Seismic Evaluation and Retrofit of Fruitvale Avenue Railroad Bridge

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SUMMARY:
Fruitvale Avenue Railroad Bridge spans the Oakland estuary, connecting Alameda and Oakland, CA. This unusual structure is a 214 foot span, steel truss with lattice framing, vertical lift bridge with concrete approaches and foundation. The objective of this project was to determine an effective method to seismically retrofit the bridge structure and foundation. Due to severe overstressing of the bridge framing under seismic loads, the best strategy for retrofitting the structure was to seismically isolate it from its foundation. Retrofitting using passive energy dissipating elements such as sliding friction devices has the characteristics most needed for an effective reduction in structure demand. The results of this project confirmed that selecting a frictional rocking system as a retrofit strategy for the bridge changed the seismic response of the bridge significantly. The demand on the structure was reduced to acceptable values.

Keyword: *Lift Bridge, Seismic Analysis, Retrofit, Base Isolation*

1. INTRODUCTION

Fruitvale Avenue Railroad Bridge, built in 1951, spans the Oakland estuary in Oakland Inner Harbor, connecting Alameda and Oakland, CA. It is located at the eastern end of the channel where the estuary becomes a narrow tidal canal. In 1989, the bridge sustained relatively minor damage due to the Loma Prieta earthquake and was subsequently repaired. The bridge is owned by and this investigation was done under contract with the U.S. Army Corps of Engineers, San Francisco District.

Figure 1.1. Fruitvale Railroad Bridge, Lift Span at Mid Position

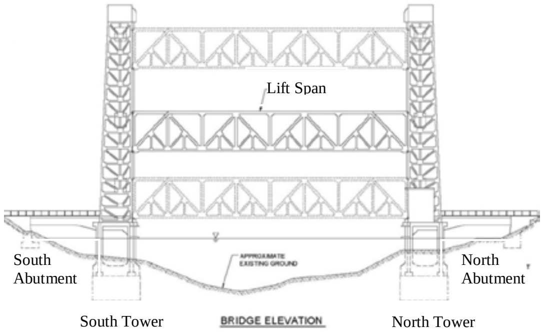


Figure 1.2. Fruitvale Railroad Bridge, Elevation View

The structure consists of a concrete foundation, steel superstructure (towers and lift span) and concrete approach spans, and mechanical and electrical systems to operate the lift span. The bridge is a single-track, steel through-truss, vertical lift bridge with a single span, and ballasted deck concrete approaches. The lift span deck uses wood ties on steel beams. It was designed to carry a Cooper E-50 live load. The lift span is 214 feet long with through trusses centered at 18 feet. Including the approach spans the bridge has an overall length of 377 feet. The 185 feet tall towers provide a maximum vertical clearance of 135 ft above Mean Higher High Water (MHHW) with the lift span fully raised, and a minimum clearance of 13 ft. above MHHW (Fig.1.1 and Fig.1.2).

A number of activities and investigations were carried out to assess the condition of the bridge and methods to retrofit the structure. This work included evaluating the condition of the concrete, both underwater and above water; evaluating the condition of the steel superstructure, electrical equipment and mechanical equipment, wire ropes, and investigating the soil conditions. Based on the visual and physical inspection performed, the underwater portions of the tower piers are in good condition. The concrete foundation above water has cracking and spalling and is in need of repair. Overall, the steel superstructure is in good condition with general coating system failure exposing members to corrosion. The counterweights, balance chain, and wire ropes were noted to be in good condition. Following the condition assessment, the bridge was analyzed to determine its performance using current code requirements. The bridge does not satisfy current vertical loading criteria and does not have adequate capacity under combined dead load and Cooper E50 train loading for which it was originally designed. The lack of capacity is primarily due to recently developed methods for calculating the capacity for the type of framing used to construct the bridge (lattice framing). Seismic analysis of the bridge was also required to determine if the bridge meets the current seismic loading requirements and if the structure would collapse from such an event. Due to the bridge's location in a high seismic risk zone, an extensive seismic analysis was performed to evaluate the global stability of the bridge. This paper presents only the seismic evaluation and retrofit of the steel superstructure of Fruitvale Avenue Railroad Bridge.

