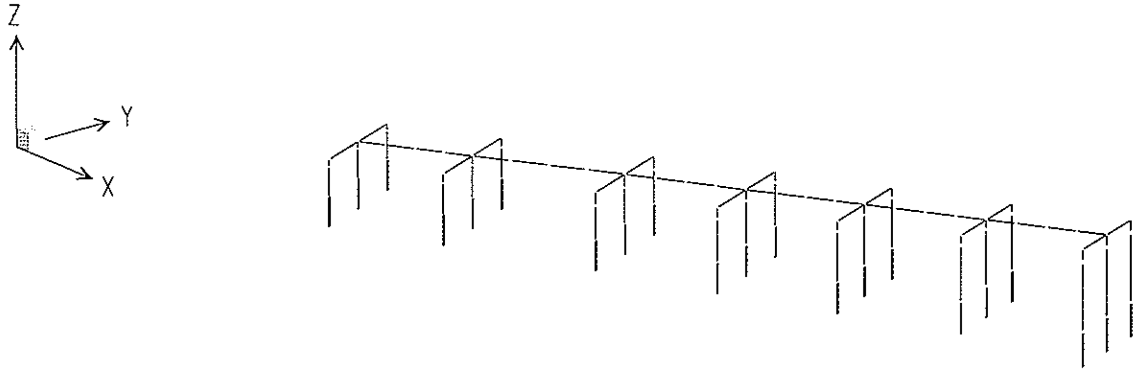
SAP2000 v7.44 - File:b3-9l - Moment 2-2 Diagram (PUSHL - Step 17) - Kip-ft Units

SAP2000

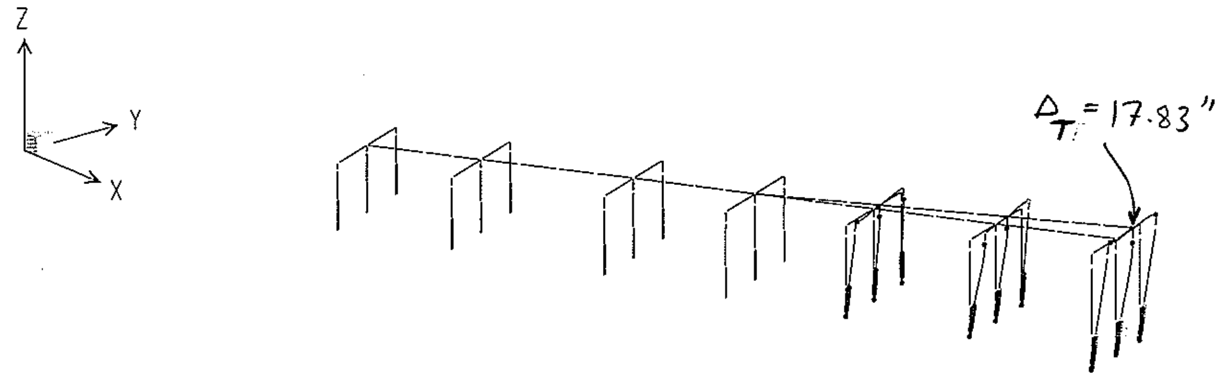
Pushover Curve 12/24/03 8:30:47



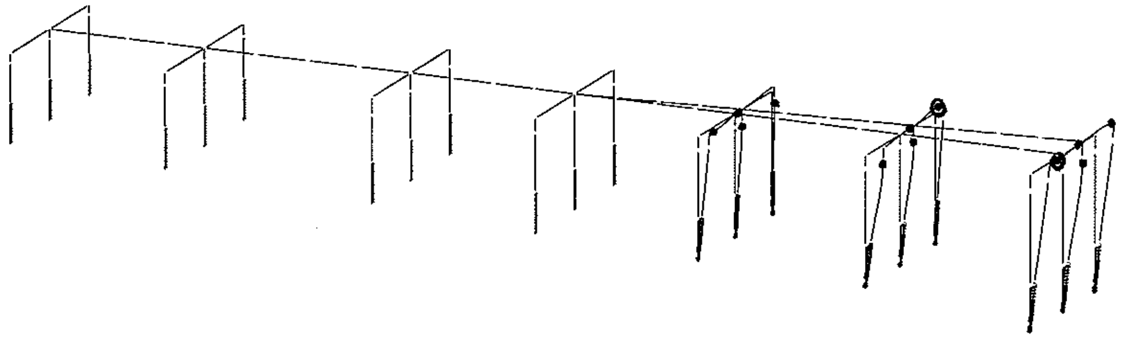
SAP2000 v7.44 - File:b3-9l - Kip-ft Units
Pushover Case PUSHL



SAP2000 v7.44 - File:b3-9t - 3-D View - Kip-ft Units



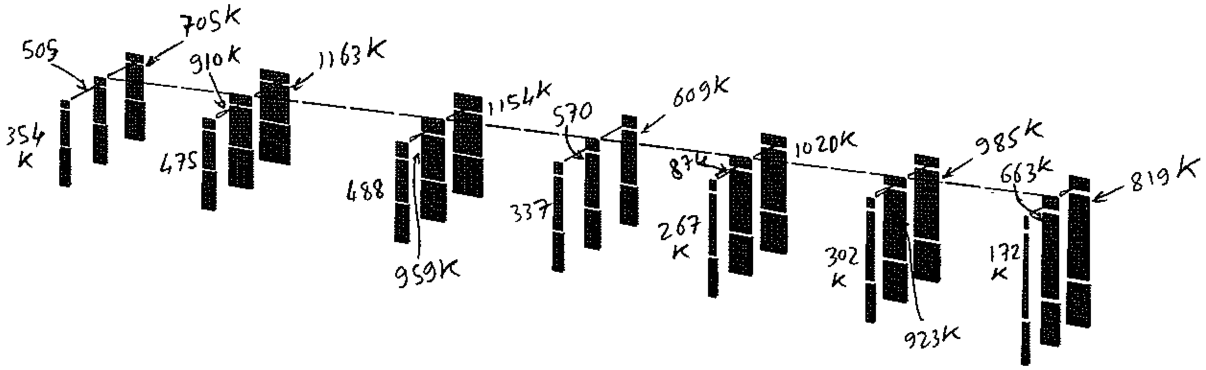
SAP2000 v7.44 - File:b3-9t - Deformed Shape (PUSHT - Step 14) - Kip-ft Units



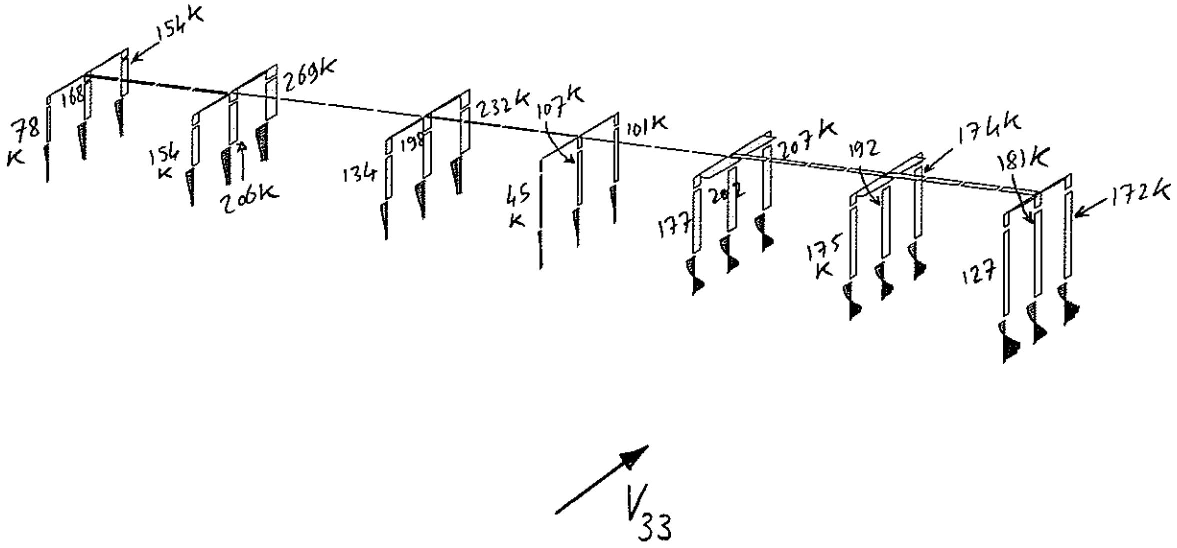
⊙ Rupture

● Plastic Hinge

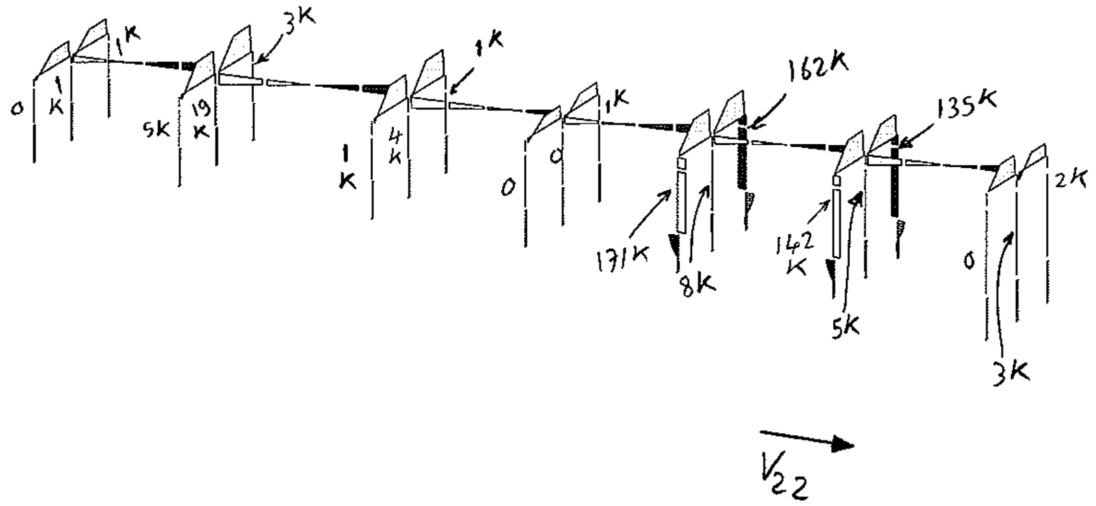
SAP2000 v7.44 - File:b3-9t - Deformed Shape (PUSHT - Step 15) - Kip-ft Units



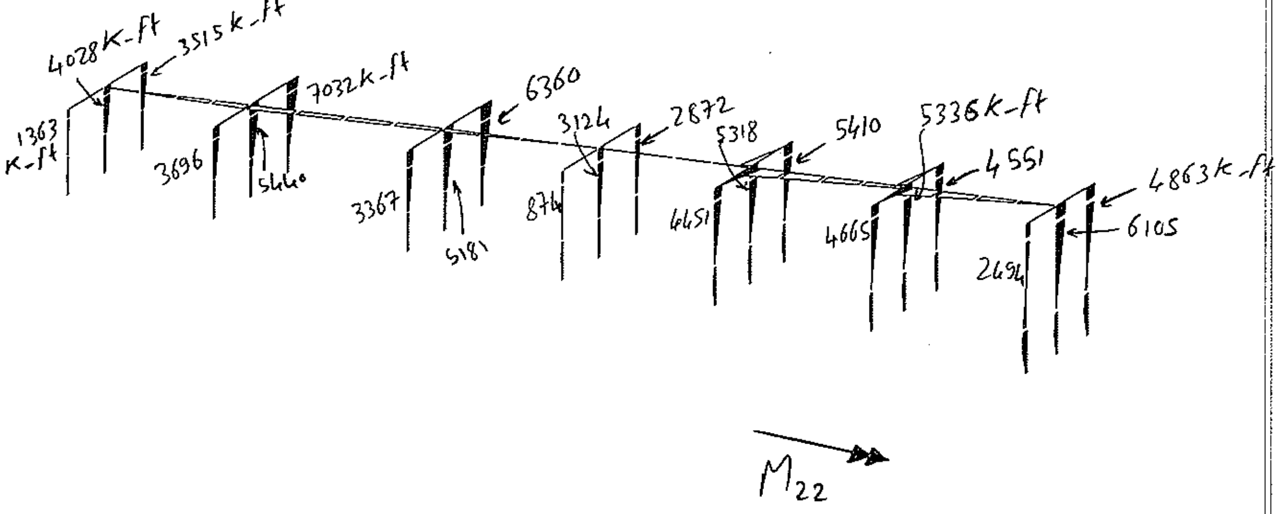
SAP2000 v7.44 - File:b3-9t - Axial Force Diagram (PUSHT - Step 14) - Kip-ft Units



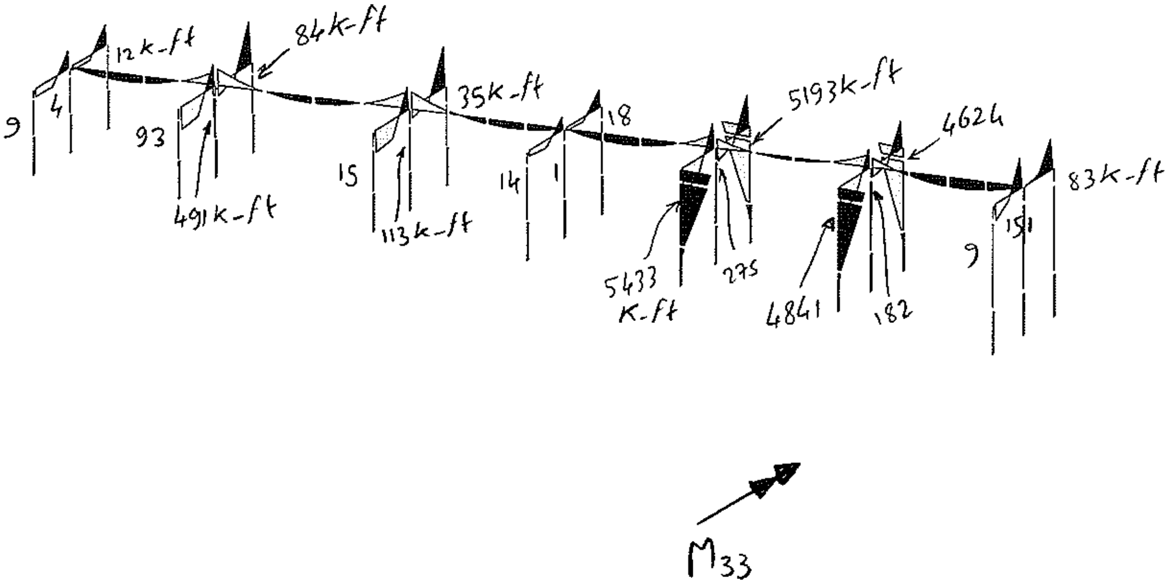
SAP2000 v7.44 - File:b3-9t - Shear Force 3-3 Diagram (PUSHT - Step 14) - Kip-ft Units



SAP2000 v7.44 - File:b3-9t - Shear Force 2-2 Diagram (PUSHT - Step 15) - Kip-ft Units



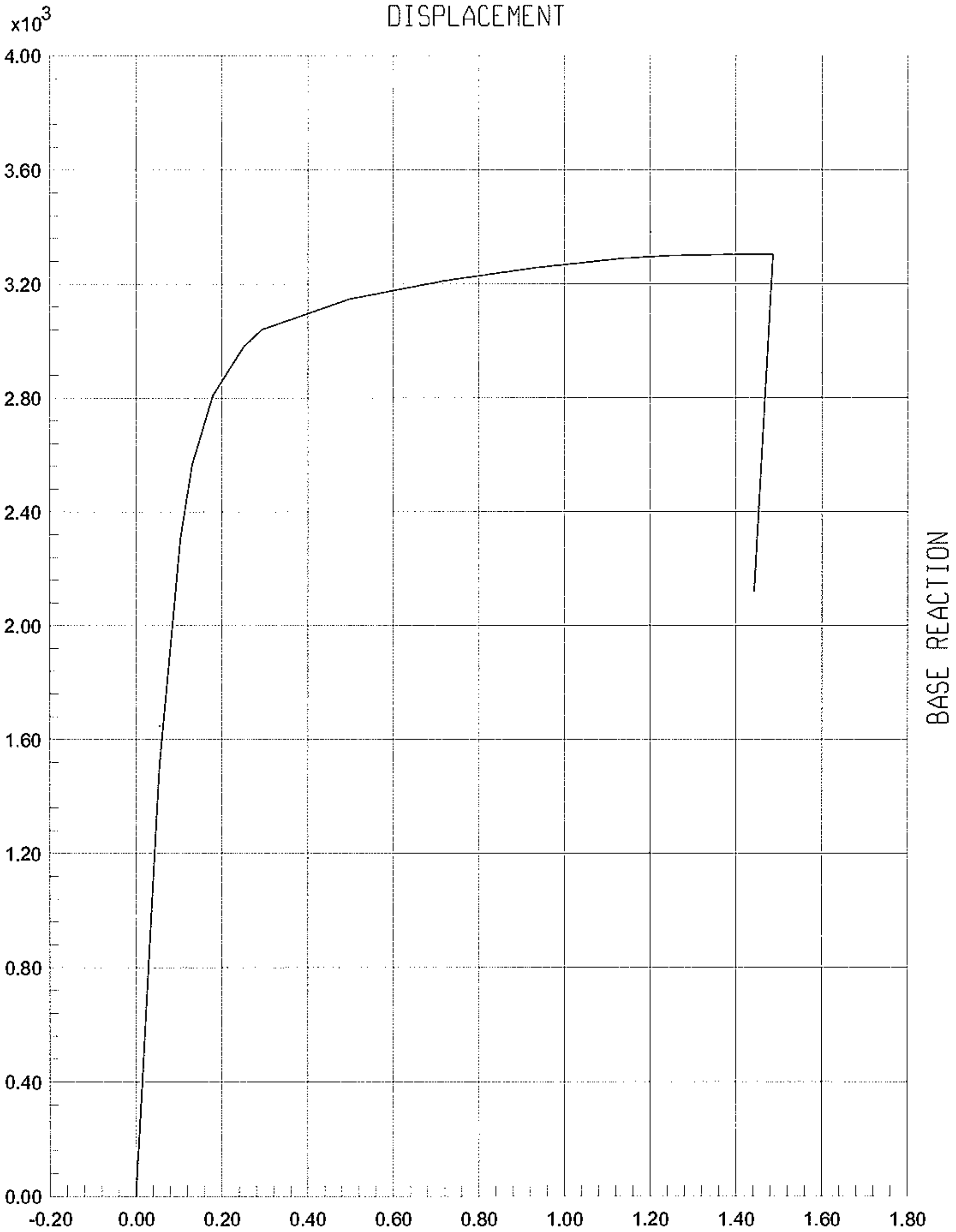
SAP2000 v7.44 - File:b3-9t - Moment 2-2 Diagram (PUSHT - Step 14) - Kip-ft Units



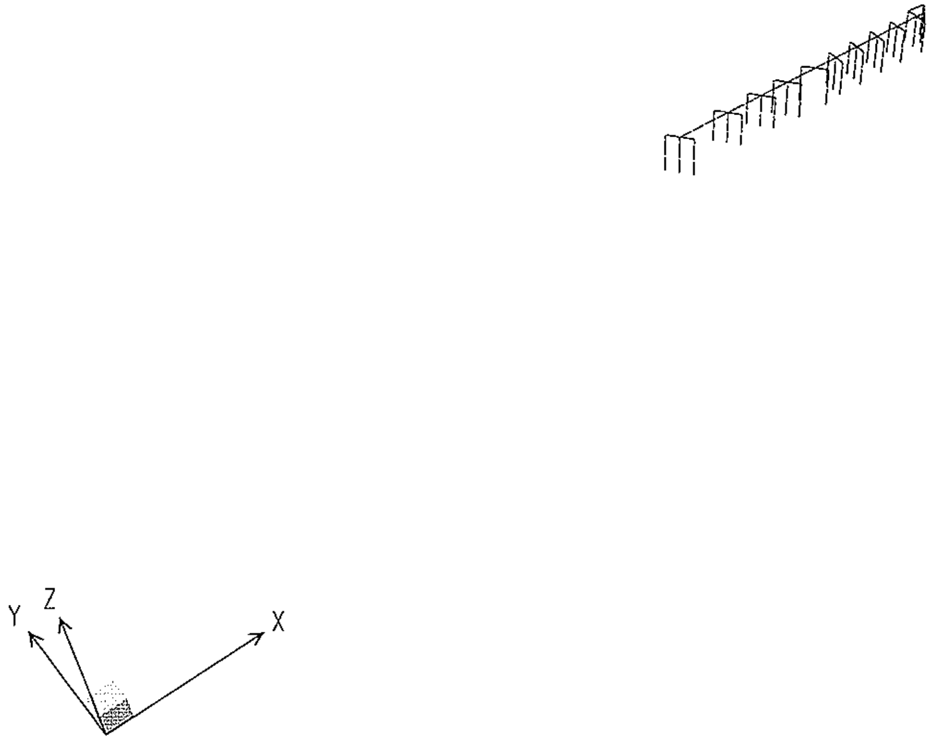
SAP2000 v7.44 - File:b3-9t - Moment 3-3 Diagram (PUSHT - Step 14) - Kip-ft Units

SAP2000

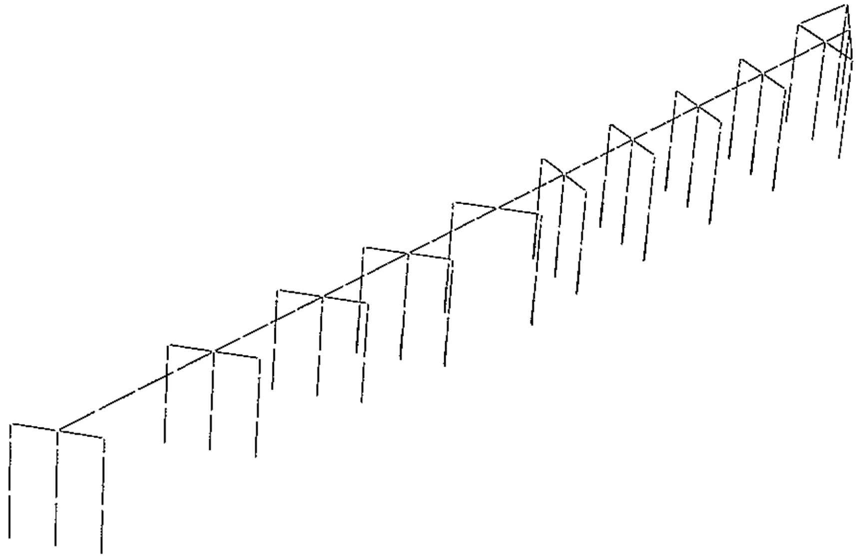
Pushover Curve 12/24/03 8:31:01



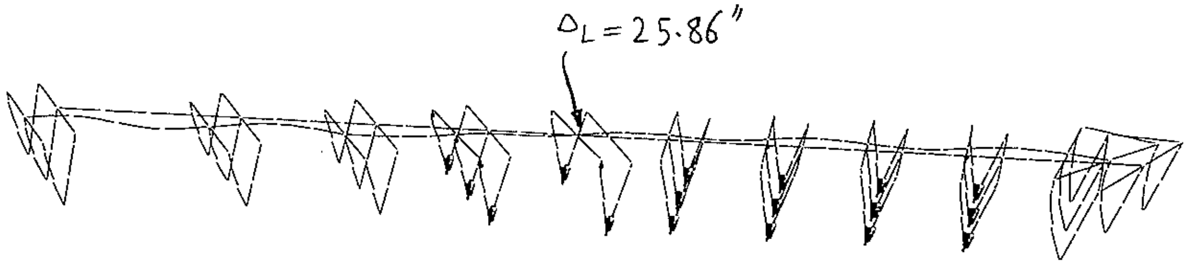
SAP2000 v7.44 - File:b3-9t - Kip-ft Units
Pushover Case PUSHT



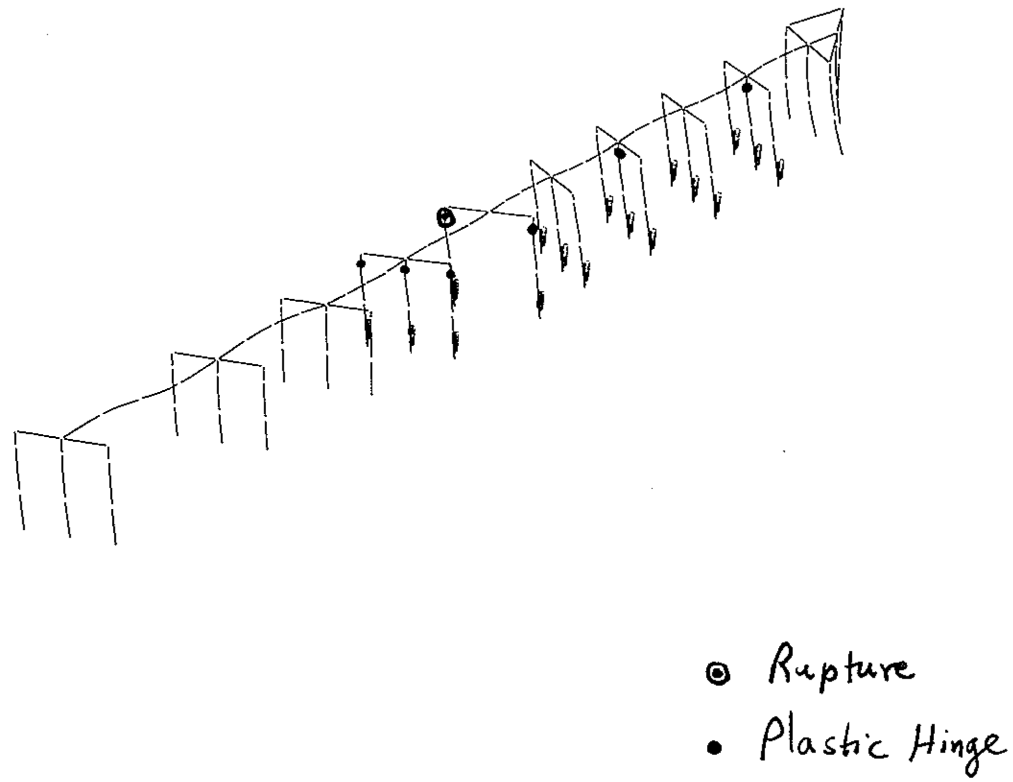
SAP2000 v7.44 - File:fr3-ll - 3-D View - Kip-ft Units



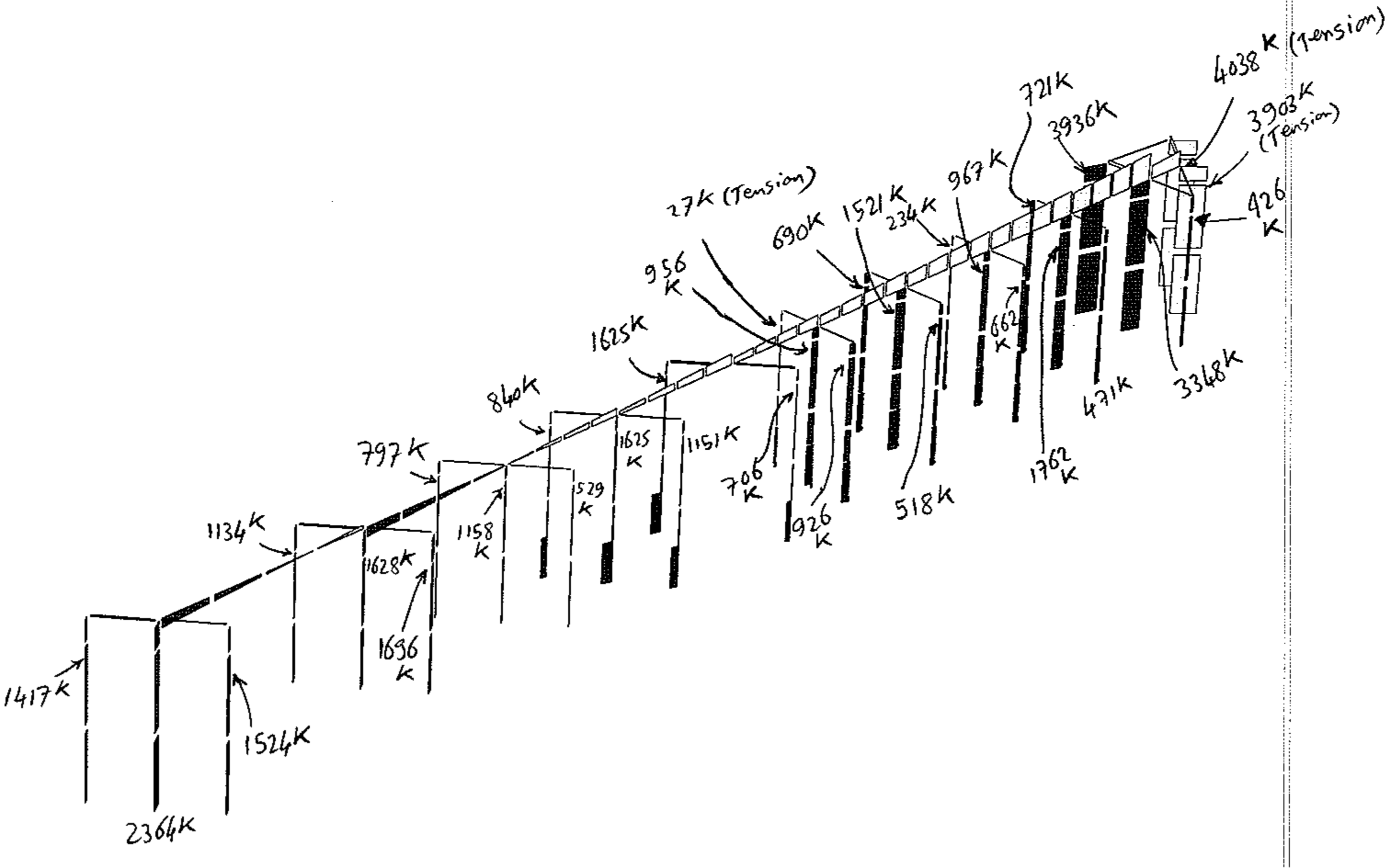
SAP2000 v7.44 - File:fr3-ll - 3-D View - Kip-ft Units



SAP2000 v7.44 - File:fr3-ll - Deformed Shape (PUSHLL - Step 5) - Kip-ft Units

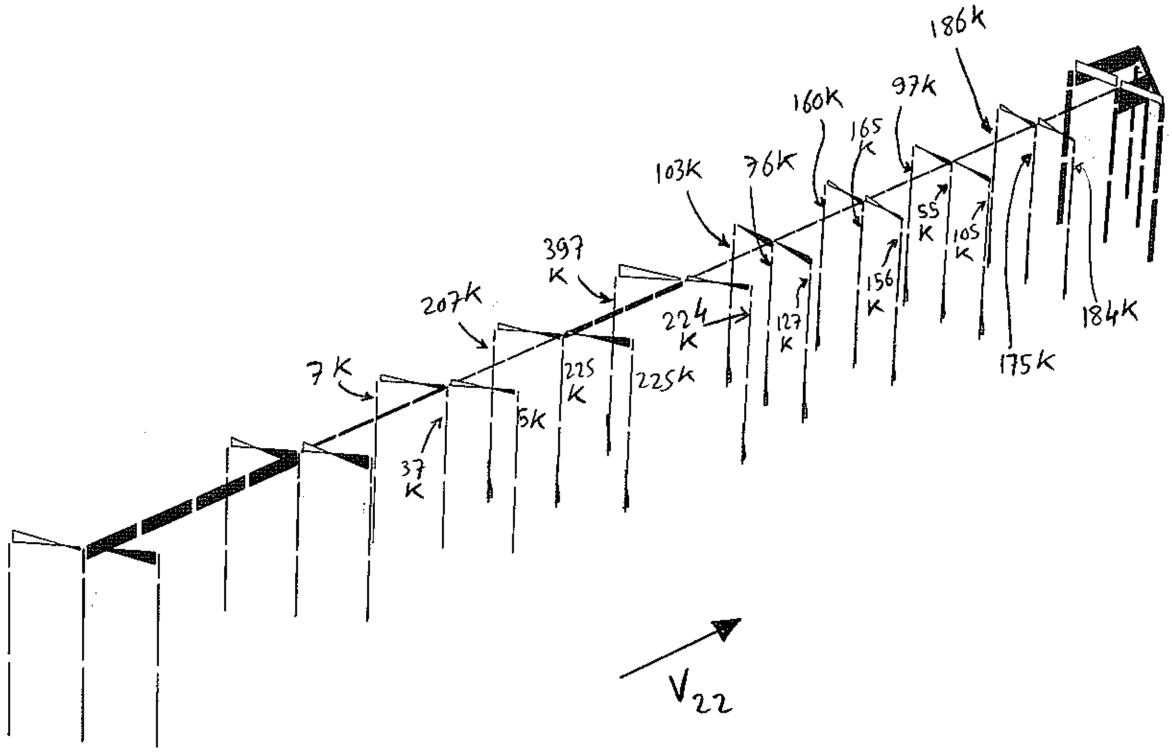


SAP2000 v7.44 - File:fr3-ll - Deformed Shape (PUSHLL - Step 5) - Kip-ft Units

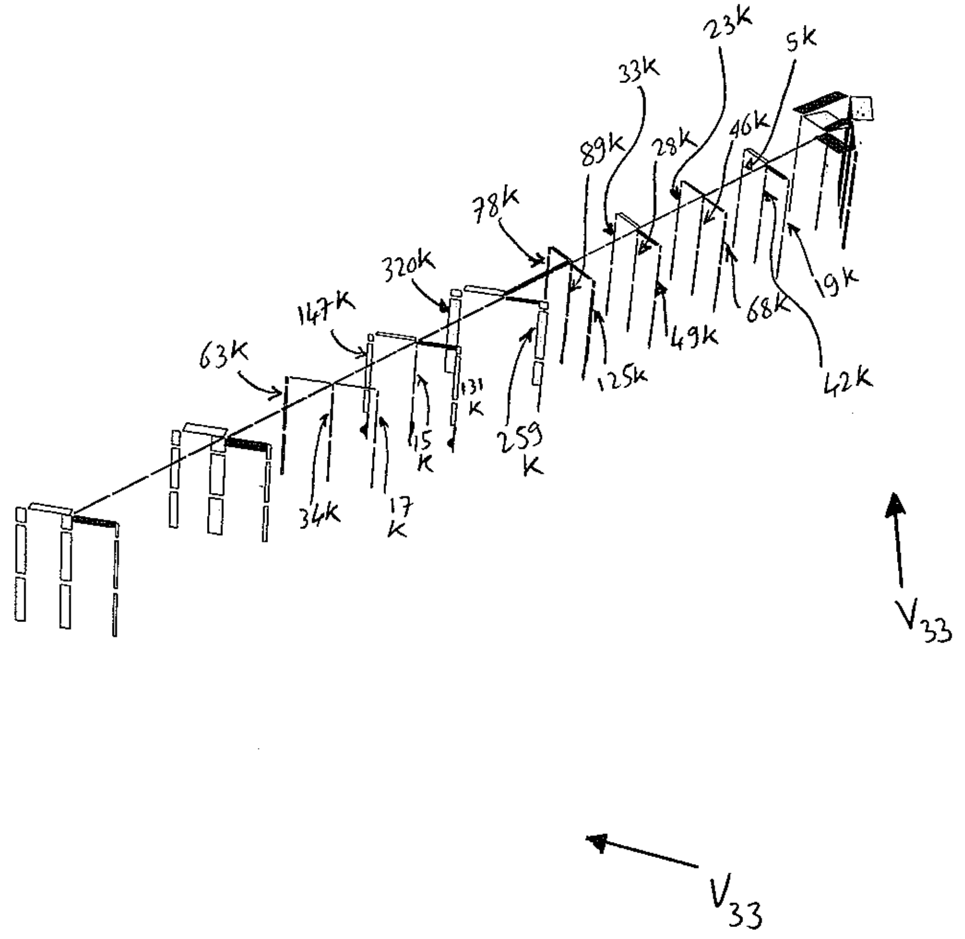


SAP2000 v7.44 - File:fr3-ll - Axial Force Diagram (PUSHLL - Step 5) - Kip-ft Units

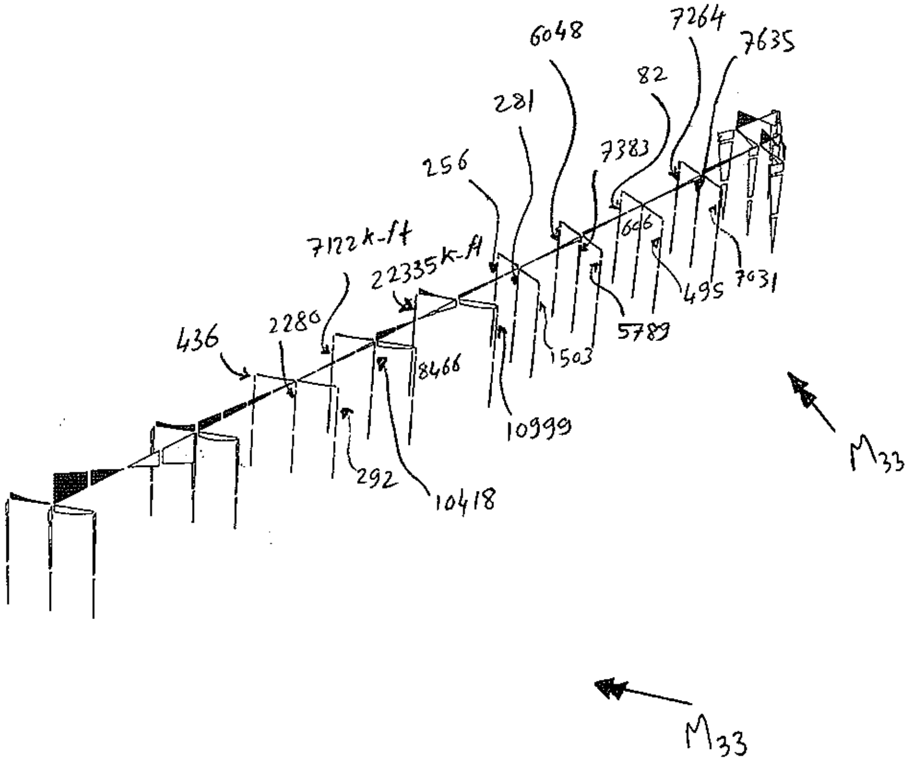
SAP2000 12/29/03 12:13:39



SAP2000 v7.44 - File:fr3-ll - Shear Force 2-2 Diagram (PUSHLL - Step 5) - Kip-ft Units



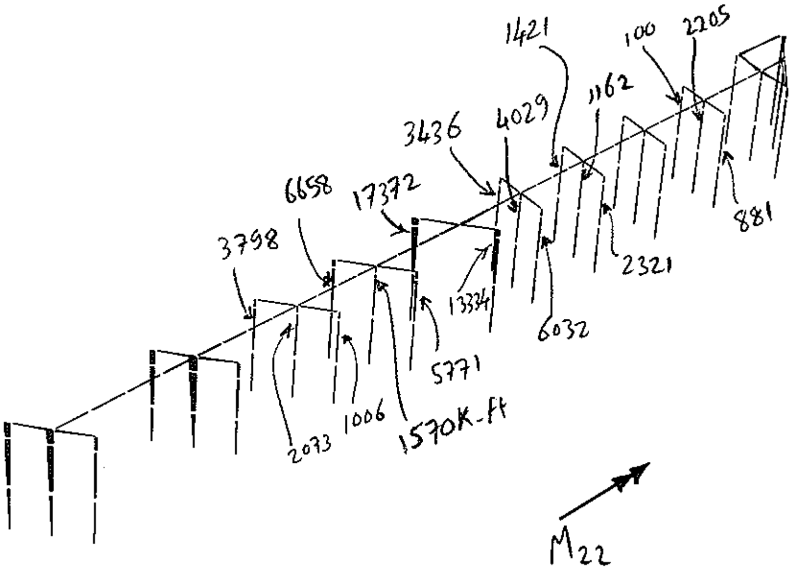
SAP2000 v7.44 - File:fr3-II - Shear Force 3-3 Diagram (PUSHLL - Step 5) - Kip-ft Units



SAP2000 v7.44 - File:fr3 ll - Moment 3-3 Diagram (PUSHLL - Step 5) - Kip-ft Units

