SEISMIC RESPONSE OF THE HWY 46/CHOLAME CREEK BRIDGE DURING THE 2004 PARKFIELD EARTHQUAKE

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Abstract

A Caltrans designed concrete slab bridge is located southwest of the 2004 M_w 6.0 Parkfield rupture zone. Peak horizontal accelerations of 1.0g and absolute displacements of 4 inches were measured with six CSMIP accelerometers on the bridge, and one free-field accelerometer east of the bridge. Ground motions resulted in longitudinal soil displacements in front of the abutments and around the bent piles due to the structure swaying back and forth. Based on our displacement analyses using the measured ground motions, we found that the current Caltrans seismic design approach results in a close match with the measured bridge displacements.

Introduction

The City of Parkfield is located within the northwest trending Cholame Valley between Hwy 101 and Interstate 5 in central California. The prominent geologic feature of the Valley is the highly active San Andreas Fault system that has ruptured multiple times in this area over the last 150 years. Moderate to large earthquakes were recorded in 1857, 1881, 1901, 1922, 1934, and 1966. Based on this high level of seismicity, the California Geologic Survey (CGS) installed over 50 strong motion accelerometer stations throughout the Cholame Valley to gather strong motion data resulting from an earthquake (Fig. 1). The accelerometers are installed in low, fiberglass huts anchored to a reinforced concrete pad bearing on the ground surface. The strong motion array is monitored and maintained by the CGS Strong Motion Instrumentation Program (CSMIP).

The State of California Department of Transportation (Caltrans) constructed the bridge along Hwy 46 in 1954, and widened it by 11 feet in 1979. The five-span, pile supported, reinforced concrete slab bridge is 130-ft long, 44-ft wide and spans Cholame Creek (Fig. 2 & 3). The original structure is supported on 16-inch diameter, CIDH piles at the bents and abutments. The widened portion is supported on driven 16-inch octagonal piles at the bents, and 10-inch square piles at the abutments. A monolithic, diaphragm abutment was used on the west end of the bridge. The east end of the original bridge is supported on a seat abutment with asbestos sliding sheets, while the widening consists of a diaphragm abutment. The 12-inch thick, reinforced concrete wingwalls are connected to the bridge deck. A 3-inch asphalt concrete overlay tops the 18-inch thick slab deck.

On the evening of September 30, 1955 the legendary actor James Dean was driving west on Hwy 46 in his silver Porsche Spyder 550 to race in Salinas, California. Unfortunately, another driver erroneously pulled directly in front of him at the intersection of Hwy 41 and Hwy 46 and Mr. Dean was killed in the resulting accident. If the other driver could have avoided the accident, James

Dean would have soon driven over the 1-year old Cholame Creek Bridge, towards an unknown future in Hollywood and beyond.

The bridge and abutments are underlain alluvial soils consisting of medium dense silty sands and stiff silty clays within the Cholame Creek drainage basin. The sedimentary Paso Robles Formation underlies the site with depth. A total of eight soil borings have been drilled at the site. Groundwater was apparently encountered at depths of 10 to 20-ft below ground surface at the time of the field investigations in 1953 and 1977, however the depth of the groundwater at the time of the earthquake is unknown. The site is classified as Type D soil conditions per Caltrans (2004).

On June 28, 1966 the San Andreas Fault ruptured to the southeast across Hwy 46, within 500 feet of the Cholame Creek Bridge. A free-field CSMIP accelerometer was in place (Cholame 2W) beside the bridge, and measured horizontal ground accelerations as high as 0.6g. Caltrans Bridge Inspection Reports after the earthquake note that the bridge deck shifted east approximately 1-inch at the seat abutment, the bridge approaches settled on the order of ½ inch, both wing walls at the east abutment had diagonal cracks, hairline cracks were present at the tops of the bent piles, and a ½-inch gap was present at the ground surface around the piles at Bent #5. The cracked wing wall on the southern edge of the east abutment was removed as part of the bridge widening in 1979.