2. SEISMIC ANALYSIS OF THE BRIDGE

The primary focus of this work was to determine the level of performance of the existing bridge under present code earthquake loading. Dynamic characteristics of the bridge were determined using eigensolution technique and these results were used to perform a response spectrum analysis of the bridge, followed by a time history analysis of the counterweight system using only the vertical acceleration time history. It was assumed that the vertical dynamic effect of the counterweights was decoupled from the bridge response during a seismic event and was added linearly to the seismic response of the bridge. Fig. 2.1 shows the schematic view of the bridge modeled in SAP 2000. Based on AREMA (American Railway Engineering and Maintenance-of-Way Association) requirements, three ground motion levels were used for seismic assessment of the bridge (Table 2.1).

Table 2.1. Bridge Response Levels for Selected Ground Motions (AREMA)

Response Level	Ground Motion Level	Expected Damage to Track, and Structure
I	1 (72 yr. event)	Very low probability of damage or speed restriction
II	2 (475 yr. event)	Moderate damage which may require temporary speed restriction
III	3 (1000 yr. event)	Heavy to severe damage which may require major rehabilitation. Track or structure may be out of service for an infinite period of time

The probabilistic seismic hazard spectra for three levels: 5% in 50 years 10% in 50 years, and 50% in 50 years (corresponding return periods are 1000, 475, and 72 years, respectively) using New Generation of Attenuation Relationships and considering near-fault directivity effects were developed. Since the Fault Normal (FN) direction is closely in line with the bridge long axis, the FN spectrum is applicable to the longitudinal direction of the bridge, and (Fault Parallel) FP spectrum is applicable to the bridge transverse direction. Fig. 2.2 shows the plots of the horizontal and vertical spectral accelerations for the 1000-, 475-, and 72-year return periods.

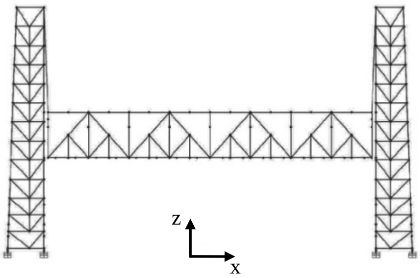


Figure 2.1. Schematic Elevation View of Fruitvale Ave Bridge, at Mid-Level Position