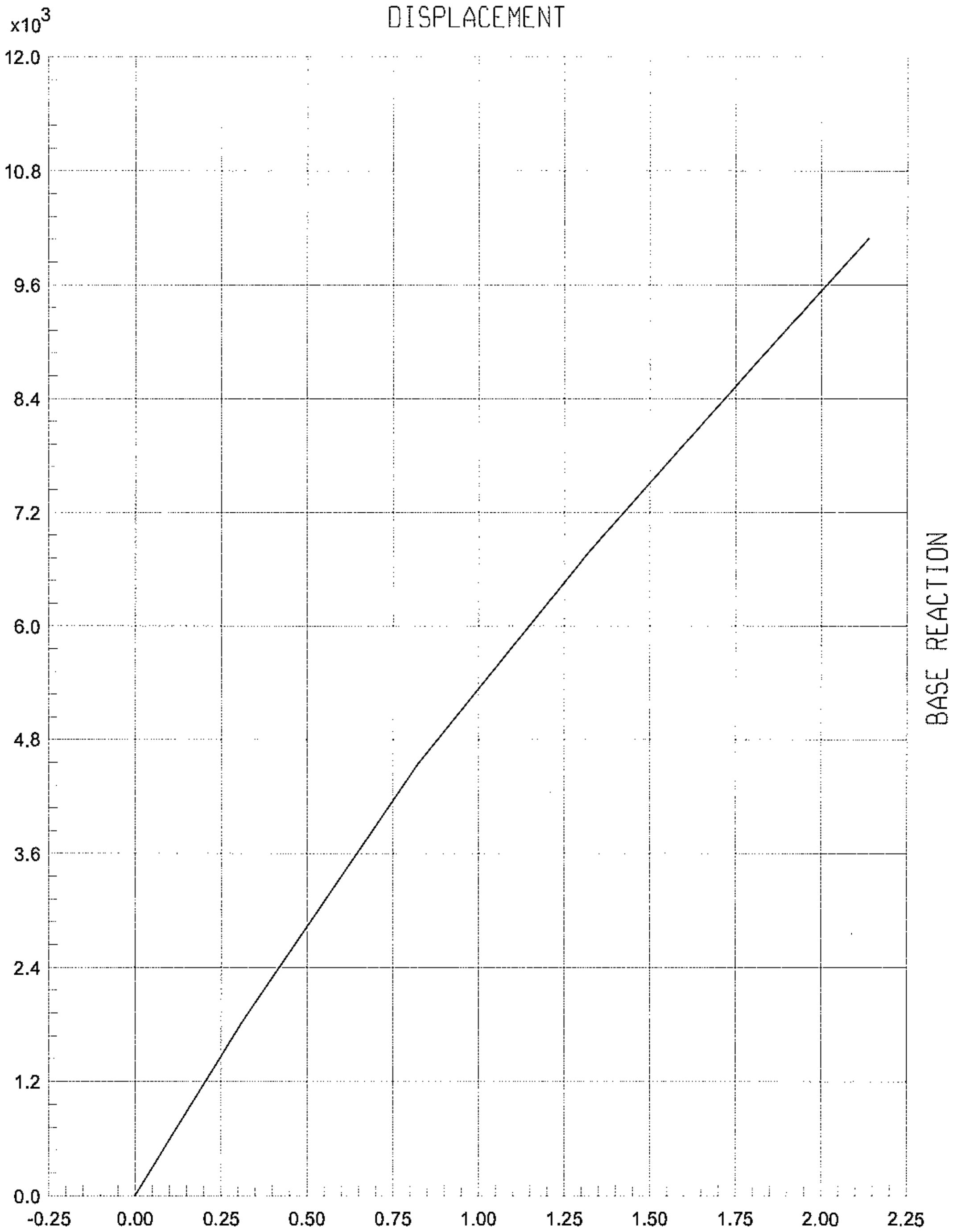
SAP2000

12/29/03 13:07:38

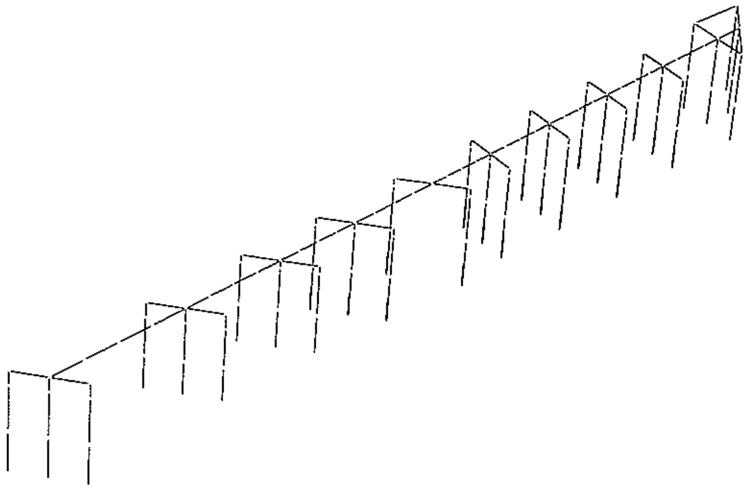


SAP2000 v7.44 · File:fr3-ll - Moment 2-2 Diagram (PUSHLL - Step 5) - Kip-ft Units

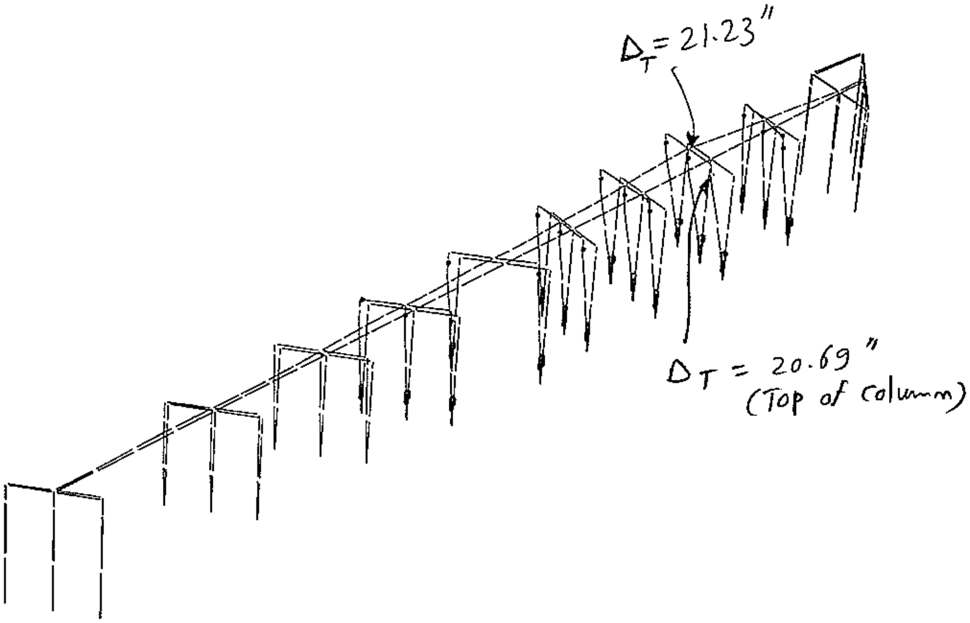
SAP2000 Pushover Curve 12/29/03 12:07:50



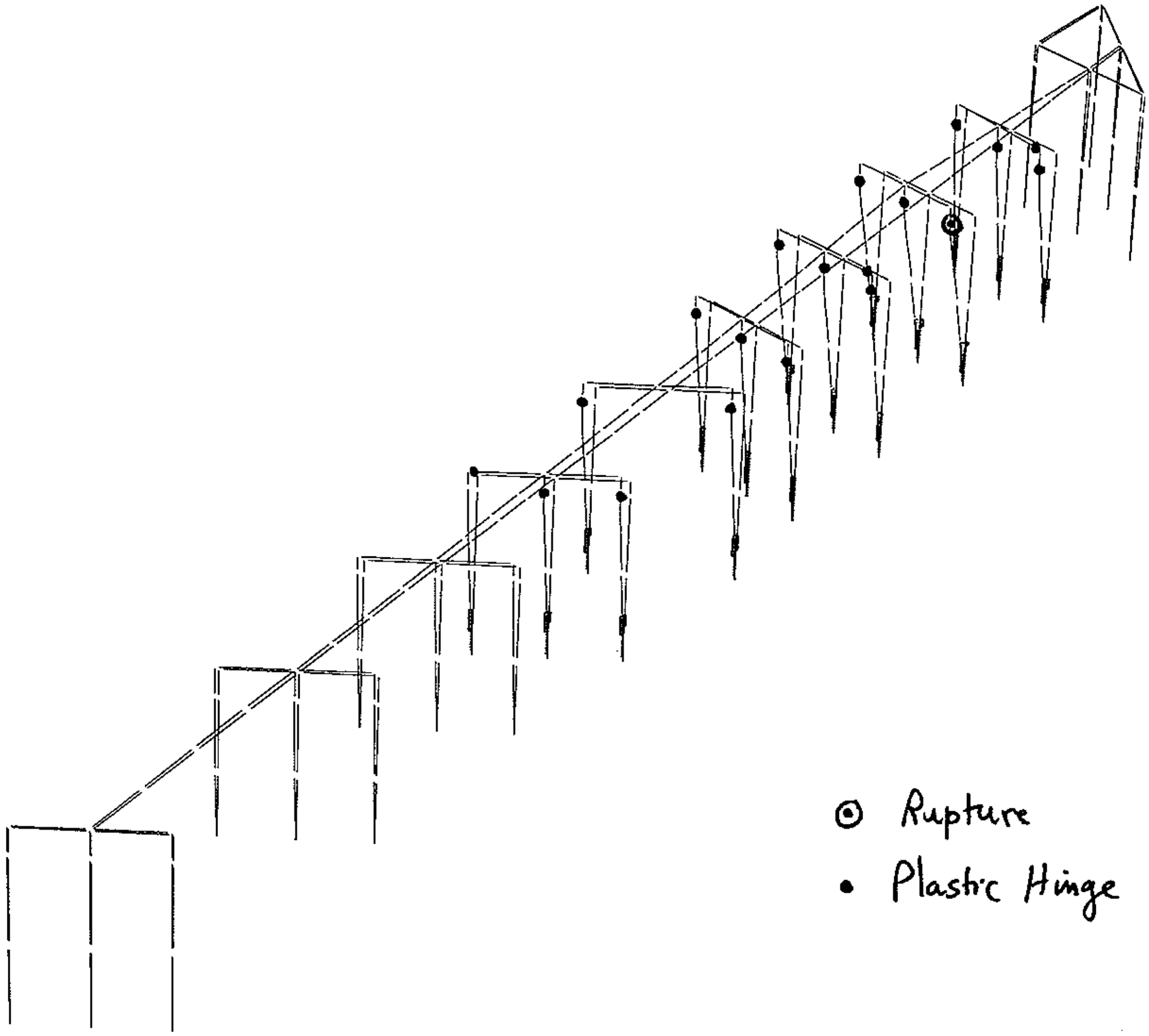
SAP2000 v7.44 - File:fr3-II - Kip-ft Units
Pushover Case PUSHLL



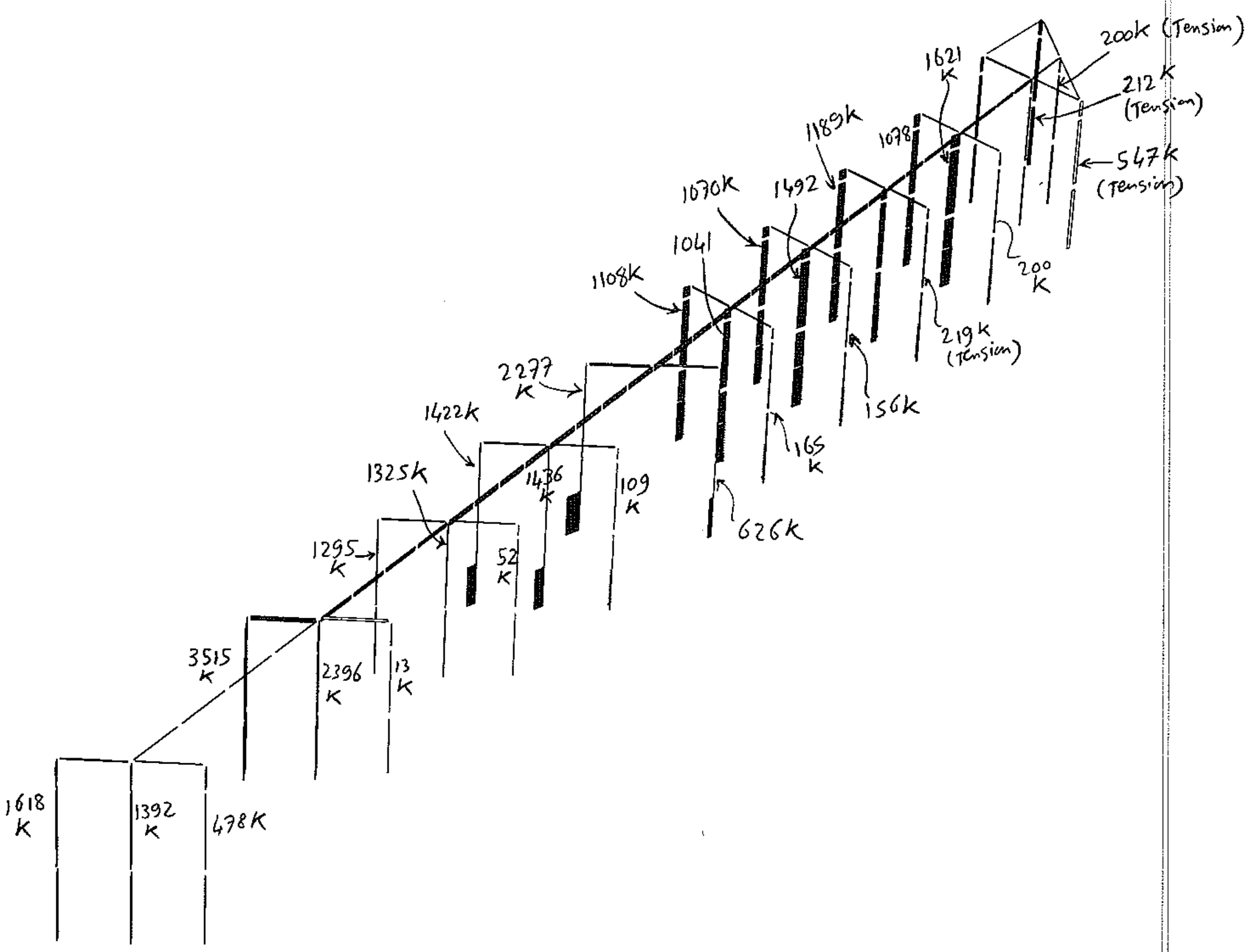
SAP2000 v7.44 - File:fr3-ft - 3-D View - Kip-ft Units



SAP2000 v7.44 - File:fr3 tt - Deformed Shape (PUSHTT - Step 6) - Kip-ft Units

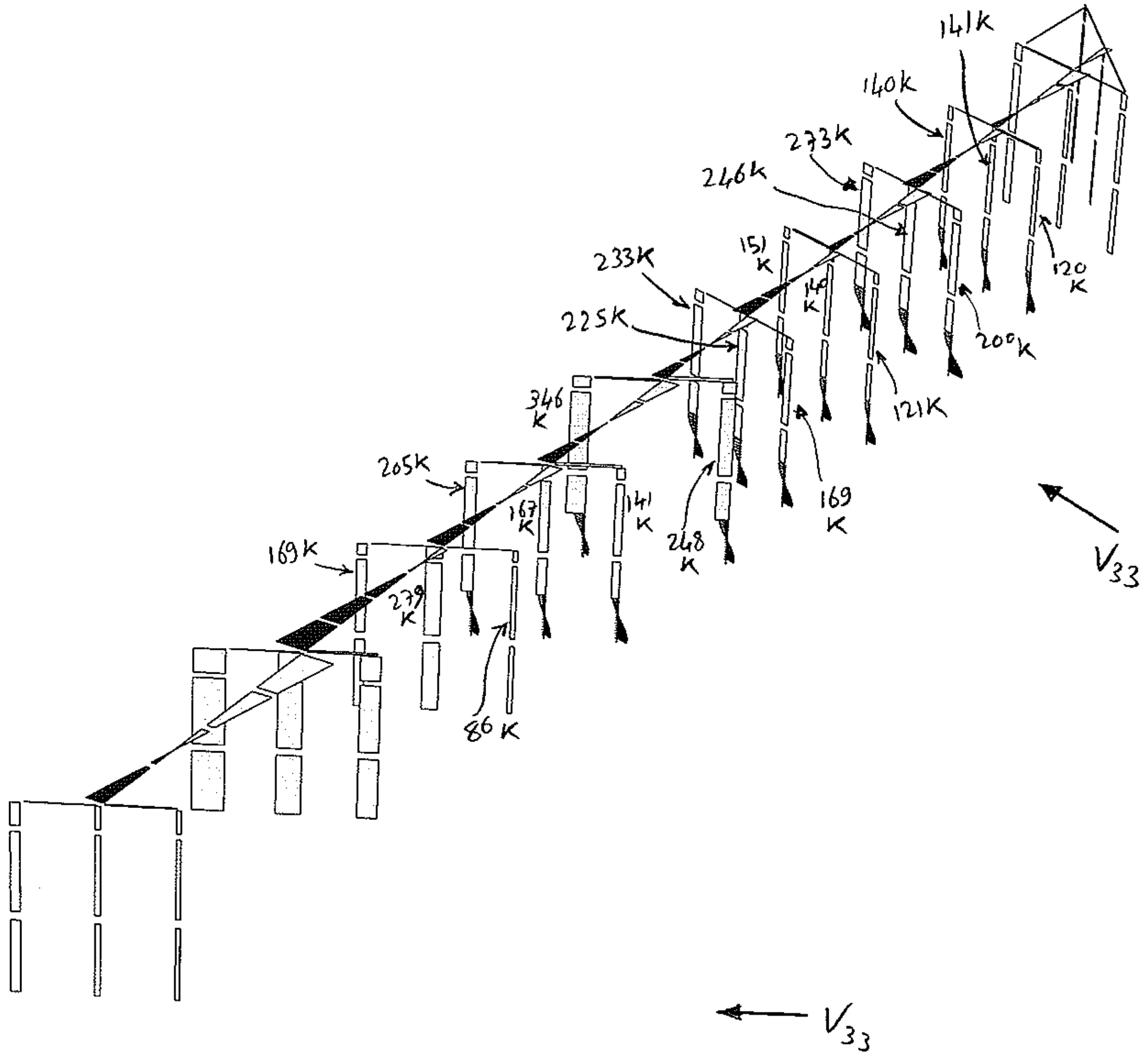


SAP2000 v7.44 - File:fr3-tt - Deformed Shape (PUSHTT - Step 6) - Kip-ft Units

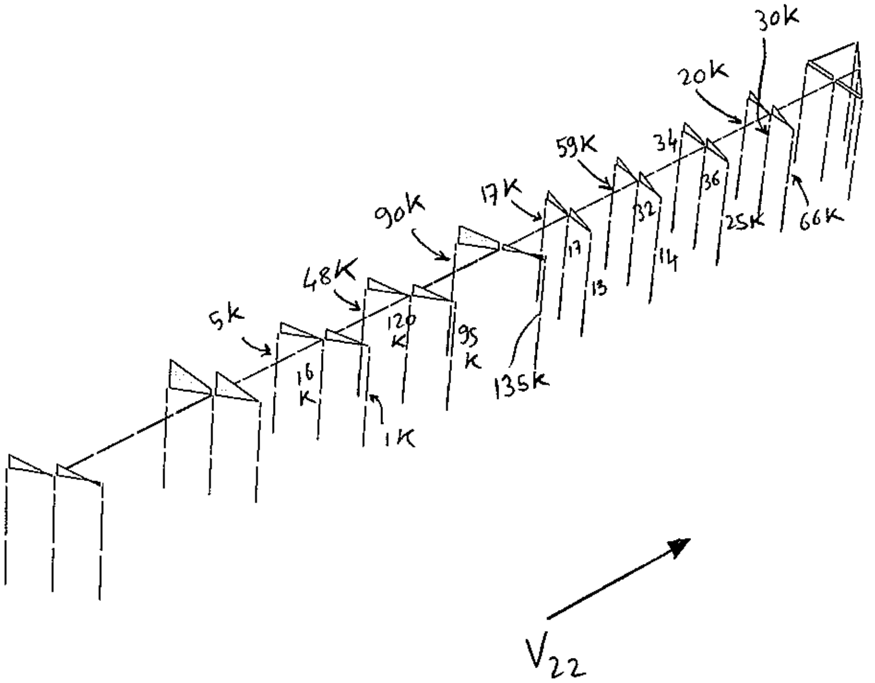


SAP2000 v7.44 - File:fr3-tt - Axial Force Diagram (PUSHTT - Step 6) - Kip-ft Units

12/29/03 13:25:04



SAP2000 v7.44 - File:fr3-tt - Shear Force 3-3 Diagram (PUSHTT - Step 6) - Kip-ft Units



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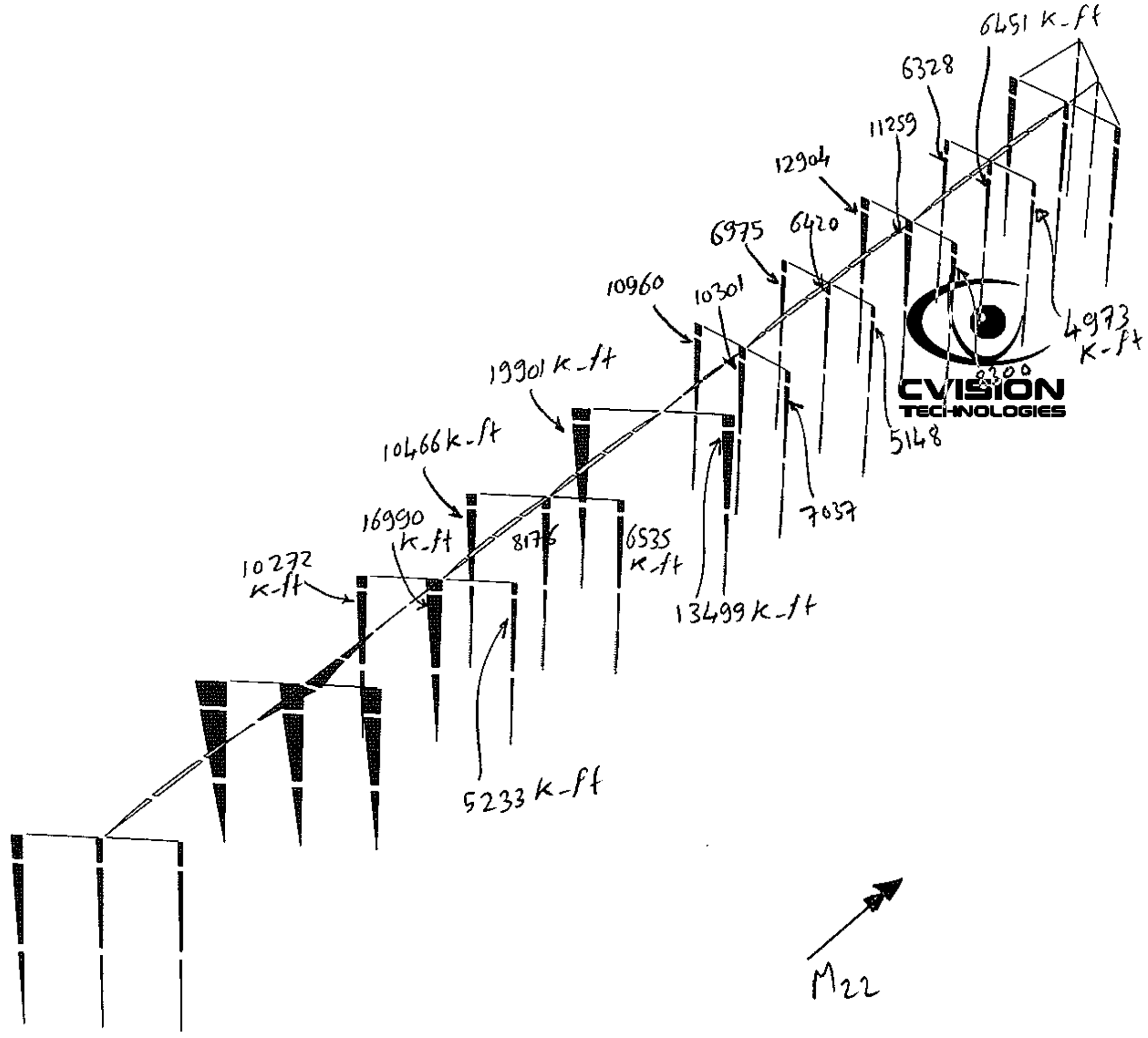
SAP2000

12/29/03 13:26:46



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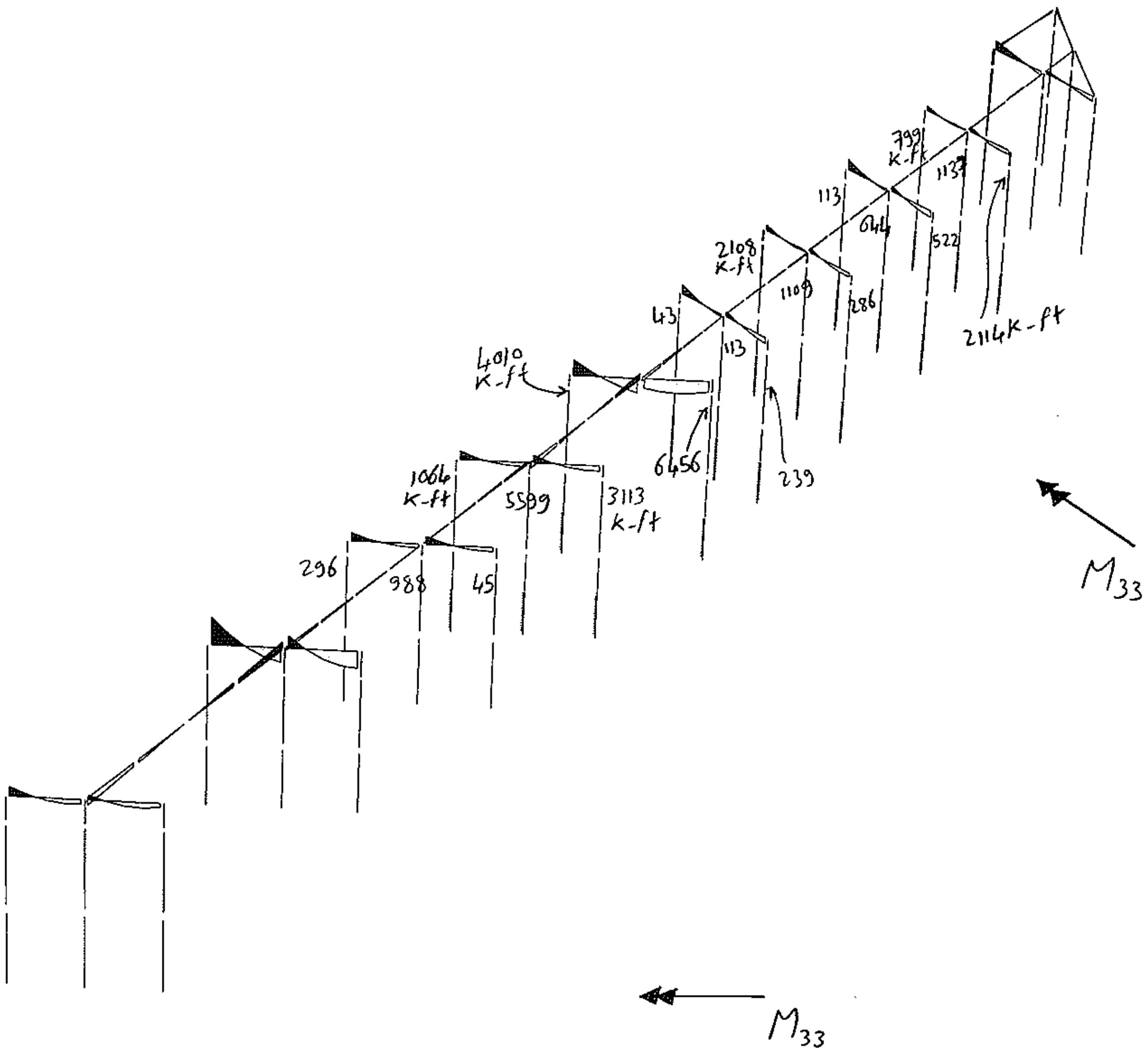

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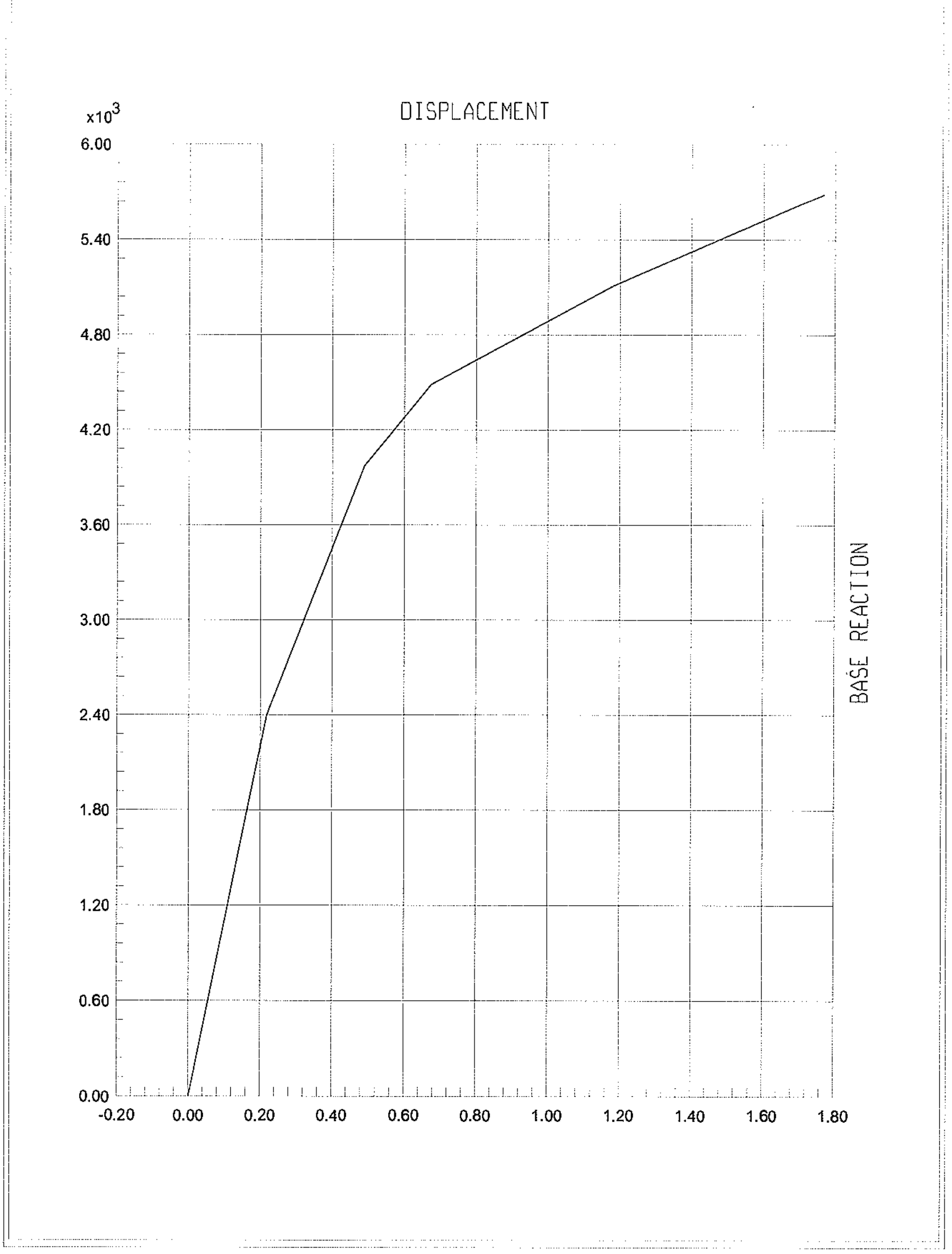
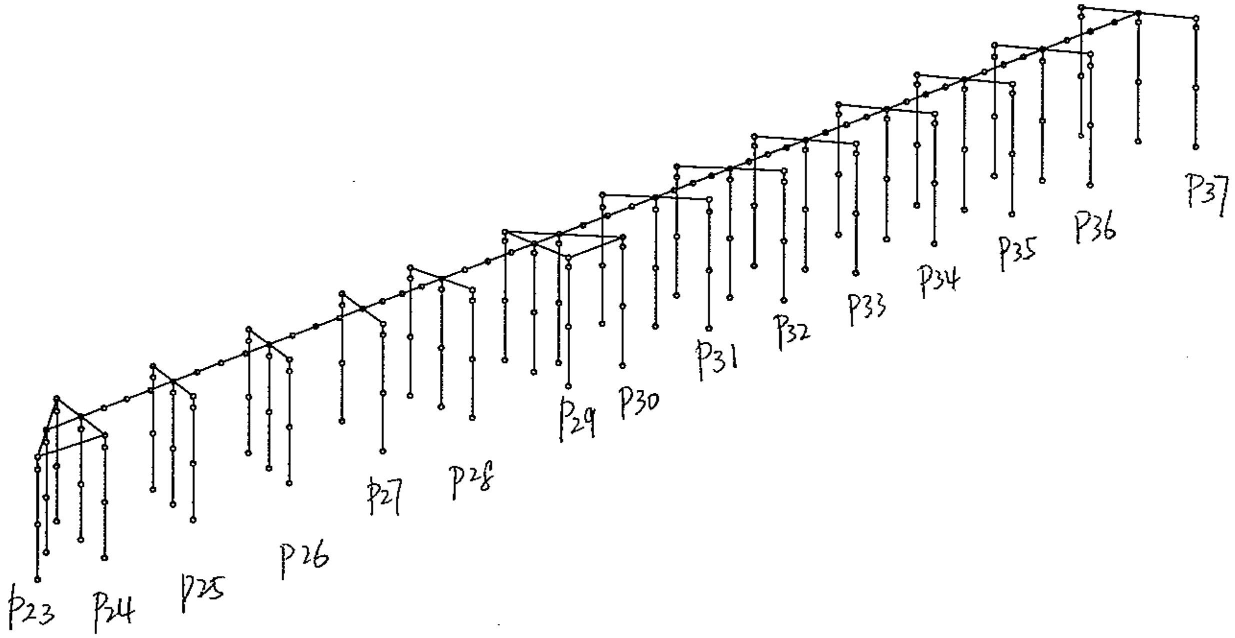
SAP2000

Pushover Curve 12/29/03 13:24:19

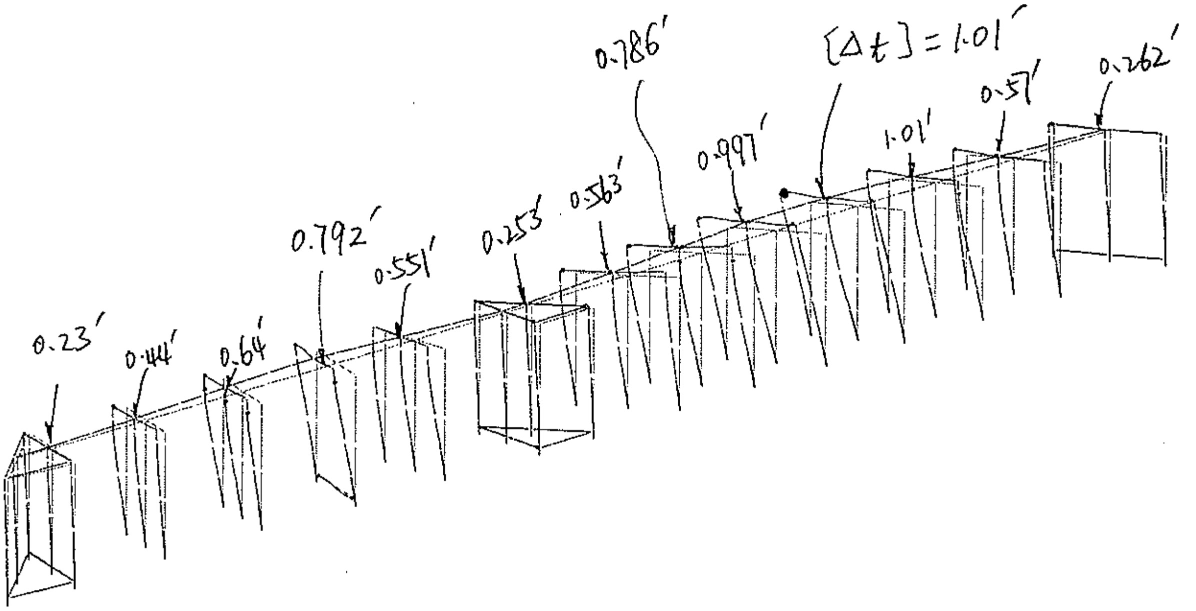


SAP2000 v7.44 - File:fr3-tt - Kip-ft Units
Pushover Case PUSHTT

6th ST Bridge, Frame-4 push-over.

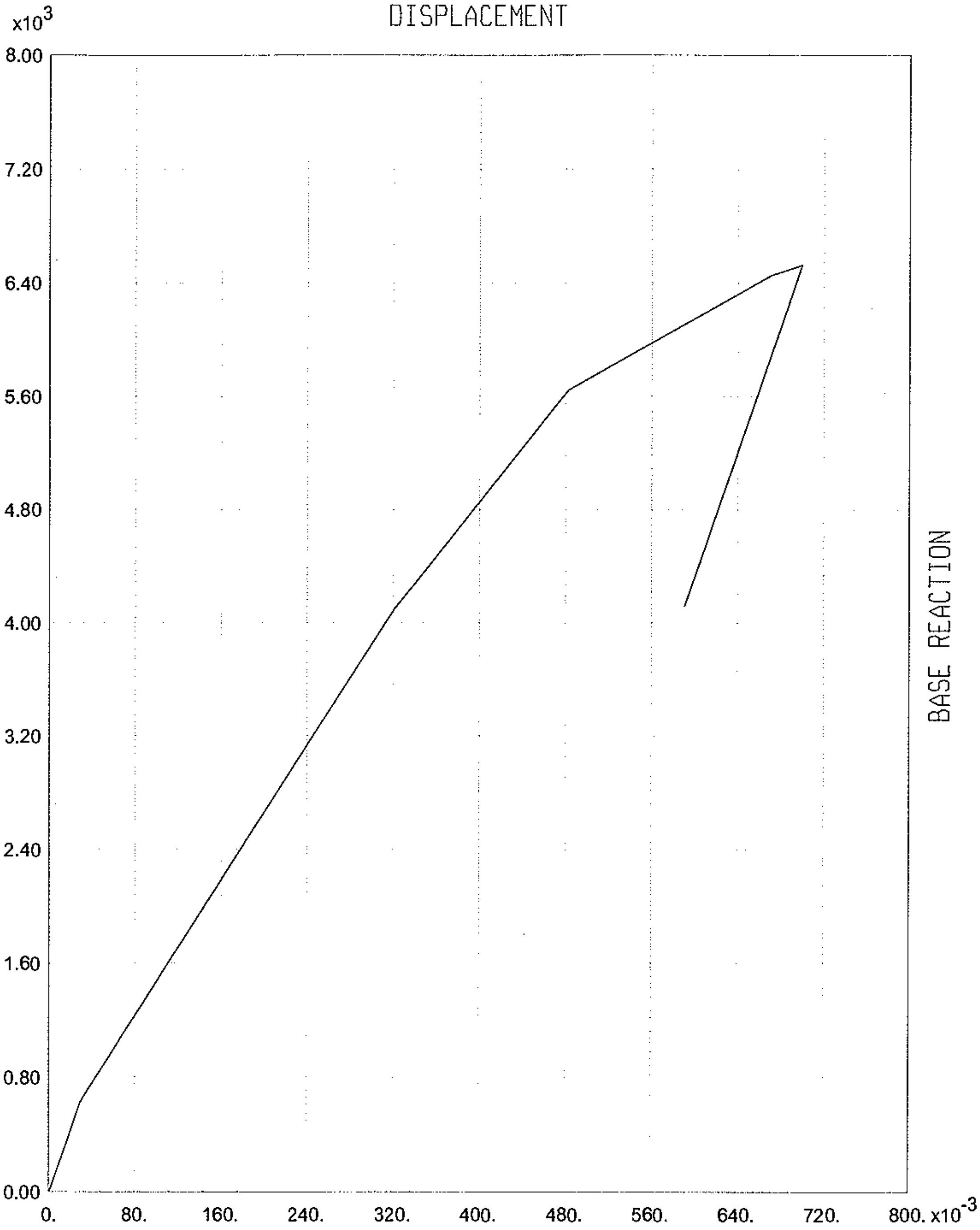


With/without P-Δ effect (same PH sequence).



SAP2000 v7.40 - File:fr4-tt - Deformed Shape (PUSHTT - Step 6) - Kip-ft Units

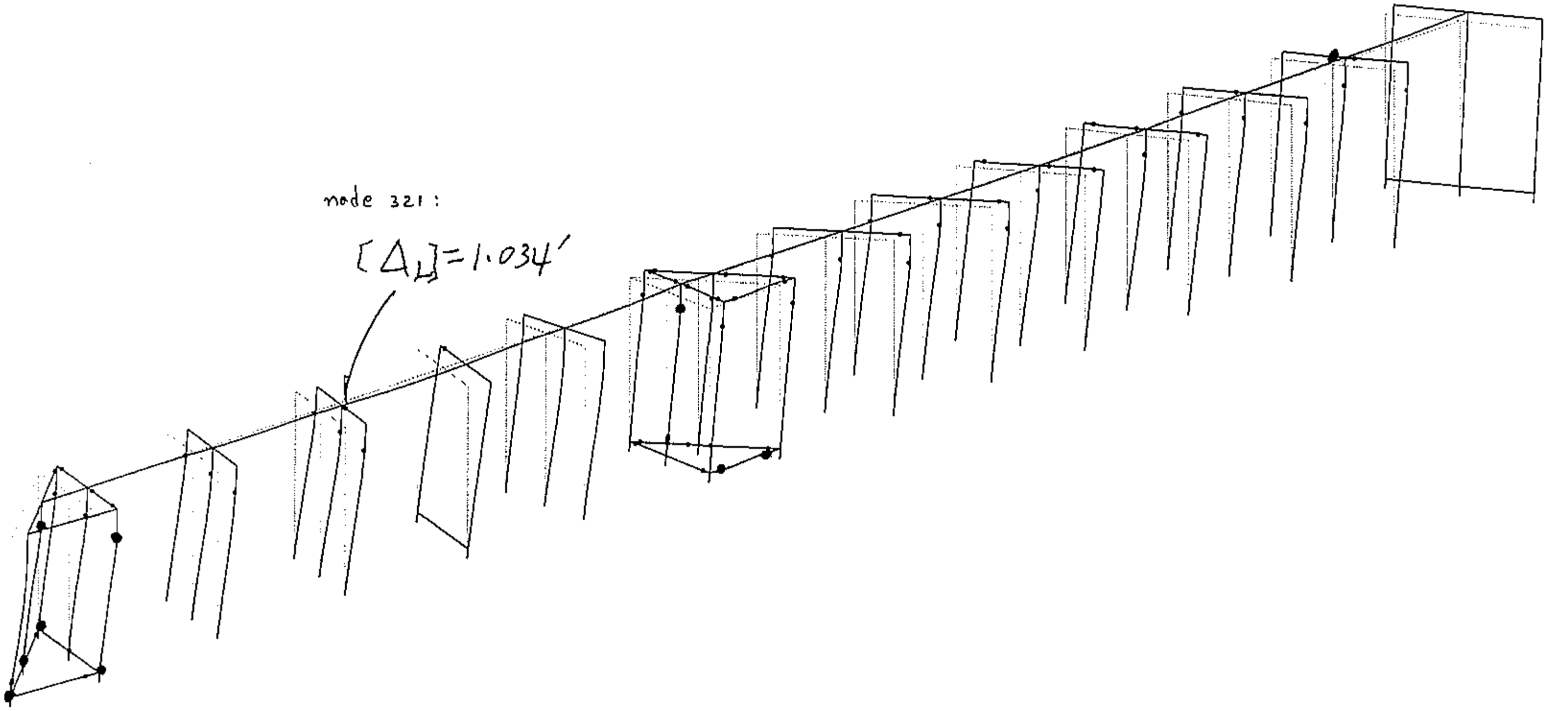
SAP2000 Pushover Curve 8/14/02 13:56:44



SAP2000 v7.40 - File:fr4-tt - Kip-ft Units
Pushover Case PUSHTT

With P-Δ efect. (L-dir push)

- controling plastic hinge.