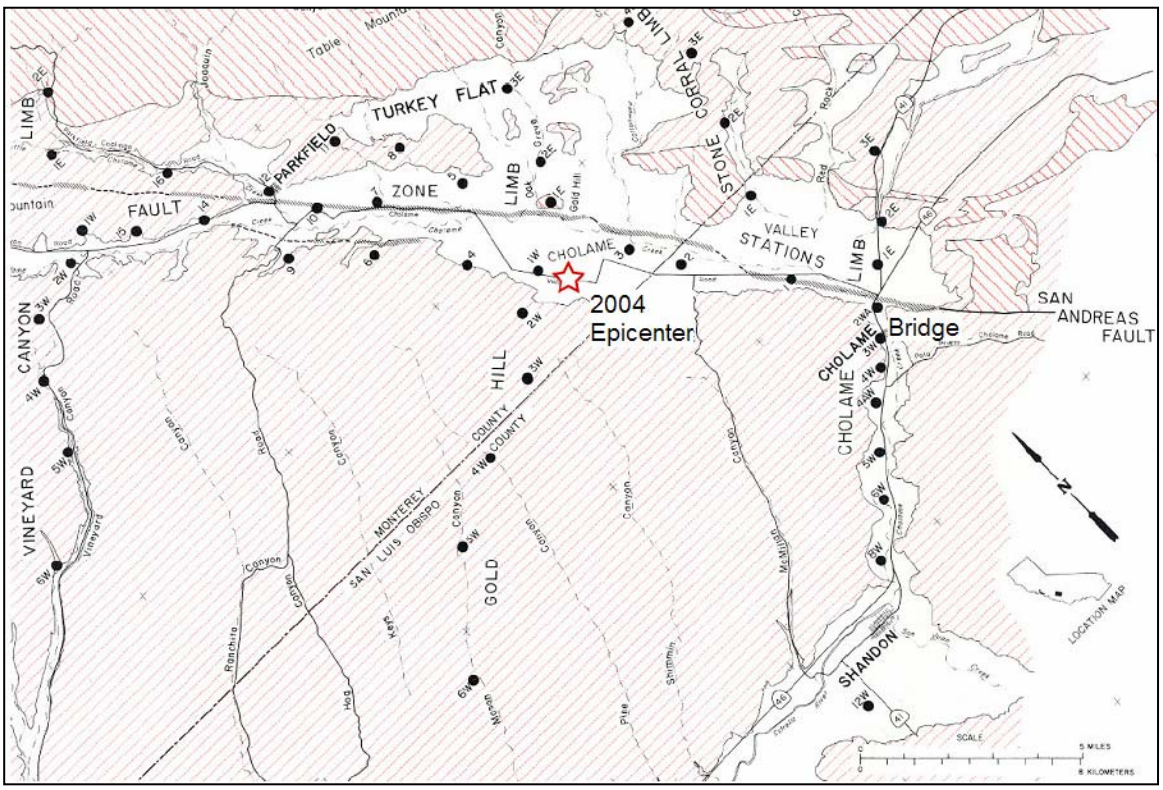

Figure 1. Bridge Location and Parkfield Strong Motion Array Locations (Original map by McJunkin and Shakal, 1989)