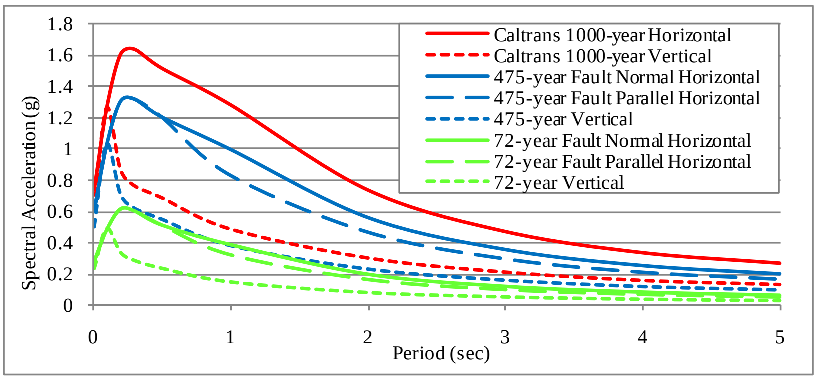


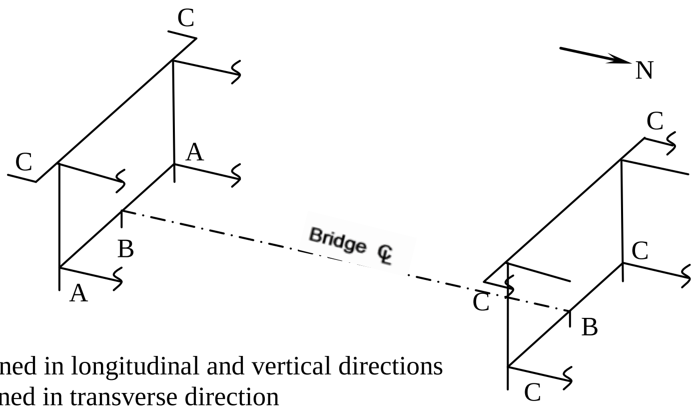
Figure 2.2. Response Spectra for 1000 yr., 475 yr., and 72 yr. Return Period Events

2.1. Member Properties and Connections

All truss members in the lift span and the towers were modeled as beam elements. The section properties of the latticed members were determined using the same approach used for latticed members in San Francisco-Oakland Bay Bridge (Duan, Reno, and Lynch).The wire ropes were modeled using beam elements with uniaxial material properties and zero strength in compression.

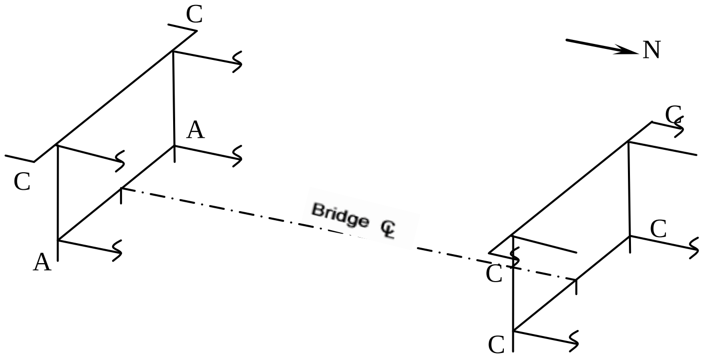
2.2. Boundary Conditions

The bridge pier foundation is very stiff and massive compared with the steel superstructure and could be simplified as a fixed-end condition in modeling. Therefore, the bridge could be modeled without its pier foundation and approach spans. However, for assessing the approach spans and pier foundations, the entire bridge was modeled. Fig. 2.3 and Fig. 2.4 illustrate the connections of the lift span to tower at the fully lowered position and the raised positions, respectively. The connections of the lift span to each tower were modeled by constraining the lift span to towers for specific degrees of freedom as shown in the figures. In the fully lowered position, the lift span is seated on chairs supported on the concrete piers. These chairs are designed to lock the span thus eliminating vertical movement and longitudinal movement (Boundary Condition A). There is also a shear key built into the concrete piers preventing transverse movement (Boundary Condition B). In the raised positions all but two corners of the lift span are restrained from sideway movement by wheels which ride along the tower columns (Boundary Condition C). There are two corners at the lift span lower level on the south end which have wheels which also restrain the lift span from longitudinal movement (Boundary Condition A).



Notes:
Boundary Condition A: restrained in longitudinal and vertical directions
Boundary Condition B: restrained in transverse direction
Boundary Condition C: restrained in transverse direction

Figure 2.3. Boundary Conditions of Lift Span-Tower Connections at Fully Lowered Position



Notes:
Boundary Condition A: restrained in longitudinal and transverse directions
Boundary Condition C: restrained in transverse direction

Figure 2.4. Boundary Conditions of Lift Span-Tower Connections at Raised Position.

2.3. Loading

To help assure acceptance of the bridge retrofit by railway companies, the AREMA Manual for Railway Engineering service design approach was used to analyze the bridge. AREMA has specific load combination requirements for vertical lift bridges. Since the future use of the bridge is uncertain and therefore to help ensure that a seismic retrofit would be acceptable to other state and load jurisdiction, the Caltrans, BART (Bay Area Rapid Transit), and California Building Code seismic requirements were reviewed to confirm that the approach selected would meet these codes. Based on AREMA special provisions for vertical lift bridges (section 6.3.15), load combinations were considered for the following conditions:
1- Bridge open (lift span raised)
2- Bridge closed (lift span lowered)
3- Bridge closed, with counterweights supported independently
Therefore, the considered load combinations for seismic analysis are as presented in Table 2.2.