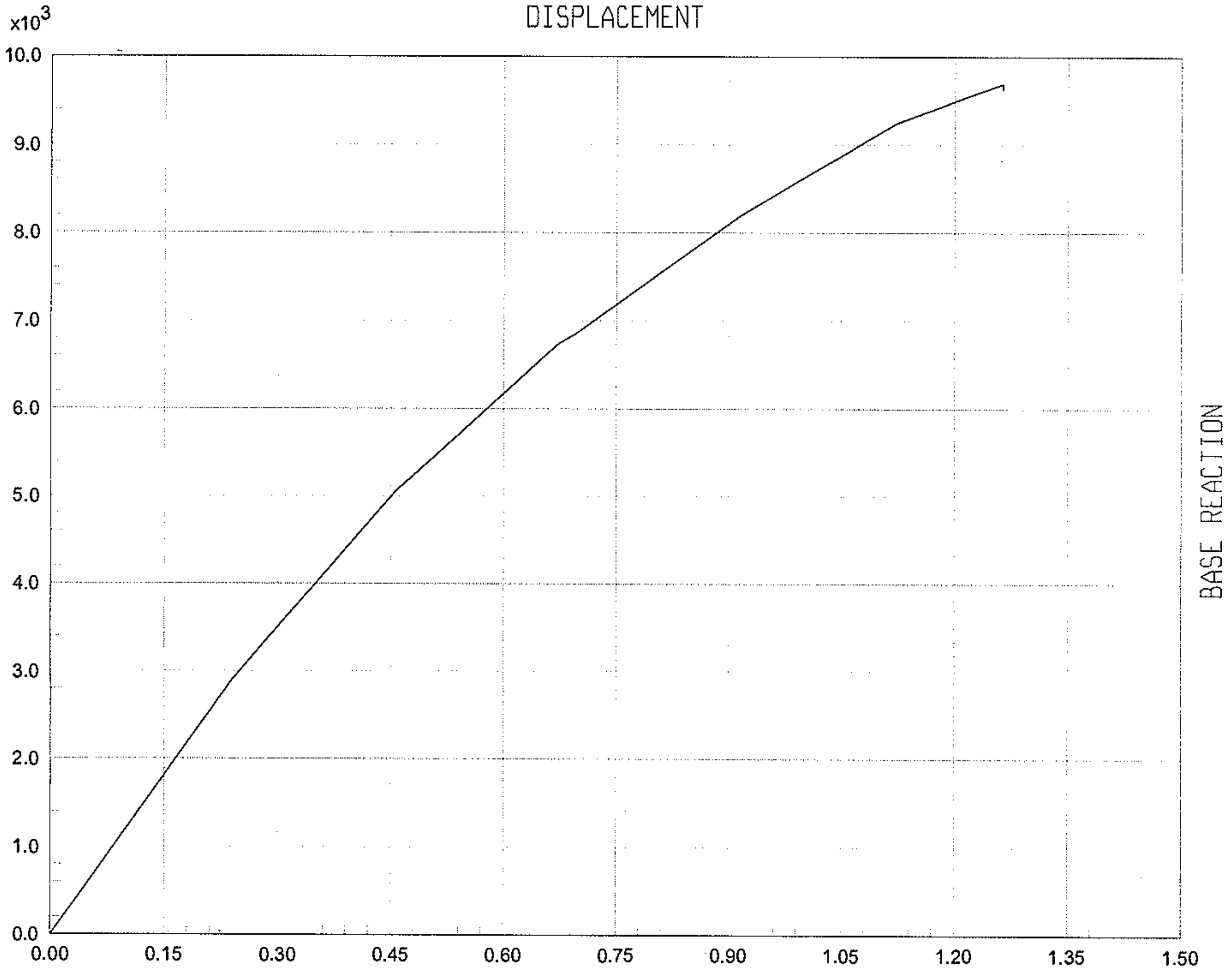
node 321:

$[\Delta_L] = 1.034'$

SAP2000 v8.0.5 Beta - File:Fr4-II~1 - Deformed Shape (PUSHLL) - Step 9 - Kip, f t, F Units

SAP2000

Pushover Curve 8/14/02 16:40:06

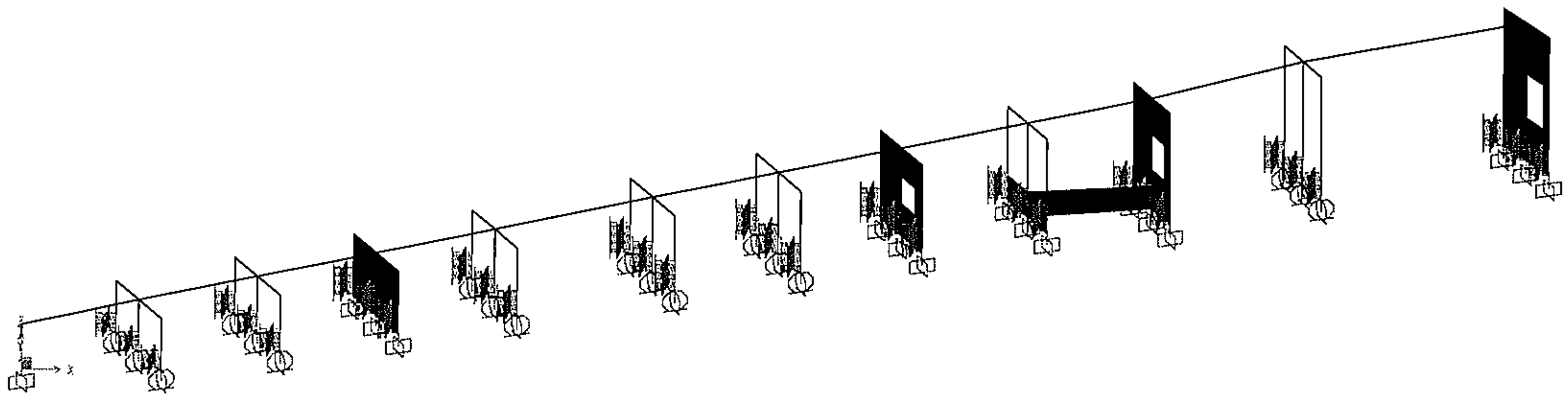


SAP2000 v8.0.5 Beta - File:Fr4-ll~1 - Kip, f t, F Units
Pushover Case PUSHLL

Approach Spans (Steel Casing Retrofit Alternative 2)

SAP2000

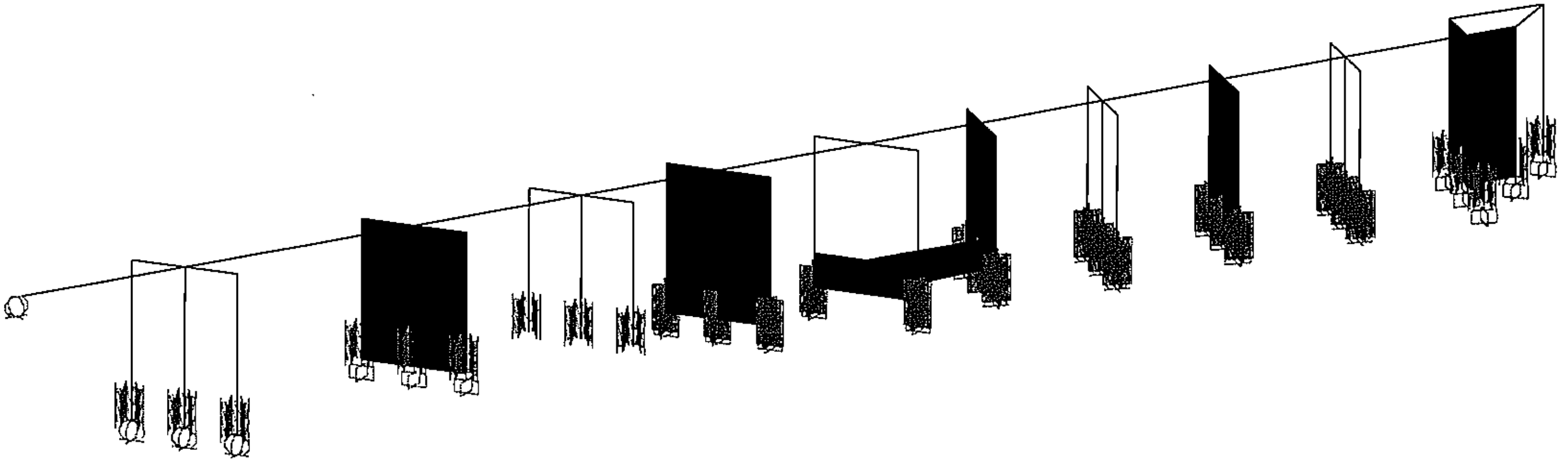
1/29/04 13:41:23



SAP2000 v8.0.9 - File:F1-NEW - 3-D View - Kip, ft, F Units

SAP2000

1/29/04 13:42:21



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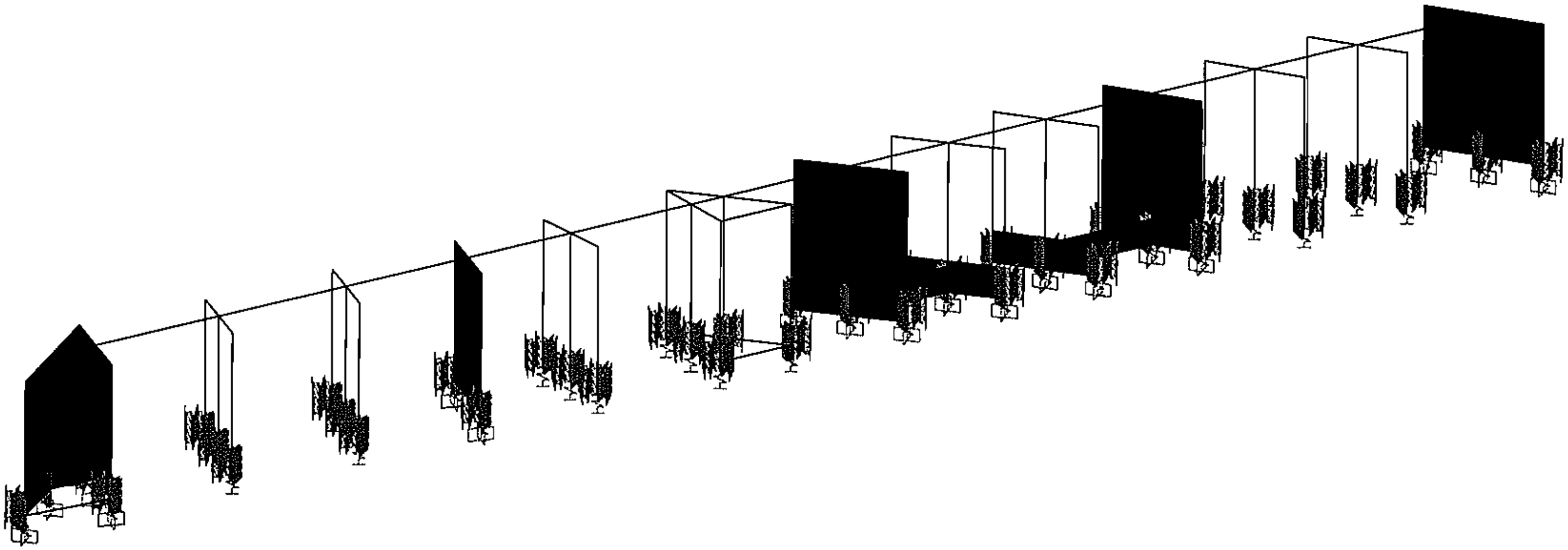
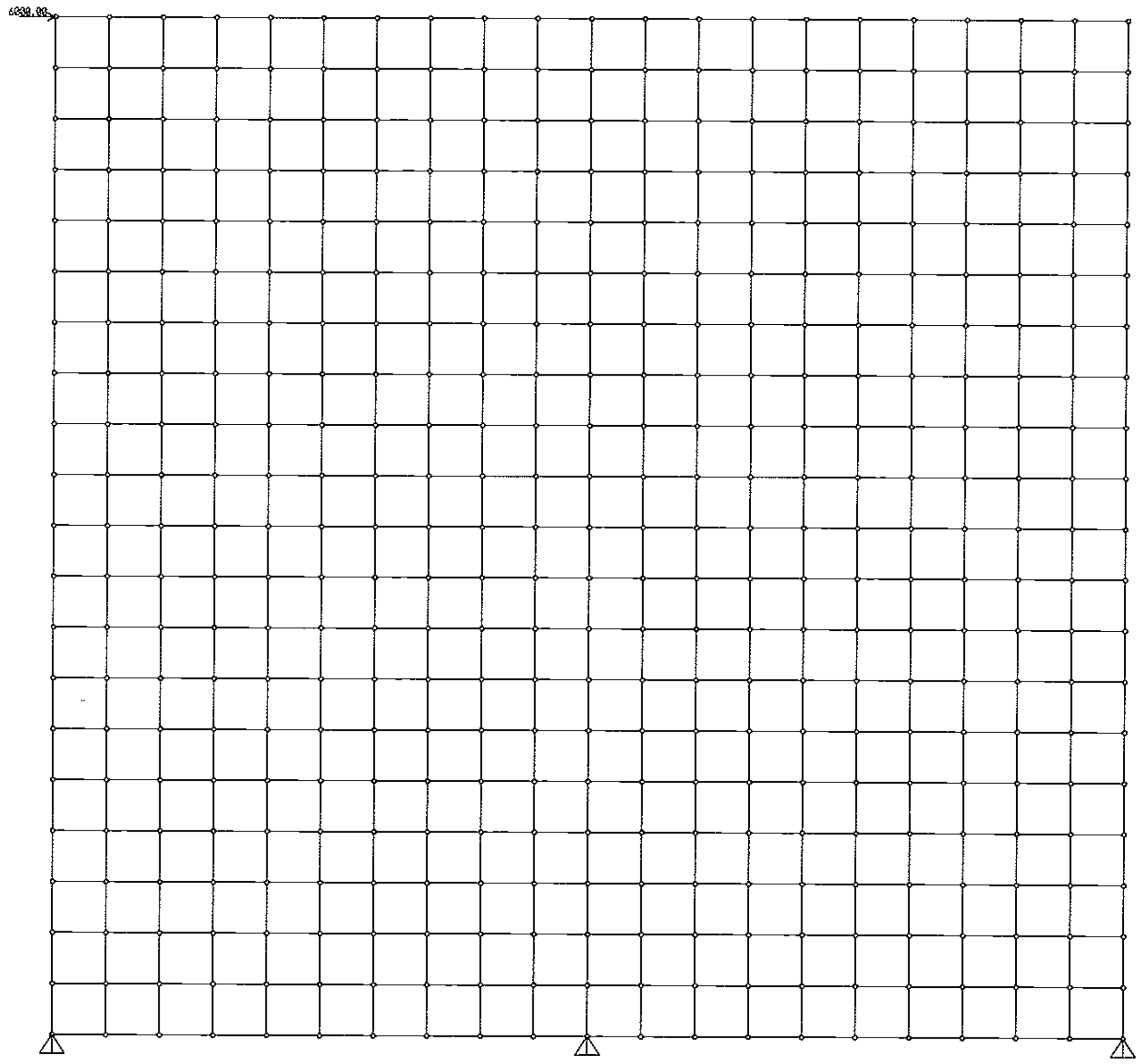
SAP2000

1/29/04 13:48:33

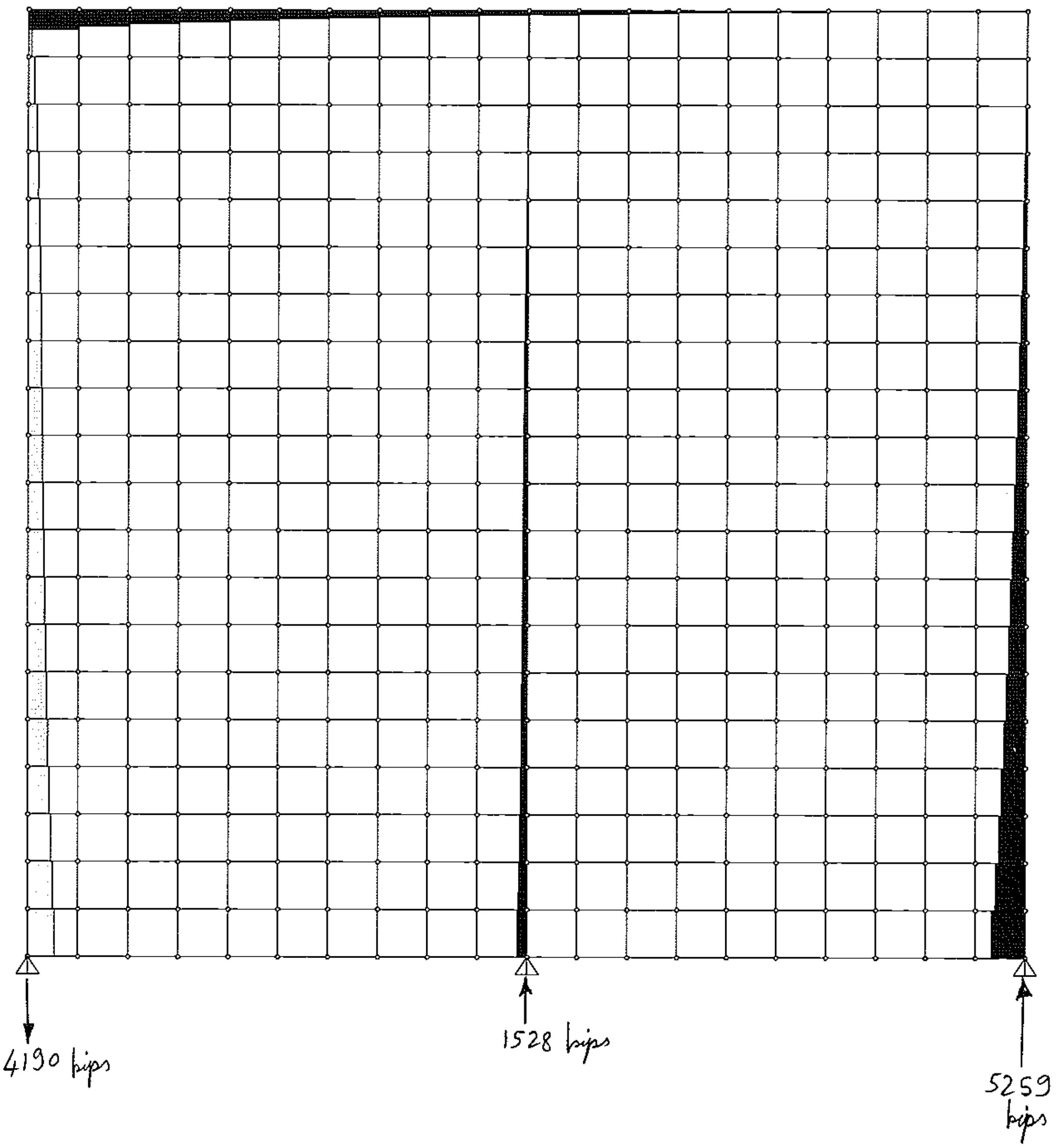


SAP2000 v8.0.9 - File:FR4-new - 3-D View - Kip, ft, F Units

Elastic Analyses of Infill Walls without and with Footings



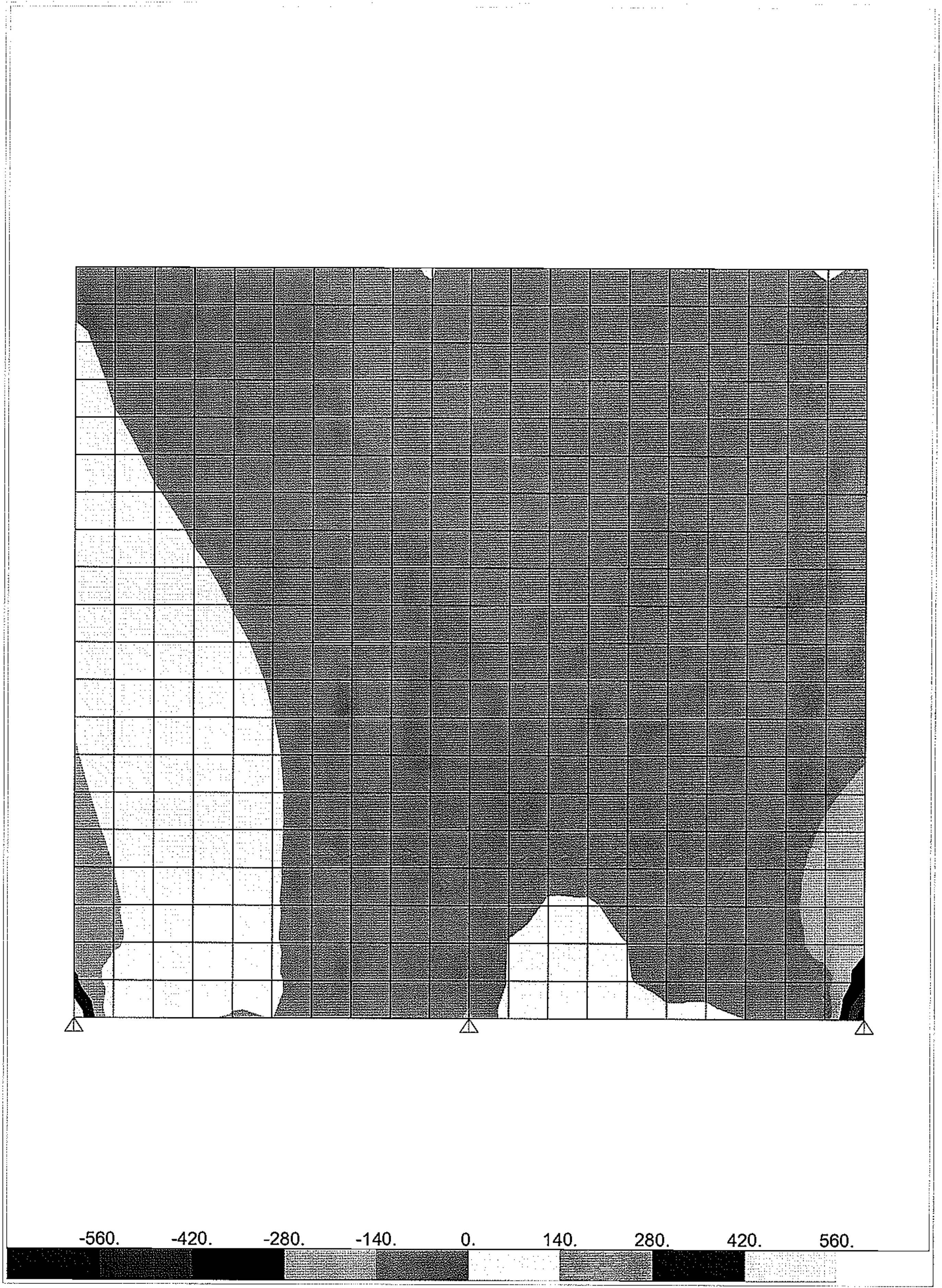
SAP2000 v7.44 - File:wallwcolumns - Joint Loads (LOAD1) - Kip-ft Units



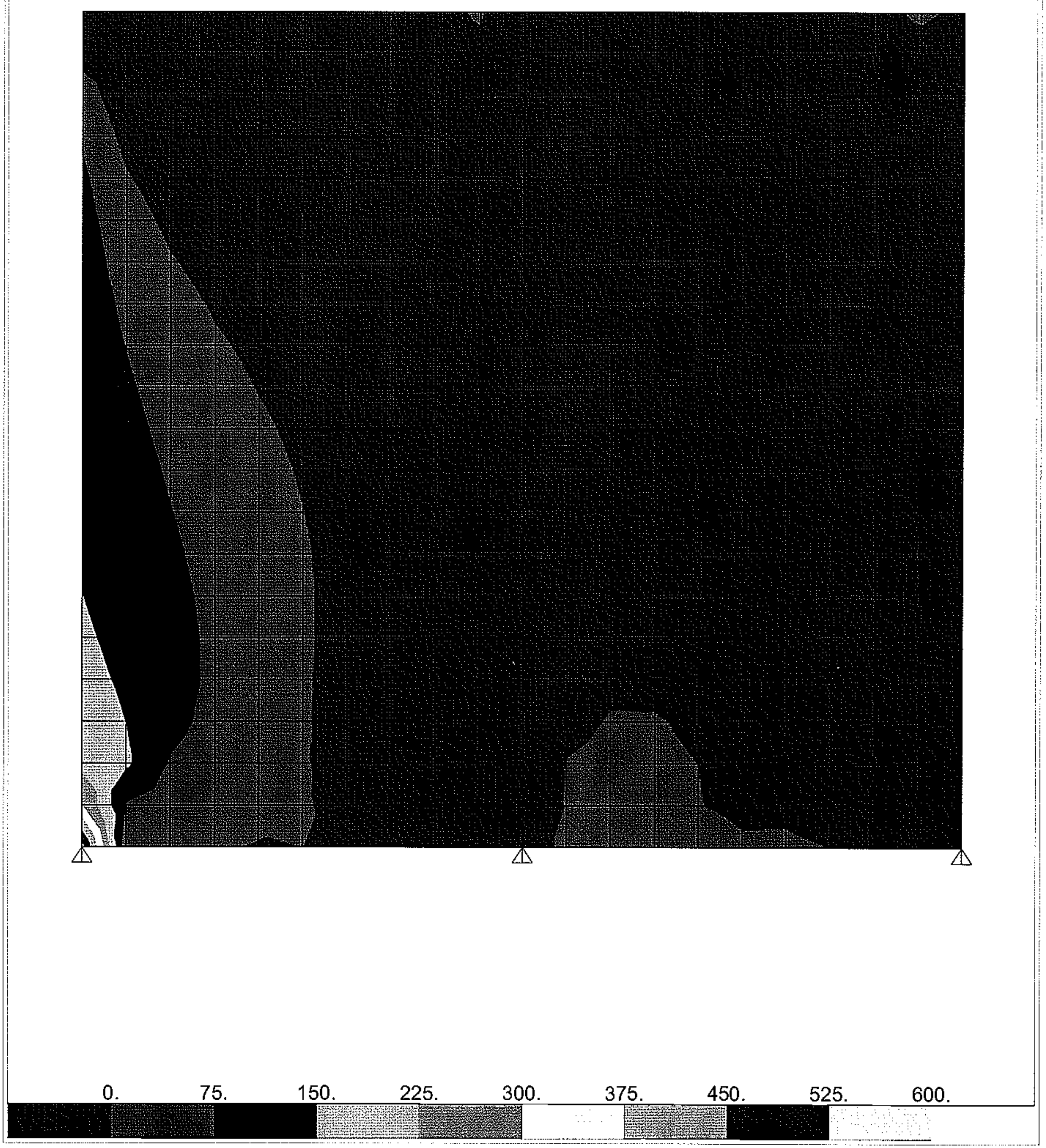
SAP2000 v7.44 - File:wallwcolumns - Axial Force Diagram (LOAD1) - Kip-ft Units

SAP2000

12/1/03 6:56:05



SAP2000 v7.44 - File:wallwcolumns - Resultant F22 Diagram (LOAD1) - Kip-ft Units



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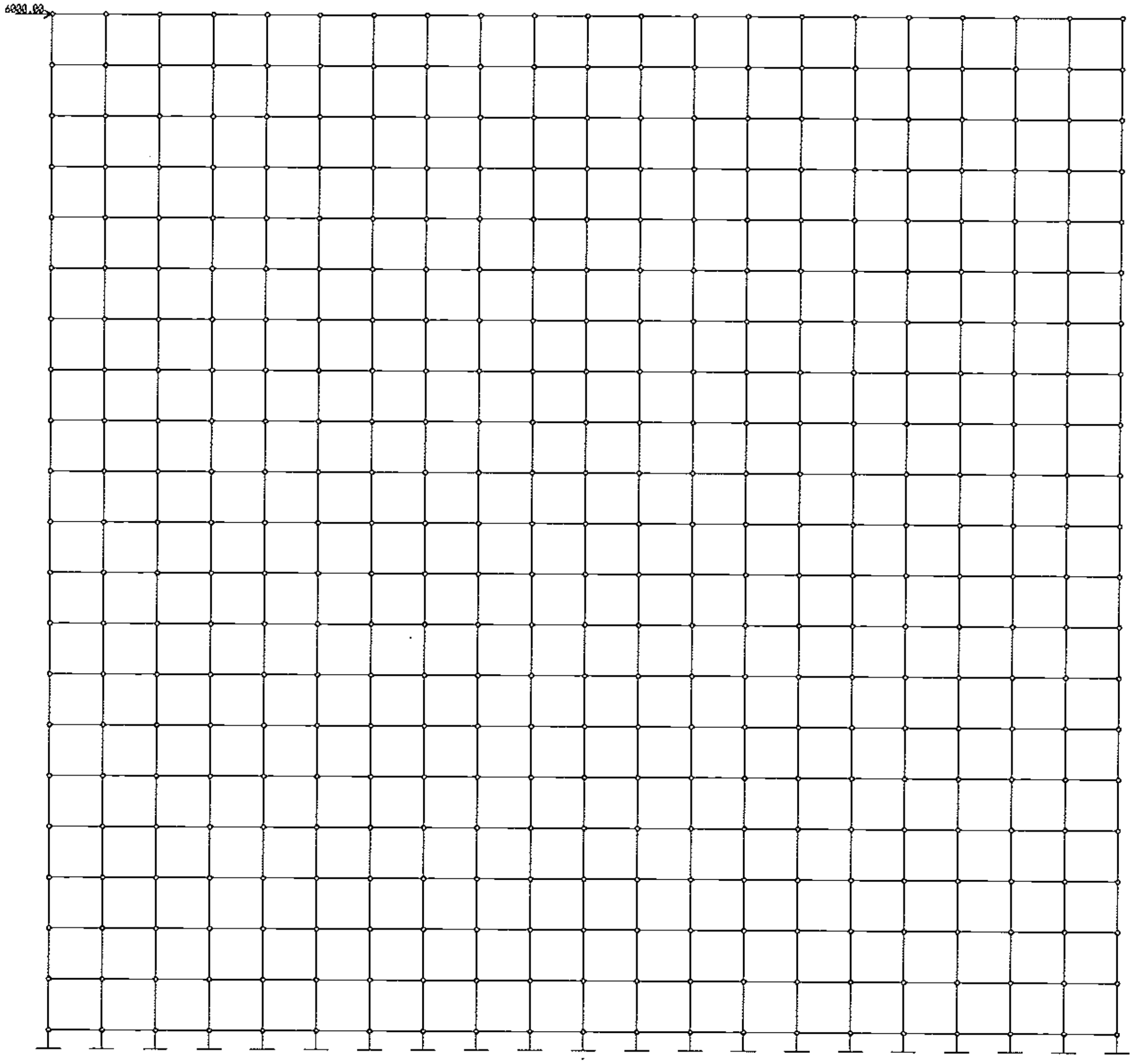
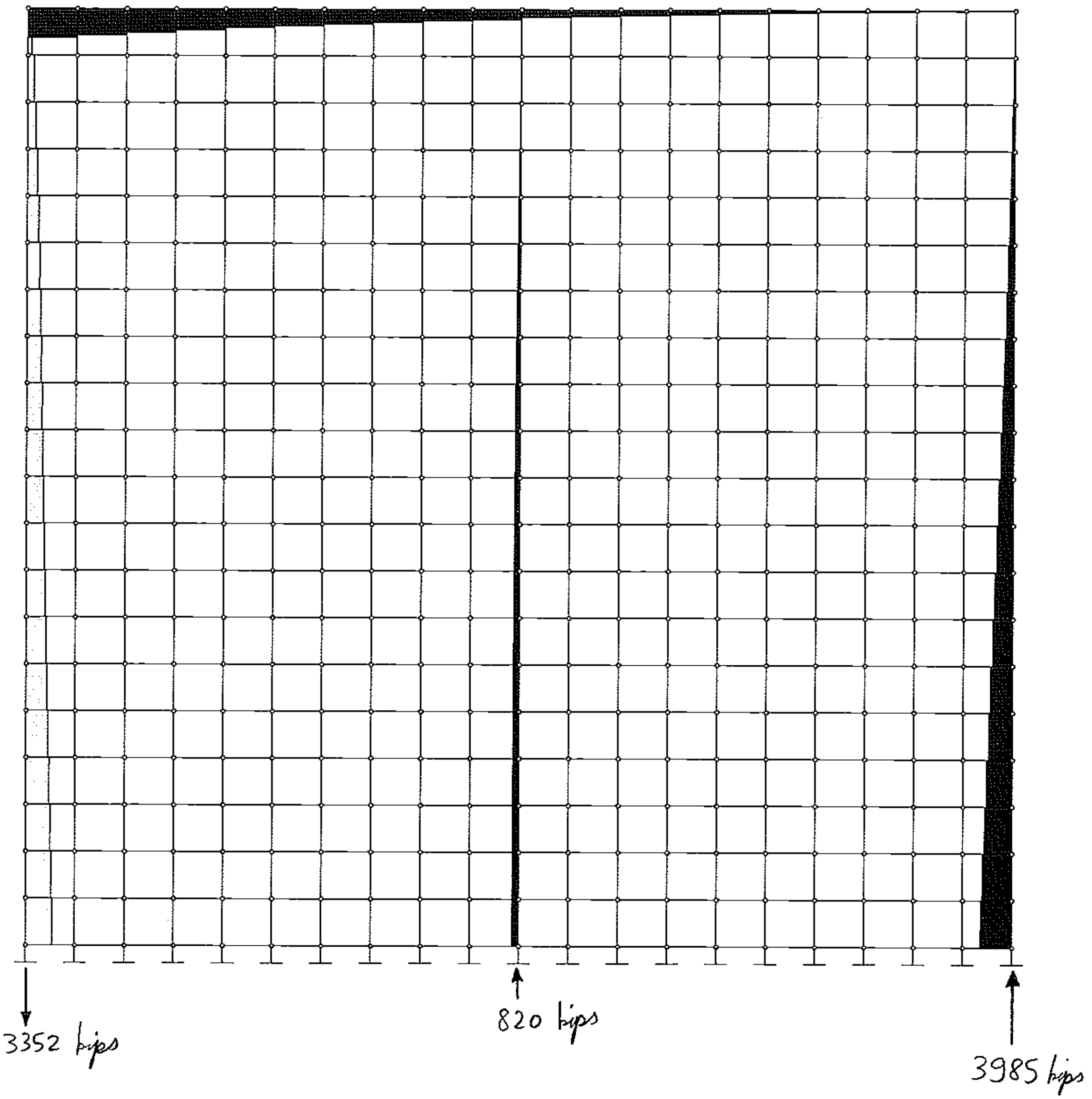
SAP2000

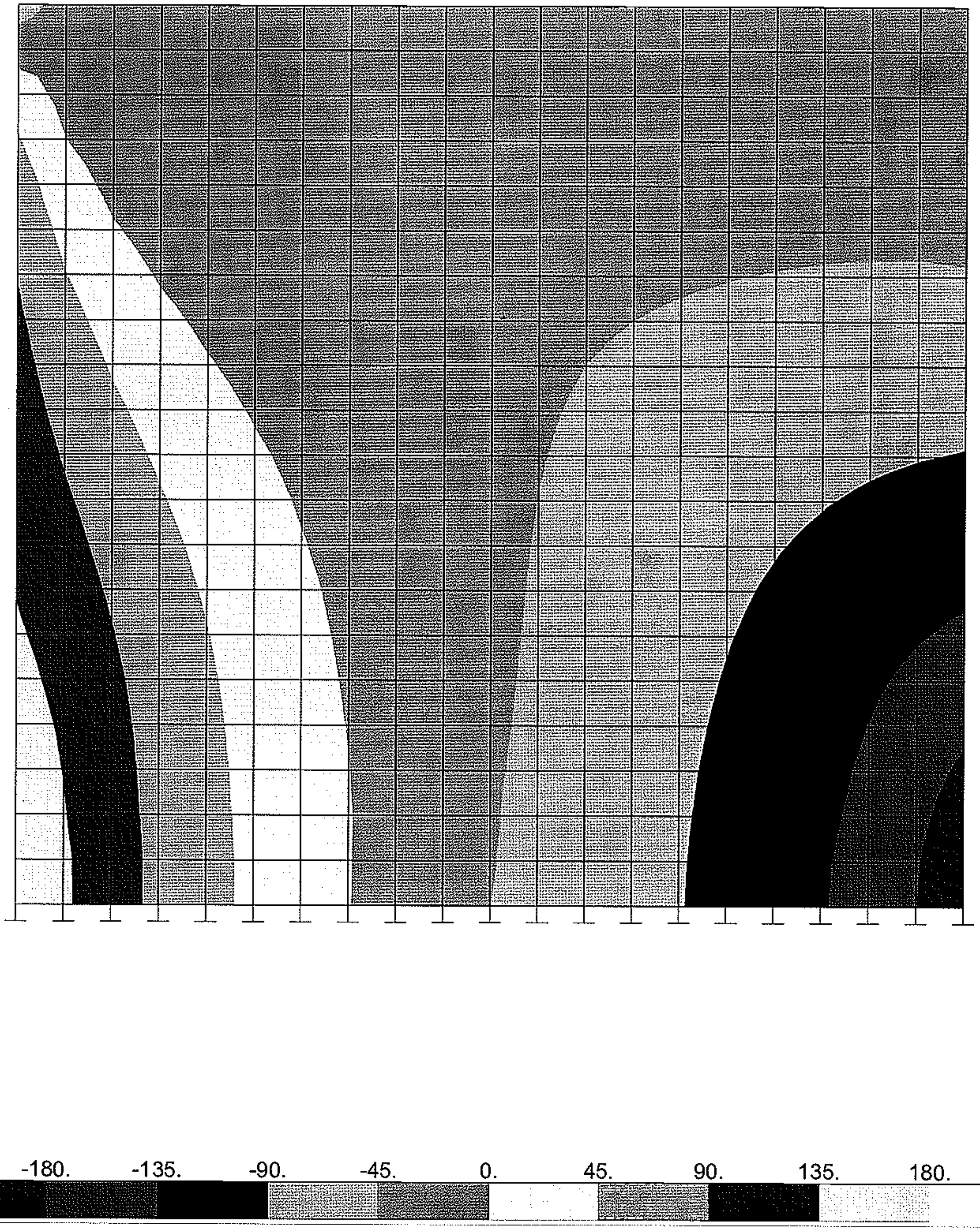
12/1/03 6:58:02



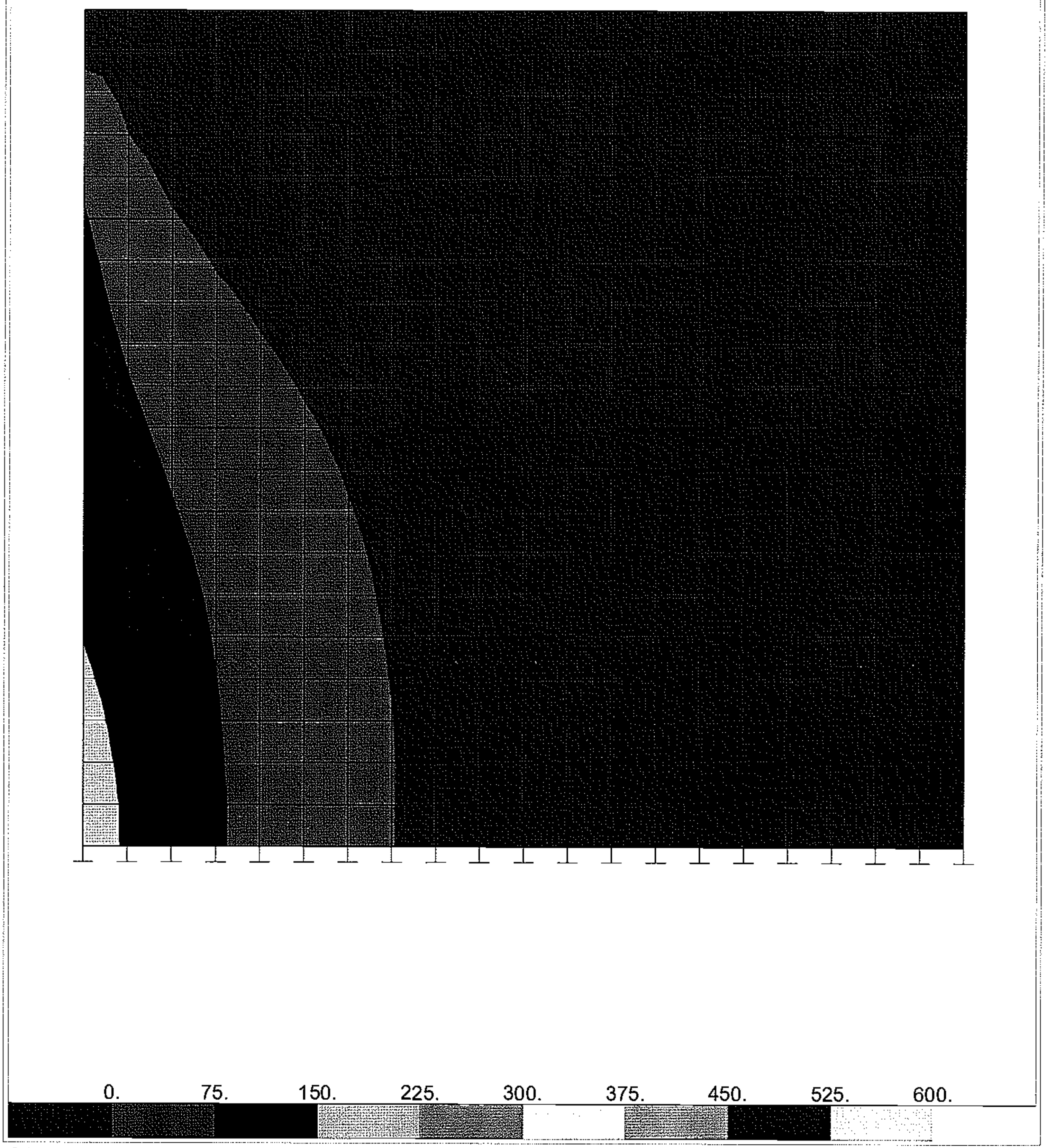
SAP2000 v7.44 - File:wallwfooting - Joint Loads (LOAD1) - Kip-ft Units