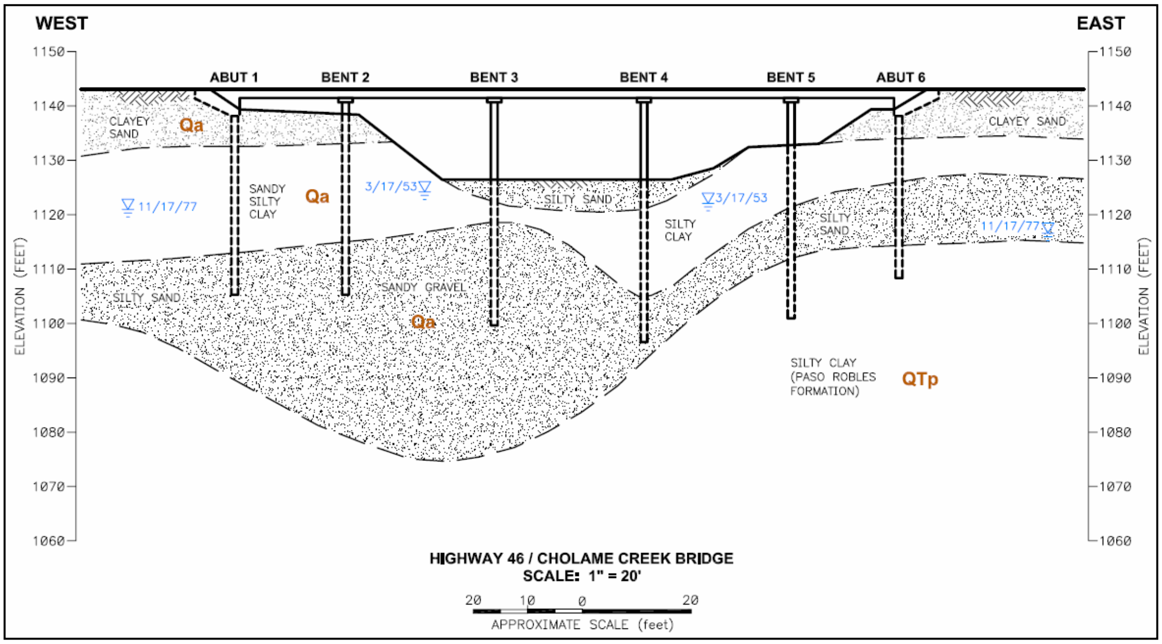

Figure 2. Bridge Elevation and Subgrade Conditions

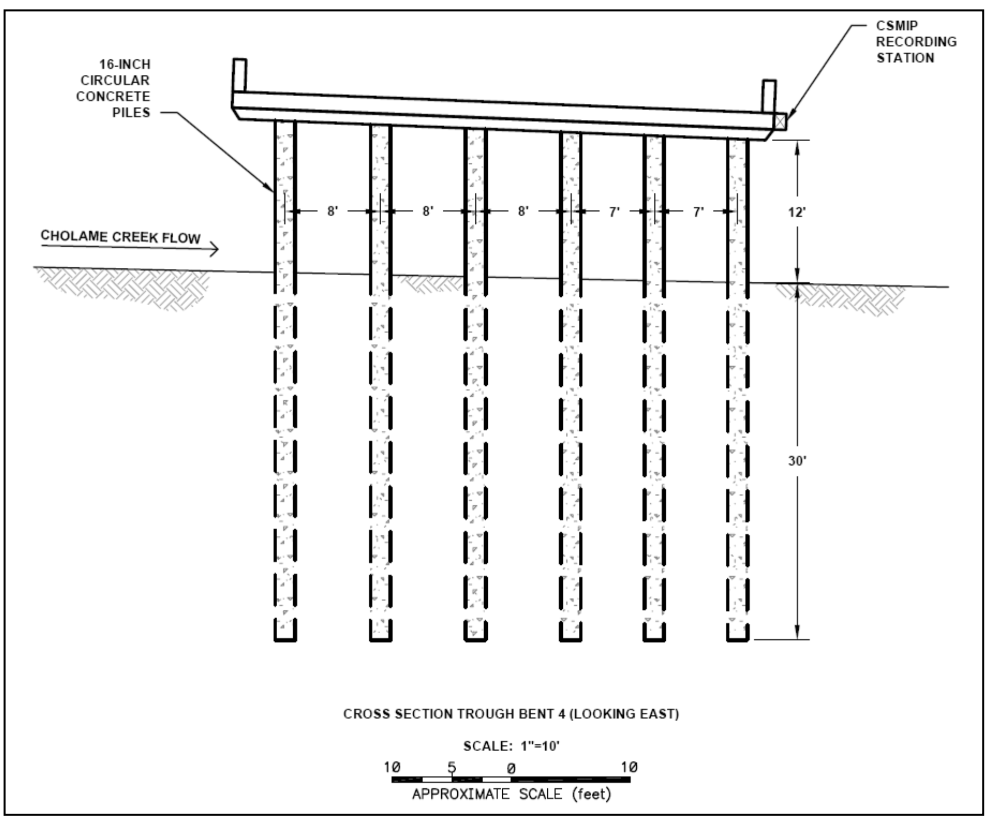

Figure 3. Bridge Cross Section

The magnitude M_w 6.5 San Simeon earthquake occurred on December 22, 2003 approximately 50 miles west of the site. The bridge deck experienced peak acceleration values of 0.2g and absolute displacements of approximately 1.4-inches during the far-field event.