Table 2.2. Considered Load Combinations for Seismic Analysis

Group A: Bridge is Open (lift span raised)	
1	D + EQ
Group B: Lift Span is Fully Lowered	
2	D + EQ
Group C: Lift Span is Fully Lowered	
3	D + EQ (counterweight independently supported)

2.4. Eigensolution

Eigenvector analysis was performed to determine the undamped, free-vibration mode shapes and natural frequencies of the bridge. The number of modes considered in the analysis to capture at least 90% mass participation in the longitudinal and transverse direction was about 30 modes. The results of the eigensolution for the first three modes of the bridge with fixed base are presented in Table 2.3. Selected mode shapes of the bridge when the lift span is at mid position are shown in Fig. 2.5.

Table 2.3. Modal Periods for Fixed Base Condition

Mode	Period (sec)		
	Lift Span-Low	Lift Span-Mid	Lift Span-High
1	1.523	1.327	1.528
2	1.469	1.180	1.485
3	1.457	1.173	1.174

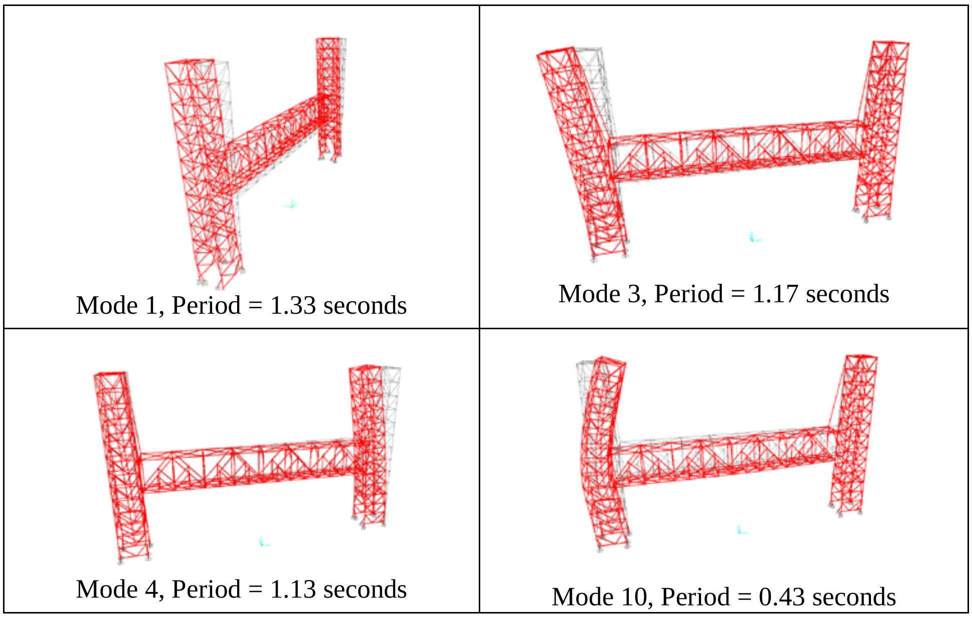


Figure 2.5. Selected Modes Shapes, Lift Span at Mid Position

2.5. Time History Analysis of the Bridge Counterweight

To consider the dynamic effect of the counterweight on the bridge towers a time history analysis of the counterweight system was performed using the vertical acceleration time history. It was assumed that the dynamic effect of the counterweight is decoupled from the seismic response of the bridge structure. Therefore, the results of the time history analysis of the counterweight were linearly combined with results of the spectral response analysis of the bridge.