SAP2000 v7.44 - File:wallwfooting - Axial Force Diagram (LOAD1) - Kip-ft Units



SAP2000 v7.44 - File:wallwfooting - Resultant F22 Diagram (LOAD1) - Kip-ft Units



SAP2000 v7.44 - File:wallwfooting - Resultant F22 Diagram (LOAD1) - Kip-ft Units

APPENDIX F

FIELD PHOTOS

Figure F.1. Overall view of the Sixth Street Viaduct looking north

Figure F.2. View of the Sixth Street Viaduct looking east from approximately Bent 10

Figure F.3. View of the arch spans over the Los Angeles River looking north

Figure F.4. View of the steel arches from the deck level looking west

Figure F.5. Span between Bents 12 and 13

Figure F.6. Typical view of girders and deck soffit

Figure F.7. Typical cracking on outside face of columns and superstructure girders

Figure F.8. Typical condition of columns in the east approach spans

Figure F.9. Typical horizontal cracks in column at level of bent cap

Figure F.10. Typical cracking pattern in bent caps and girders

Figure F.11. Cracking in webs of superstructure longitudinal girders

Figure F.12. Severe cracking in bent cap and deck soffit

Figure F.13. Reopened crack at surface of column with two generations (colors) of epoxy injection

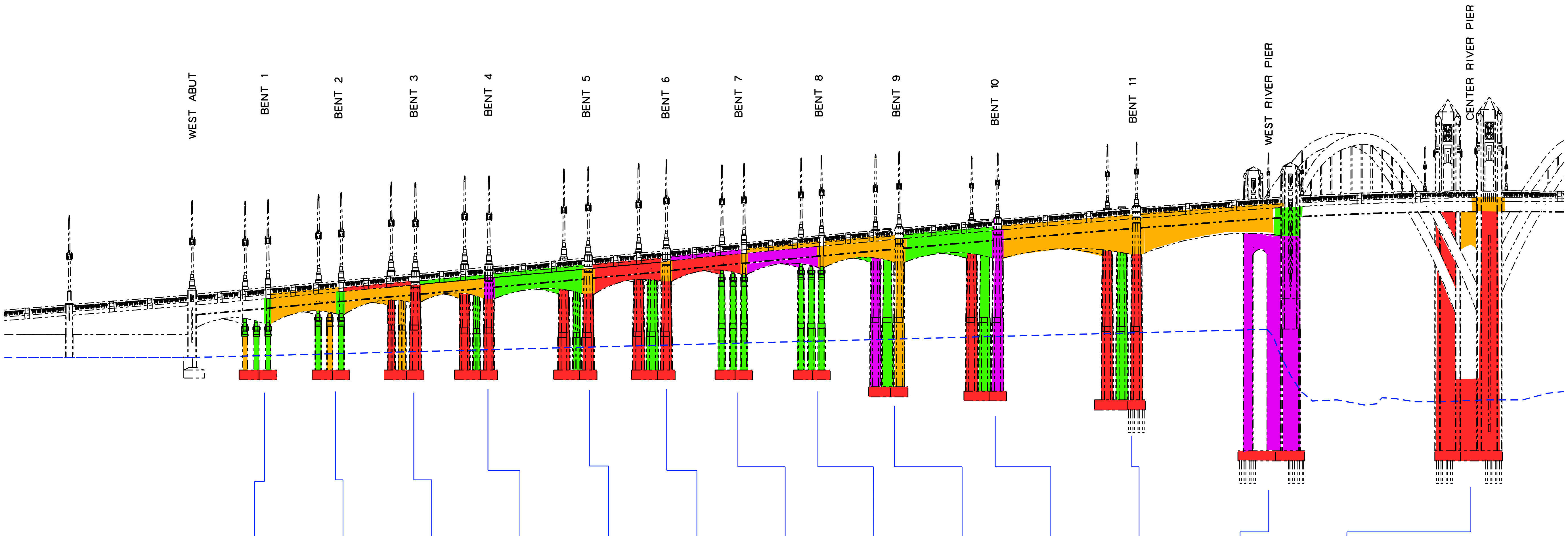
Figure F.14. Cracks in column below ground level

Figure F.15. Water leaking from the deck at an expansion joint

Figure F.16. Water leaking from a typical horizontal column crack at bent cap

APPENDIX G

MATERIAL TESTING AND SAMPLING



BENT#	1		2		3		4		5		6		7		8		9		10		11		W-P	C-P	BENT#
NORTH COLUMN	M		L		S		S		S		S		L		L		M-S		S		S		M-S	S	NORTH COLUMN
NORTH BENT CAP	L		L		S		M-S		M		M		M		M		M		S		M		L	M	NORTH BENT CAP
MIDDLE COLUMN	L		M		M		L		L		L		L		L		L		L		L		-	-	MIDDLE COLUMN
SOUTH BENT CAP	L		L		S		M-S		M		M		M		M		M		S		M		L	M	SOUTH BENT CAP
SOUTH COLUMN	L		L		S		S		S		S		L		L		M		M-S		S		M-S	S	SOUTH COLUMN
FOOTING	S		S		S		S		S		S		S		S		S		S		S		S	S	FOOTING
GIRDERS		M		M		M		L		S		S		M-S		M		L		M		M			GIRDERS
DECK		M		S		L		L		S		M-S		M		M		L		M		M			DECK

LEGEND:

- 🟩 L = LIGHT DAMAGE
- 🟧 M = MODERATE
- 🟪 M-S = MODERATE - SEVERE
- 🟥 S = SEVERE

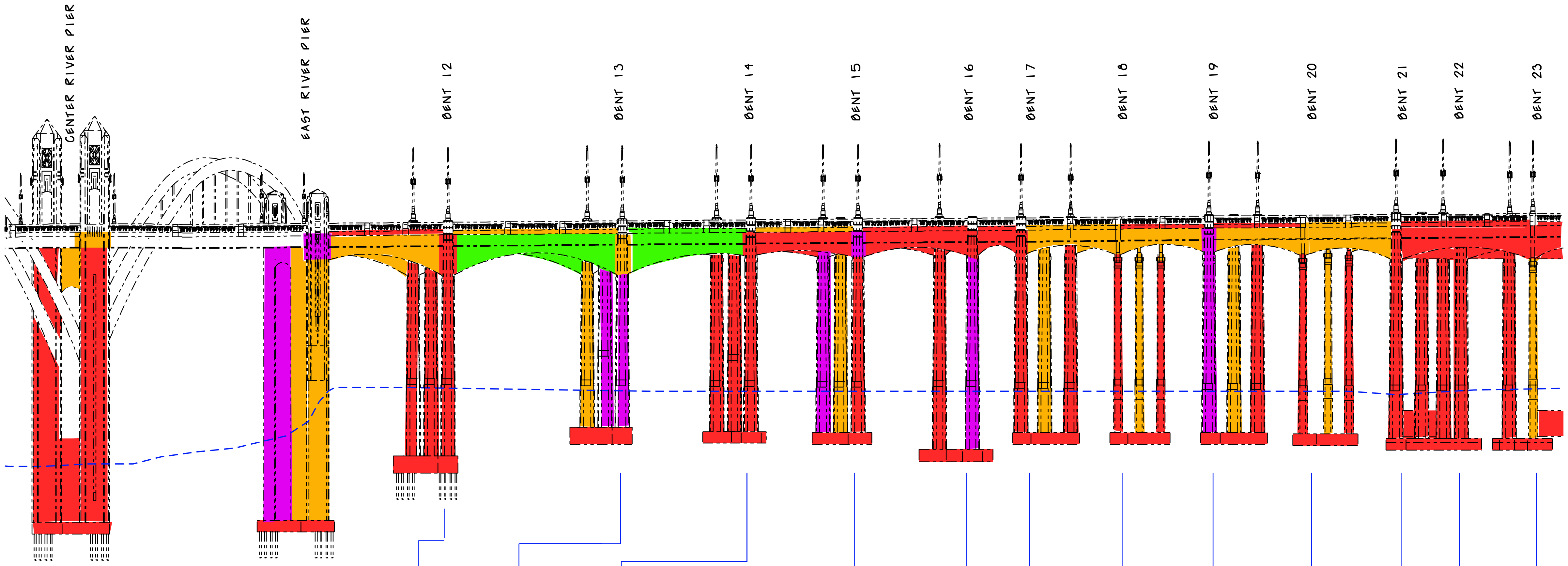
DESIGNED
DRAWN
CHECKED
SUPERVISED
PROJECT ENGR.
R. E. NO.
ASST.DIV./DIST.ENGR.
R. E. NO.
DATE

SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
MATERIAL CONDITION . I

NO.	REVISION DESCRIPTION	DIV./DIST. ENGR.	DATE

CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER
DATE 20
DIV./DIST.ENGR.
R. E. NO.

SCALES HORIZ. 1" = 40'-0" VERT. 1" = 20'-0"
SHEET OF SHEETS INDEX NUMBER



BENT•	E-P		12		13		14		15		16		17		18		19		20		21		22		23	BENT•
NORTH COLUMN	M-S		S		M		S		M-S		S		S		S		S		S		S		S		M	NORTH COLUMN
NORTH BENT CAP	M-S		S		M		S		M-S		S		S		M		S		M		S		S		S	NORTH BENT CAP
MIDDLE COLUMN	-		S		M-S		S		M		-		M		M		M		M		S		S		S	MIDDLE COLUMN
SOUTH BENT CAP	M-S		S		M		S		M-S		S		S		M		S		M		S		S		S	SOUTH BENT CAP
SOUTH COLUMN	M		S		M-S		S		S		M-S		S		S		M-S		S		S		S		M	SOUTH COLUMN
FOOTING	S		S		S		S		S		S		S		S		S		S		S		S		S	FOOTING
GIRDERS		M		L		L		S		S		S		M		M		M		M		S		S		GIRDERS
DECK		S		M		L		M		S		S		M		S		S		M		M		S		DECK

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🟩 L = LIGHT DAMAGE

🟧 M = MODERATE

🟪 M-S = MODERATE - SEVERE

🟥 S = SEVERE

SCALES HORIZ. [illegible]" = [illegible]0'-0"
VERT. 1" = 20'-0"

SHEET OF SHEETS INDEX NUMBER

CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER

DATE ____________ 20 ____

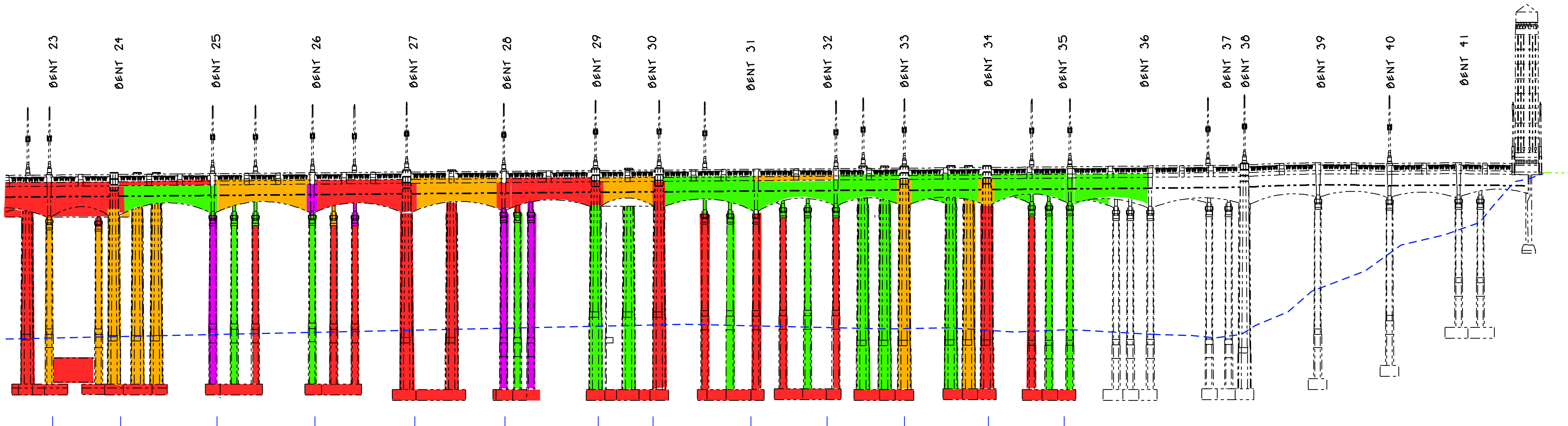
DIV./DIST.ENGR. R. E. NO.

NO.	REVISION DESCRIPTION	DIV./DIST. ENGR.	DATE

SIXTH STREET VIADUCT
OVER THE
LOS ANGELES RIVER
MATERIAL CONDITION

.2

	DATE
DESIGNED	
DRAWN	
CHECKED	
SUPERVISED	
PROJECT ENGR.	
R. E. NO.	
ASST.DIV./DIST.ENGR.	
R. E. NO.	



BENT•	23		24		25		26		27		28		29		30		31		32		33		34		35		BENT•
NORTH COLUMN	M		M		S		S		S		M-S		M		M		S		S		L		L		S		NORTH COLUMN
NORTH BENT CAP	S		S		L		M-S		S		S		S		S		L		L		M		M		L		NORTH BENT CAP
MIDDLE COLUMN	S		M		L		S		-		L		L		L		L		L		L		M		L		MIDDLE COLUMN
SOUTH BENT CAP	S		S		L		M-S		S		S		S		S		L		L		M		M		L		SOUTH BENT CAP
SOUTH COLUMN	M		M		M-S		L		S		M-S		L		S		S		S		M		S		L		SOUTH COLUMN
FOOTING	S		S		S		S		S		S		S		S		S		S		S		S		S		FOOTING
GIRDERS		S		L		M		S		M		S		M		L		L		L		L		L		L	GIRDERS
DECK		S		S		M		S		M		S		M		L		M		L		L		L		L	DECK

LEGEND:

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🟥 S = SEVERE

SIXTH STREET VIADUCT OVER THE LOS ANGELES RIVER MATERIAL CONDITION

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NO.
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DIV./DIST. ENGR.
DATE

CITY OF LOS ANGELES
GARY L. MOORE, P.E. CITY ENGINEER
DATE ________ 20 ____
DIV./DIST.ENGR.
R. E. NO.

SCALES HORIZ. 1" = 40'-0" VERT. 1" = 20'-0"
SHEET OF SHEETS
INDEX NUMBER

6th St. Viaduct
Los Angeles, CA

Comparison of Visual Survey and Core Condition Ratings

Below Grade Cores Not Included

Legend

🟫 S = Severe Deterioration
🟪 MS = Moderate to Severe Deterioration
🟥 M = Moderate Deterioration
🟧 LM = Light to Moderate Deterioration
🟨 L = Light Deterioration
🟩 N = No cracking in core

* **Poor correlation**, difference of 3 or more in visual vs core rating,
i.e. S to LM, MS to L, and M to N.

Overall 111/125 89% With good correlation, i.e. similar visual and core ratings.

Locations With Poor Correlation

West Side	1/43	2%	With visual rating worse than core rating.
West Side	1/43	2%	With visual rating better than core rating.
East Side	1/76	1%	With visual rating worse than core rating.
East Side	11/76	14%	With visual rating better than core rating.

Columns	32/74	43%	With Visual rating better than core rating.
Bent Caps	6/15	40%	With Visual rating worse than core rating.
Girders	11/18	61%	With Visual rating worse than core rating.
	1/18	6%	With Visual rating better than core rating.
Deck	10/18	56%	With Visual rating worse than core rating.
	2/18	11%	With Visual rating better than core rating.

Core located immediately above grade.

One girder and deck core are in the same bay.

Summary of Correlation of Visual Survey and Core Distress Ratings

	Good Correlation	Poor Correlation	Visual distress **equals** Core distress	Visual distress **worse than** Core distress	Visual distress **less than** Core distress
Columns	64	10	24	18	32
Bent Caps	14	1	7	6	2
Girders	16	2	6	11	1
Decks	17	1	6	10	2
Total	111	14	43	45	37
Percentage	89%	11%	34%	36%	30%

Cores 89 to 137 were taken and inspected in the retrofit strategy phase

West Side

																							*														*						
Bent #	1	1	2	2	3	3	3.2	4	4	5	5	5	5	5.5	5.5	6	6	6	6	6.5	6.5	6.5	6.5	6.5	6.5	7	7	8	8	8.5	8.5	8.5	8.5	8.8	8.8	9	9	9	9	9	9	10	10
Ele. Code	c	c	c	c	c	c	g	c	c	c	c	bc	c	d	d	bc	c	bc	c	d	g	g	d	g	g	c	c	c	c	g	d	g	g	g	d	c	bc	c	bc	c	c	c	c
Core Distress Rating	L	M	N	N	S	S	MS	S	S	S	S	L	S	M	S	MS	S	S	S	L	M	N	N	S	S	N	N	N	N	M	M	N	N	MS	MS	MS	MS	N	S	M	MS	MS	S
Visual Surey Rating	L	L	L	L	M	S	S	MS	MS	MS	MS	LM	S	S	S	M	S	S	S	M	S	L	M	S	S	L	L	L	L	M	M	L	L	MS	S	S	M	L	S	S	S	MS	S
Core Count	94	95	93	100	98	55	59	102	101	130	131	39	56	42	43	50	34	49	57	41	31	33	40	32	64	29	30	28	99	37	38	27	35	60	58	46	51	36	47	89	90	91	92
Core ID						P7	C25					E6	C29	C24	P6 / C25	P10	C23	C27	P8	C15	C13	C14	CL4	P4	C30	C12	E4	P5		CL5	E5	C27	C28	P11	E7	P9	E8	C26	C31				
Grid	A	E	A	E	A	E	E	A	E	A	E	D - E	E	D- E	D- E	D - E	A	D - E	E	A- B	A	B	A- B	A	C	A	A	A	E	A	A - B	A	B	E	D-E	E	D	C	D-E	A	E	A	E
Joint?	No	No	No	No	Yes	Yes	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes	Yes	Yes	Yes	No	No

River Piers

	C. river pier	C. river pier	C. river pier	C. river pier	W. river pier	W. river pier
	c	c	c	c	c	c
	S	S	S	S	MS	MS
	M	M	M	M	M	M
	61	62	63	65	134	135
	C33	E9	P12	C32		
	E	E	A	A	A	E
	No	No	No	No	Yes	Yes

East Side

																																			*	*			*			*		*				*	*
Bent #	14	14	15	15	15.8	16	16	16	17	17	17	17.5	17.5	17.5	17.5	17.5	17.5	18	18	19	19	20	20	21	21	21	21	21	22	22	23	22.5	22.5	24	24	25	25	25	26	26	26	26	26	26.2	26.5	26.5	27	27	28	28
Ele. Code	c	c	c	c	g	c	c	bc	c	c	bc	d	g	g	d	d	d	c	c	c	c	c	c	c	c	bc	c	c	c	bc	c	d	d	c	c	c	c	bc	c	c	bc	c	c	d	g	g	c	c	c	c
Core Distress Rating	S	S	S	MS	S	MS	S	S	S	S	S	M	N	N	N	S	S	S	S	MS	S	S	S	S	S	S	S	S	S	N	M	L	M	S	S	MS	S	N	L	S	M	N	S	S	N	N	S	S	MS	MS
Visual Surey Rating	M	S	S	S	S	S	S	S	S	S	S	M	LM	L	LM	M	M	S	M	S	S	S	S	S	MS	S	S	S	S	LM	M	LM	S	M	M	L	L	L	L	L	MS	L	L	S	MS	L	M	M	L	L
Core Count	137	125	136	126	48	123	127	52	18	21	53	24	19	20	25	22	23	105	124	104	13	120	121	1	2	4	5	6	122	3	108	7	8	110	109	118	119	68	54	132	66	44	69	67	26	45	103	107	111	106
Core ID					C36			P13	C16	C20	C37	CL3	C18	C19	E3	C17	P3				C11			C1	C2	E1	CL1	P1		C3		C4	C5					P17	C39		P14	C21	P18	C38	C22	C35/P4				
Grid	A	E	A	E	B	A	E	B-C	E	A	B-C	C-D	D	E	C-D	D-E	D-E	A	E	A	E	A	E	A	C	A-B	A	E	E	C	A	A-B	A-B	A	E	A	E	D - E	A	E	B-C	A	C	B-C	B	A	A	E	A	E
Joint?	Yes	Yes	No	No	No	No	No	No	Yes	Yes	Yes	No	No	No	No	No	No	No	No	Yes	Yes	No	No	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes	No	No

East Side

										*	*	*									*				*	
Bent #	28	28.2	28.8	29	29	29	30	30	31	31	32	32	33	33	33	33.5	33.5	33.5	33.5	33.5	33.5	34	34	34.7	35	35
Ele. Code	bc	d	d	c	bc	bc	c	c	c	c	c	c	c	c	c	g	g	g	d	d	g	c	c	d	c	bc
Core Distress Rating	S	S	MS	L	M	S	S	M	S	S	S	S	M	L	M	L	N	N	N	N	S	S	L	N	S	N
Visual Surey Rating	S	S	S	M	S	S	MS	MS	M	L	L	L	M	M	M	L	L	L	L	L	L	M	M	L	L	L
Core Count	76	75	74	115	72	73	116	112	114	133	129	128	14	113	15	17	9	10	11	12	16	117	96	71	97	70
Core ID	C42	A1	C41		P20	C40							E2		C6	CL2	C7	C8	C10	C9	P2			E10		P19
Grid	A - B	B - C		A	D-E	D-E	A	E	A	E	A	E	A	E	A	B	C	B	B-C	B-C	B	A	E	D - E	E	E
Joint?	No	No	Near Joint	No	Yes	Yes	No	No	Yes	Yes	No	No	No	Yes	No	No	No	No	No	No	No	Yes	Yes	No	No	No

6th Street Viaduct, Los Angeles, CA
Preliminary Visual Survey

Legend

S=Severe	M-S	M=Moderate	L-M	L=Light
🟫	🟪	🟥	🟧	🟨

------- = Joint in Deck

Bent	Joint In Deck	North Column	North Bent Cap	Middle Column	South Bent Cap	South Column	Girders	Deck	Photos
1		L	L	L	L	L			K2.10
							M	M	
2		L	L	M	L	L			K2.9
							M	S	
3	-------	S	S	M	S	M			K2.5, 8, 11
							M	L	K2.4
4		M-S	M-S	L	M-S	M-S			K1.89, K2.2-3
							L	L	
5		M-S	M	L	M	M-S			K1.85, 86, 88
							S	S	
6	-------	S	M	L	M	M-S			K1.84, 1.87
							S	M-S	
7		L	M	L	M	L			K1.83, D3.61
							M-S	M	K2.1
8		L	M	L	M	L			K1.80-81
							M	M	
9	-------	S	M	L	M	S			K1.77-78, K1.82, K2.13, K6.113-114
							L	L	K1.78
10		S	S	L	S	M-S			K1.73-76, K2.14
							M	M	
11		S	M	L	M	S			K1.69-71
West Pier	-------	M	L		L	M			K1.72
								M	
Center Pier		M	M		M	M			K1.67, D2.73-74
								M	D2.75-76
East Pier	-------	M-S	M-S		M-S	M			K1.65
							M	S	K1.66, 63, 68
12		S	S	S	S	S			K1.64, 68
							L	M	K1.61-62
13		M	M	M-S	M	M-S			
							L	L	
14	-------	S	S	S	S	M			K1.60, D3.55-56
							S	M	
15		S	M-S	M	M-S	S			K1.59
							S	S	
16		S	S	NO COL	S	S			K1.58
							S	S	
17	-------	S	S	M	S	S			K1.57
							M	M	
18		M	M	M	M	S			K1.56
							M	S	
19	-------	S	S	M	S	S			K1.54-55, D3.51
							M	S	
20		S	M	M	M	S			K1.53
							M	M	
21	-------	-	S	M-S	S	S			K1.52
							S	M	
22		S	S	M	S	S			K1.48, 51
							S	S	K1.50
23		-	S	M	S	M			K1.49
							S	S	
24	-------	S	S	M	S	S			K1.47
							L	S	
25		L	L	L	L	L			K1.46
							M	M	
26		L	M-S	L	M-S	L			K1.43, 45
							S	S	K1.33
27	-------	M	S	-	S	M			K1.32, 36
							M	M	K1.29
28		L	S	L	S	L			
							S	S	K1.24
29		-	S	L	S	M			K1.28
							-	-	
30		M-S	S	L	S	M-S			K1.23, 26
							L	L	K1.22, 25
31	-------	L	L	L	L	M			K1.21, 37
							L	M	K1.20
32		L	L	L	L	L			K1.19, 38
							L	L	
33	-------	M	M	L	M	M			K1.39, D3.32
							L	L	
34	-------	M	M	M	M	M			
							L	L	
35		L	L	L	L	L			
							L	L	
36		L	L	L	L	L			
							L	L	
37	-------	M	M	M	M	M			
38		L		L					
39		L		L					
	S	14	15	2	15	12	10	12	80
	M-S	4	4	2	4	6	1	1	22
	M	8	13	12	13	12	12	14	84
	L	13	8	21	8	10	12	10	82
		39	40	37	40	40	35	37	

Sixth St. Viaduct
Los Angeles, CA

Core Sample Distress Ratings and Distribution By Span And Element Cored

Legend

N	No Cracking
L	Light Distress
M	Moderate Distress
MS	Moderate to Severe Distress
S	Severe Distress

East Side of Viaduct

Bent #	12		13		14		15		16		17		18		19		20		21		22		23		24		25		26		27		28		29		30		31		32		33		34		35
Columns Cored					S		S		S		S		S		S		S		S		S		M		M		S		S		S		MS		M		S		S		S		M		S		S
					S		MS		MS		S		S		MS		S		S				M				MS		S		S		MS		L				S		S		M		L		
																			S										L														L				
																			S										N																		
Bent Caps Cored									S		S																N		M				S		S												N
																			S		N														M												
Girders Cored																																												S			
								S				N																		N														L			
												N																		N														N			
																																												N			
Deck Cores												S																																N			
												S										M								S				S										N		N	
												M										L												M/S													
												N																																			
Below Grade																			S		S		S																								
																							S																								
Grade Beams																			S	S			S																								
																							S																								
Footings																			S				S																								
																			S				S																								
Total Cores	88 Cores From East Side of Viaduct																																														

West Side of Viaduct

Bent #	1		2		3		4		5		6		7		8		9		10		11
Columns Cored	M		N		S		S		S		S		N		N		MS		S		
	L		N		S		S		S		S		N		N		MS		MS		
									S								M				
																	N				
Bent Caps Cored									L		S						S				
											MS						MS				
Girders Cored						MS						S				MS					
												M				MS					
												N				N					
												N				N					
Deck Cores																					
										S		L				MS					
										M		N				M					
Below Grade																					
Grade Beams																					
Footings																					
Total Cores	43 Cores From West Side of Viaduct																				

River Spans

W		C		E
MS		S		
MS		S		
		S		
		S		

6 Cores From River Piers

Summary

	N	L	LM	M	MS	S	Total
Column Cores							
	8	5		8	11	42	74
	11%	7%	0%	11%	15%	57%	
Bent Caps							
	3	1		2	2	7	15
	20%	7%	0%	13%	13%	47%	
Girders							
	10	1		1	3	3	18
	56%	6%	0%	6%	17%	17%	
Decks							
	5	2		4	2	5	18
	28%	11%	0%	22%	11%	28%	
Beow Grade							
	0	0		0	0	12	12
	0%	0%	0%	0%	0%	100%	
Total	26	9	0	15	18	69	137
	19%	7%		11%	13%	50%	

APPENDIX H

GEOTECHNICAL MEMOS

URS

Memorandum

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6th Street Bridge over LA River
URS Job No. 57-00255002.01 00102

TO:	**Dan Weddell (WKA) By E-mail** **Renee Au (URS) By E-mail**
FROM:	**Farid Motamed**
DATE:	**June 7, 2002**
SUBJECT:	**Preliminary Geotechnical Memorandum**

Site Location and Description

The subject site is the 6th Street Bridge (No. 53C-1880) where it crosses the LA River in Los Angeles, California. The site's coordinates are 118.038 W, 34.227 N. The site is located approximately 1 mile northwest of the intersection of I-5 and I-10. The 6th Street Bridge, which connects to Whittier Boulevard, was built in 1932 to create an entrance to the City of Los Angeles from the East Los Angeles. The total length of the structure is 3,168 feet running from San Mateo Street in the west to the I-5 in the east. The bridge crosses several local streets (Santa Fe Avenue, Mesquite Avenue, South Mission Road, and South Clarence Street), I-101 in the east, and the LA River. The Bridge location and its general vicinity are shown in Figure 1.

The bridge foundations are footings generally embeded between 15 and 27 feet. Footing sizes vary; details of the foundation dimensions are presented in Table 2.

Site Conditions

Based on the borings and laboratory testing performed previously (Dames & Moore 1997), the subsurface conditions at the site include up to 5 feet of fill soils consisting of medium dense silty sand. The fill is immediately underlain by dense to very dense, native, alluvium comprising alternating layers of sands, gravelly sands and gravels. The alluvium is further underlain by firm and hard, dark gray clayey silt to the maximum depth explored, 175 feet. The clayey silt material generally grades to include fine sand and shell fragments and is interpreted to be the Repetto Member of the Fernando Formation. The fill soils are not expected within the Los Angeles River Channel.

One downhole shear wave velocity study was conducted beneath the bridge (November, 1996, Ryland Associates, Inc.) to determine shear wave velocities at intervals in the upper 150 feet of earth materials. A representative boring log and downhole test plot are presented in Appendix A.

Groundwater was not measured during the previous subsurface investigations; City of Los Angeles (1996) reports that based on Los Angeles County Department of Public Works monitoring wells in the area the depth to groundwater is expected to be more than 150 feet

Seismicity and Faulting

As it is the case with most of southern California, the site is located within an active seismic area. Due to its location, the site may experience severe seismic shakings in the future.

Based on the California Seismic Hazard Map (Caltrans, 1996) as shown on Figure 2, several significant faults surround the subject site. These include the Elysian Park Seismic Zone (EPK), Malibu Coast-Santa Monica-Hollywood-Raymond (MMR), Newport-Inglewood-Rose Canyon/E (NIE), Eagle Rock (ERK), Charnock (CNK), Verdugo (VDO), San Fernando-Sierra Madre-Duarte (SSD) and San Andreas/C (SAC). Fault parameters and distances are presented in the following table.

Fault	Maximum Credible Earthquake[1]	Estimated Closest Distance from Site (km)[1]	Fault Type[1]
Elysian Park Seismic Zone (EPK)	7	<1	Reverse
Malibu Coast-Santa Monica-Hollywood-Raymond (MMR)	7.5	10	Reverse/Oblique
Newport-Inglewood-RoseCanyon/E (NIE)	7	13	Strike-Slip
Eagle Rock (ERK)	6	14	Unknown
Charnock (CNK)	6.5	17	Strike-Slip
Verdugo (VDO)	6.75	19	Reverse/Oblique

[1] Obtained from California Seismic Hazard Map (1996)

Seismic Parameters and ARS Curve

Based on the California Seismic Hazard Map (Caltrans, 1996), the controlling earthquake has a magnitude of 7, resulting in a peak horizontal rock acceleration of 0.6g at the site.

For developing seismic spectra using ATC-32 (1996) the seismic parameters below may be used:

Peak Rock Acceleration (g)	0.6g
Soil Profile Type (As per ATC-32)	D
Maximum Credible Event Magnitude	7.25 (+/-0.25)

Since the structure is within 15 kilometers of an active fault, the spectral acceleration should be magnified as per Caltrans Seismic Design Criteria. In Addition, since the style of faulting is reverse, the response spectra should be increased an additional 20 percent over all periods.

The recommended enveloped ARS curve is provided in Figure 3. Figure 4 presents ARS curves contributed by each individual fault listed in the above fault table. The spectral ordinates of recommended ARS are provided in Table 1.

Probabilistic Seismic Hazard Analysis

A probabilistic seismic hazard analysis (PSHA) is a mathematical process based on probability and statistics that is used to estimate the mean number of events per year in which the level of some ground parameter (peak ground acceleration and spectral acceleration in this investigation) at the project site exceeds a specified value. This mean number of events per year refers to annual frequency of exceedance. The inverse of this number is called the "average return period" (ARP), which is expressed in terms of years.

The key elements of a PSHA are:

- Defining the location, geometry, and characteristics of earthquake sources relative to the site;
- Estimating the recurrence of earthquakes of various magnitudes, up to the maximum magnitude, on each source;
- Selecting appropriate attenuation relationships, which relate the variation of the earthquake ground motion parameter with earthquake distance and magnitude based upon the site geology and subsurface characterization; and,
- Performing the mathematical calculations, which combine individual seismic source probabilities, to obtain annual probabilities of the selected ground motion parameter being exceeded at the site.

For the project, PSHA is conducted to correlate various PGAs to their respective ARPs. A PGA versus ARP curve is developed in terms of nearby faults within 62 miles (100 km) using several attenuation relationships (Abrahanson & Silva, 1997; Sadigh, 1997; Idriss, 1993). Soil type is determined based on available information and classified to be dense to very dense soil. It should be noted that computed PGAs are the values for ground surface (outcropping) and deconvolution may be needed depending on whether soil amplification or de-amplification is considered in structural analysis. The computed Peak Horizontal Acceleration (PHA) versus ARP curve is presented in Figure 5. The computed uniform hazard response spectrum is also shown on Figure 6. The ordinates of the figures are presented in Appendix B.

Since rupture directivity effects cause spatial variations in ground motion amplitude and duration around faults and cause differences between the strike-normal and strike-parallel components of horizontal ground motion amplitude, which also have spatial variation around the fault (Somerville et al., 1997), the computed average uniform hazard response spectra are corrected by multiplying the near-source modification factors. Polarization of horizontal components of ground motion is further corrected to fault-normal and fault-parallel. Dominant scenario for deriving the factors of rupture directivity was evaluated by deaggregating the hazard in terms of magnitude, distance and hazard. Figures showing the deaggragated hazard contributions in terms of magnitudes and distances are presented in Appendix B. The controlling earthquake, estimated based on modal method, is a magnitude 6.7 having a distance around 5.0 km. The near source modification factors and factors of fault-normal to average and fault parallel to average are presented in Appendix B.

Liquefaction

Liquefaction is a phenomenon whereby saturated, granular soils lose their inherent shear strength due to excess pore water pressure build-up such as that generated during repeated cyclic loading from an earthquake. A low relative density and loose consistency of the granular materials, shallow ground-water table, long duration and high acceleration of seismic shaking are some of the factors favorable to cause liquefaction.

Due to the substantial depth of groundwater, it is our opinion that liquefaction potential is considered low.

Scour

Scour is not considered a design issue at this site. The LA River Channel, in which some of the support locations are located, is a concrete lined channel.

Corrosion

Based on the granular soil types, corrosion is not considered a design issue at the site.

Preliminary As-Built Capacities for the Existing Foundations

Based on the soil data available from the previous field investigation, preliminary as-built bearing capacities, and foundation stiffness values for seismic evaluation are calculated and presented in Table 2. Unit P-y curves are also presented in Figure 7.

The capacities, stiffness values and P-y curves are prepared using the following geotechnical parameters:

Friction Angle:	39 degrees
Cohesion:	nil
Unit Weight:	120 pcf
Poisson Ratio:	0.3
Shear Modulus:	3.08 x 10^6 psf

References

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Somerville, P.G., Smith, N.F., Graves, R.W., and Abrahamson, N.A. (1997), "Modification of Empirical Strong Ground Motion Attenuation Relations to Include the Amplitude and Duration Effects of Rupture Directivity," Seismic Research Letters 68, 199-222.

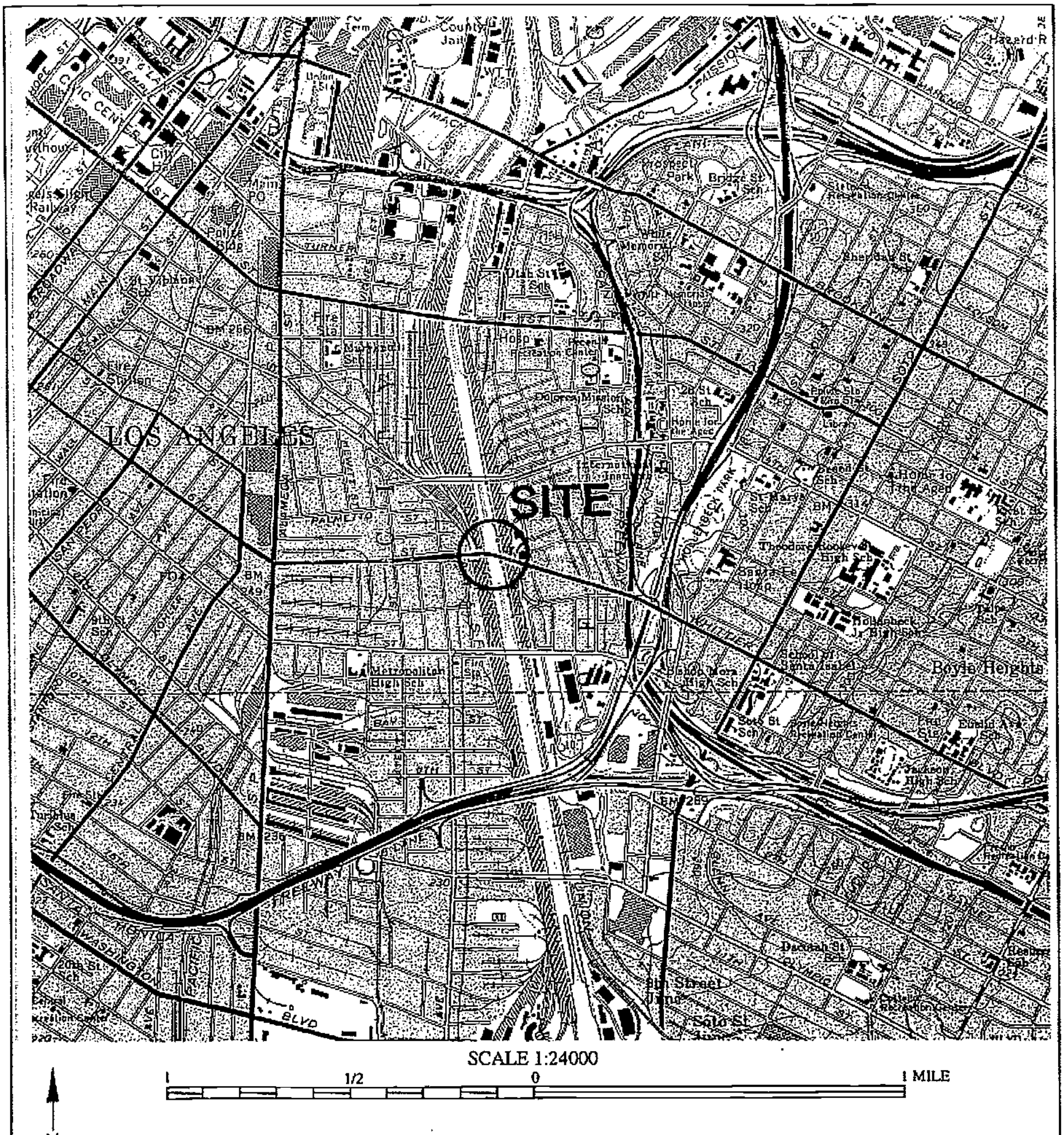
Table 1. ARS Ordinates

Period (sec)	Sa (g)	Period (sec)	Sa (g)
0.010	0.720	0.533	1.766
0.030	0.716	0.650	1.711
0.101	1.524	0.787	1.608
0.109	1.572	0.868	1.553
0.121	1.631	0.998	1.451
0.126	1.650	1.162	1.237
0.136	1.669	1.307	1.087
0.148	1.683	1.411	0.987
0.154	1.703	1.575	0.867
0.177	1.808	1.819	0.726
0.208	1.876	2.068	0.616
0.236	1.893	2.537	0.459
0.271	1.900	2.954	0.365
0.309	1.896	3.203	0.320
0.355	1.884	3.463	0.287
0.386	1.868	4.000	0.223
0.442	1.829		

Table2. 6th Street Bridge - Foundation Bearing Capacities Vertical Vibration Soil Spring Coefficients