The 2004 M_w 6.0 Parkfield Earthquake

On September 28, 2004 the highly anticipated Parkfield earthquake finally occurred. The right-lateral, strike-slip San Andreas Fault ruptured to the northwest (away from the Cholame Creek Bridge), with discontinuous surface rupture occurring over 20 miles along the fault. Surface rupture on the order of 1 to 2 inches was noted, and evidence of liquefaction was observed (sand boils) in two locations along Cholame Creek northwest of the bridge. While the active trace of the fault is only 500 feet to the east, the fault rupture zone did not pass by the site. The earthquake epicenter is shown on Fig. 1. Free-field acceleration time histories were recorded 200-feet east of the bridge at Cholame 2W (Station 36228), as well as on the southern edge of the bridge deck at the west abutment, center and east abutment (Station 36668). The Cholame 2W and bridge time histories recorded by the CSMIP are shown on Fig. 4 and 5. These time histories can be downloaded off the CSMIP web site.

Free-field motions were recorded in the north-south (360 deg) and east-west (90 deg) directions, while the bridge motions were recorded in the transverse and longitudinal bridge directions. As the bridge has little skew (4 degrees), and the centerline (~ N63°E) is essentially perpendicular to the strike of the San Andreas Fault, the longitudinal bridge direction is approximately fault-normal and the transverse bridge direction is approximately fault parallel.

The free-field, fault normal ground motion at the bridge included a directivity pulse with a peak displacement of 5.2 inches (13.3 cm) at t = 2.94 seconds, peak velocity of –21.9 in/sec (-55.5 cm/sec) at t = 3.24 seconds, and a peak acceleration of 0.53g at 3.34 seconds. These are relatively high values for a M_w 6.0 earthquake. It was interesting that the peak displacement preceded both the peak velocity and peak acceleration. Another interesting observation was that the displacement pulse occurred towards the fault to the east. Thus demonstrating that near-field pulses can come from both directions, regardless of the location of the project relative to the fault rupture zone. A summary of the peak accelerations, velocities and displacements recorded at Stations 36228 and 36668 are listed in Table I.

Table I. Summary of Peak Ground Motions at Bridge During 2004 M_w Parkfield Earthquake

Station	Acceleration (g's)	Velocity (in/sec)	Displacement (inches)
Cholame 2W, 90 deg	0.60 @ 3.34 sec	-24.9 @ 3.22 sec	4.9 @ 3.00 sec
Cholame 2W, 360 deg	0.37 @ 3.12 sec	-17.4 @ 3.00 sec	3.0 @ 2.84 sec
Bridge, Longitudinal	1.05 @ 3.34 sec	-20.4 @ 3.15 sec	4.3 @ 2.94 sec
Bridge, Transverse East Abutment	0.99 @ 3.36 sec	-17.1 @ 3.26 sec	2.0 @ 3.13 sec

Caltrans inspected the bridge the day of the earthquake and noted that the ground motions resulted in longitudinal soil displacements of 1 to 2-inches in front of the abutments, and approximately ½ inch around the rows of bent piles, due to the structure swaying back and forth. Structural damage consisted of diagonal cracking of the northern wing wall at the east abutment, minor cracking around the perimeter of all the bent piles at the connection to the bridge deck, and transverse cracking through the asphalt concrete at each bridge approach. Caltrans concluded that the damage was not serious and repairs were not necessary.