Fig. 2.6 shows the three sets of spectrum-compatible time histories of the vertical component for the 1000 year event. The time history analyses were performed for three different positions of the lift span and for 3 different earthquake levels. Three positions were used to simulate the changes in length of the wire ropes. When the lift span is at the highest position, the wire rope supporting the span is the shortest. In this situation, the wire rope will be stiffer than when it has a long length (lift span in mid or lowest positions). For example, the wire rope stiffness decreases by a factor of almost 14 from approximately 41 million lb/ft with the counterweight at the top to 3 million lb/ft when at the bottom position. The counterweight system was modeled in SAP by using link elements. Each link element was defined by a stiffness (spring) and a damping ratio (dashpot). The schematic of the model is shown in Fig. 2.7.

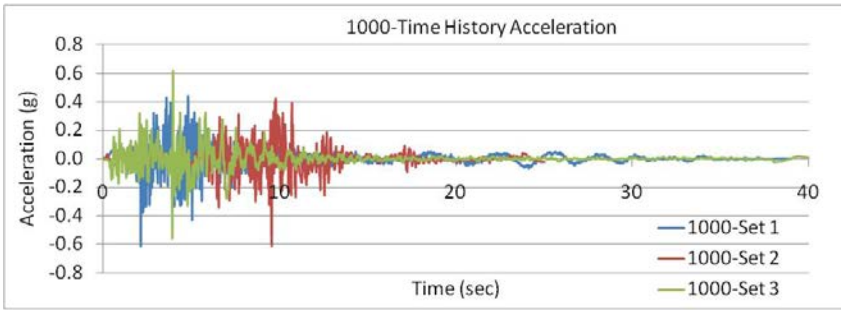


Figure 2.6. Spectrum-Compatible Time Histories for Vertical Components (1000-year)

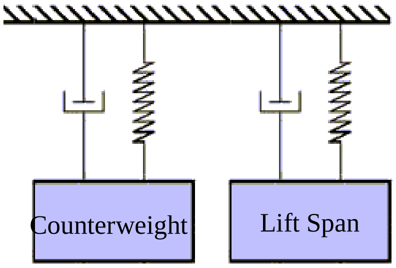


Figure 2.7. Schematic of Counterweight System Model

The maximum force produced in each position was similar for each of the earthquake return periods (72-, 475-, and 1000-year). The results showed that the force increased over the dead weight approximately by 22%, 54%, and 65% for 72-, 475-, and 1000-year events, respectively.

2.6. Response Spectrum Analysis:

The response spectrum analysis was performed considering a combination of 100% of the forces in one direction, 30% of the forces for the perpendicular direction, and 100% of vertical direction as follows:
Load Case 1: 100% x + 30% y +100% z
Load Case 2: 30% x + 100% y +100% z
The maximum displacements at the top of the tower for 475-year event are presented in Table 2.4.

Table 2.4. Maximum Displacements at Top of the Tower-Response Spectra Analyses

Load Case	Lift-Span-fully lowered (inch)	Lift-Span-mid position (inch)	Lift-Span-fully raised (inch)
Load Case 1	20.4 (in x direction)	22.1 (in x direction)	21.4 (in x direction)
Load Case 2	16.5 (in y direction)	18.5 (in y direction)	18.3 (in y direction)

Members were checked for the above-mentioned load combinations. For the 72- year event, about 50% of the tower vertical members (corner columns) are shown to yield. It was observed that when the lift span is at the fully lowered position the number of members with utilization ratios above 1.0 is less than when the lift span is at mid and fully raised position. For 475-year and 1000-year events, nearly 80% to 100% of towers legs, tower diagonal braces and horizontal members are overstressed. The top and bottom diagonal braces in the lift span also have demand/capacity ratios greater than 1.0. It should be noted that, some of the tower leg members close to the base of the towers experience very high utilization ratios (> 8.0). The maximum utilization ratio (demand force / capacity) in the wire ropes (total of 8 ropes) from Earthquake and Dead Load is 0.4 and it occurs when the lift span is at the fully raised position. Therefore, the capacity of the wire ropes is adequate.