Bents	Columns	Column		Usable Footing				Ultimate Bearing Capacity Qult (psf)	Allowable Bearing Capacity Qall (psf)	t-z		
		Width	Length	Width	Length	Area	Depth			Soil Stiffness Coefficient Vertical Vibration (kips/ft)	Lower-Bound Soil Stiffness Coefficient Vertical Vibration Cap Displacement (ft)	Upper-Bound Soil Stiffness Coefficient Vertical Vibration Cap Displacement (ft)
Bent 1	North Column	7.7	7.3	12.3	11.9	147	15	30000	10000	1.13E+05	3.90E-02	1.99E-01
	Middle Column	6.0	5.5	10.6	10.1	107	15	30000	10000	9.66E+04	3.34E-02	1.70E-01
	South Column	7.7	7.3	12.3	11.9	147	15	30000	10000	1.13E+05	3.90E-02	1.99E-01
Bent 2	North Column	7.7	7.3	12.3	11.9	147	15	30000	10000	1.13E+05	3.90E-02	1.99E-01
	Middle Column	6.0	5.5	10.6	10.1	107	15	30000	10000	9.66E+04	3.34E-02	1.70E-01
	South Column	7.7	7.3	12.3	11.9	147	15	30000	10000	1.13E+05	3.90E-02	1.99E-01
Bent 3	North Column	7.7	7.3	12.3	11.9	147	15	30000	10000	1.13E+05	3.90E-02	1.99E-01
	Middle Column	6.0	5.5	10.6	10.1	107	15	30000	10000	9.66E+04	3.34E-02	1.70E-01
	South Column	7.7	7.3	12.3	11.9	147	15	30000	10000	1.13E+05	3.90E-02	1.99E-01
Bent 4	North Column	7.8	7.3	12.4	11.9	149	15	30000	10000	1.14E+05	3.93E-02	2.00E-01
	Middle Column	6.0	5.5	10.6	10.1	107	15	30000	10000	9.66E+04	3.34E-02	1.70E-01
	South Column	7.8	7.3	12.4	11.9	149	15	30000	10000	1.14E+05	3.93E-02	2.00E-01
Bent 5	North Column	7.8	9.1	12.7	14.1	180	15	30000	10000	1.25E+05	4.32E-02	2.21E-01
	Middle Column	8.5	6.5	13.5	11.5	155	15	30000	10000	1.16E+05	4.01E-02	2.05E-01
	South Column	7.8	9.1	12.7	14.1	180	15	30000	10000	1.25E+05	4.32E-02	2.21E-01
Bent 6	North Column	7.8	6.7	13.0	11.9	155	15	30000	10000	1.16E+05	4.00E-02	2.04E-01
	Middle Column	6.5	6.7	11.7	11.9	139	15	30000	10000	1.10E+05	3.79E-02	1.94E-01
	South Column	7.8	6.7	13.0	11.9	155	15	30000	10000	1.16E+05	4.00E-02	2.04E-01
Bent 7	North Column	6.5	6.0	11.1	10.6	118	15	30000	10000	1.01E+05	3.50E-02	1.79E-01
	Middle Column	6.5	6.0	11.1	10.6	118	15	30000	10000	1.01E+05	3.50E-02	1.79E-01
	South Column	6.5	6.0	11.1	10.6	118	15	30000	10000	1.01E+05	3.50E-02	1.79E-01
Bent 8	North Column	6.5	6.0	11.1	10.6	118	16	30000	10000	1.01E+05	3.50E-02	1.79E-01
	Middle Column	6.5	6.0	12.9	12.4	159	16	30000	10000	1.17E+05	4.06E-02	2.07E-01
	South Column	6.5	6.0	12.9	12.4	159	16	30000	10000	1.17E+05	4.06E-02	2.07E-01
Bent 9	North Column	7.9	8.8	12.5	13.4	169	21	30000	10000	1.21E+05	4.18E-02	2.13E-01
	Middle Column	5.5	6.7	13.3	14.5	192	21	30000	10000	1.29E+05	4.46E-02	2.28E-01
	South Column	11.1	8.8	17.7	15.5	274	21	30000	10000	1.54E+05	5.33E-02	2.72E-01
Bent 10	North Column	9.5	8.0	9.5	8.0	76	23	30000	10000	8.12E+04	2.81E-02	1.43E-01
	Middle Column	9.5	6.5	16.4	13.4	221	23	30000	10000	1.38E+05	4.78E-02	2.44E-01
	South Column	9.5	8.0	9.5	8.0	76	23	30000	10000	8.12E+04	2.81E-02	1.43E-01
Bent 11	North Column	7.8	8.1	7.8	8.1	63	27	30000	10000	7.39E+04	2.55E-02	1.30E-01
	Middle Column	10.8	6.5	20.1	15.7	316	27	30000	10000	1.66E+05	5.72E-02	2.92E-01
	South Column	7.8	8.1	7.8	8.1	63	27	30000	10000	7.39E+04	2.55E-02	1.30E-01

FIGURES



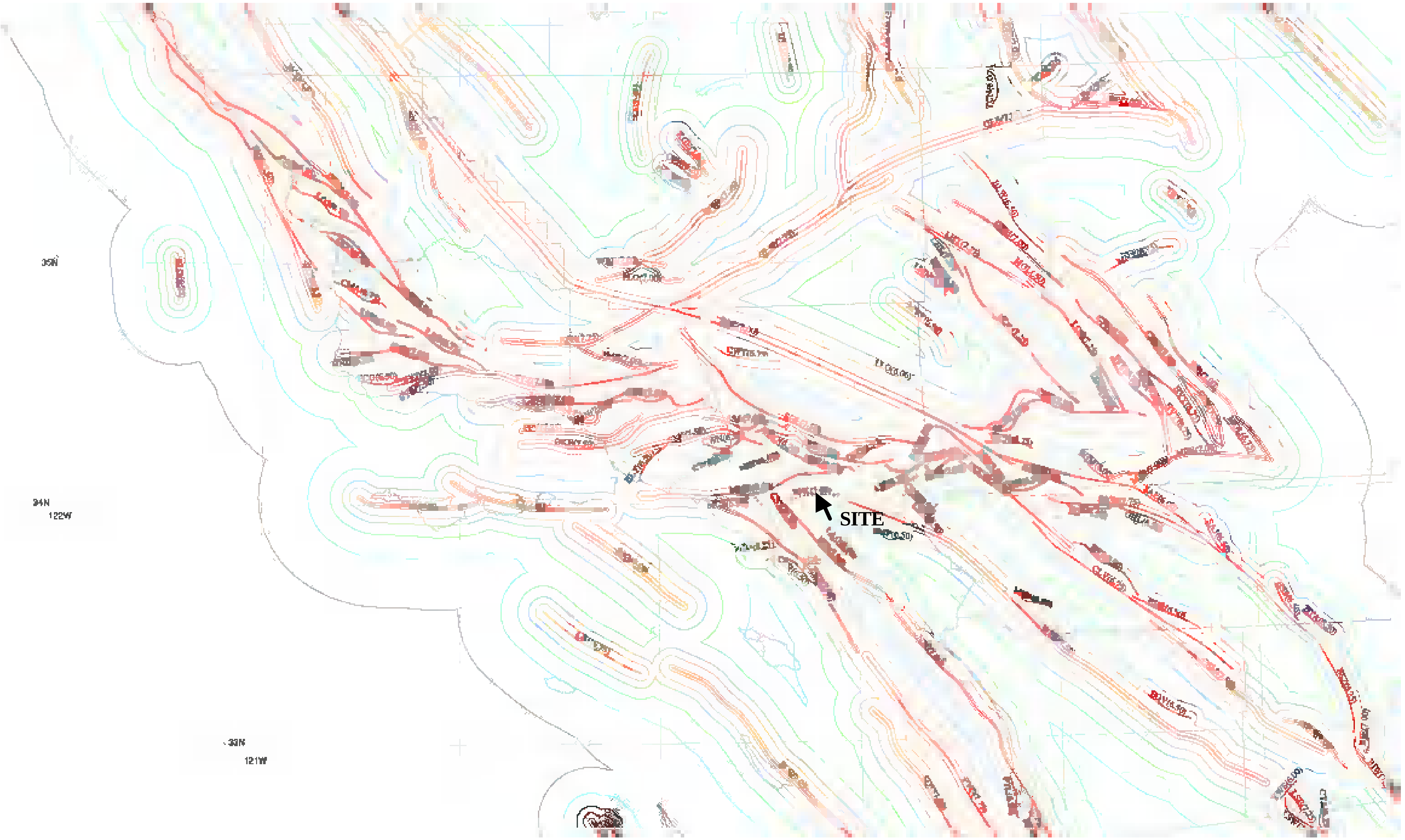
VICINITY MAP

SIXTH STREET BRIDGE
OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA

FOR WEI KOO & ASSOCIATES

REFERENCE: USGS 7.5 Minute Series Topographic Map,
"Los Angeles, California". Quadrangle, 1966, Photorevised 1981.

URS
FIGURE 1



N
W
E
S

LEGEND:

- 0.7g Peak Acceleration Contour
- 0.6g Peak Acceleration Contour
- 0.5g Peak Acceleration Contour
- 0.4g Peak Acceleration Contour
- 0.3g Peak Acceleration Contour
- 0.2g Peak Acceleration Contour
- 0.1g Peak Acceleration Contour
- Special Seismic Source (SSS)
- Faults with Fault Codes (MCE)
- State Highways
- County Boundary
- Latitude & Longitude

SCALE

0 50 100 150 200 250 Kilometers

REGIONAL FAULT MAP

6TH BRIDGE OVER LA RIVER
LOS ANGELES, CALIFORNIA

FOR W. Koo & Associates / City of Los Angeles

URS

REFERENCE : California Seismic Hazard Map, Caltrans, 1996

FIGURE 2

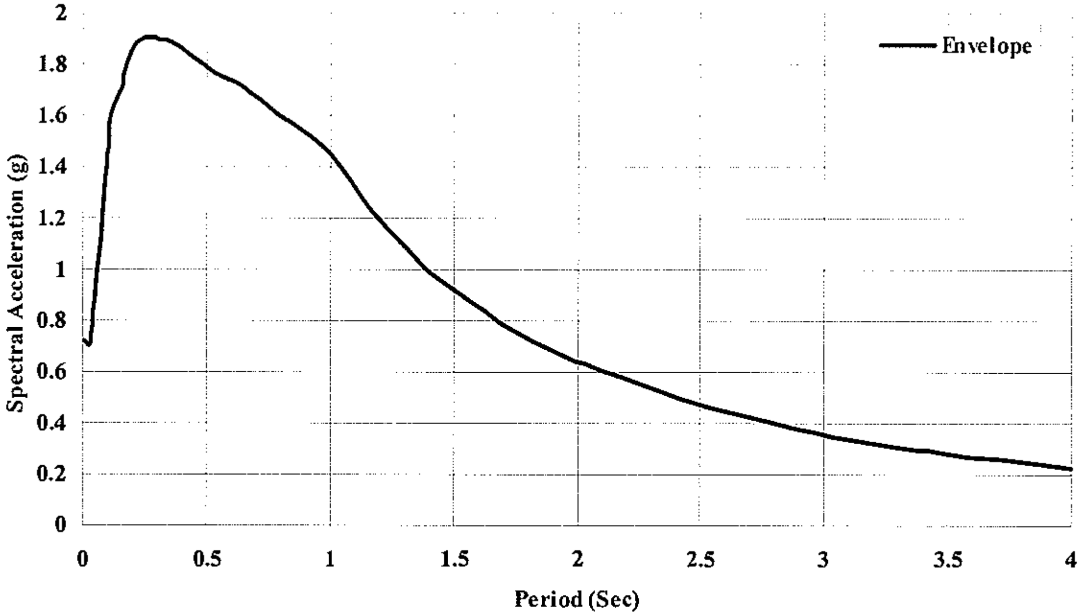


Figure 3. Recommended Enveloped ARS Curve

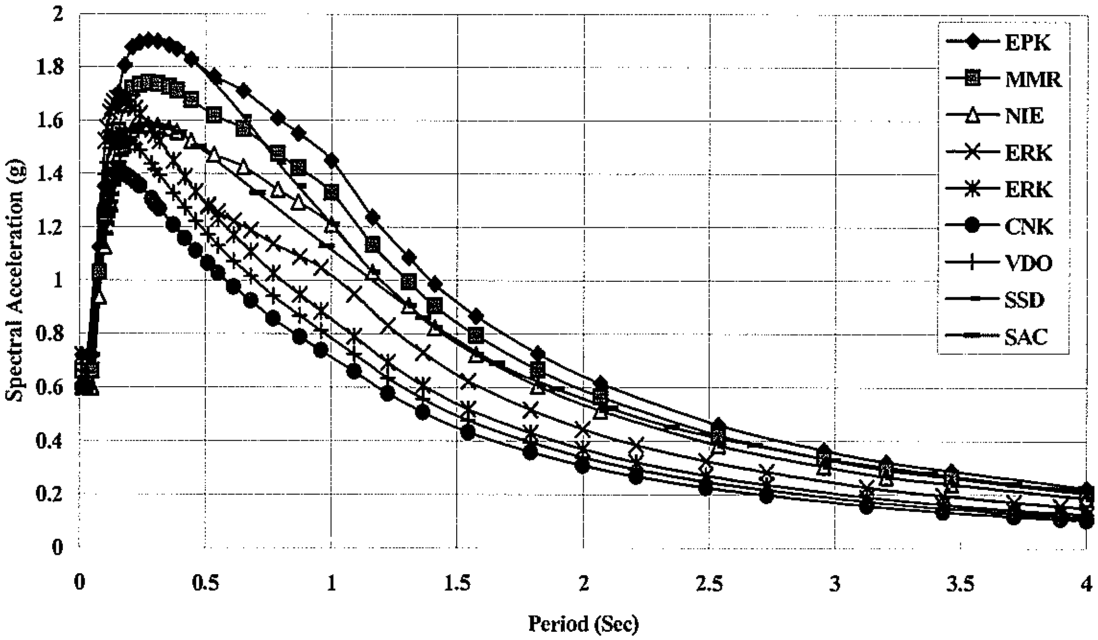


Figure 4. ARS Curves for each fault

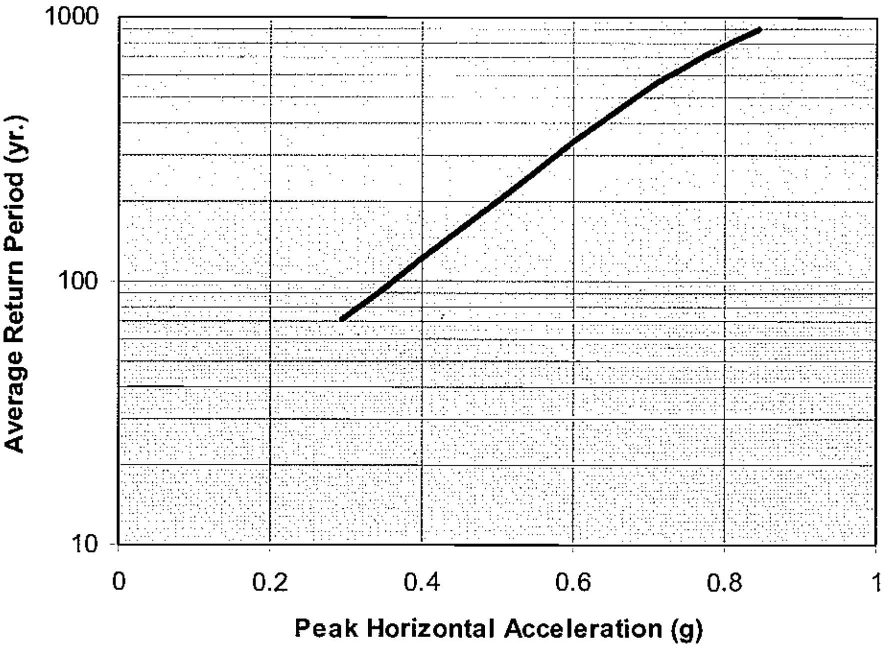


Figure 5. ARP vs. PHA

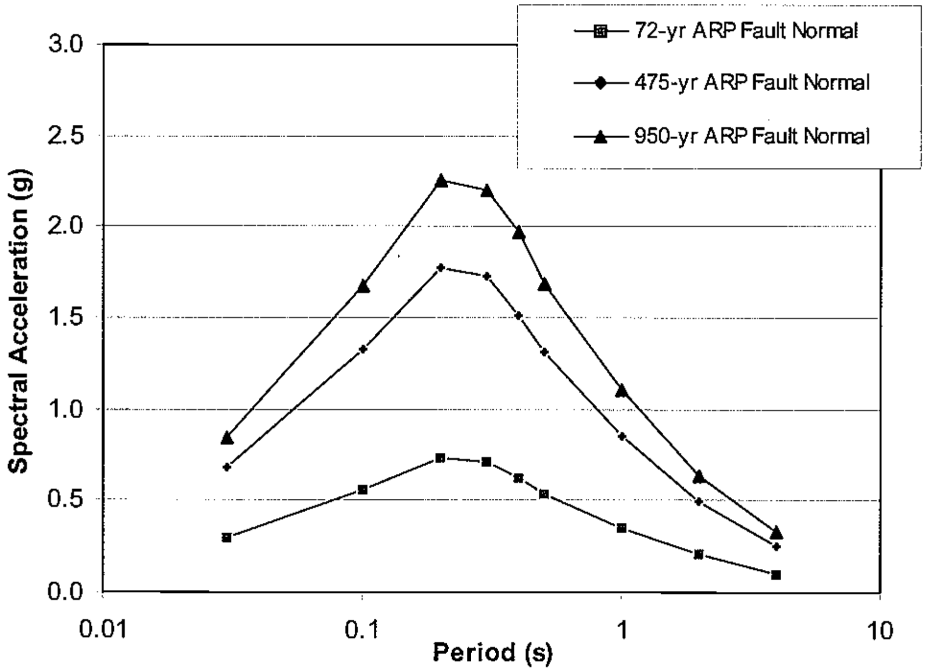


Figure 6. Uniform Hazard Response Spectrum

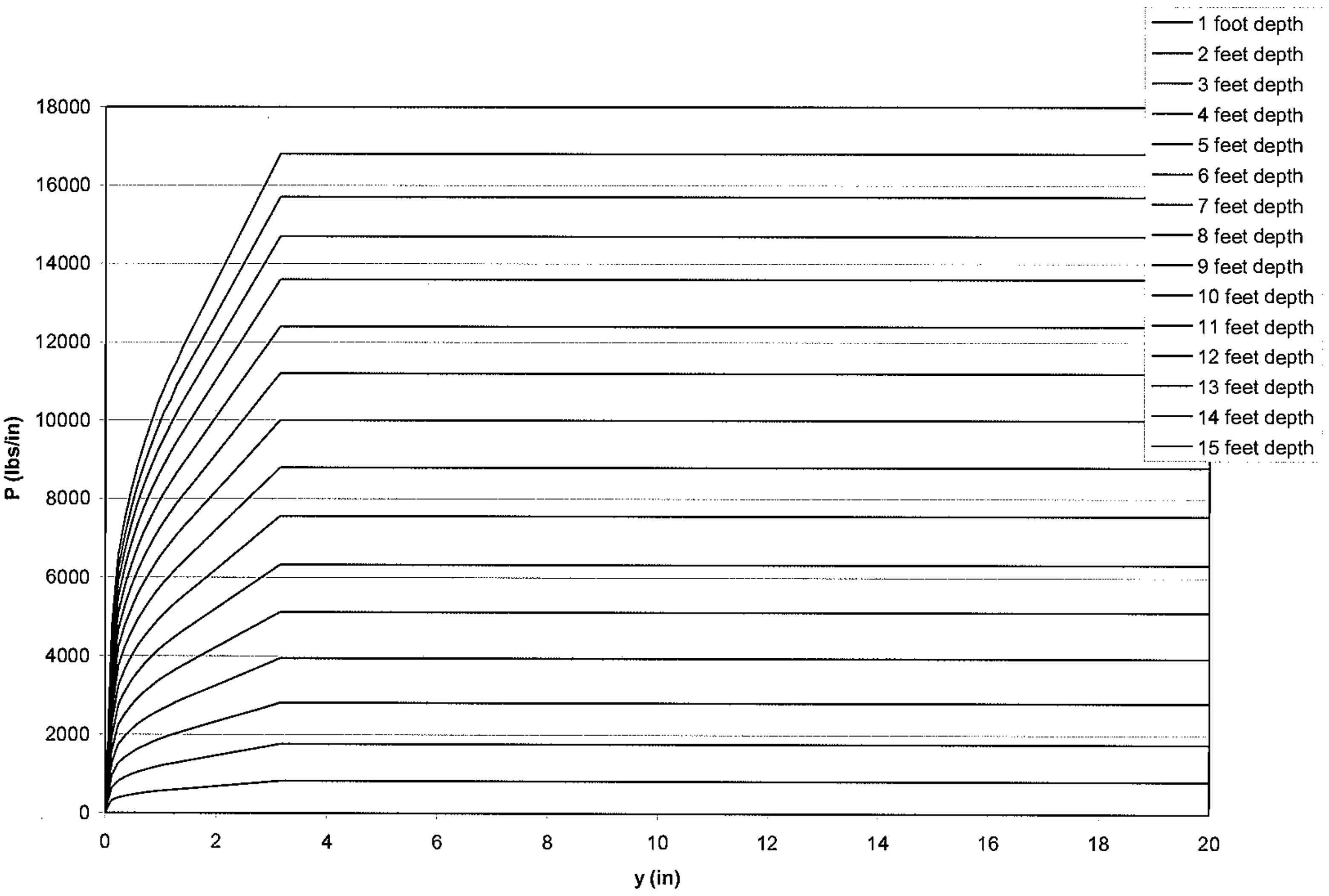
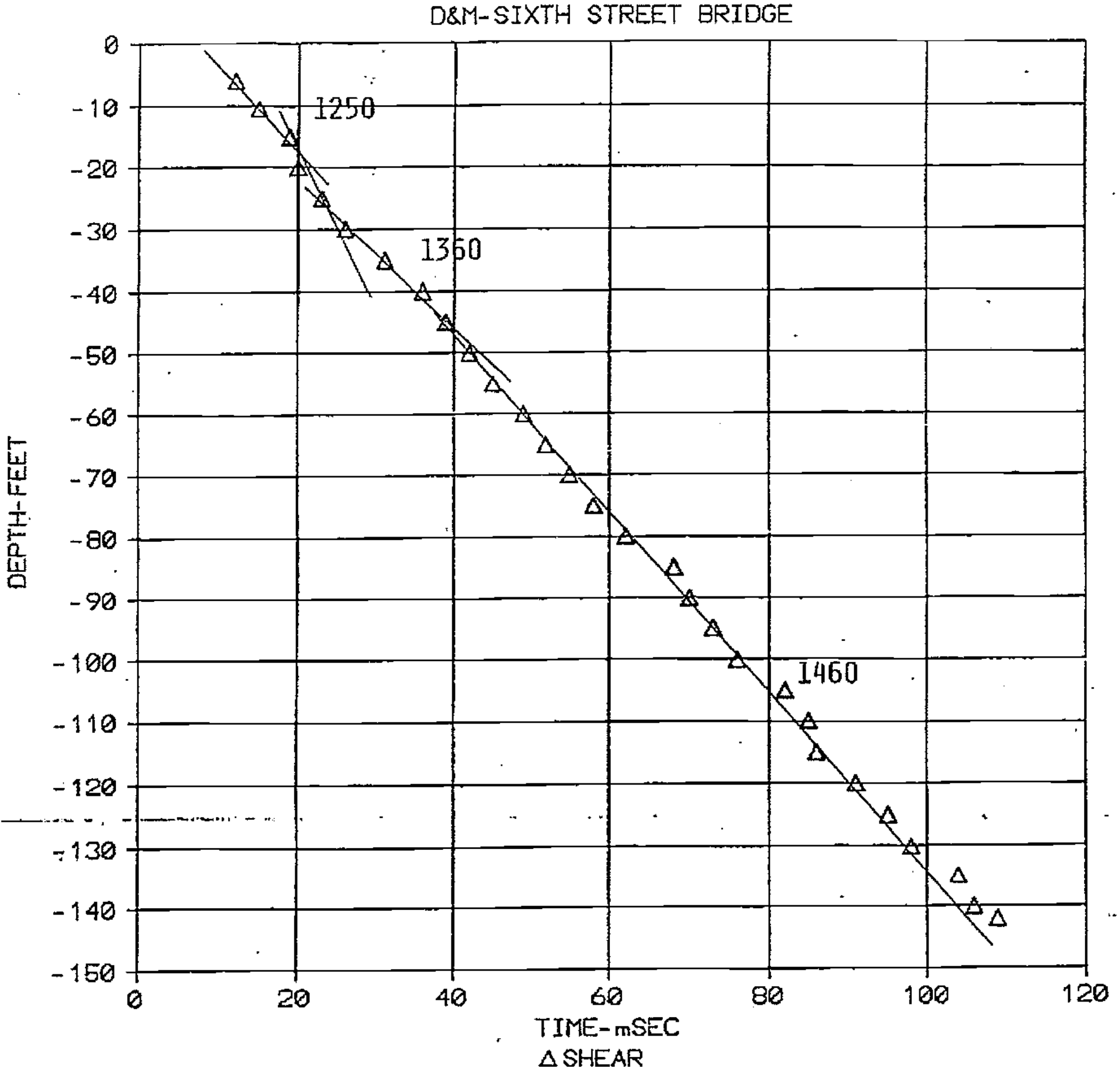


Figure 7. Unit P-y Curves

APPENDIX A
LOGS OF EXPLORATORY BORINGS



RYLAND ASSOCIATES, INC.

BORING B-1

DEPTH IN FEET	LABORATORY TEST DATA									BLOWS/FOOT	SAMPLE TYPE	SYMBOL	DESCRIPTION
	TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS		STRENGTH TEST DATA				MOISTURE CONTENT (%)	DRY DENSITY (PCF)				
		LIQUID LIMIT (%)	PLASTICITY INDEX (%)	TYPE OF TEST	NORMAL OR CONFINING PRESSURE (PSF)	SHEAR STRENGTH (PSF)	DEVIATOR STRESS (PSF)						
0												SM	**FILL:** Brown silty fine to medium SAND (medium dense) Encountered piece of brick fragment at 3 feet
5								15		10	▣	SP SM	**ALLUVIUM:** Brown fine SAND with silt (dense)
10								12		33	▣	GP	Brown sandy fine to coarse GRAVEL (dense)
15								11		52	▣		
20										54	▣	SP SM	Brown fine to medium SAND with silt (dense)
25								19		49	▣	GP	Brown fine to coarse GRAVEL with fine to medium sand (very dense)
30								14		75	▣		
35								11		50/6"	▣		Grades to include occassional cobble
40								13		50/3"	▣		
45								10		50/6"	▣	SM	Brown silty fine SAND with occassional sandy silt lenses.(very dense)
50	-200 (50)							23		66	▣		
55								20		50/3"	▣		Grades gray and greenish-gray Grades with trace fine gravel
60								17		50/2"	▣		
65													

LOG OF BORING

6ST STREET VIADUCT OVER THE LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR CITY OF LOS ANGELES GEOTECHNICAL SERVICES

Dames & Moore
FIGURE 2

BORING B-1 Cont'd

DEPTH IN FEET	LABORATORY TEST DATA									BLOWS/FOOT	SAMPLE TYPE	SYMBOL	DESCRIPTION
	TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS		STRENGTH TEST DATA				MOISTURE CONTENT (%)	DRY DENSITY (PCF)				
		LIQUID LIMIT (%)	PLASTICITY INDEX (%)	TYPE OF TEST	NORMAL OR CONFINING PRESSURE (PSF)	SHEAR STRENGTH (PSF)	DEVIATOR STRESS (PSF)						
65								13		50/6"	◩	SP	Gray fine to medium SAND with some gravel (very dense)
70	-200 (5)							10		50/6"	◩		Grades with coarse sand and with more gravel
												GP	Gray fine to coarse GRAVEL with fine to coarse sand (very dense)
75								12		50/6"	◩		
80								10		50/6"	◩		
85	-200 (96)							30		38	◩	ML	FERNANDO FORMATION - Repetto Member: Dark gray clayey SILT with trace fine sand and shell fragments (hard)
90								30		48	◩		
95	-200 (95)									53	◩		
100								32		79	◩		Grades with more fine sand
105	-200 (96)							29		48	◩		Grades to include more clay
110								30		48	◩		
115								29		43	◩		
120	-200 (93)							27		46	◩		
125	CON							28		38	■		
130													

LOG OF BORING

6ST STREET VIADUCT OVER THE LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR CITY OF LOS ANGELES GEOTECHNICAL SERVICES

Dames & Moore
FIGURE 2 (Cont.)

BORING B-1 Cont'd

DEPTH IN FEET	LABORATORY TEST DATA									BLOWS/FOOT	SAMPLE TYPE	SYMBOL	DESCRIPTION
	TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS		STRENGTH TEST DATA				MOISTURE CONTENT (%)	DRY DENSITY (PCF)				
		LIQUID LIMIT (%)	PLASTICITY INDEX (%)	TYPE OF TEST	NORMAL OR CONFINING PRESSURE (PSF)	SHEAR STRENGTH (PSF)	DEVIATOR STRESS (PSF)						
130								26		65	■	ML	cont'd **FERNANDO FORMATION:** Dark gray clayey SILT with trace fine sand and shell fragments (hard)
135	-200 (92)							28		55	◪		
140								28		60	◪		
145								28		52	◪		
150	-200 (93)							29		56	◪		
155													
160													
165													
170													
175													
180													
185													
190													
195													

NOTES:

1. Boring initiated on 11-21-96 and completed on 11-22-96.
2. Boring drilled to 150 feet using a rotary wash drilling rig.
3. Water level was not measured in the boring.
4. Standard Penetration Tests performed at the depths indicated.
5. Relatively undisturbed samples obtained using a Dames & Moore Type U sampler driven using a 300-pound hammer and dropping 30 inches.
6. 2-inch diameter Schedule 40 PVC pipe installed to 150 feet in the boring.
7. Annular space between pipe and soil backfilled with a lean mix of cement-bentonite grout upon completion.

LOG OF BORING

6ST STREET VIADUCT OVER THE LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR CITY OF LOS ANGELES GEOTECHNICAL SERVICES

Dames & Moore
FIGURE 2 (Cont.)

APPENDIX B
PROBABILISTIC SEISMIC HAZARD ANALYSIS

Table 1 Fault-Normal Spectral Ordinates for 72-Yr ARP
6th Street Bridge

Critical Damping = 5%

Period (s)	Frequency (Hz)	Sa (g)	Sv (cm/s)	Sv (in/s)	Sd (cm)	Sd (in)
0.03	33.33	0.294	1.377	0.542	0.007	0.003
0.10	10.00	0.563	8.790	3.459	0.140	0.055
0.20	5.00	0.735	22.951	9.032	0.731	0.287
0.30	3.33	0.714	33.443	13.160	1.597	0.628
0.40	2.50	0.629	39.283	15.458	2.501	0.984
0.50	2.00	0.541	42.233	16.620	3.361	1.323
1.00	1.00	0.347	47.464	18.678	7.554	2.973
2.00	0.50	0.205	46.839	18.432	14.909	5.867
4.00	0.25	0.099	34.349	13.517	21.867	8.605

PGA = 0.294 g

LEGEND

Sa = Pseudo-Absolute Spectral Acceleration
Sv = Pseudo-Relative Spectral Velocity
Sd = Relative Spectral Displacement

Table 2 Fault-Normal Spectral Ordinates for 475-Yr ARP
6th Street Bridge

Critical Damping = 5%

Period (s)	Frequency (Hz)	Sa (g)	Sv (cm/s)	Sv (in/s)	Sd (cm)	Sd (in)
0.03	33.33	0.681	3.190	1.255	0.015	0.006
0.10	10.00	1.328	20.734	8.159	0.330	0.130
0.20	5.00	1.777	55.489	21.836	1.766	0.695
0.30	3.33	1.727	80.891	31.832	3.862	1.520
0.40	2.50	1.516	94.678	37.257	6.027	2.372
0.50	2.00	1.319	102.968	40.520	8.194	3.224
1.00	1.00	0.858	117.411	46.203	18.686	7.353
2.00	0.50	0.493	112.414	44.237	35.783	14.081
4.00	0.25	0.256	88.682	34.898	56.457	22.217

PGA = 0.681 g

LEGEND

Sa = Pseudo-Absolute Spectral Acceleration
Sv = Pseudo-Relative Spectral Velocity
Sd = Relative Spectral Displacement

Table 3 Fault-Normal Spectral Ordinates for 950-Yr ARP

6th Street Bridge

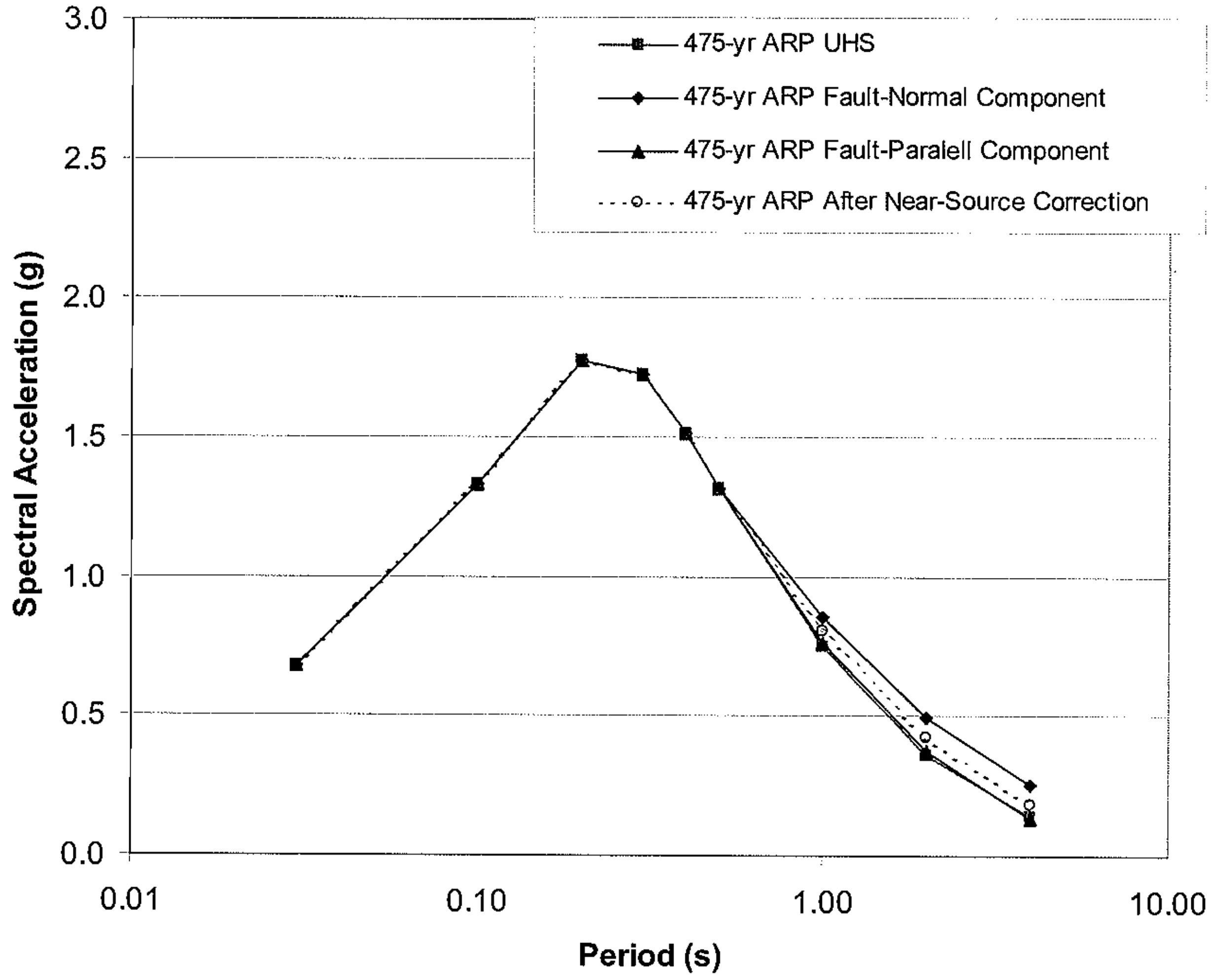
Critical Damping = 5%

Period (s)	Frequency (Hz)	Sa (g)	Sv (cm/s)	Sv (in/s)	Sd (cm)	Sd (in)
0.03	33.33	0.845	3.958	1.558	0.019	0.007
0.10	10.00	1.674	26.136	10.285	0.416	0.164
0.20	5.00	2.254	70.384	27.697	2.240	0.882
0.30	3.33	2.200	103.046	40.551	4.920	1.936
0.40	2.50	1.968	122.906	48.366	7.824	3.079
0.50	2.00	1.691	132.009	51.948	10.505	4.134
1.00	1.00	1.103	150.823	59.351	24.004	9.446
2.00	0.50	0.633	144.265	56.771	45.921	18.071
4.00	0.25	0.326	113.039	44.483	71.963	28.319

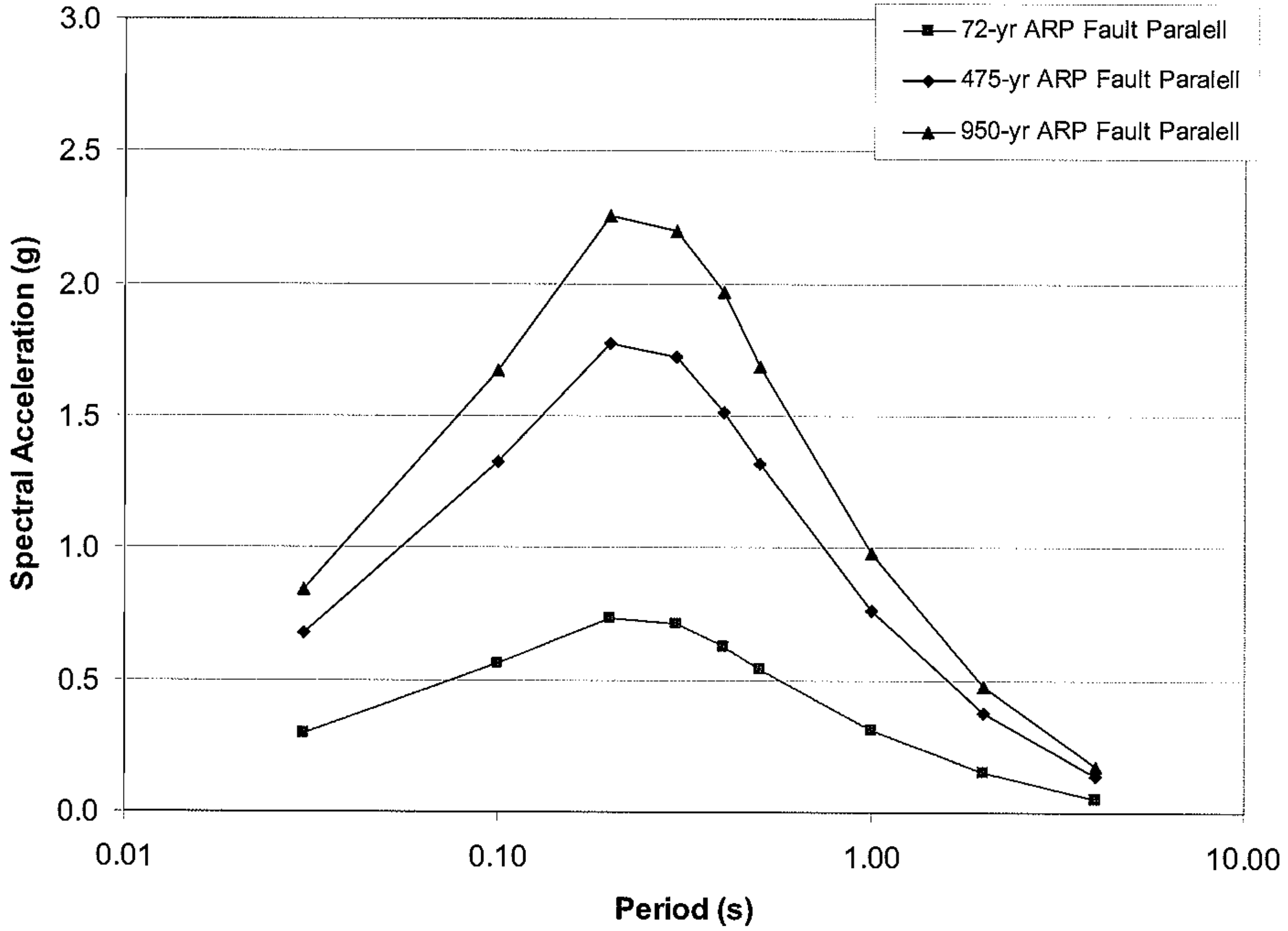
PGA = 0.845 g

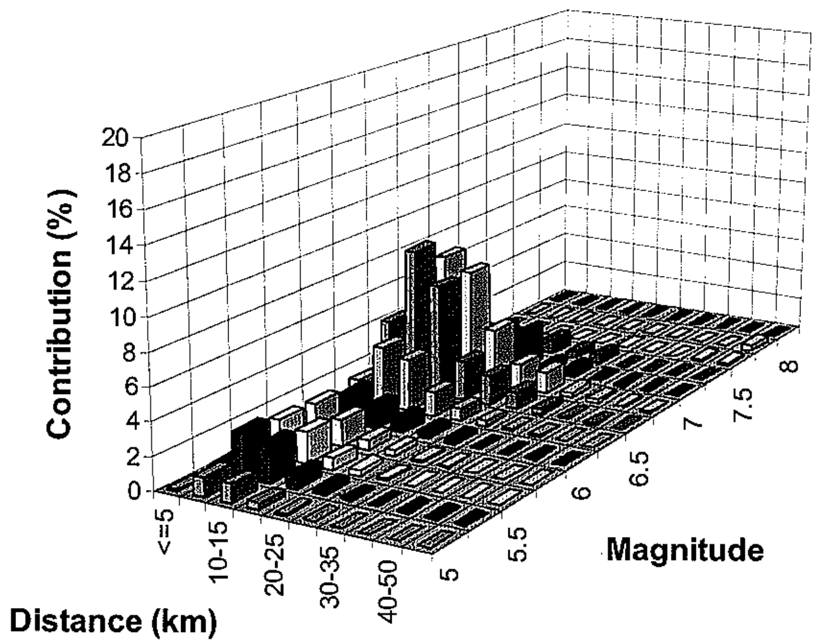
LEGEND

Sa = Pseudo-Absolute Spectral Acceleration
Sv = Pseudo-Relative Spectral Velocity
Sd = Relative Spectral Displacement



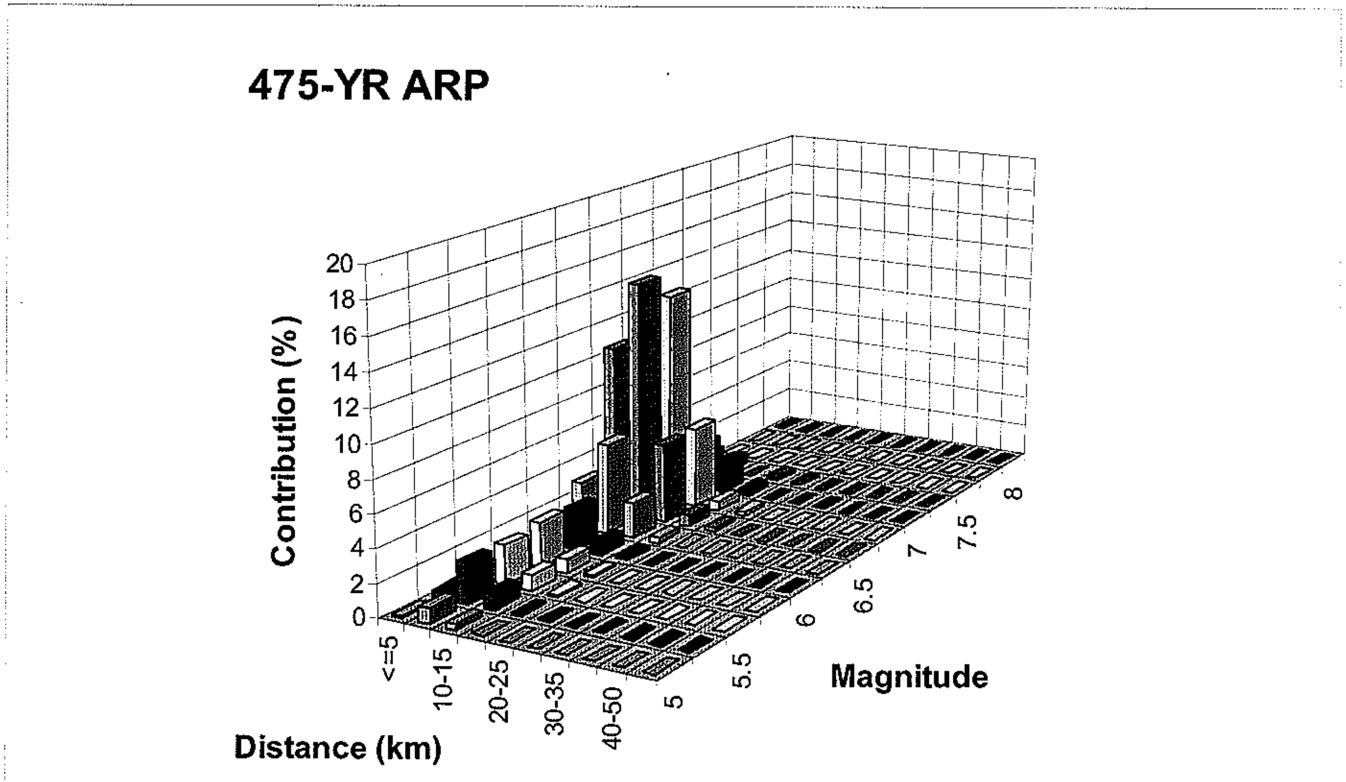
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Spectral Acceleration (g)
0.01
0.10
1.00
10.00
Period (s)
475-yr ARP UHS
475-yr ARP Fault-Normal Component
475-yr ARP Fault-Paralell Component
475-yr ARP After Near-Source Correction