In summary, the structure does not satisfy code requirements for the seismic loading, and some members exhibit very high utilization ratios. To resolve this situation, different retrofit approaches were investigated.

3. RETROFIT CONCEPTS

The wide-ranging extent of work needed for a conventional retrofit such as supplemental piers, significant strengthening of the tower members by building up sections, adding bracing and reconstructing connections, led to investigating seismic-isolation of the towers as a retrofit solution. Retrofits using controlled rocking with friction energy dissipators have the characteristics most needed for an effective reduction in structure demand. Releasing of the tower-to-pier foundation anchorage connections' tensile capacity (or allowing the connections to fail) would enable the steel towers to rock on their foundation, effectively increasing its period and partially isolating the pier. Adding passive energy dissipation devices at the uplift location would restrain the uplift displacements while providing additional energy dissipation. Sliding bolted joints are relatively simple to construct and provide many cycles of ductile energy dissipation with little or no strength degradation. As the towers rock back and forth, the friction connections serve as hysteretic friction dampers. A capacity design procedure and conservative assessment of maximum force demands are needed to ensure that non-ductile elements can remain elastic and that all inelastic action occurs in the specially detailed ductile structural elements. The anchorage connections should be designed to be capable of transferring the horizontal base shear. To ensure the satisfactory seismic performance of this retrofit approach, a number of design constraints should be included such as: 1) Tower drift limits to prevent excessive P-Δ effects on seismic behavior, and system overturning instability; and 2) Maximum developed dynamic force within tower and foundation (capacity check). A schematic diagram of the proposed friction connection for the latticed tower column is shown in Fig. 3.1 and Fig. 3.2.

The slots allow the tower legs to move vertically relative to the plate anchored to the foundation. The tension force in the bolt determines the friction between the plate and the tower leg, and therefore the force required for the tower leg to move in a controlled manner and dissipate energy. The bolts also restrain horizontal movement of the tower legs and allow rotation.

To model this connection, nonlinear kinematic springs were connected to the bottom of the towers to allow movement in the vertical direction. The tower legs were restrained from movement in horizontal directions. The links provided 50 kips of resistance in tension at each tower leg. The links are very stiff in compression. The hysteresis diagram for the spring at the bottom of the southeast tower column is shown in Fig. 3.3.