72-yr ARP Fault Paralell
475-yr ARP Fault Paralell
950-yr ARP Fault Paralell
Spectral Acceleration (g)
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.01
0.10
1.00
10.00
Period (s)

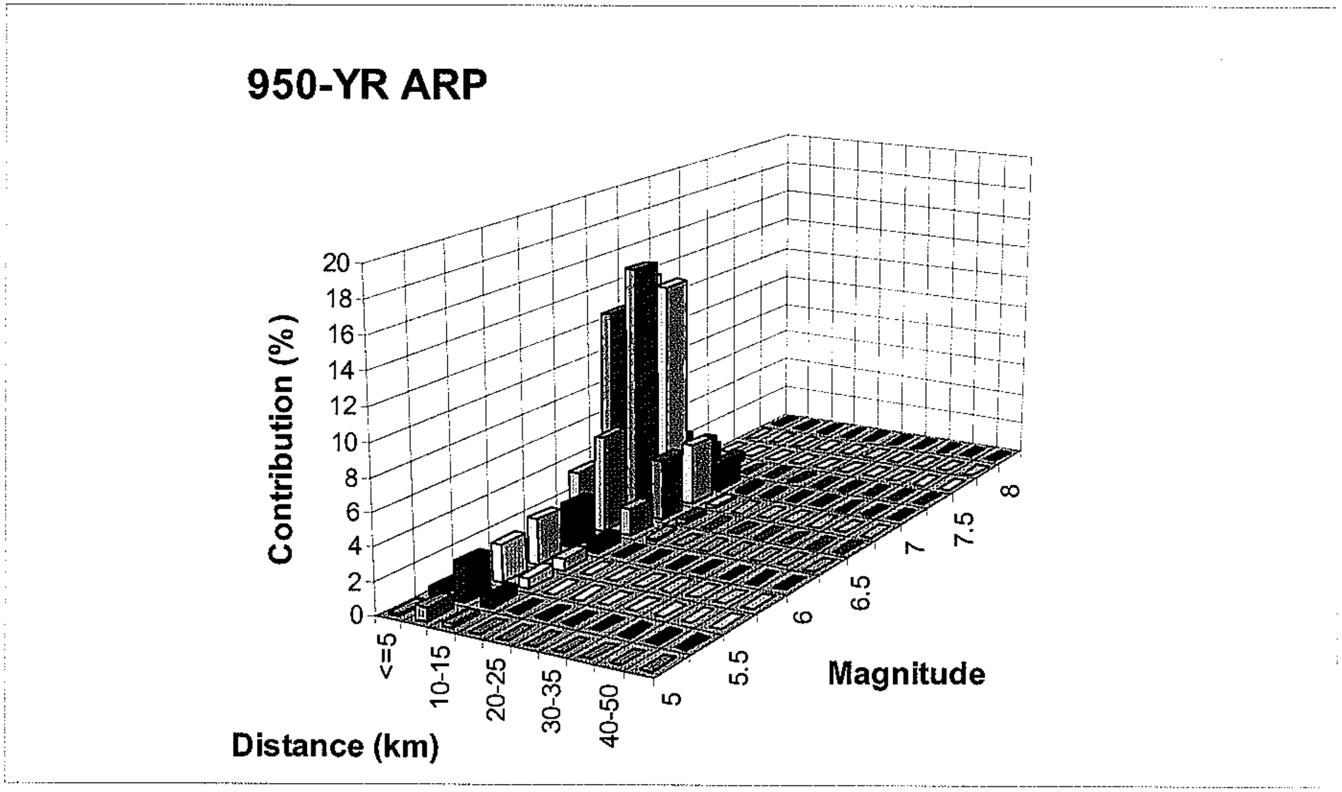


72-YR ARP

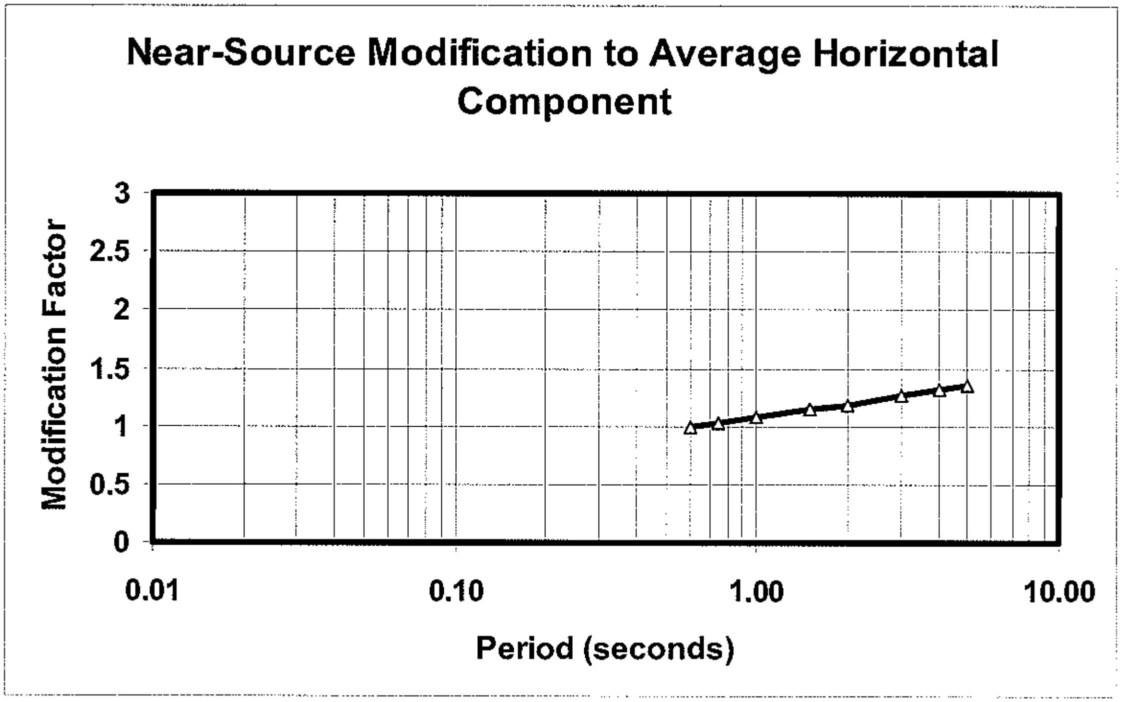
Deaggregation for Determining Controlling Earthquake

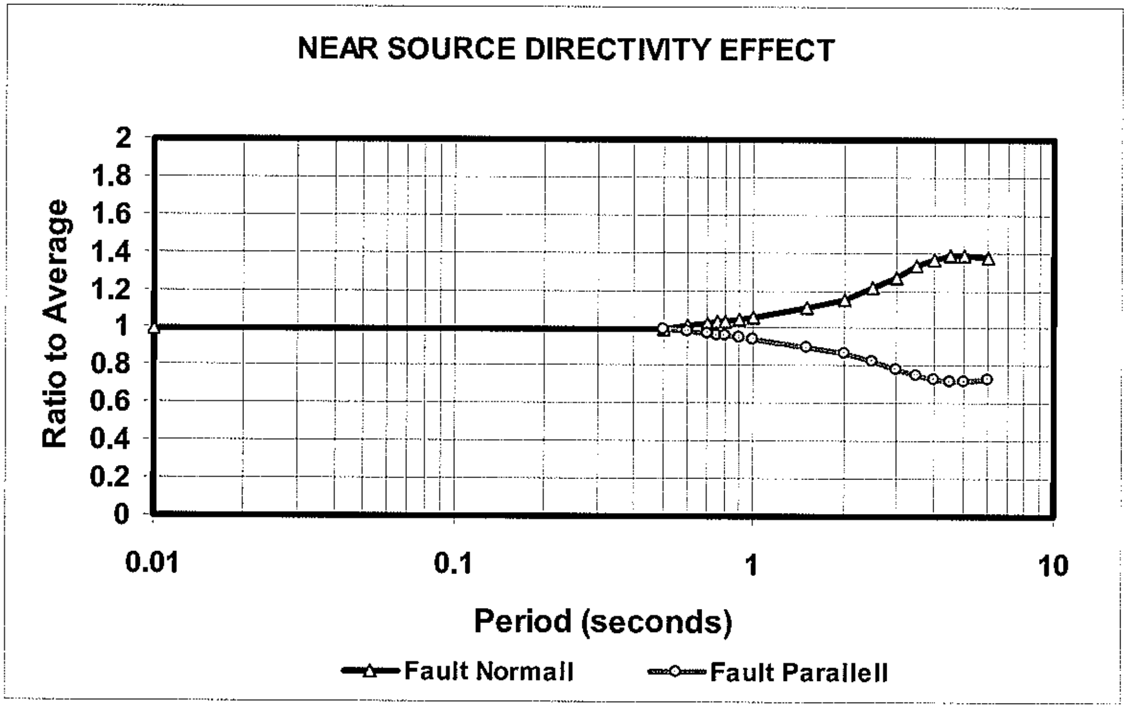


Deaggregation for Determining Controlling Earthquake



Deaggregation for Determining Controlling Earthquake


Near-Source Modification to Average Horizontal Component
Modification Factor
3
2.5
2
1.5
1
0.5
0
0.01
0.10
1.00
10.00
Period (seconds)


NEAR SOURCE DIRECTIVITY EFFECT
Ratio to Average
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.01
0.1
1
10
Period (seconds)
Fault Normall
Fault Parallell

Earth Mechanics, Inc.
Geotechnical & Earthquake Engineering

July 17, 2003

W. Koo & Associates
600 The City Parkway West, Suite 310
Orange, CA 92868

Attention: Mr. Dan Weddell

Subject: Earthquake Time Histories for Sixth Street Viaduct
City of Los Angeles

Dear Mr. Weddell:

In accordance with our agreement on Task Order No.02-EMI-A, Earth Mechanics, Inc. (EMI) has generated multiple support ground motion time histories for the subject project.

Project Understanding

It is our understanding that WKA plans to conduct time history analyses, in addition to conventional response spectrum (ARS) design analysis for the Sixth Street Viaduct over L.A. River. The bridge structure has significantly different column heights and dissimilar hunched concrete and arched bridge deck that warrants multiple support time history analysis for certain feature of the bridge response.

Design ARS Curve

Other consultant developed the design ARS curve adopted for the response spectrum analysis of the structure. WKA provided EMI this design ARS curve to be used for generation of time histories for the horizontal components; we assumed the vertical design spectrum to be 2/3 of the horizontal spectrum. From our reviews, it appears that this ARS curve resembles Caltrans standard ARS curve with the following characteristics:

Magnitude 7
Soil Type C
PBA of 0.7 g
Increase in spectral acceleration for close proximity to fault

The design ARS curve is included at the end of this memo for reference. We feel that the adopted ARS curve seems reasonable considering the fact that no soil boring was conducted to further refine the ground motion parameters.

17660 Newhope Street, Suite E, Fountain Valley, California 92708 Tel: (714) 751-3826 Fax: (714) 751-3928

Reference Time Histories

To generate ground motion time histories, we selected the recorded motions from the 1994 Northridge Earthquake. The selected ground motions have three components: two horizontals and one vertical motion.

We then generated three component time histories by modifying the selected time histories so that their spectra are similar to the design spectra. This is done by gradually modifying the motion through an iterative process so that the response spectrum of the modified time history is compatible with the target spectrum. Various methods have been developed to perform the spectrum matching. A commonly used method adjusts the Fourier amplitude spectrum based on the ratio of the target response spectrum to the time history response spectrum while keeping the Fourier phase of the reference history fixed. An alternative approach for spectral matching adjusts the time history in the time domain by adding wavelets to the reference time history. In this study, the frequency domain method was used.

Plots of the three reference time histories are provided with this memo.

Multiple Support Motions

After the reference time histories are obtained, multiple support ground motions were generated for all the bents. The multiple support motions were derived considering the wave passage effect only; i.e., the incoherency effect due to scattering of seismic wave is not included.

An average propagation speed of 2.5 km per second was used to determine the arrival time of earthquake wave at each group of the bents. The following bents were grouped together in order to develop the multiple support time histories:

Bents 1 2 3 4 Arrival Time: 0.036 sec

Bents 5 6 7 8 Arrival Time: 0.072 sec

Bents 9 10 11 Arrival Time: 0.108 sec

East, Middle, West Arrival Time: 0.144 sec (Main Channel)

Bents 12 13 14 15 Arrival Time: 0.180 sec

Bents 16 17 18 19 Arrival Time: 0.216 sec

Bents 20 21 22 23 Arrival Time: 0.252 sec

Bents 24 25 26 27 Arrival Time: 0.288 sec

Bents 28 29 30 31 Arrival Time: 0.324 sec

Bents 32 33 34 35 Arrival Time: 0.36 sec

Bents 36 37 38 39 Arrival Time: 0.396 sec

Bents 40 41 42 43 Arrival Time: 0.432 sec

Acceleration and displacement time histories of the multiple support motions in an electronic format.

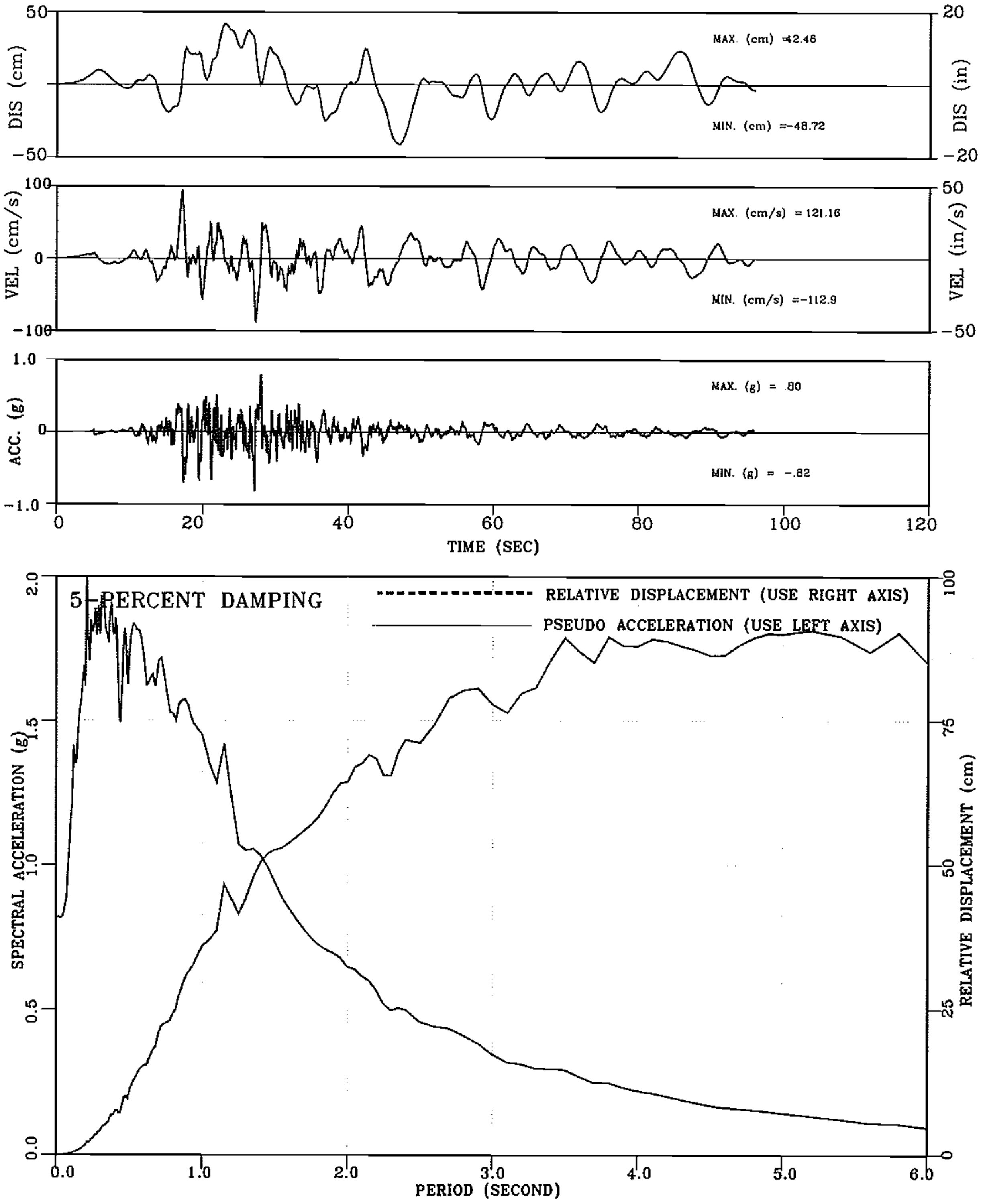
If you have any question regarding this project, please do not hesitate to contact me.

Sincerely,
EARTH MECHANICS, INC.

Hubert Law

Hubert Law
Project Manager

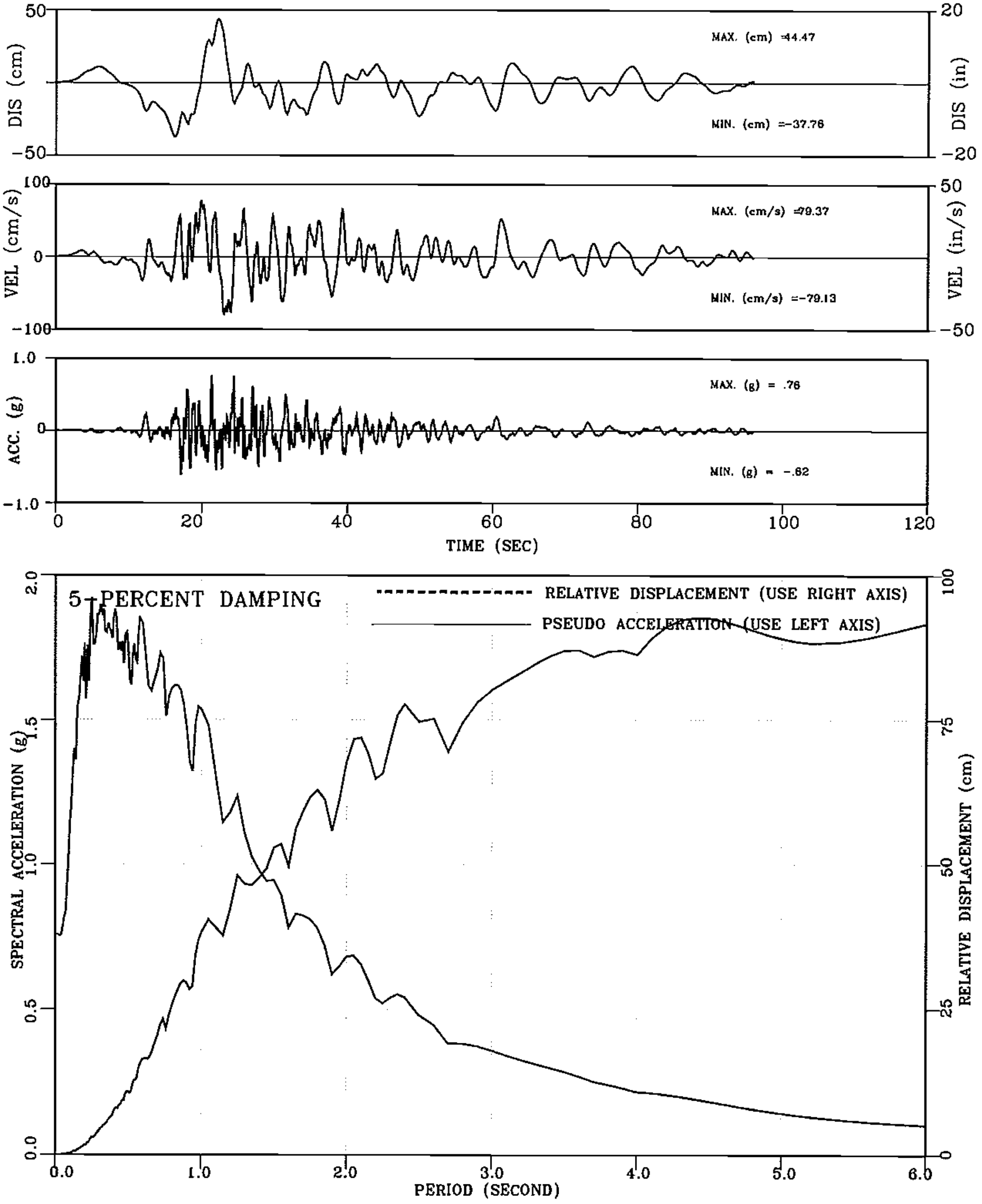
Attachment:
- Three Reference Ground Motions
- Memo from WKA of the Design ARS Curve



Earth Mechanics, Inc.

Geotechnical and Earthquake Engineering

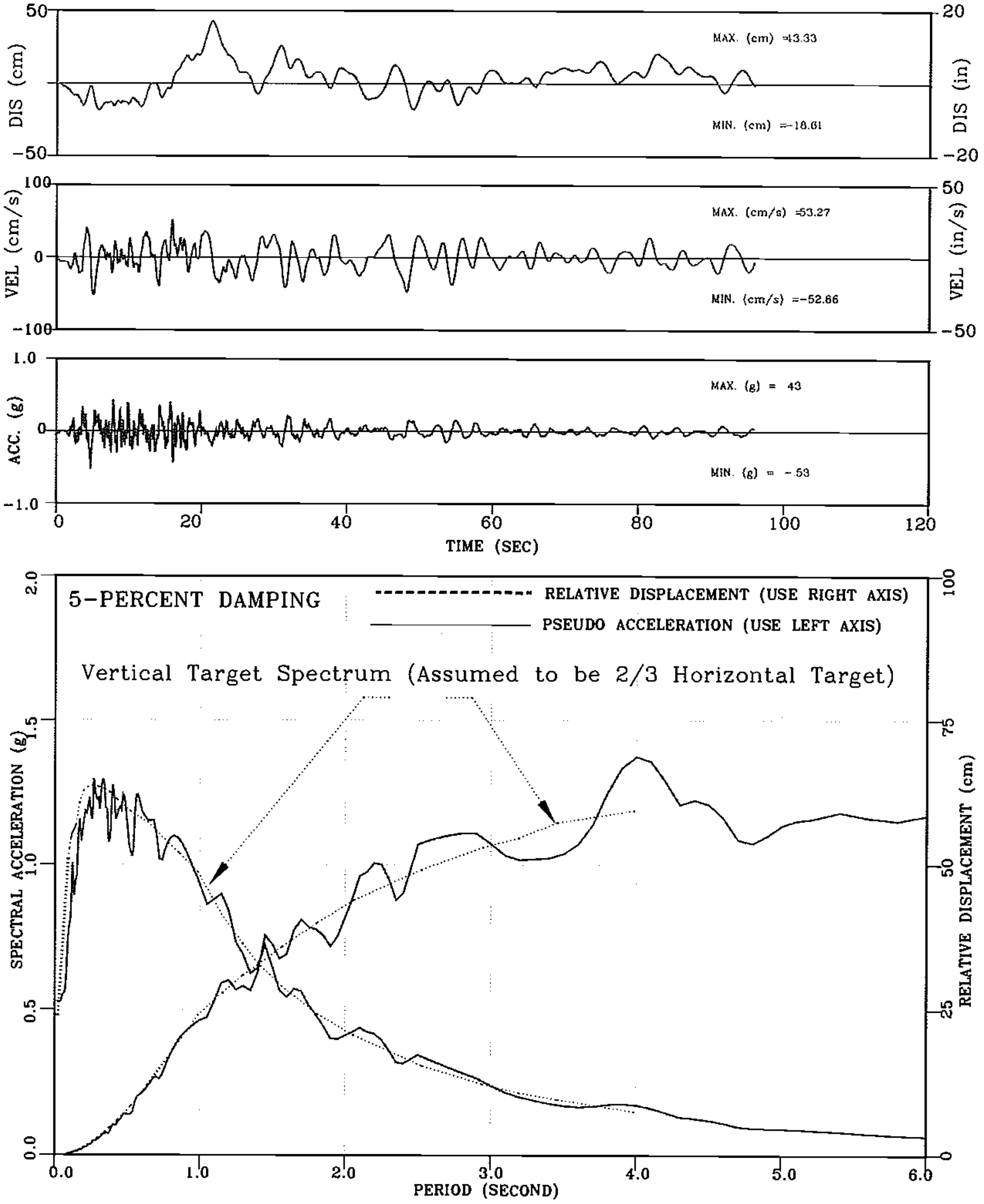
SPECTRUM COMPATIBLE HORIZONTAL MOTIONS
FOR 6TH STREET BRIDGE
COMPONENT NO. 1



Earth Mechanics, Inc.

Geotechnical and Earthquake Engineering

SPECTRUM COMPATIBLE HORIZONTAL MOTIONS
FOR 6TH STREET BRIDGE
COMPONENT NO. 2



Earth Mechanics, Inc.

Geotechnical and Earthquake Engineering

SPECTRUM COMPATIBLE HORIZONTAL MOTIONS FOR 6TH STREET BRIDGE
Vertical Component Motion

W. Koo & Associates
Structural Engineers
600 The City Parkway West, •Suite 310•Orange CA 92868
Tel (714) 456-0280 Fax (714) 456-0295

Send To Hewbert Law
Date 02/04/2003
Company EMI
Fax No. (714) -751-3928
Sent By Xiaoyun Wu
Project No.
Reference 6th Street Bridge

No. of Pages (including transmittal sheet) 2

This original transmission will ☐ will not ☒ be sent by Mail. If you do not receive all pages transmitted herein, please call the fax operator at (714)456-0280

Hewbert,

Attached is the standard ARS curve we currently used for analysis of the 6th street bridge structure. It is closed to the curve for 475-year ARP. Should you have any question or need any other information please let me know.

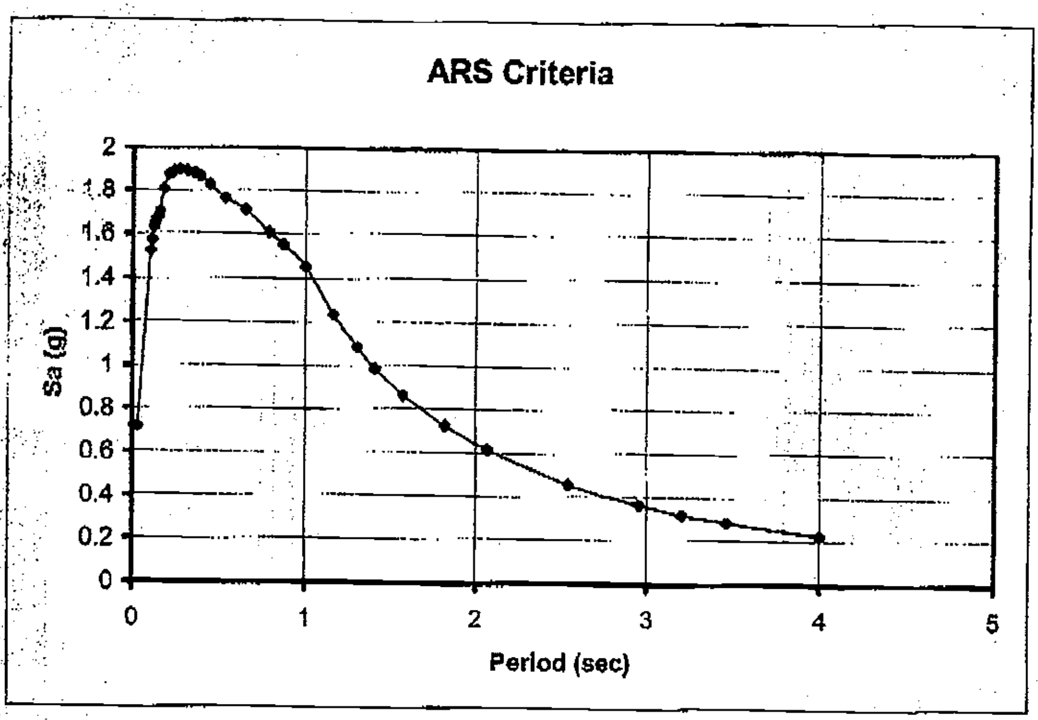
Thank you.

Xiaoyun Wu

CC: Wei Koo/ WKA
CC: Dan/ WKA

FILE

FAX TRANSMITTAL



Period (sec)	Sa (g)	Period (sec)	Sa (g)
0.01	0.72	0.533	1.766
0.03	0.716	0.65	1.711
0.101	1.524	0.787	1.608
0.109	1.572	0.868	1.553
0.121	1.631	0.998	1.451
0.126	1.65	1.162	1.237
0.136	1.669	1.307	1.087
0.148	1.683	1.411	0.987
0.154	1.703	1.575	0.867
0.177	1.808	1.819	0.726
0.208	1.876	2.068	0.616
0.236	1.893	2.537	0.459
0.271	1.9	2.954	0.365
0.309	1.896	3.203	0.32
0.355	1.884	3.463	0.287
0.386	1.868	4	0.223
0.442	1.829		

Sixth Street Retrofit Project: WKA

East, Center and West Piers: t-z curve in compression (use 75 % of load resistance for tension) Pile: 15" Diameter Pile Concrete Pile 4/25/2003

Layer	Soil Type	Top El. ft	Bottom El. ft	Eff Unit Wt. kcf	Eff. Stress at Bottom ksf	c ksf	phi degree	Unit Skin Friction ksf	t_{max} k/ft	z_c ft	t1 kip/ft	z1 ft	t2 kip/ft	z2 ft	t3 kip/ft	z3 ft	t4 kip/ft	z4 ft
1	sand	237	204	0.120	3.96	0.00	35	0.98										
2	sand	204	202	0.120	4.20	0.00	38	2.21	8.70	0.013	4.61	0.001	6.52	0.003	7.91	0.006	8.70	0.013
3	sand	202	200	0.120	4.44	0.00	38	2.35	9.21	0.013	4.88	0.001	6.91	0.003	8.38	0.006	9.21	0.013
4	sand	200	198	0.120	4.68	0.00	38	2.48	9.72	0.013	5.15	0.001	7.29	0.003	8.85	0.006	9.72	0.013
5	sand	198	196	0.120	4.92	0.00	38	2.61	10.23	0.013	5.42	0.001	7.67	0.003	9.31	0.006	10.23	0.013
6	sand	196	194	0.120	5.16	0.00	38	2.74	10.74	0.013	5.69	0.001	8.06	0.003	9.78	0.006	10.74	0.013
7	sand	194	192	0.120	5.40	0.00	38	2.87	11.26	0.013	5.97	0.001	8.44	0.003	10.24	0.006	11.26	0.013
8	sand	192	190	0.120	5.64	0.00	38	3.00	11.77	0.013	6.24	0.001	8.83	0.003	10.71	0.006	11.77	0.013

Skin Friction Compression kips
17
18
19
20
21
23
24

143

q-u curve in compression (use 0 for tension)

Nr	Ultimate Bearing Cap ksf	q_{max} kip	u_c ft	q1 kip	u1 ft	q2 kip	u2 ft	q3 kip	u3 ft	q4 kip	u4 ft
80	451.2	154.29	0.063	69.43	0.006	97.20	0.016	121.89	0.031	154.29	0.063

End Bearing Compression kips
154

Note:

No ground water was encountered in some near by borings

Plan shows three different pile types including tappered piles. We used the following for design:

for skin friction, 15" diameter was used

for end bearing, 8" butt diamter was used

Typical pile data: pile top at El. 204', pile tip at El. 190'

Pile driving data indicates safe bearing values ranging from 50 to 100 tons using ENR equation (I.e., ultimate pile capacity ranges from 200 to 400 tons)

The provided t-z curves are for compression only. For tension, use 75% of the load resistance

Total Compression (ki
298

Total Tension (kips
107

APPENDIX I

GEOTECHNICAL & ENVIRONMENTAL INVESTIGATION (EXCAVATIONS FOR EVALUATION OF SUBSTRUCTURE)

URS

Report
Geotechnical & Environmental Investigation Excavations for Evaluation of Substructure Sixth Street Bridge Over Los Angeles River Los Angeles, California

Prepared for
Wei Koo & Associates
600 City Parkway Way West, Suite 310
Orange, CA 92868

URS Job No. 57-00255002.01
November 28, 2001

URS

911 Wilshire Boulevard, Suite 800
Los Angeles, CA 90017
213-996-2200 Fax: 213-996-2374

URS

November 28, 2001

Wei Koo & Associates
600 City Parkway Way West, Suite 310
Orange, California 92868.

Attention: Mr. Wei Koo

Re: Report
Geotechnical and Environmental Investigation
Excavations for Evaluation of Substructure
Sixth Street Bridge Over Los Angeles River
Los Angeles, California
For Wei Koo & Associates/City of Los Angeles
URS Job No. 57-00255002.01

Dear Mr. Koo

INTRODUCTION

This report presents our results and recommendations based on our limited geotechnical and environmental investigation in support of the evaluation of the substructure at the Sixth Street Bridge over Los Angeles River. The area investigated was below the 6th Street Bridge, between Mission Road and Anderson Street, Los Angeles, California.

The purpose of the investigation was to evaluate the soil conditions and stability of the proposed temporary 20-foot deep excavations near bents (piers 19, 21, 23, and 26). These locations were determined by your office and indicated to us during our field meeting on November 19, 2001. Presence of contamination was also investigated. It is our understanding that the excavations are temporary and will be backfilled within 24 hours. The excavations are required in order to expose the concrete pier walls for inspection and concrete sampling by coring. The location of the site is shown in the Vicinity Map, Figure 1.

Wei Koo & Associates provided this office with as-built plans dated February 1931 prepared for the City of Los Angeles.

This report includes our geotechnical and environmental (contamination) recommendations for the temporary excavations to facilitate concrete coring and inspection at various levels of the bridge piers. Conclusions and recommendations presented in this report are based on subsurface conditions encountered at the locations of our explorations and our experience on similar projects performed on other facilities. Soil and groundwater data obtained during our field explorations were observed and interpreted at our boring locations only. Conditions may vary between boring locations, and should not be extrapolated to other areas without our prior review.

FIELD INVESTIGATION

The subsurface field investigation was initiated on November 19, 2001 and completed on November 20, 2001 under the supervision of a URS representative. Geotechnical and environmental exploration included drilling and sampling four (4) borings to a depth of approximately 31 feet below the existing ground surface (bgs). The locations of the borings are shown on the Plot Plan, Figure 2. The borings were drilled using a truck-mounted drilling rig, equipped with 8-inch diameter hollow-stem augers. The borings were logged and sampled by a URS field-representative who maintained a detailed log of the soils encountered in the borings and visually classified the soils in accordance with the Unified Soil Classification System (USCS). A description of the USCS and Key to the Log of Boring is presented in Figure 3. Graphic logs of the borings showing the contacts between the different soil types encountered and other pertinent information were prepared using the gINT software and are presented in Figures 4 through 7.

Relatively undisturbed soils samples were collected at regular intervals from all boring locations. The undisturbed samples were obtained using a Dames & Moore Type-U Sampler. Geotechnical soil samples were obtained at 5-foot intervals and environmental samples were obtained at the surface and at depths of 10 feet, 20 feet, and 30 feet below ground surface. The sampler was driven 18 inches into the subsurface soils using a 140-pound hammer falling 30 inches. The number of blows (blow counts) to drive the samplers in to the subsurface soils were recorded at 6-inch intervals and the blow counts required to drive the samplers the final 12 inches is recorded on the boring logs.

The cuttings from the drilling operations were placed in drums and left at the site pending removal. The borings were then backfilled using slurry-cement grout.

A portable Photo Ionization Detector (PID) was used to screen for potential presence of hydrocarbons in the borings during the drilling activities.

GEOTECHNICAL LABORATORY TESTING

Geotechnical soil samples obtained from the borings were carefully sealed and packaged in the field to reduce moisture loss and disturbance. The samples were subsequently delivered to our Los Angeles laboratory where they were further examined and classified. Selected representative samples were tested to evaluate moisture content, in-situ dry density, fines content, and shear strength of the soils. All tests discussed below were performed in accordance with the latest American Society of Testing and Materials (ASTM) guidelines.

Moisture Content and Density Tests (ASTM D 2216 and D 2937)

The results of these tests are used to compute existing soil overburden pressure and correlation with existing data. The moisture content and dry density tests were performed in accordance with ASTM Test Methods D 2216 and 2937, respectively. The results of these tests are presented on the Logs of Borings (Figures 4 through 7).

Percent Passing #200 Sieve (Fines Content) (ASTM D 1140)

Percent passing no. 200 sieve tests were performed on selected soils samples obtained from the borings. These tests were performed to aid in classification of the soils and were performed in accordance with ASTM Test Method D 1140. The results of the tests are presented on the Logs of Borings (Figures 4 through 7).

Sieve Analysis (ASTM D 422)
Sieve analysis was performed on a selected soil sample obtained from the borings. The test was performed to aid in classification of the soils and was performed in accordance with ASTM Test Method D 422. The results of the test are presented on the Logs of Borings and in Figure 8.

Direct Shear Test (ASTM D 3080)
Consolidated-drained (saturated) direct shear tests were performed on selected undisturbed samples to evaluate shear strength parameters of the site soils. The direct shear tests were performed in accordance with ASTM Test Method D 3080. The results of these tests are presented in Figures 9 through 13.

ENVIRONMENTAL ANALYTICAL TESTING

Soil samples from the surface and at depths of 10, 20, and 30 feet bgs were selected from the borings for analytical testing. The samples were carefully sealed and packaged in the field and were subsequently delivered to Calscience Environmental Laboratories, Inc. of Garden Grove, California where they were tested for Total Petroleum Hydrocarbons (TPH) by EPA Method 8015M and Title 22 metals by EPA Method 6010/7000 Series. The results of the analytical tests from Calscience Environmental Laboratories are discussed below and presented in the Appendix.

Total Petroleum Hydrocarbons (TPH)

TPH was analyzed for the carbon chain range of C7 through C44. TPH as gasoline is typically detected within the carbon chain range of C7 through C12; TPH as diesel fuel is typically detected within the carbon chain range of C10 through C22; TPH as waste oil and other heavy products such as lubricating oil is typically detected within the carbon chain range of C18 through C44.

A summary of the TPH analytical data is provided in Table 1. TPH (C7 – C44) was detected in all four borings, B-1 through B-4, at depths generally above 10 feet bgs. TPH (C7 – C44) was detected at concentrations ranging from 25 milligrams per kilogram (mg/kg) (boring B-1 at 10 feet bgs) to 420 mg/kg (boring B-4 at the surface). More specifically, TPH was only detected within the carbon chain range of C17 through C44, which is generally associated with waste oils and other heavier fuels. There was a notable lack of low molecular weight, volatile petroleum hydrocarbons. The measured TPH concentrations decreased significantly with increasing depth.

Metals

Metals detected in soil samples collected from borings B-1 through B-4 included arsenic, barium, beryllium, total chromium, cobalt, copper, lead, mercury, molybdenum, nickel, selenium, thallium, vanadium, and zinc. Refer to Table 2 for a summary of Title 22 Metals analytical results. Concentrations of metals detected were all below their respective Preliminary Remediation Goals for industrial soils, as established by the United States Environmental Protection Agency (USEPA, 2000).

The reported concentrations of lead were elevated in the surface samples collected at all four boring locations. This finding is consistent with the lead impacts that are typically associated with atmospheric deposition in urbanized areas.

The reported concentration of copper in the surface sample from boring B-4 was particularly elevated (1,010 mg/kg compared with a range of 5.37 to 78.1 mg/kg in the balance of the soil samples). This result indicates a localized, surface-derived copper impact in the vicinity of boring B-4.

The reported concentration of barium in the 30-foot bgs sample from boring B-1 was particularly elevated (480 mg/kg compared with a range of 38.6 to 109 mg/kg in the balance of the soil samples). There is no apparent reason for elevated barium in the deep sample from boring B-1.

SITE CONDITIONS

SUBSURFACE SOIL CONDITIONS

Based on the borings and laboratory testing performed during the current study, the site of the proposed excavations is generally underlain by coarse-grained soils consisting predominantly of sands, silty sands, and sands with silt to the maximum depth explored except in boring B-3 where very stiff sandy silt was encountered at 27 feet below the existing ground surface (bgs). The consistency of the coarse-grained soils in the upper 20 feet varies from loose to medium dense with pockets of dense soils and below 20 feet varies from medium dense to very dense.

ENVIRONMENTAL CONDITIONS

During drilling and sampling, a portable Photo Ionization Detector (PID) was used to screen for potential presence of hydrocarbons in the soil samples. The PID readings ranged between 0 and less than 2 parts per million (ppm). The PID readings are shown on the logs of borings.

Based on the analytical results, surface soils (0 to 10 feet bgs) are impacted with low levels of TPH and lead. There is also shallow copper impacts at the location of boring B-4 and apparent Barium impact in deeper soil at the location of boring B-1.