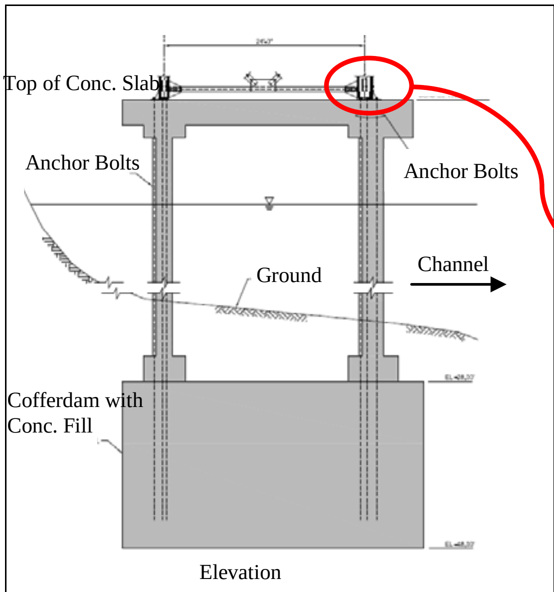


Figure 3.1. Foundation Anchorage Elevation

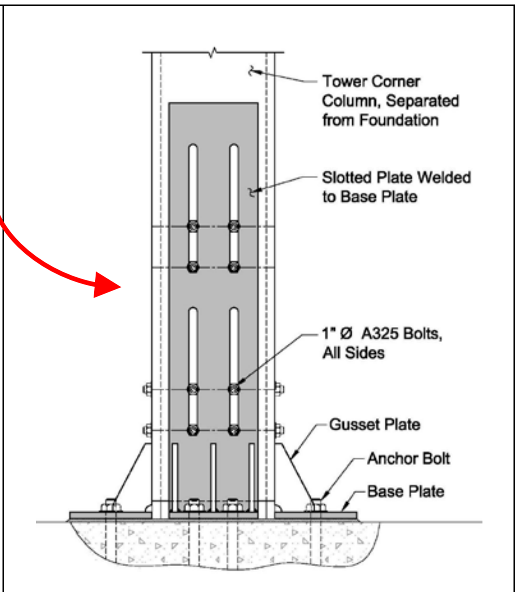


Figure 3.2. Tower Base Friction Connection

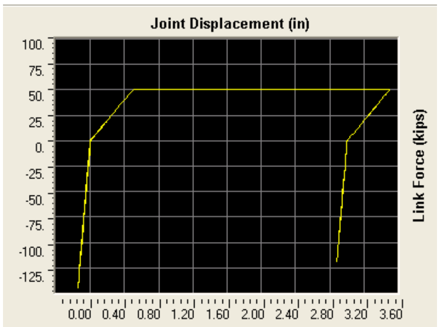


Figure 3.3. Retrofit Model Spring Hysteresis

Eigenvector analysis was performed on the retrofit model to determine the mode shapes and the mass participation corresponding periods for each mode shape. The periods of the structure are presented in Table 3.1. As can be seen in the table, there is over 80% cumulative mass participation for the x direction at mode 3. For the y direction, there is 90% participation at mode 1.

Table 3.1. Retrofit Model-Modal Mass Participation and Periods

Mode	Periods (sec)	Cumulative Mass Participation	
		UX	UY
1	5.16	0.0%	89.0%
2	4.54	52.5%	89.0%
3	3.96	82.9%	89.0%

To verify that the connection will respond as designed a series of time history analyses were performed. The scope of this project did not include the development of horizontal time history data. Therefore, a Contingency-Level Earthquake, 475-year return period horizontal time history from another project located in Long Beach, California was used. The unscaled and scaled by a factor of 1.25 of the horizontal time histories were considered (Fig. 3.4 and Fig.3.5). A series of sensitivity analyses on stiffness and tensile resistance of the connection were conducted to generate design force and displacement envelopes to assure the slip connection at the base of towers would function properly. As expected, the softer model underwent greater drift with lower base shear and member forces than the stiffer model. It was found that a connection with 50 kips ultimate friction capacity and stiffness of 100 kip/inch generates member forces and displacements within acceptable ranges.

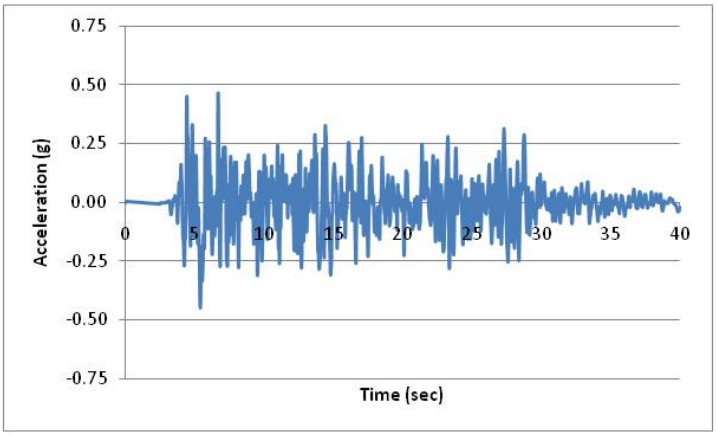


Figure 3.4. Horizontal Time History for Y-Direction Load Case

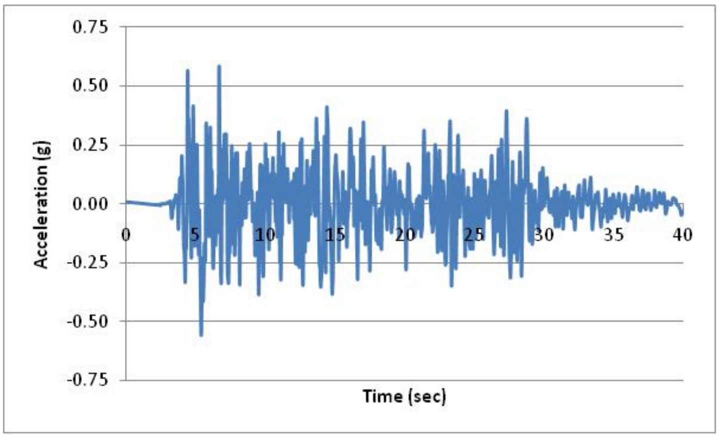


Figure 3.5. Horizontal Time History for X-Direction Load Case

The results showed that the maximum displacements at the top of towers are reduced from the fixed base response spectra analyses (Table 3.2). The maximum rotation at the base of towers is less than 2 degrees. The utilization ratios for the members are reduced significantly from the previous response spectra analysis. For example, the utilization ratio in one of the tower vertical members decreased from 8.26 to 1.38. This member can be retrofitted for this demand force by adding steel plates. The significant reduction in utilization ratios indicates that the resulting system response complied with the design intent. Selecting a rocking system as a retrofit strategy for the bridge would change the response of the bridge significantly. The effect of reducing the demand in the structure by allowing the bridge to rock using friction-base connections has the additional benefit of reducing the loads applied to the foundation. Since the studied model does not include the foundation system in the modeling, the amount of reduction is not evaluated.

Table 3.2. Absolute Displacement at top of Tower, 475-year Time History Analysis in X and Y Direction (Lift-Span-mid position)

	Analysis in X Direction* (longitudinal)		Analysis in Y Direction (Transverse)	
Direction	North Tower Displ.	South Tower Displ.	North Tower Displ.	South Tower Displ.
X (N-S)	12 inch	19.7 inch	4.5 inch	4.5 inch
Y(E-W)	0.1 inch	0.1 inch	15 inch	16.1 inch
Z (Vertical)	2.2 inch	3.2 inch	2.9 inch	3.1 inch

*The time history was scaled by a factor of 1.25

4. CONCLUSIONS

Fruitvale Avenue Railroad Bridge (designed in 1949), spans the Oakland estuary, and connects Alameda and Oakland, CA. In 1989, the bridge sustained some damage due to the Loma Prieta earthquake and was subsequently repaired. However, the bridge has remained closed. The objective of this study was to determine if the bridge meets the current seismic criteria and to develop an effective method to seismically retrofit the bridge structure and foundation.

Due to severe overstressing of the bridge framing under seismic loads, the best strategy for retrofitting the structure (rather than replacing it) is to seismically isolate it from its foundation. Adding passive energy dissipation devices at the uplift location would limit the uplift displacements while providing additional energy dissipation. It should be noted that the Loma Prietra earthquake caused the anchor bolts at the tower bases to pull out and stretch. This retrofit approach allows the bridge to respond in a similar manner, able it controlled rocking, while reducing member demands.

Nonlinear inelastic time history analysis was performed to assess the seismic behavior of the retrofitted bridge. A capacity based design procedure was used in sizing bolts and sliding plates. A series of sensitivity analyses on stiffness and tensile resistance of the connection were conducted to generate design force and displacement envelopes to assure the slip connection at the base of the towers would function properly. The results confirmed that selecting a rocking system as a retrofit strategy for the bridge changed the response of the bridge to the seismic loads significantly, and the demand on the structure was reduced to acceptable values.

ACKNOWLEDGEMENT

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