CURRENT GROUNDWATER LEVELS

Groundwater was not encountered in the borings to the maximum depth drilled during the current investigation (approximately 31 feet below ground surface). The Los Angeles County Department of Regional Planning (1990) reported the site to be outside mapped shallow and perched groundwater zones.

DISCUSSION

In general the geotechnical soil conditions are uniform throughout the four boring locations. As such in selecting two locations for excavations, all four locations are feasible provided the recommendations outlined below are followed.

Based on the results of the environmental (analytical) testing, all four borings encountered TPH and Lead in the upper 10 feet. In addition, soils in Borings B-1 and B-4 were impacted with Barium and Copper. As such, excavations at locations of Borings B-2 and B-3 (Bents 21 and 23) would be preferable.

RECOMMENDATIONS

TEMPORARY EXCAVATIONS

All excavations shall comply with the current California and Federal OSHA requirements, as applicable. All cuts greater than 5 feet in depth should be sloped and/or shored. For excavations, the subsurface soils shall be classified as Type C in accordance with OSHA regulations.

Excavations during construction should be carried out in such a manner that failure and excessive ground movement do not occur. Where spacing permits, open excavations may be considered for the temporary excavations.

In general, unsupported slopes, for temporary construction excavation should be limited to a gradient of no steeper than 1:1 (horizontal to vertical) and a height of 10 feet. Alternatively, temporary excavations to a depth of 20 feet may be achieved by either.

- Limiting the gradient to a slope of no steeper than 1.5:1 (horizontal to vertical); or
- Using a series of 10-foot high 1:1 (horizontal to vertical) excavations with a 5-foot wide (measured horizontally) terrace separating the sloped portions.

A geotechnical engineer should observe excavations in the field; field conditions may dictate shorter and flatter excavations.

Surcharge loads from vehicles, and stockpiled materials should be kept away from top of temporary excavations a distance equal to at least one half of the excavation depth. Surface drainage should be controlled along the top of the temporary excavations to prevent excessive wetting and erosion of excavation faces. Sloughing of surface soils within the excavation area may be expected at the site, therefore workers should be adequately protected from such sloughing.

Where there is insufficient space for open excavations, shoring should be used to support the excavation.

TEMPORARY SHORING

All shoring systems must be designed and constructed in accordance with applicable requirements and regulations of the City, County, State and Federal agencies.

Shoring systems should consist of soldier piles and a lagging retention system; either, internally, or cantilevered. Typical soldier piles consist of steel H-sections installed in pre-drilled holes. If this method is used, the holes shall be backfilled below the planned bottom of the excavation with structural concrete and with sand/cement slurry above. Center to center horizontal spacing between soldier piles shall be limited to a maximum of 8 feet. Where applicable, treated timber lagging shall be installed as the excavation descends to prevent running sand conditions in the predominantly dry and granular soils. Solid sheeting shall be used for containing potentially loose sandy cohesionless soils.

Any space between the lagging and the face of the excavation shall be filled with lean concrete with provisions for weep-holes to reduce the potential for buildup of hydrostatic pressure. Runoff shall be prevented from entering shored excavations. As excavations are backfilled, the timber lagging shall be removed. Timber lagging shall not be left buried.

Voids created by the removal of shoring elements shall be backfilled with slurry (1.5 sacks of cement per cubic yard of slurry).

The shoring system shall be designed to resist lateral earth pressures plus any additional horizontal pressures imposed by foundations of adjacent structures or other live loads such as vehicular load. Cantilevered shoring shall be designed to resist the pressure exerted by an equivalent fluid weighing 30 pounds per cubic feet (pcf). Thirty percent of any uniform areal surcharges placed adjacent to the shoring will act as a uniform horizontal pressure against the shoring. The above pressures do not include any hydrostatic pressures; it is assumed that drainage will be provided by weep-holes or cracks in the lagging. The above values assume a uniform and level grade behind the shoring system.

Soldier piles must extend below the excavation bottom to provide lateral resistance by passive soil pressure. Allowable passive pressures shall be taken as equivalent to the pressure exerted by a fluid weighing 300 pcf. The passive earth pressure shall be limited to 3,000 psf. To account for three-dimensional effects, the lateral pressure may be assumed to act on an area twice the pile width as long as soldier piles are spaced at least three pile diameters apart. The geotechnical engineer should review the design of any shoring required and be present during all soldier piles installation and testing activities. Depending upon the location of shoring in relation to existing adjacent structures, recommendations for design and construction of other special shoring systems will be provided on a case-by-case basis after the design constraints have been discussed with the structural engineer.

FILLS AND BACKFILLS

It is our understanding that the excavation will be backfilled with soil. If the backfilled soil is to be relied upon for lateral or vertical resistance, then the fill should be compacted as specified below.

Due to the levels of Total Petroleum Hydrocarbons and Lead in the soils in the upper 10 feet, the soils from this interval should not be used as backfill. The soils below 10 feet are suitable for use as backfill.

It might become necessary to import fill materials for other reasons. All imported fill and backfill materials, except as required specifically, should be predominantly granular in nature (no more than 35 percent mostly non-plastic fine materials passing the No. 200 sieve, with a Expansion Index of less than 20) and free of organic and inorganic debris.

All fills placed for support of structural loads (structural fills) should be brought to within ±3% of the optimum moisture content in-place, and compacted to at least 95 percent of the maximum dry density per ASTM D-1557. In general, fills should be placed in loose lifts not exceeding 8 inches in loose thickness, brought to within ±3% of the optimum moisture content in-place, and compacted using mechanical compaction equipment.

All fill and backfill materials shall be observed and tested by the geotechnical engineer prior to their use in order to evaluate their suitability. All imported soil should be inspected and approved at the borrow site by the geotechnical engineer and tested prior to its import.

ENVIRONMENTAL RECOMMENDATIONS

Based on the analytical results, shallow soils (less than 10 feet bgs) are impacted with relatively low levels of hydrocarbons and lead. There is also a shallow copper impact at the location of boring B-4 and an apparent barium impact in deeper soil at the location of boring B-1. The TPH and metals impacted soil at the site will need to be managed appropriately.

The shallow impacted soils that are excavated from the site could be segregated from deeper material, for off-site treatment/disposal at a licensed facility. Stockpiled soil should be covered or located beneath the bridge structure to avoid the generation of impacted stormwater.

It is understood that the City of Los Angeles intends to undertake two excavations in the vicinity of the soil borings. It is therefore recommended that the excavations should avoid locations B-1 and B-4 so that the barium and copper impacted soil at these respective locations may be left undisturbed.

The impacted soil at the site represents a potential hazard that should be accounted for during planning for the excavation and foundation inspection work. The impacts are relatively minor, and may be managed by health and safety controls that include the use of appropriate (Level D) personal protective equipment.

GEOTECHNICAL CONSTRUCTION SERVICES

Geotechnical construction services are an important and necessary continuation of this investigation, and it is important that a qualified geotechnical engineer be retained to perform such services.

Observation and testing should be performed by the project geotechnical engineer during the excavation to ensure that material encountered during the excavation are similar to the material encountered during the investigation and that any structural backfill is properly compacted.

LIMITATIONS

URS warrants that our services are performed within the limits prescribed by our clients, with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either express or implied, is included or intended in this report.

- o0o -

It has been a pleasure to assist you with this project. Should you have any questions regarding this report, please contact us. The following are attached and complete this report:

Table 1	Summary of Soil Analytical Data – TPH by EPA Method 8015M
Table 2	Summary of Soil Analytical Data – Title 22 Metals by EPA Methods 6010/7000 Series
Figure 1	Vicinity Map
Figure 2	Plot Plan
Figure 3	Key to Log of Boring
Figures 4 through 7	Logs of Borings - Borings B-1 through B-4
Figure 8	Particle Size Distribution Curves
Figures 9 through 13	Direct Shear Test Results
Appendix	Analytical Report by Cal science Environmental Laboratories, Inc.

Respectfully submitted,

URS

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Senior Project Engineer

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No. 50350
Exp. 6/30/05

Murray Wallis Ph.D.
Senior Environmental Manager

Reviewed By

Garry Lay, P.E. G.E.
Principal Engineer

REFERENCES

Los Angeles County Department of Regional Planning (1990), Shallow and Perched Groundwater, dated December.

Naval Facilities Engineering Command (NAVFAC)(1986), Foundation and Earth Structures, Design Manual 7.02.

United States Environmental Protection Agency, Region 9, Preliminary Remediation Goals, November 22, 2000: www.epa.gov/region09/waste/sfund/prg/

1997 Uniform Building Code, Volume 2

1999-City of Los Angeles Building Code, Volume 1 & 2 (1999-LABC).

TABLE 1
SUMMARY OF SOIL ANALYTICAL DATA
Total Petroleum Hydrocarbons (TPH) by EPA Method 8015M
Sixth Street Bridge Over Los Angeles River
Los Angeles, California

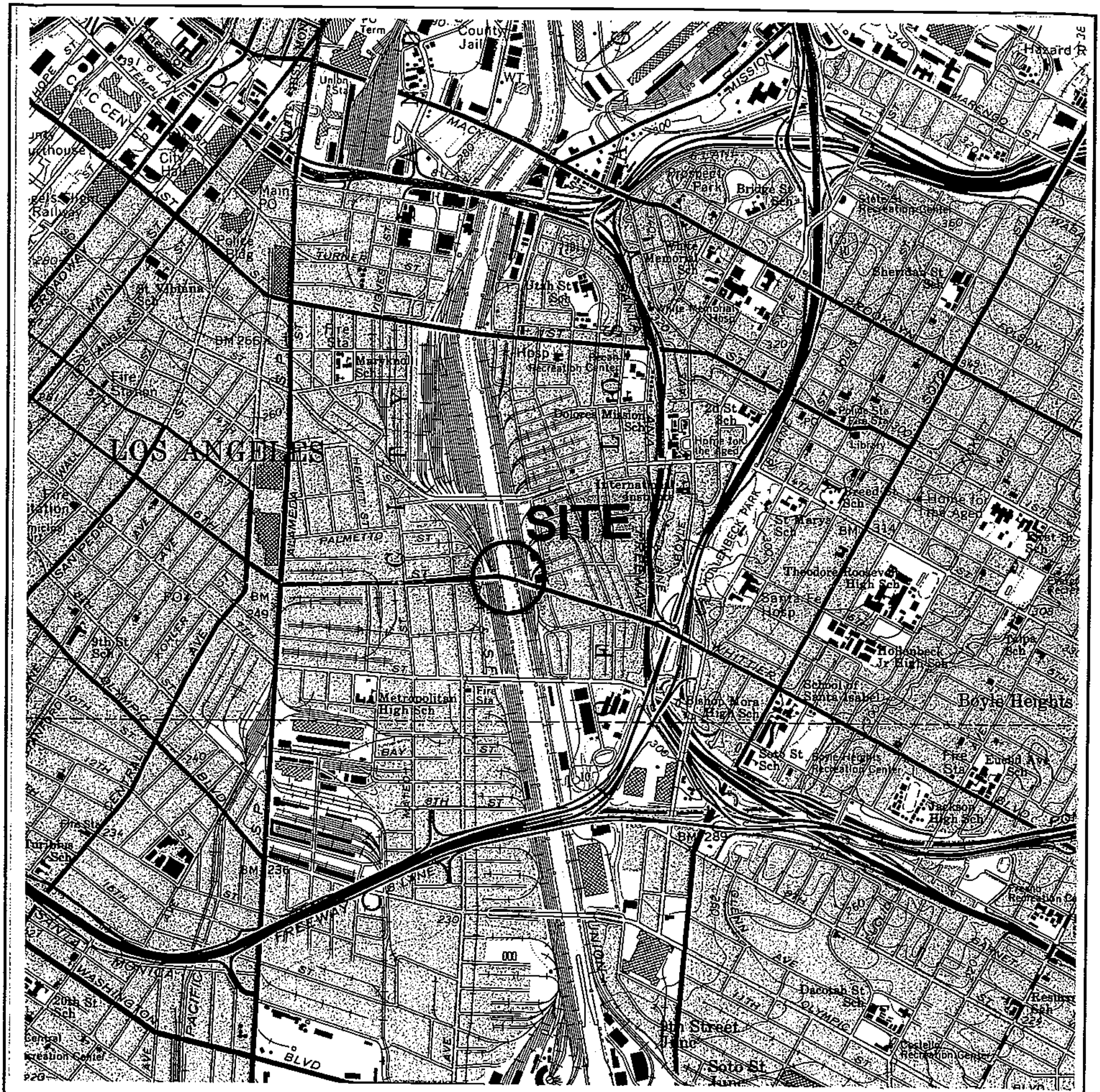
Soil Boring I.D.	Date of Sampling	Sample Depth (feet bgs)	Total Petroleum Hydrocarbons (TPH) (mg/kg)			
			Carbon Chain C7 - C10	Carbon Chain C11 - C20	Carbon Chain C21 - C44	Reported Total Carbon Chain C7 - C44
B-1	19-Nov-01	Surface	ND	8.4	105.9	110
	19-Nov-01	10	ND	3.41	21.7	25
	19-Nov-01	20	ND	ND	ND	ND > 5.0
	19-Nov-01	30	ND	0.404	2.81	ND > 5.0
B-2	19-Nov-01	Surface	ND	2.1	464	470
	19-Nov-01	10	ND	0.59	4.047	ND > 5.0
	19-Nov-01	20	ND	0.37	3.8	ND > 5.0
	19-Nov-01	30	ND	ND	ND	ND > 5.0
B-3	19-Nov-01	Surface	ND	2.3	114.8	120
	19-Nov-01	10	ND	ND	0.31	ND > 5.0
	19-Nov-01	20	ND	ND	0.472	ND > 5.0
	19-Nov-01	30	ND	ND	ND	ND > 5.0
B-4	19-Nov-01	Surface	ND	ND	416.4	420
	19-Nov-01	10	ND	ND	32.83	33
	19-Nov-01	20	ND	ND	0.75	ND > 5.0
	19-Nov-01	30	ND	ND	0.70	ND > 5.0

ND = not detected above the laboratory reporting limit
bgs = below ground surface
mg/kg = milligrams per kilogram

TABLE 2
SUMMARY OF SOIL ANALYTICAL DATA
Title 22 Metals by EPA Method 6010/7000 Series
Sixth Street Bridge Over Los Angeles River
Los Angeles, California

Soil Boring I.D.	Date of Sampling	Sample Depth (feet bgs)	Title 22 Metals (mg/kg)																
			Antimony	Arsenic	Barium	Beryllium	Cadmium	Chromium (Total)	Cobalt	Copper	Lead	Mercury	Molybdenum	Nickel	Selenium	Silver	Thallium	Vanadium	Zinc
B-1	19-Nov-01	Surface	ND	1.84	101	0.368	ND	24.5	7.24	19.1	10.8	ND	1.55	15.2	ND	ND	ND	25.9	50.6
	19-Nov-01	10	ND	0.799	68.4	ND	ND	16.5	5.25	14.0	5.76	ND	1.14	10.3	ND	ND	ND	22.7	35.8
	19-Nov-01	20	ND	ND	60.6	ND	ND	13.3	5.79	7.64	1.92	0.110	ND	6.41	ND	ND	ND	28.6	27.8
	19-Nov-01	30	ND	5.74	480	0.729	ND	19.7	17.8	29.2	5.29	0.119	4.29	11.1	ND	ND	ND	55.9	45.5
B-2	19-Nov-01	Surface	ND	1.50	65.8	ND	ND	8.73	5.07	10.2	18.4	ND	ND	5.89	ND	ND	ND	23.6	47.3
	19-Nov-01	10	ND	ND	44.7	ND	ND	6.90	4.09	5.37	2.48	ND	ND	4.02	ND	ND	ND	17.4	22.4
	19-Nov-01	20	ND	ND	42.7	ND	ND	9.49	4.06	17.9	1.92	ND	0.538	4.77	ND	ND	ND	18.7	27.1
	19-Nov-01	30	ND	ND	38.6	ND	ND	5.28	3.54	5.29	1.36	ND	ND	3.60	ND	ND	ND	16.4	17.5
B-3	19-Nov-01	Surface	ND	3.38	109	0.303	ND	13.4	6.11	27.3	52.4	0.125	0.997	10.8	ND	ND	0.869	25.8	84.9
	19-Nov-01	10	ND	ND	95.6	0.526	ND	16.4	9.44	23.5	5.65	ND	ND	11.1	ND	ND	0.754	37.0	46.0
	19-Nov-01	20	ND	ND	51.5	ND	ND	10.1	4.65	11.8	1.89	ND	0.291	5.77	ND	ND	ND	19.8	26.1
	19-Nov-01	30	ND	ND	104	0.356	ND	16.7	9.25	17.8	3.19	ND	ND	11.4	1.88	ND	ND	37.6	48.6
B-4	19-Nov-01	Surface	ND	ND	81.5	0.252	ND	12.9	6.16	1,010	51.8	ND	0.394	8.49	ND	ND	ND	25.0	66.8
	19-Nov-01	10	ND	ND	46.2	ND	ND	7.39	4.32	78.1	4.36	0.0902	0.318	4.98	ND	ND	ND	16.8	27.2
	19-Nov-01	20	ND	ND	58.5	ND	ND	13.8	5.59	16.2	4.49	ND	0.311	7.70	ND	ND	ND	24.0	34.7
	19-Nov-01	30	ND	ND	56.9	0.275	ND	10.4	5.23	21.7	3.25	ND	0.380	6.27	ND	ND	ND	24.5	31.1

ND = not detected above the laboratory reporting limit
bgs = below ground surface
mg/kg = milligrams per kilogram



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VICINITY MAP

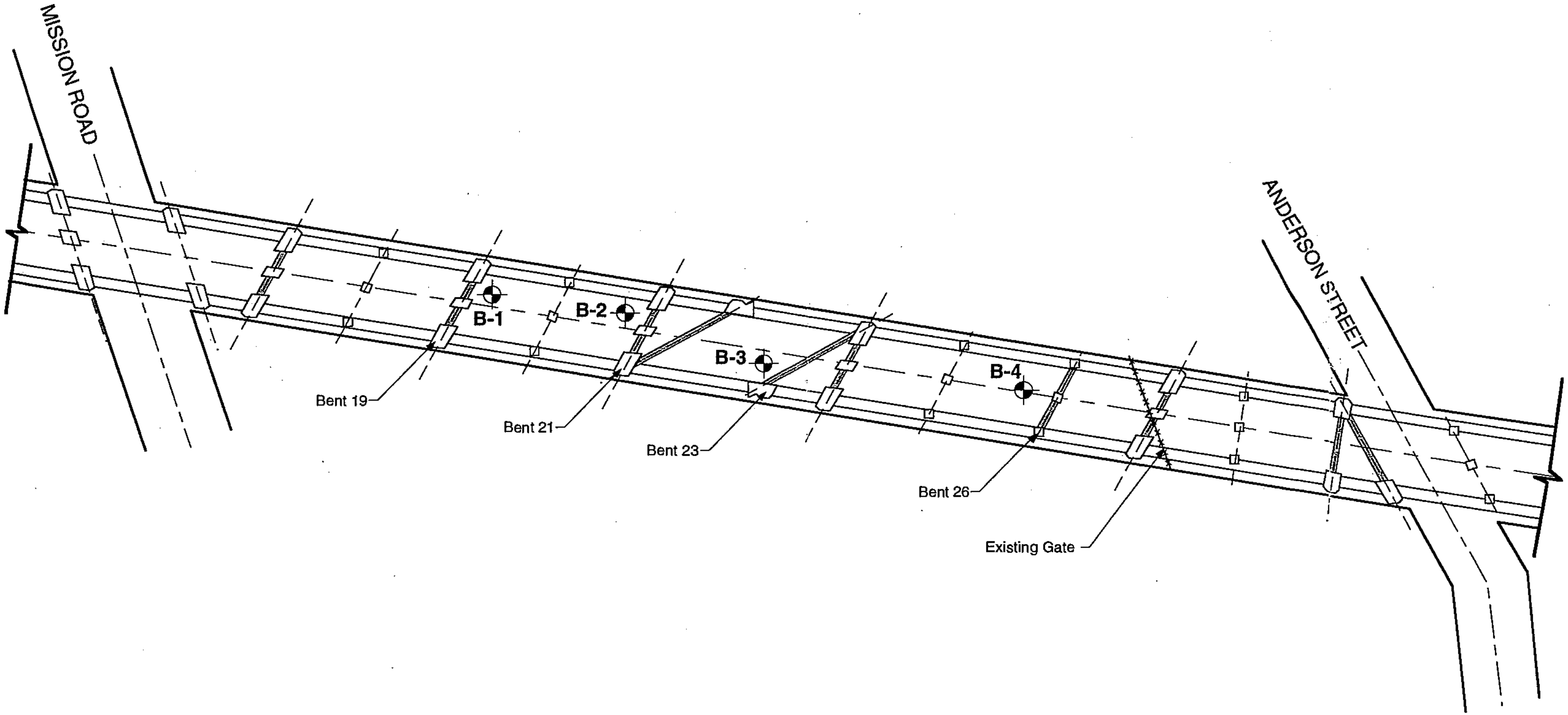
SIXTH STREET BRIDGE
OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA

FOR WEI KOO & ASSOCIATES

REFERENCE: USGS 7.5 Minute Series Topographic Map,
"Los Angeles, California". Quadrangle, 1966, Photorevised 1981.

URS

FIGURE 1



NOT TO SCALE

LEGEND

B-4 ⌖ Boring Location and Designation

PLOT PLAN
SIXTH STREET BRIDGE OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA

FOR: WEI KOO & Associates

FIGURE 2

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	*GRAVEL AND GRAVELLY SOILS* MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	*CLEAN GRAVELS* (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	*CLEAN SANDS* (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM	SILTY SANDS, SAND - SILT MIXTURES
			SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	*SILTS AND CLAYS*	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Dual symbols are used to indicate gravels or sand with 5-12% fines and soils with fines classifying as CL-ML. Symbols separated by a slash indicate borderline soil classifications.

Other Material Symbols

▨ Asphalt

▤ Siltstone

Sampler and Symbol Descriptions

■ Dames & Moore Type-U sample

◩ Standard Penetration Test

☐ No Recovery

Bk☒ Bulk sample

☒ Disturbed Type-U Sample

☐ Pitcher Tube Sample

▯ Shelby Tube Sample

▮ Rock Core Sample

▼ Approximate depth of perched water or groundwater

Note: Number of blows required to advance driven sample 12" (or length noted) is recorded.

Laboratory and Field Test Abbreviations

CBR	California Bearing Ratio Test
COL	Collapse Potential test (test result in parentheses)
COMP	Compaction test
CON	Consolidation test
CORR	Corrosivity test
DSCD	Consolidated drained direct shear test
EI	Expansion Index test (test result in parentheses)
LL=29	Liquid limit (Atterberg limits test)
PI=11	Plasticity Index (Atterberg limits test)
PP	Pocket Penetrometer test (test result in parentheses)
R-Value	Resistance Value test
SA	Sieve Analysis (-200 result in parentheses)
HA	Hydrometer Analysis (-200 result in parentheses)
SE	Sand Equivalent test (test result in parentheses)
SWELL	Swell Load test (test result in parentheses)
TV	Torvane test (test result in parentheses)
-200	Percent passing #200 sieve (test result in parentheses)
+4	Percent retained #4 sieve (test result in parentheses)

KEY TO LOG OF BORING

SIXTH STREET BRIDGE OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR: WEI KOO & ASSOCIATES

URS

FIGURE 3

Date(s) Drilled	11/19/01	Logged By	Joseph Gratzer
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8-inch O.D.
Drill Rig Type	CME 75	Hammer Data	140 lbs/30-inch Drop
Sampling Method(s)	Dame & Moore Type-U		
Approximate Groundwater Depth and Date Measured	No Groundwater Encountered		
Comments	None		

Boring B-1

Sheet 1 of 1

Job Number	57-00255002.01
Total Depth Drilled (ft)	31.0
Approximate Ground Surface Elevation(ft)	250 feet MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	PID Readings (ppm)	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
250	0					SM	Brown silty fine to coarse SAND with trace fine gravel, dry				
	5	■	1	20			Grades moist, loose	0	4	97	+4 (3) -200 (21) DSCD
						SP	Brown fine to coarse SAND with fine gravel, dry, medium dense				
240	10	■	2	29				0	2	108	+4 (34) -200 (3) DSCD
						SM	Brown silty fine to medium SAND, moist, medium dense				
	15	■	3	48				1	5	97	
230	20	■	4	60			Grades fine	1	5	105	+4 (0) -200 (13) DSCD
						SP-SM	Brown fine SAND with silt, dry, medium dense				
	25	■	5	65				0	3	111	
220	30	■	6	50/5"			Grades moist, very dense	0	4	113	
	35										
210	40										

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

SIXTH STREET BRIDGE
OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR: WEI KOO & ASSOCIATES

URS

FIGURE 4

Date(s) Drilled	11/19/01	Logged By	Joseph Gratzer
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8-inch O.D.
Drill Rig Type	CME 75	Hammer Data	140 lbs/30-inch Drop
Sampling Method(s)	Dame & Moore Type-U		
Approximate Groundwater Depth and Date Measured	No Groundwater Encountered		
Comments	None		

Boring B-2

Sheet 1 of 1

Job Number	57-00255002.01
Total Depth Drilled (ft)	31.5
Approximate Ground Surface Elevation(ft)	250 feet MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	PID Readings (ppm)	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
250	0					SP	Brown fine to coarse SAND with gravel, dry				
	5	■	1	27			Grades medium dense	0	2	98	Rig Chatter at 5 feet
						SM	Brown silty fine SAND, moist, medium dense				
240	10	■	2	50				0	5	102	
	15	■	3	24			Grades fine to coarse, with fine gravel	3	4	110	+4 (36) -200 (18) DSCD
						SP-SM	Brown fine to medium SAND with silt, moist, very dense				
230	20	■	4	50/4"				2	4	108	
	25	■	5	50/5"				2	5	113	
220	30	■	6	50/4"				0	5	118	
						ML	Brown fine sandy SILT, moist, very dense				
	35										
210	40										

Report: DMG14; Project File: G:\GINTW\PROJECTS\6THSTR-1.GPJ; Data Template:DMLA.GDT Printed: 11/27/01

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

SIXTH STREET BRIDGE
OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR: WEI KOO & ASSOCIATES

FIGURE 5

Date(s) Drilled	11/20/01	Logged By	Joseph Gratzer
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8-Inch O.D.
Drill Rig Type	CME 75	Hammer Data	140 lbs/30-inch Drop
Sampling Method(s)	Dame & Moore Type-U		
Approximate Groundwater Depth and Date Measured	No Groundwater Encountered		
Comments	None		

Boring B-3

Sheet 1 of 1

Job Number	57-00255002.01
Total Depth Drilled (ft)	31.5
Approximate Ground Surface Elevation(ft)	250 feet MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	PID Readings (ppm)	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
250	0					SP-SM	Brown fine SAND with silt, dry				
	5	■	1	31			Grades medium dense	2	1	108	
240	10	■	2	50/5"			Grades fine to coarse, moist, very dense	2	5	116	
	15	■	3	60			Grades with trace fine gravel, medium dense	1	4	115	
						SM	Brown silty fine to coarse SAND with trace fine gravel, moist, very dense				
230	20	■	4	50/5"				1	6	109	
	25	■	5	50/6"			Grades with some fine gravel, dense	0	4	111	+4 (19) SA (21)
						ML	Brown fine sandy SILT, very moist, dense to very dense				
220	30	■	6	50/5"				0	20	101	
	35										
210	40										

Report: DMG14; Project File: G:\GINTW\PROJECTS\6THSTR~1.GPJ; Data Template:DMLA.GDT Printed: 11/27/01

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

SIXTH STREET BRIDGE
OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR: WEI KOO & ASSOCIATES

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FIGURE 6

Date(s) Drilled	11/20/01	Logged By	Joseph Gratzer
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8-inch O.D.
Drill Rig Type	CME 75	Hammer Data	140 lbs/30-inch Drop
Sampling Method(s)	Dame & Moore Type-U		
Approximate Groundwater Depth and Date Measured	No Groundwater Encountered		
Comments	None		

Boring B-4

Sheet 1 of 1

Job Number	57-00255002.01
Total Depth Drilled (ft)	31.5
Approximate Ground Surface Elevation(ft)	250 feet MSL

Elevation (ft)	Depth (ft)	SAMPLES			Graphic Log	USCS	MATERIAL DESCRIPTION	PID Readings (ppm)	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
		Type	Number	Blows per 6 inches							
250	0					SP	Brown fine to coarse SAND with trace fine gravel, dry				
	5	■	1	49			Grades medium dense	0	2	102	
240	10	■	2	80			Grades dense	1	2	104	
	15	■	3	70			Grades light brown, medium dense	0	2	121	
						SM	Brown silty fine SAND, moist, medium dense				
230	20	■	4	67				0	6	100	
	25	■	5	42			Grades with fine gravel	3	4	114	+4 (16) -200 (21) DSCD
220	30	■	6	70			Grades fine to coarse, with trace fine gravel	1	5	116	
	35										
210	40										

Report: DMG14; Project File: G:\GINTW\PROJECTS\6THSTR-1.GPJ; Data Template:DMLA.GDT Printed: 11/28/01

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

SIXTH STREET BRIDGE
OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR: WEI KOO & ASSOCIATES

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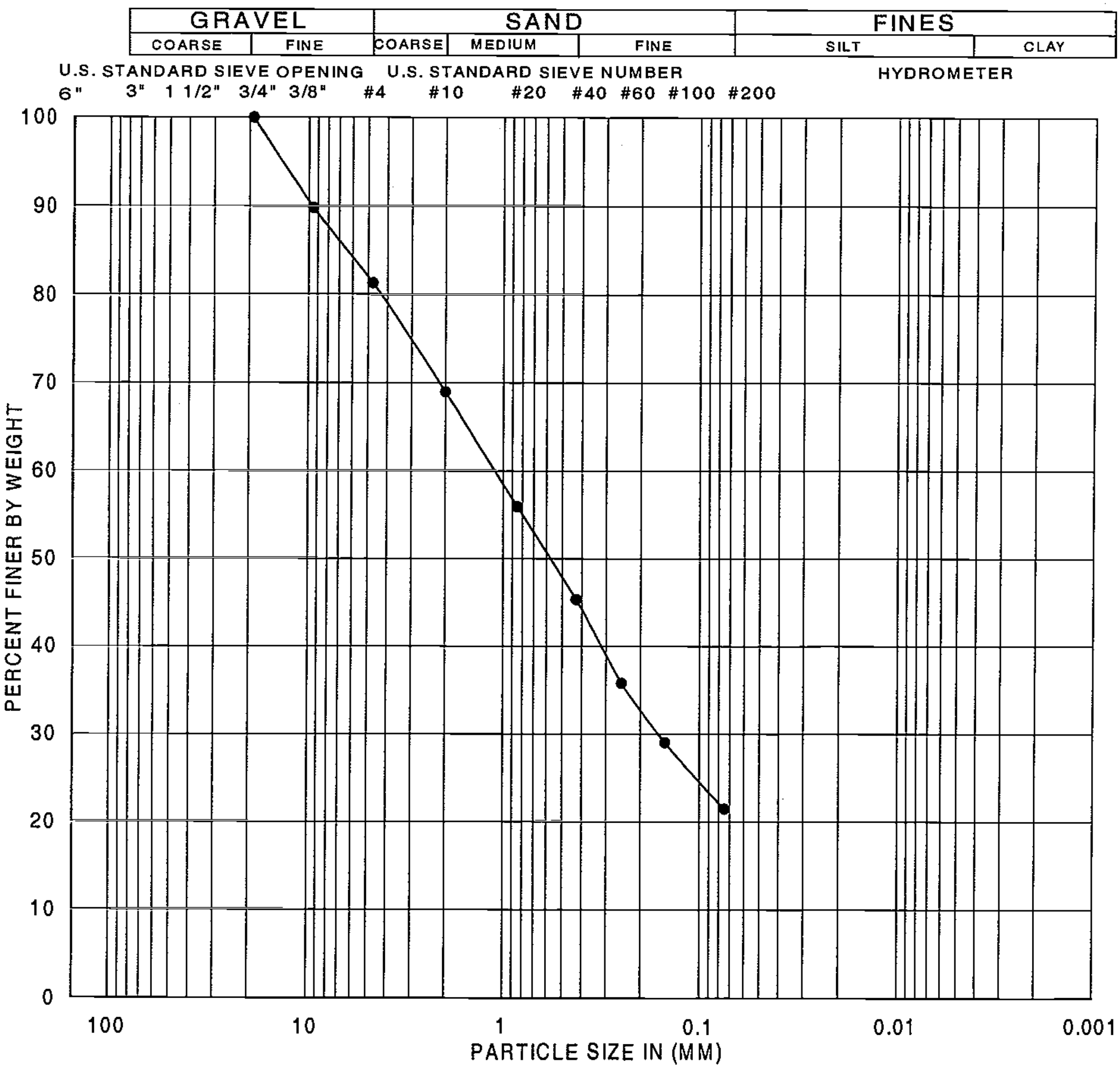
FIGURE 7

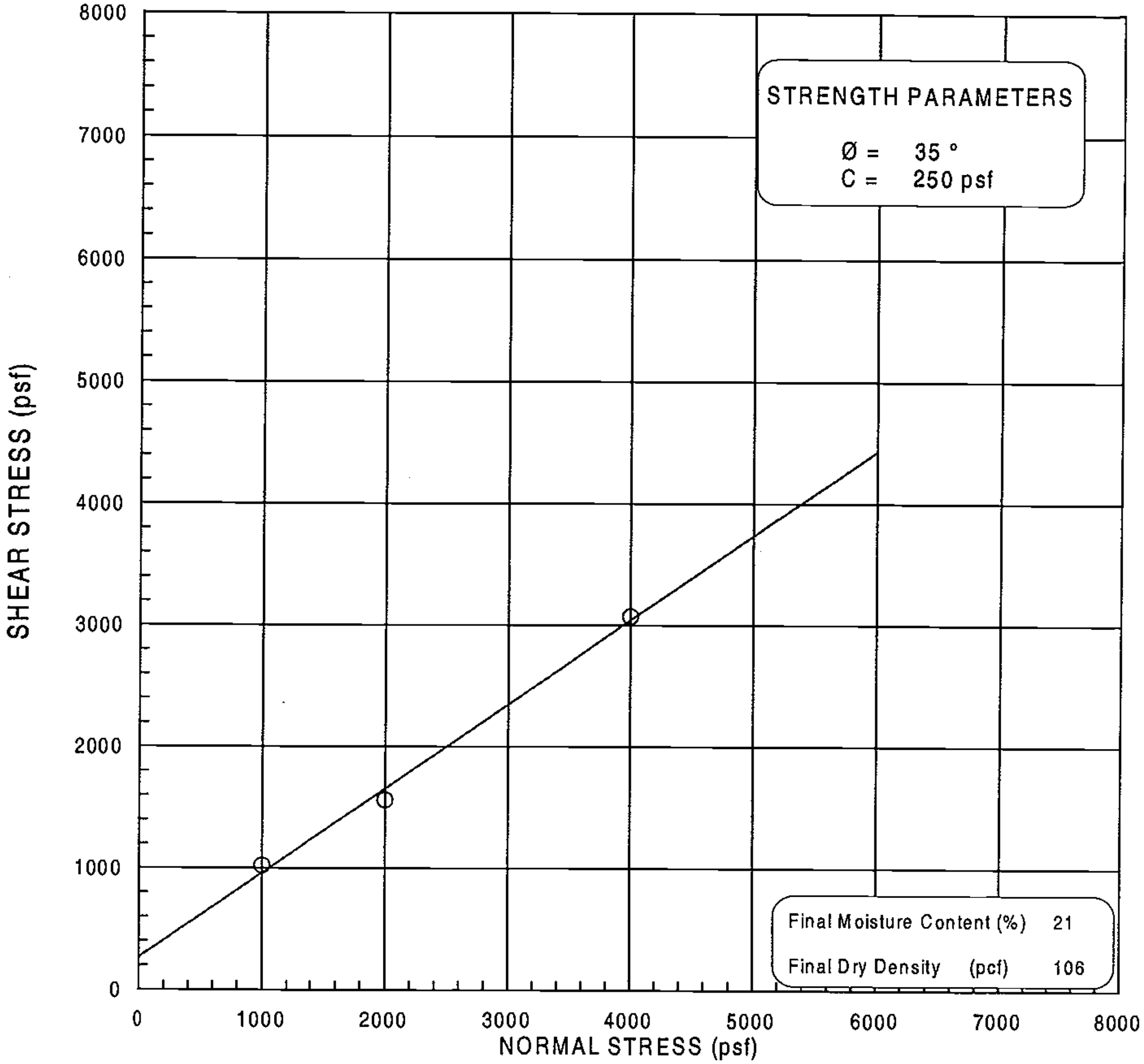
PARTICLE-SIZE DISTRIBUTION CURVE



Symbol	Boring No.	Sample No.	Depth (ft.)	GR:SA:FI (%)	Sample Description (USCS Symbol)
●	B-3	5	25.0	19:60:21	Brown silty fine to coarse SAND (SM) with some fine gravel

SIXTH STREET BRIDGE
OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR: WEI KOO & ASSOCIATES

FIGURE 8



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	SHEAR STRESS (psf)
B-1	1	5	0.01	○ 1000	1020
				○ 2000	1560
				○ 4000	3072

Sample Description: Brown silty fine to coarse SAND (SM) with trace fine gravel

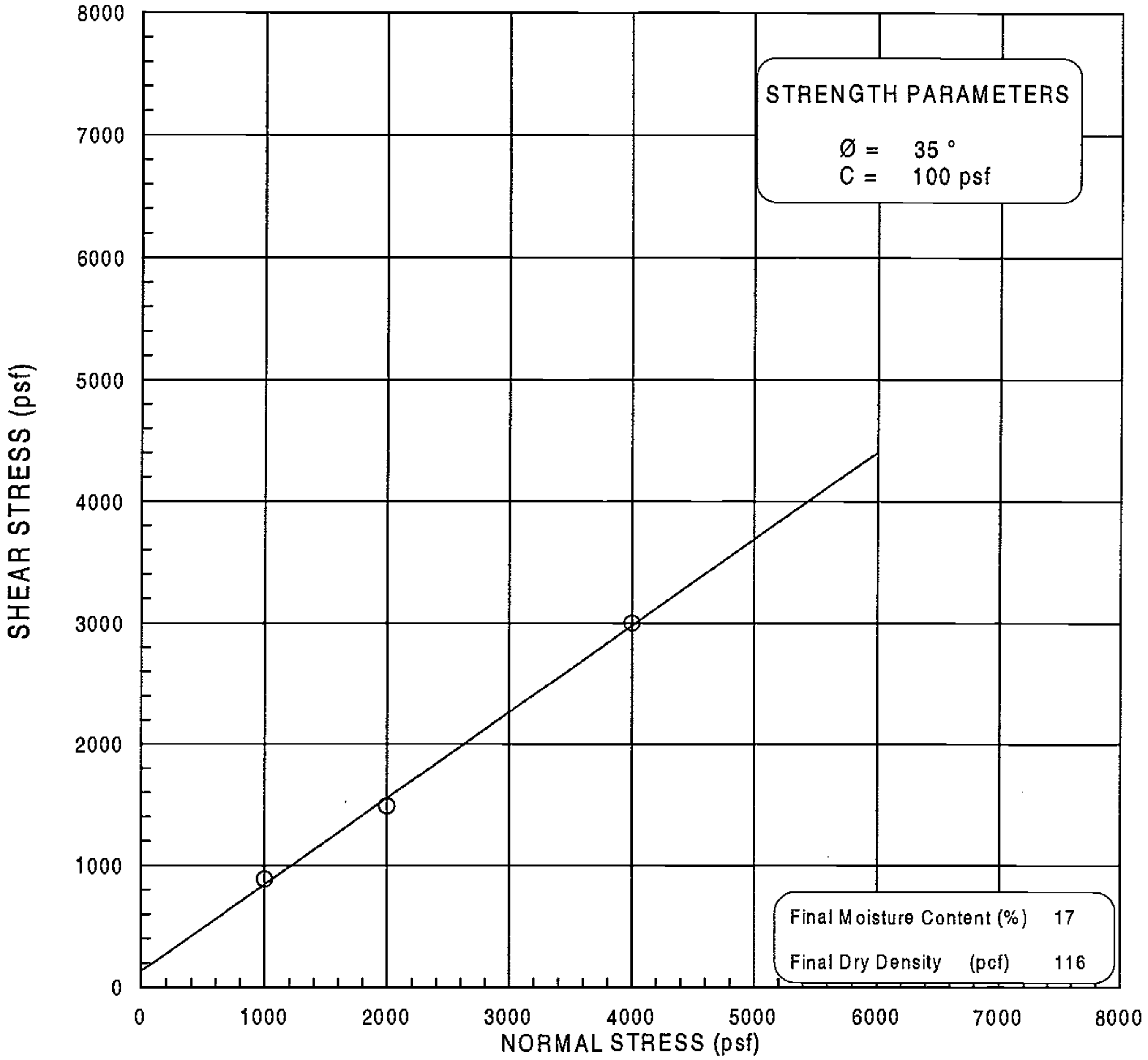
DIRECT SHEAR TEST RESULTS

CONSOLIDATED DRAINED
ASTM D 3080

SIXTH STREET BRIDGE
OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR: WEI KOO & ASSOCIATES

URS

FIGURE 9



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	SHEAR STRESS (psf)
B-1	2	10	0.01	○ 1000	888
				○ 2000	1488
				○ 4000	3000

Sample Description: Brown fine to coarse SAND (SP) with fine gravel

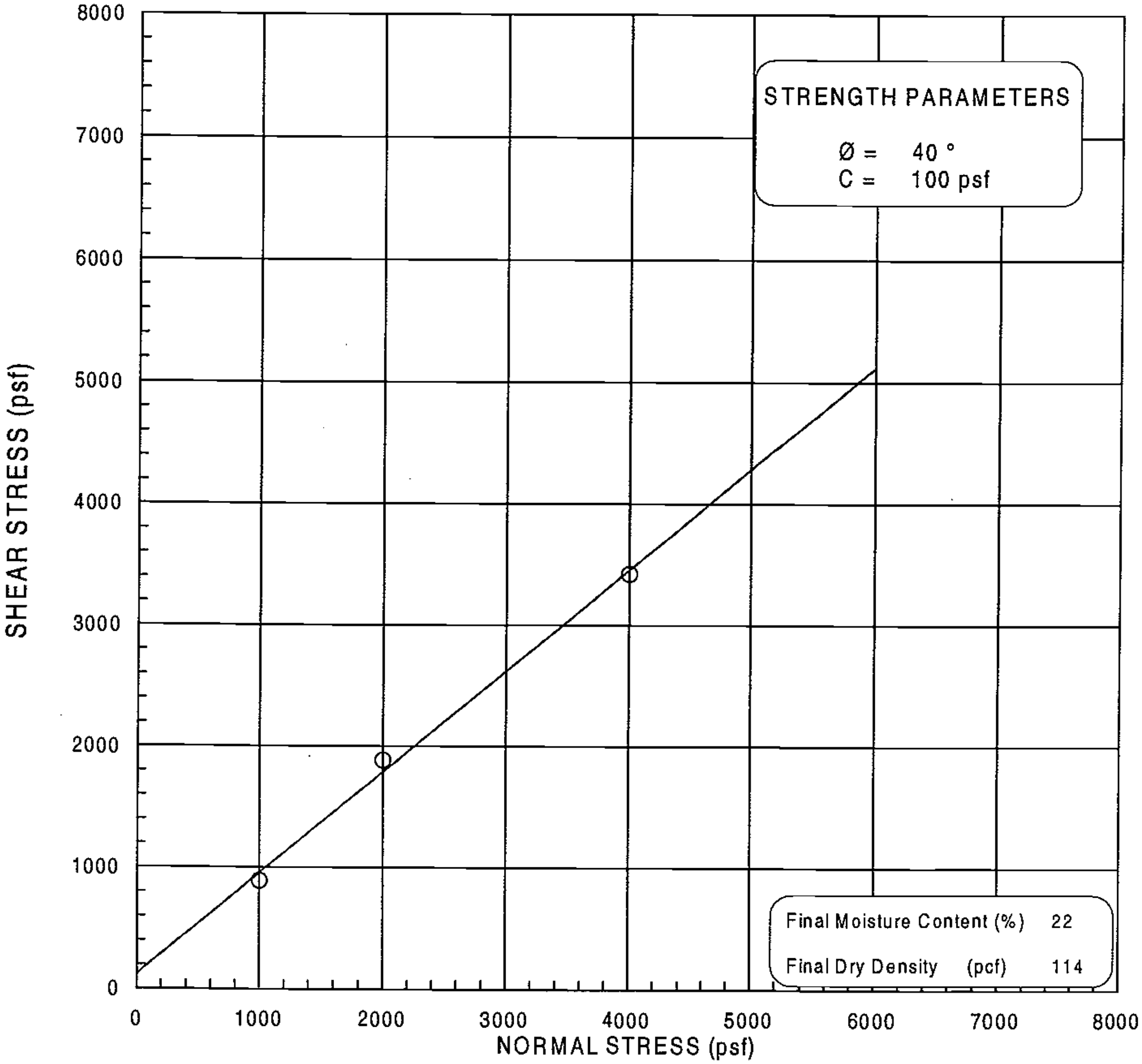
DIRECT SHEAR TEST RESULTS

CONSOLIDATED DRAINED
ASTM D 3080

SIXTH STREET BRIDGE
OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR: WEI KOO & ASSOCIATES

URS

FIGURE 10



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	SHEAR STRESS (psf)
B-1	4	20	0.01	○ 1000	888
				○ 2000	1884
				○ 4000	3420

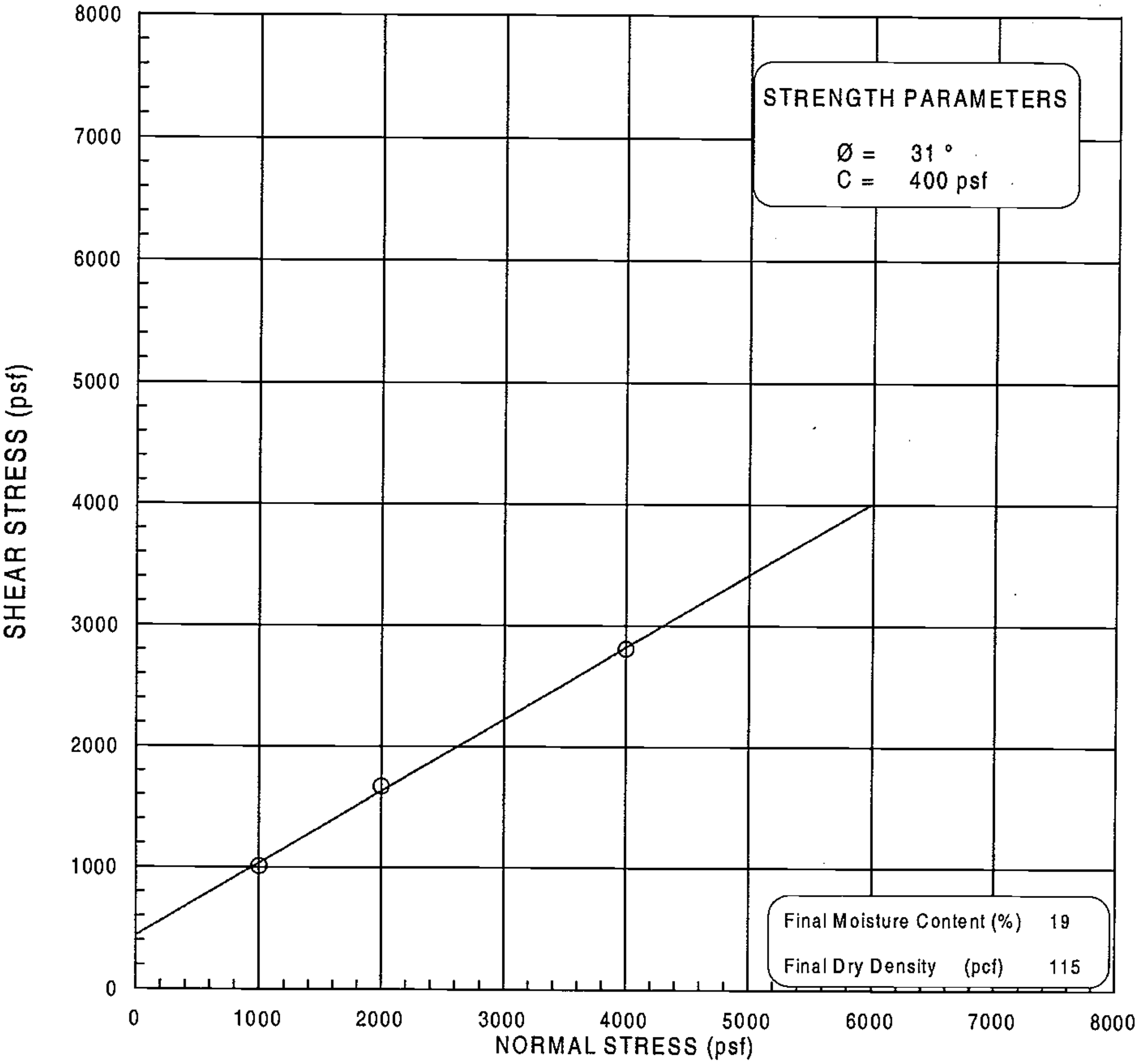
Sample Description: Brown silty fine to medium SAND (SM)

DIRECT SHEAR TEST RESULTS

CONSOLIDATED DRAINED
ASTM D 3080

SIXTH STREET BRIDGE
OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR: WEI KOO & ASSOCIATES

FIGURE 11



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	SHEAR STRESS (psf)
B-2	3	15	0.01	○ 1000	1008
				○ 2000	1668
				○ 4000	2808

Sample Description: Brown silty fine to coarse SAND (SM) with fine gravel

DIRECT SHEAR TEST RESULTS

CONSOLIDATED DRAINED
ASTM D 3080

SIXTH STREET BRIDGE
OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR: WEI KOO & ASSOCIATES

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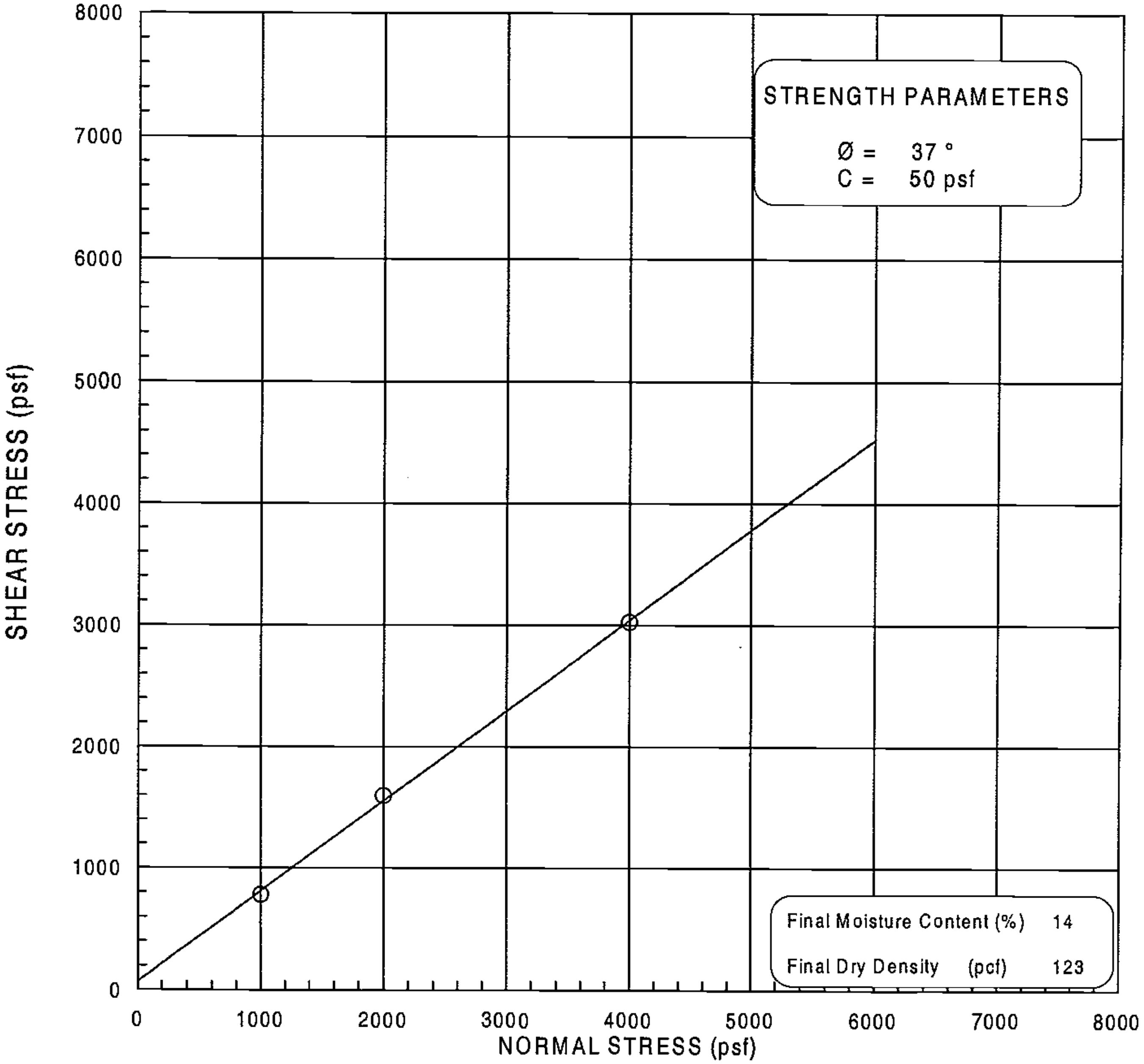
FIGURE 12



BORING NO.	SAMPLE NO.	DEPTH (ft)	STRAIN RATE (in/min)	NORMAL STRESS (psf)	SHEAR STRESS (psf)
B-4	5	25	0.01	○ 1000	780
				○ 2000	1596
				○ 4000	3024

Sample Description: Brown silty fine SAND (SM)

DIRECT SHEAR TEST RESULTS

CONSOLIDATED DRAINED
ASTM D 3080

SIXTH STREET BRIDGE
OVER LOS ANGELES RIVER
LOS ANGELES, CALIFORNIA
FOR: WEI KOO & ASSOCIATES

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FIGURE 13

Calscience Environmental Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received:	11/20/01
Work Order No:	01-11-1097
Preparation:	Total Digestion
Method:	EPA 6010B / EPA 7471A

Project: 6th Street Bridge

Page 1 of 6

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-1-Surface	01-11-1097-1	11/19/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 5:36:30 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	ND	0.0835	1		mg/kg
Arsenic	1.84	0.75	1		mg/kg	Molybdenum	1.55	0.25	1		mg/kg
Barium	101	0.500	1		mg/kg	Nickel	15.2	0.2	1		mg/kg
Beryllium	0.368	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	24.5	0.2	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	7.24	0.25	1		mg/kg	Vanadium	25.9	0.2	1		mg/kg
Copper	19.1	0.5	1		mg/kg	Zinc	50.6	1.0	1		mg/kg
Lead	10.8	0.5	1		mg/kg						

B-1-10	01-11-1097-2	11/19/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 5:39:33 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	ND	0.0835	1		mg/kg
Arsenic	0.799	0.750	1		mg/kg	Molybdenum	1.14	0.25	1		mg/kg
Barium	68.4	0.5	1		mg/kg	Nickel	10.3	0.2	1		mg/kg
Beryllium	ND	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	16.5	0.2	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	5.25	0.25	1		mg/kg	Vanadium	22.7	0.2	1		mg/kg
Copper	14.0	0.5	1		mg/kg	Zinc	35.8	1.0	1		mg/kg
Lead	5.76	0.50	1		mg/kg						

B-1-20	01-11-1097-3	11/19/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 5:42:36 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	0.110	0.083	1		mg/kg
Arsenic	ND	0.750	1		mg/kg	Molybdenum	ND	0.250	1		mg/kg
Barium	60.6	0.5	1		mg/kg	Nickel	6.41	0.25	1		mg/kg
Beryllium	ND	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	13.3	0.2	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	5.79	0.25	1		mg/kg	Vanadium	28.6	0.2	1		mg/kg
Copper	7.64	0.50	1		mg/kg	Zinc	27.8	1.0	1		mg/kg
Lead	1.92	0.50	1		mg/kg						

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

Calscience Environmental Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received:	11/20/01
Work Order No:	01-11-1097
Preparation:	Total Digestion
Method:	EPA 6010B / EPA 7471A

Project: 6th Street Bridge

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-1-30	01-11-1097-4	11/19/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 5:45:40 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	0.119	0.083	1		mg/kg
Arsenic	5.74	0.75	1		mg/kg	Molybdenum	4.29	0.25	1		mg/kg
Barium	480	0.500	1		mg/kg	Nickel	11.1	0.2	1		mg/kg
Beryllium	0.729	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	19.7	0.2	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	17.8	0.2	1		mg/kg	Vanadium	55.9	0.2	1		mg/kg
Copper	29.2	0.5	1		mg/kg	Zinc	45.5	1.0	1		mg/kg
Lead	5.29	0.50	1		mg/kg						

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-2-Surface	01-11-1097-5	11/19/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 5:48:45 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	ND	0.0835	1		mg/kg
Arsenic	1.50	0.75	1		mg/kg	Molybdenum	ND	0.250	1		mg/kg
Barium	65.8	0.5	1		mg/kg	Nickel	5.89	0.25	1		mg/kg
Beryllium	ND	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	8.73	0.25	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	5.07	0.25	1		mg/kg	Vanadium	23.6	0.2	1		mg/kg
Copper	10.2	0.5	1		mg/kg	Zinc	47.3	1.0	1		mg/kg
Lead	18.4	0.5	1		mg/kg						

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-2-10	01-11-1097-6	11/19/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 5:51:50 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	ND	0.0835	1		mg/kg
Arsenic	ND	0.750	1		mg/kg	Molybdenum	ND	0.250	1		mg/kg
Barium	44.7	0.5	1		mg/kg	Nickel	4.02	0.25	1		mg/kg
Beryllium	ND	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	6.90	0.25	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	4.09	0.25	1		mg/kg	Vanadium	17.4	0.2	1		mg/kg
Copper	5.37	0.50	1		mg/kg	Zinc	22.4	1.0	1		mg/kg
Lead	2.48	0.50	1		mg/kg						

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

Calscience Environmental Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received: 11/20/01
Work Order No: 01-11-1097
Preparation: Total Digestion
Method: EPA 6010B / EPA 7471A

Project: 6th Street Bridge

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-2-20	01-11-1097-7	11/19/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 6:01:01 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	ND	0.0835	1		mg/kg
Arsenic	ND	0.750	1		mg/kg	Molybdenum	0.538	0.250	1		mg/kg
Barium	42.7	0.5	1		mg/kg	Nickel	4.77	0.25	1		mg/kg
Beryllium	ND	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	9.49	0.25	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	4.06	0.25	1		mg/kg	Vanadium	18.7	0.2	1		mg/kg
Copper	17.9	0.5	1		mg/kg	Zinc	27.1	1.0	1		mg/kg
Lead	1.92	0.50	1		mg/kg						

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-2-30	01-11-1097-8	11/19/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 6:04:03 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	ND	0.0835	1		mg/kg
Arsenic	ND	0.750	1		mg/kg	Molybdenum	ND	0.250	1		mg/kg
Barium	38.6	0.5	1		mg/kg	Nickel	3.60	0.25	1		mg/kg
Beryllium	ND	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	5.28	0.25	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	3.54	0.25	1		mg/kg	Vanadium	16.4	0.2	1		mg/kg
Copper	5.29	0.50	1		mg/kg	Zinc	17.5	1.0	1		mg/kg
Lead	1.36	0.50	1		mg/kg						

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-3-Surface	01-11-1097-9	11/19/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 6:07:02 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	0.125	0.083	1		mg/kg
Arsenic	3.38	0.75	1		mg/kg	Molybdenum	0.997	0.250	1		mg/kg
Barium	109	0.500	1		mg/kg	Nickel	10.8	0.2	1		mg/kg
Beryllium	0.303	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	13.4	0.2	1		mg/kg	Thallium	0.869	0.750	1		mg/kg
Cobalt	6.11	0.25	1		mg/kg	Vanadium	25.8	0.2	1		mg/kg
Copper	27.3	0.5	1		mg/kg	Zinc	84.9	1.0	1		mg/kg
Lead	52.4	0.5	1		mg/kg						

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

Calscience
Environmental
Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received:	11/20/01
Work Order No:	01-11-1097
Preparation:	Total Digestion
Method:	EPA 6010B / EPA 7471A

Project: 6th Street Bridge

Page 4 of 6

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix	Date Prepared:	Date Analyzed:	QC Batch ID:
B-3-10	01-11-1097-10	11/20/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 6:10:02 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	ND	0.0835	1		mg/kg
Arsenic	ND	0.750	1		mg/kg	Molybdenum	ND	0.250	1		mg/kg
Barium	95.6	0.5	1		mg/kg	Nickel	11.1	0.2	1		mg/kg
Beryllium	0.526	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	16.4	0.2	1		mg/kg	Thallium	0.754	0.750	1		mg/kg
Cobalt	9.44	0.25	1		mg/kg	Vanadium	37.0	0.2	1		mg/kg
Copper	23.5	0.5	1		mg/kg	Zinc	46.0	1.0	1		mg/kg
Lead	5.65	0.50	1		mg/kg						

B-3-20	01-11-1097-11	11/20/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 6:13:02 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	ND	0.0835	1		mg/kg
Arsenic	ND	0.750	1		mg/kg	Molybdenum	0.291	0.250	1		mg/kg
Barium	51.5	0.5	1		mg/kg	Nickel	5.77	0.25	1		mg/kg
Beryllium	ND	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	10.1	0.2	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	4.65	0.25	1		mg/kg	Vanadium	19.8	0.2	1		mg/kg
Copper	11.8	0.5	1		mg/kg	Zinc	26.1	1.0	1		mg/kg
Lead	1.89	0.50	1		mg/kg						

B-3-30	01-11-1097-12	11/20/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 6:16:02 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	ND	0.0835	1		mg/kg
Arsenic	ND	0.750	1		mg/kg	Molybdenum	ND	0.250	1		mg/kg
Barium	104	0.500	1		mg/kg	Nickel	11.4	0.2	1		mg/kg
Beryllium	0.356	0.250	1		mg/kg	Selenium	1.88	0.75	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	16.7	0.2	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	9.25	0.25	1		mg/kg	Vanadium	37.6	0.2	1		mg/kg
Copper	17.8	0.5	1		mg/kg	Zinc	48.6	1.0	1		mg/kg
Lead	3.19	0.50	1		mg/kg						

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

Calscience Environmental Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received: 11/20/01
Work Order No: 01-11-1097
Preparation: Total Digestion
Method: EPA 6010B / EPA 7471A

Project: 6th Street Bridge

Page 5 of 6

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-4-Surface	01-11-1097-13	11/20/01	Solid	11/20/01	11/21/01	011120lca7

Comment(s): Mercury was analyzed on 11/20/01 6:19:03 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	ND	0.0835	1		mg/kg
Arsenic	ND	0.750	1		mg/kg	Molybdenum	0.394	0.250	1		mg/kg
Barium	81.5	0.5	1		mg/kg	Nickel	8.49	0.25	1		mg/kg
Beryllium	0.252	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	12.9	0.2	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	6.16	0.25	1		mg/kg	Vanadium	25.0	0.2	1		mg/kg
Copper	1010	0.500	1		mg/kg	Zinc	66.8	1.0	1		mg/kg
Lead	51.8	0.5	1		mg/kg						

B-4-10	01-11-1097-14	11/20/01	Solid	11/20/01	11/21/01	011120lca7

Comment(s): Mercury was analyzed on 11/20/01 6:22:04 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	0.0902	0.0835	1		mg/kg
Arsenic	ND	0.750	1		mg/kg	Molybdenum	0.318	0.250	1		mg/kg
Barium	46.2	0.5	1		mg/kg	Nickel	4.98	0.25	1		mg/kg
Beryllium	ND	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	7.39	0.25	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	4.32	0.25	1		mg/kg	Vanadium	16.8	0.2	1		mg/kg
Copper	78.1	0.5	1		mg/kg	Zinc	27.2	1.0	1		mg/kg
Lead	4.36	0.50	1		mg/kg						

B-4-20	01-11-1097-15	11/20/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 6:25:06 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	ND	0.0835	1		mg/kg
Arsenic	ND	0.750	1		mg/kg	Molybdenum	0.311	0.250	1		mg/kg
Barium	58.5	0.5	1		mg/kg	Nickel	7.70	0.25	1		mg/kg
Beryllium	ND	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	13.8	0.2	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	5.59	0.25	1		mg/kg	Vanadium	24.0	0.2	1		mg/kg
Copper	16.2	0.5	1		mg/kg	Zinc	34.7	1.0	1		mg/kg
Lead	4.49	0.50	1		mg/kg						

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

Calscience Environmental Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received: 11/20/01
Work Order No: 01-11-1097
Preparation: Total Digestion
Method: EPA 6010B / EPA 7471A

Project: 6th Street Bridge

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-430	01-11-1097-16	11/20/01	Solid	11/20/01	11/21/01	011120lcs7

Comment(s): Mercury was analyzed on 11/20/01 6:28:09 PM with batch 011120lcs4

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Mercury	ND	0.0835	1		mg/kg
Arsenic	ND	0.750	1		mg/kg	Molybdenum	0.380	0.250	1		mg/kg
Barium	56.9	0.5	1		mg/kg	Nickel	6.27	0.25	1		mg/kg
Beryllium	0.275	0.250	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Silver	ND	0.250	1		mg/kg
Chromium (Total)	10.4	0.2	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Cobalt	5.23	0.25	1		mg/kg	Vanadium	24.5	0.2	1		mg/kg
Copper	21.7	0.5	1		mg/kg	Zinc	31.1	1.0	1		mg/kg
Lead	3.25	0.50	1		mg/kg						

Method Blank	099-04-007-1,218	N/A	Solid	11/20/01	11/20/01	011120lcs4

Parameter	Result	RL	DF	Qual	Units
Mercury	ND	0.0835	1		mg/kg

Method Blank	097-01-002-2,929	N/A	Solid	11/20/01	11/20/01	011120lcs7

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
Antimony	ND	0.750	1		mg/kg	Molybdenum	ND	0.250	1		mg/kg
Arsenic	ND	0.750	1		mg/kg	Nickel	ND	0.250	1		mg/kg
Barium	ND	0.500	1		mg/kg	Selenium	ND	0.750	1		mg/kg
Beryllium	ND	0.250	1		mg/kg	Silver	ND	0.250	1		mg/kg
Cadmium	ND	0.500	1		mg/kg	Thallium	ND	0.750	1		mg/kg
Chromium (Total)	ND	0.250	1		mg/kg	Vanadium	ND	0.250	1		mg/kg
Cobalt	ND	0.250	1		mg/kg	Zinc	ND	1.00	1		mg/kg
Copper	ND	0.500	1		mg/kg	Lead	ND	0.500	1		mg/kg

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

Calscience Environmental Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received:	11/20/01
Work Order No:	01-11-1097
Preparation:	Ext. + D/I
Method:	TPH - Carbon Range

Project: 6th Street Bridge

Page 1 of 8

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-1 Surface	01-11-1097-1	11/19/01	Solid	11/20/01	11/22/01	01112006sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	5.9		1		mg/kg
C8	ND		1		mg/kg	C23-C24	6.0		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	18		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	17		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	19		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	11		1		mg/kg
C17-C18	6.0		1		mg/kg	C41-C44	29		1		mg/kg
C19-C20	2.4		1		mg/kg	C7-C44 Total	110	5	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	105	45-149	

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-1-10	01-11-1097-2	11/19/01	Solid	11/20/01	11/21/01	01112006sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	5.7		1		mg/kg
C8	ND		1		mg/kg	C23-C24	2.4		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	ND		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	ND		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	1.5		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	3.8		1		mg/kg
C17-C18	0.31		1		mg/kg	C41-C44	8.3		1		mg/kg
C19-C20	3.1		1		mg/kg	C7-C44 Total	25	5	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	117	45-149	

**Calscience
Environmental
Laboratories, Inc.**

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received:	11/20/01
Work Order No:	01-11-1097
Preparation:	Ext. + D/I
Method:	TPH - Carbon Range

Project: 6th Street Bridge

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-1-20	01-11-1097-3	11/19/01	Solid	11/20/01	11/21/01	01112006sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	ND		1		mg/kg
C8	ND		1		mg/kg	C23-C24	ND		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	ND		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	ND		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	ND		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	ND		1		mg/kg
C17-C18	ND		1		mg/kg	C41-C44	ND		1		mg/kg
C19-C20	ND		1		mg/kg	C7-C44 Total	ND	5.0	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	91	45-149	

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-1-30	01-11-1097-4	11/19/01	Solid	11/20/01	11/21/01	01112006sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	0.93		1		mg/kg
C8	ND		1		mg/kg	C23-C24	1.1		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	0.78		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	ND		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	ND		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	ND		1		mg/kg
C17-C18	0.034		1		mg/kg	C41-C44	ND		1		mg/kg
C19-C20	0.37		1		mg/kg	C7-C44 Total	ND	5.0	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	122	45-149	

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

Calscience Environmental Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received: 11/20/01
Work Order No: 01-11-1097
Preparation: Ext. + D/I
Method: TPH - Carbon Range

Project: 6th Street Bridge

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-2-Surface	01-11-1097-5	11/19/01	Solid	11/20/01	11/22/01	01112006sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		5		mg/kg	C21-C22	11		5		mg/kg
C8	ND		5		mg/kg	C23-C24	20		5		mg/kg
C9-C10	ND		5		mg/kg	C25-C28	75		5		mg/kg
C11-C12	ND		5		mg/kg	C29-C32	79		5		mg/kg
C13-C14	ND		5		mg/kg	C33-C36	70		5		mg/kg
C15-C16	ND		5		mg/kg	C37-C40	78		5		mg/kg
C17-C18	ND		5		mg/kg	C41-C44	130		5		mg/kg
C19-C20	2.1		5		mg/kg	C7-C44 Total	470	25	5	D	mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	118	45-149	

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-2-10	01-11-1097-6	11/19/01	Solid	11/20/01	11/21/01	01112006sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	1.4		1		mg/kg
C8	ND		1		mg/kg	C23-C24	1.6		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	1.0		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	ND		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	0.047		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	ND		1		mg/kg
C17-C18	ND		1		mg/kg	C41-C44	ND		1		mg/kg
C19-C20	0.59		1		mg/kg	C7-C44 Total	ND	5.0	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	95	45-149	

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

Calscience Environmental Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received: 11/20/01
Work Order No: 01-11-1097
Preparation: Ext. + D/I
Method: TPH - Carbon Range

Project: 6th Street Bridge

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-2-20	01-11-1097-7	11/19/01	Solid	11/20/01	11/21/01	01112005sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	2.2		1		mg/kg
C8	ND		1		mg/kg	C23-C24	1.2		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	0.40		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	ND		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	ND		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	ND		1		mg/kg
C17-C18	ND		1		mg/kg	C41-C44	ND		1		mg/kg
C19-C20	0.37		1		mg/kg	C7-C44 Total	ND	5.0	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	124	45-149	

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-2-30	01-11-1097-8	11/19/01	Solid	11/20/01	11/21/01	01112005sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	ND		1		mg/kg
C8	ND		1		mg/kg	C23-C24	ND		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	ND		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	ND		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	ND		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	ND		1		mg/kg
C17-C18	ND		1		mg/kg	C41-C44	ND		1		mg/kg
C19-C20	ND		1		mg/kg	C7-C44 Total	ND	5.0	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	123	45-149	

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

Calscience Environmental Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received:	11/20/01
Work Order No:	01-11-1097
Preparation:	Ext. + D/I
Method:	TPH - Carbon Range

Project: 6th Street Bridge

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-3-Surface	01-11-1097-9	11/19/01	Solid	11/20/01	11/22/01	01112006sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	2.5		1		mg/kg
C8	ND		1		mg/kg	C23-C24	4.3		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	19		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	19		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	22		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	20		1		mg/kg
C17-C18	ND		1		mg/kg	C41-C44	28		1		mg/kg
C19-C20	2.3		1		mg/kg	C7-C44 Total	120	5	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	93	45-149	

B-3-10	01-11-1097-10	11/20/01	Solid	11/20/01	11/21/01	01112006sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	ND		1		mg/kg
C8	ND		1		mg/kg	C23-C24	ND		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	0.23		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	0.080		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	ND		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	ND		1		mg/kg
C17-C18	ND		1		mg/kg	C41-C44	ND		1		mg/kg
C19-C20	ND		1		mg/kg	C7-C44 Total	ND	5.0	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	121	45-149	

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

Calscience Environmental Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received:	11/20/01
Work Order No:	01-11-1097
Preparation:	Ext. + D/I
Method:	TPH - Carbon Range

Project: 6th Street Bridge

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-3-20	01-11-1097-11	11/20/01	Solid	11/20/01	11/21/01	01112006sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	ND		1		mg/kg
C8	ND		1		mg/kg	C23-C24	ND		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	0.40		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	0.072		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	ND		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	ND		1		mg/kg
C17-C18	ND		1		mg/kg	C41-C44	ND		1		mg/kg
C19-C20	ND		1		mg/kg	C7-C44 Total	ND	5.0	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	111	45-149	

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-3-30	01-11-1097-12	11/20/01	Solid	11/20/01	11/21/01	01112006sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	ND		1		mg/kg
C8	ND		1		mg/kg	C23-C24	ND		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	ND		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	ND		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	ND		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	ND		1		mg/kg
C17-C18	ND		1		mg/kg	C41-C44	ND		1		mg/kg
C19-C20	ND		1		mg/kg	C7-C44 Total	ND	5.0	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	117	45-149	

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

Calscience Environmental Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received:	11/20/01
Work Order No:	01-11-1097
Preparation:	Ext. + D/I
Method:	TPH - Carbon Range

Project: 6th Street Bridge

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-4-Surface	01-11-1097-13	11/20/01	Solid	11/20/01	11/22/01	0111200ssa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		5		mg/kg	C21-C22	ND		5		mg/kg
C8	ND		5		mg/kg	C23-C24	3.4		5		mg/kg
C9-C10	ND		5		mg/kg	C25-C28	30		5		mg/kg
C11-C12	ND		5		mg/kg	C29-C32	69		5		mg/kg
C13-C14	ND		5		mg/kg	C33-C36	74		5		mg/kg
C15-C16	ND		5		mg/kg	C37-C40	100		5		mg/kg
C17-C18	ND		5		mg/kg	C41-C44	140		5		mg/kg
C19-C20	ND		5		mg/kg	C7-C44 Total	420	25	5	D	mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	77	45-149	

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-4-10	01-11-1097-14	11/20/01	Solid	11/20/01	11/21/01	0111200ssa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	0.93		1		mg/kg
C8	ND		1		mg/kg	C23-C24	1.1		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	2.3		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	5.5		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	3.7		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	7.3		1		mg/kg
C17-C18	ND		1		mg/kg	C41-C44	12		1		mg/kg
C19-C20	ND		1		mg/kg	C7-C44 Total	33	5	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	125	45-149	

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

Calscience Environmental Laboratories, Inc.

ANALYTICAL REPORT

URS Corporation
2020 East 1st Street, Suite 400
Santa Ana, CA 92705-4032

Date Received:	11/20/01
Work Order No:	01-11-1097
Preparation:	Ext. + D/I
Method:	TPH - Carbon Range

Project: 6th Street Bridge

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-4-20	01-11-1097-15	11/20/01	Solid	11/20/01	11/21/01	01112006sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	0.63		1		mg/kg
C8	ND		1		mg/kg	C23-C24	ND		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	0.12		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	ND		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	ND		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	ND		1		mg/kg
C17-C18	ND		1		mg/kg	C41-C44	ND		1		mg/kg
C19-C20	ND		1		mg/kg	C7-C44 Total	ND	5.0	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	95	45-149	

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
B-4-30	01-11-1097-16	11/20/01	Solid	11/20/01	11/21/01	01112006sa

Parameter	Result	RL	DF	Qual	Units	Parameter	Result	RL	DF	Qual	Units
C7	ND		1		mg/kg	C21-C22	ND		1		mg/kg
C8	ND		1		mg/kg	C23-C24	ND		1		mg/kg
C9-C10	ND		1		mg/kg	C25-C28	0.54		1		mg/kg
C11-C12	ND		1		mg/kg	C29-C32	0.16		1		mg/kg
C13-C14	ND		1		mg/kg	C33-C36	ND		1		mg/kg
C15-C16	ND		1		mg/kg	C37-C40	ND		1		mg/kg
C17-C18	ND		1		mg/kg	C41-C44	ND		1		mg/kg
C19-C20	ND		1		mg/kg	C7-C44 Total	ND	5.0	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	122	45-149	

Client Sample Number:	Lab Sample Number:	Date Collected:	Matrix:	Date Prepared:	Date Analyzed:	QC Batch ID:
Method Blank	098-03-002-1,479	N/A	Solid	11/20/01	11/20/01	01112006sa

Parameter	Result	RL	DF	Qual	Units
TPH as Diesel	ND	5.0	1		mg/kg

Surrogates:	REC (%)	Control Limits	Qual
Decachlorobiphenyl	117	45-149	

RL - Reporting Limit , DF - Dilution Factor , Qual - Qualifiers

CALSCIENCE ENVIRONMENTAL LABORATORIES, INC.
7440 LINCOLN WAY
GARDEN GROVE, CA 92841-1432
TEL: (714) 895-5494 • FAX: (714) 894-7501

CHAIN OF CUSTODY RECORD

Date November 19, 2001

Page 1 of 2

LABORATORY CLIENT: URS CORP

ADDRESS: 911 Wilshire Blvd, Ste 800

CITY Los Angeles STATE CA ZIP 90017

TEL (213) 996-2200 FAX: E-MAIL:

CLIENT PROJECT NAME / NUMBER: 6th Street Bridge

PROJECT CONTACT: Murray Wallis

SAMPLER(S): (SIGNATURE)

P.O. NO.:

LAB USE ONLY 11-7093

COOLER RECEIPT

TEMP = ______ °C

TURNAROUND TIME *
☐ SAME DAY ☐ 24 HR ☐ 48 HR ☐ 72 HR ☐ 5 DAYS ☐ 10 DAYS

SPECIAL REQUIREMENTS (ADDITIONAL COSTS MAY APPLY)
☐ RWQCB REPORTING ☐ ARCHIVE SAMPLES UNTIL ___/___/___

SPECIAL INSTRUCTIONS
* Need results by 10AM NOV 26, as discussed with Mary Carroll

REQUESTED ANALYSES

LAB USE ONLY	SAMPLE ID	LOCATION/DESCRIPTION	SAMPLING DATE	SAMPLING TIME	MATRIX	NO. OF CONT.	TPH (G)	TPH Full Carbon Chain	BTEX / MTBE (8021B)	HALOCARBONS (8021B)	VOCs (8260B) + MtBE	VOCs (5035 / 8260B) EnCore	SVOCs (8270C)	PEST (8081A)	PCBs (8082)	EDB / DBCP (504.1) or (8011)	CAC, T22 METALS (6010B)	PNAs (8310)	VOCs (TO-14A) or (TO-15)	CH₄ / TGNMO (25.1)	FIXED GASES (25.1) or (D1946)
1	B-1-surface		11-19-01	15:20	Soil	1		X									X				
2	B-1-10			15:45	Soil	1															
3	B-1-20			16:00	Soil	1															
4	B-1-30			1615	Soil	1															
5	B-2-surface			1635	Soil	1															
6	B-2-10			1655	Soil	1															
7	B-2-20			1710	Soil	1															
8	B-2-30			1720	Soil	1															
9	B-3-Surface			1745	Soil	1															
10	B-3-10		11-20-01	730	Soil	1															

Relinquished by: (Signature)	Received by: (Signature)	Date:	Time:
	KONGKA SAETUNG #3015	11/20/01	1115
X1 KONGKA SAETUNG	X2 UNIVERSAL COURIER.	11/20/01	12:25
	Received for Laboratory by: (Signature)	11/20/01	1225

DISTRIBUTION: White with final report, Green to File, Yellow and Pink to Client.

10/01/00 Revisio

APPENDIX J

SEISMIC RETROFIT STRATEGY MEETING MINUTES

MEETING MINUTES

Meeting Location: Caltrans HQ, 1801 30th Street
Conference Room 212, Sacramento
Project: Sixth Street
Date: April 26, 2004
Meeting Purpose: **Sixth Street Retrofit Strategy Technical Meeting**

Attendees:

Name	Organization	Phone#
Scott Straub	Caltrans SLA	(916) 227-8339
Jim Wu	City of LA BOE	(213) 847-9446
John Koo	City of LA BOE	(213) 847-5625
Dan Weddell	W. Koo & Associates	(714) 456-0280
Earl Seaberg	Caltrans SLA	(916) 227-8745
Fadel Alameddine	Caltrans OEE	(916) 227-8512
Shannon Mlcoch	Caltrans DLA/HQ	(916) 653-6750
Reza Fereshtehnejad	Caltrans DLA/HQ	(916) 651-6876
Xiaoyun Wu	W. Koo & Associates	(714) 456-0280
Rob Dowell	DH Engineering	(619) 265-8717
Sami Megally	W. Koo & Associates	(714) 456-0280
Wei Koo	W. Koo & Associates	(714) 456-0280

The City of Los Angeles and their consultants presented a summary of the seismic retrofit strategy developed for the Sixth Street Viaduct over the Los Angeles River. The presentation included a summary of previous work completed, as-built analysis, and five retrofit strategies. Analysis of the main river spans was also presented along with the proposed retrofit strategy. Finally, two replacement options were presented for comparison with the retrofit strategy costs. The discussion during the presentation is given below.

Item	Discussion
1.	Earl wanted to know the construction schedule and how it relates to continuing ASR degradation. John, City's goal is to fix the bridge as soon as possible.
2.	Earl Asked if the City has been tracking the ASR progression? The City has not kept track of ASR damage propagation by measuring cracks or other means.
3.	Fadel noted that the shear stress is relatively small, why do a few columns fail in shear? Why are some different than others? Is shear failure the main reason for retrofitting the columns? Dan answered that only a few columns fail in shear. The columns that fail in shear are all rated severe concrete degradation. For this concrete state, it was assumed that the outer 18" of concrete is not effective, thus only the middle portion of the concrete resists shear. Thus, the smaller diameter middle columns exhibited shear stress because of the greater reduction in relative area. The larger outer columns with severely deteriorated concrete still have a significant middle cross section to resist the shear. With fully competent concrete, shear would not be a problem. The majority of columns are retrofitted with steel casings to enhance the rotation ductility of the columns.
4.	Fadel asked why footing retrofits are needed for Alternative 4. Typically footing retrofits are to be avoided or at least reduced. Wei, The footing retrofits are required to control the longitudinal displacements. The number of footing retrofits may be reduced in the final design if this alternative is chosen, but the footing costs are small compared to the overall cost if alternative 4, which is much greater than alternative 2.

	Thus this alternative is unlikely to be chosen, even if the footing costs were lowered.
5.	Fadel asked the reason for infill shear walls on the outer river piers. Rob explained that the infill walls activate the two river edge piers, reducing the overall displacement of the structure. This also reduces the demands on the center river pier below their capacity. Reducing displacement demands on the superstructure is less expensive than upgrading all of the riveted steel superstructure elements.
6.	John Koo said that the City of Los Angeles wants to move the project along with alternative 5 or 6. He also mentioned that 6B will change the City's skyline which may not be acceptable to some people.
7.	Shannon asked about the cost of ROW. Dan and Wei, Alternative 6A ROW is not significant since the structure footing is well within the ROW and the edge of bridge should be at the edge of the ROW line. Alternative 6B may require some ROW for footing work, but not along the entire length of the bridge.
8.	Shannon asked about the cost contaminated soil and environmental mitigation measures. Dan, The contaminated soil mitigation costs are included in the construction cost estimates. Environmental mitigation is not included in any alternative.
9.	Reza asked if the structure is on the EBL and whether the replacement alternatives would remove the structure from the EBL. Dan, yes Sixth Street is on the FHWA EBL due to roadway width. The replacement options would remove the structure from the EBL.
10.	Shannon said she does not see the point to address the retrofit without ASR issues. That being said, it appears that only Alternatives 5 & 6 are viable.
11.	Scott will discuss the report with Fadel this week and offer written comments next week.
12.	City to incorporate comments before the report is sent to FHWA.
13.	Earl suggested a smaller presentation be made to FHWA. Don't overwhelm them with one big presentation.

Minutes prepared by City of Los Angeles Bridge Improvement Program Date April 27, 2004

cc: Participants