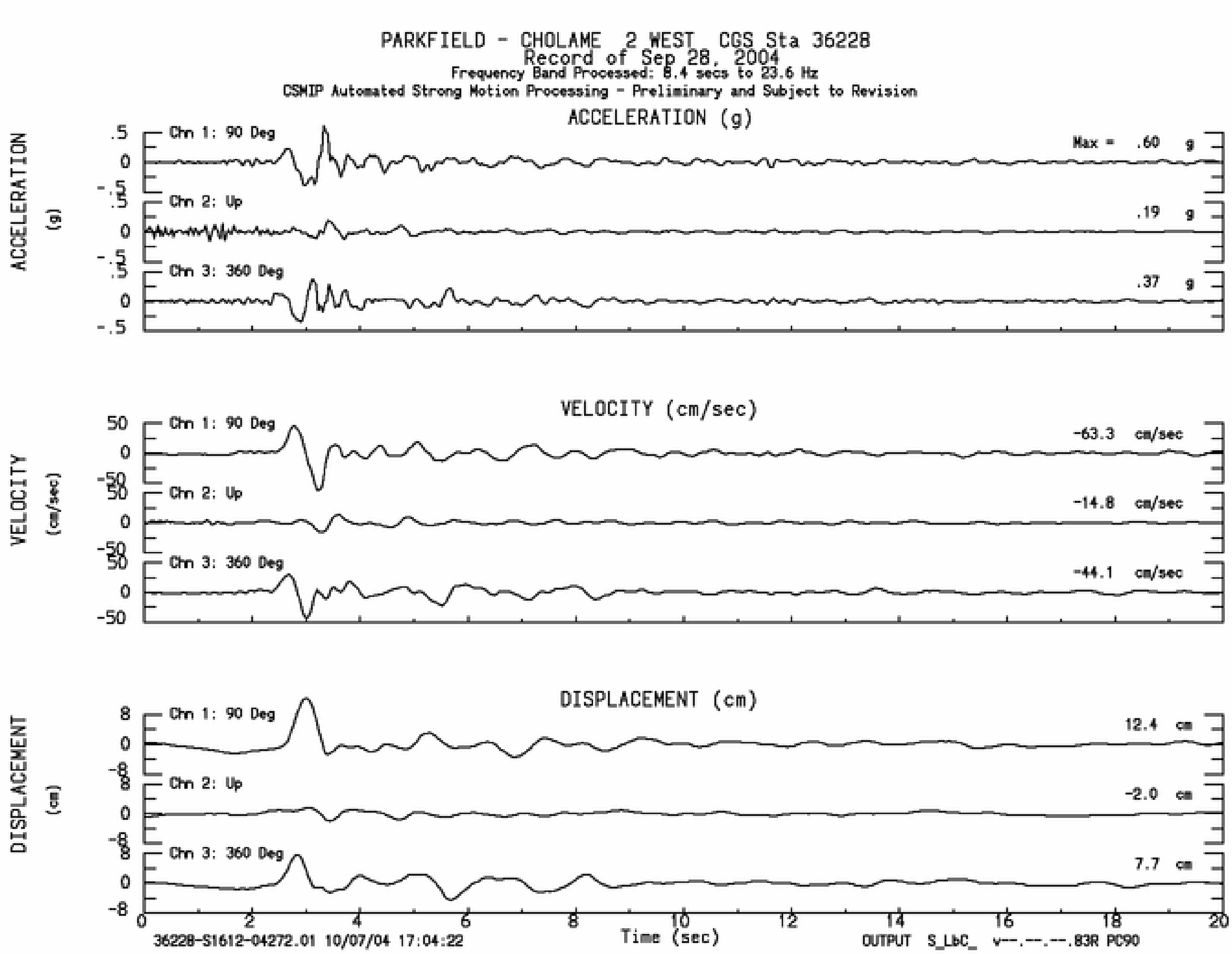

Figure 4 Cholame 2W Time Histories Recorded by CSMIP

During our August 26, 2005 site visit we noted circumferential, hairline cracks around the tops of all the piles. Epoxy repairs had been made on each of the Bent 2 piles. We reviewed all the Caltrans bridge inspection reports since bridge construction in 1954. Pile top cracks are noted in the March 22, 1971 report for nearly all the piles, indicating that the original cast-in-place piles were damaged in the 1966 seismic event. No pile cracking is noted for the new driven piles from November 1979 to June 2004. However, pile top cracking is noted for both the original and bridge widening piles in the September 28, 2004 report immediately after the recent earthquake. Thus, the pile top cracking observed in our field visit likely resulted from the two M_w 6.0 seismic events.

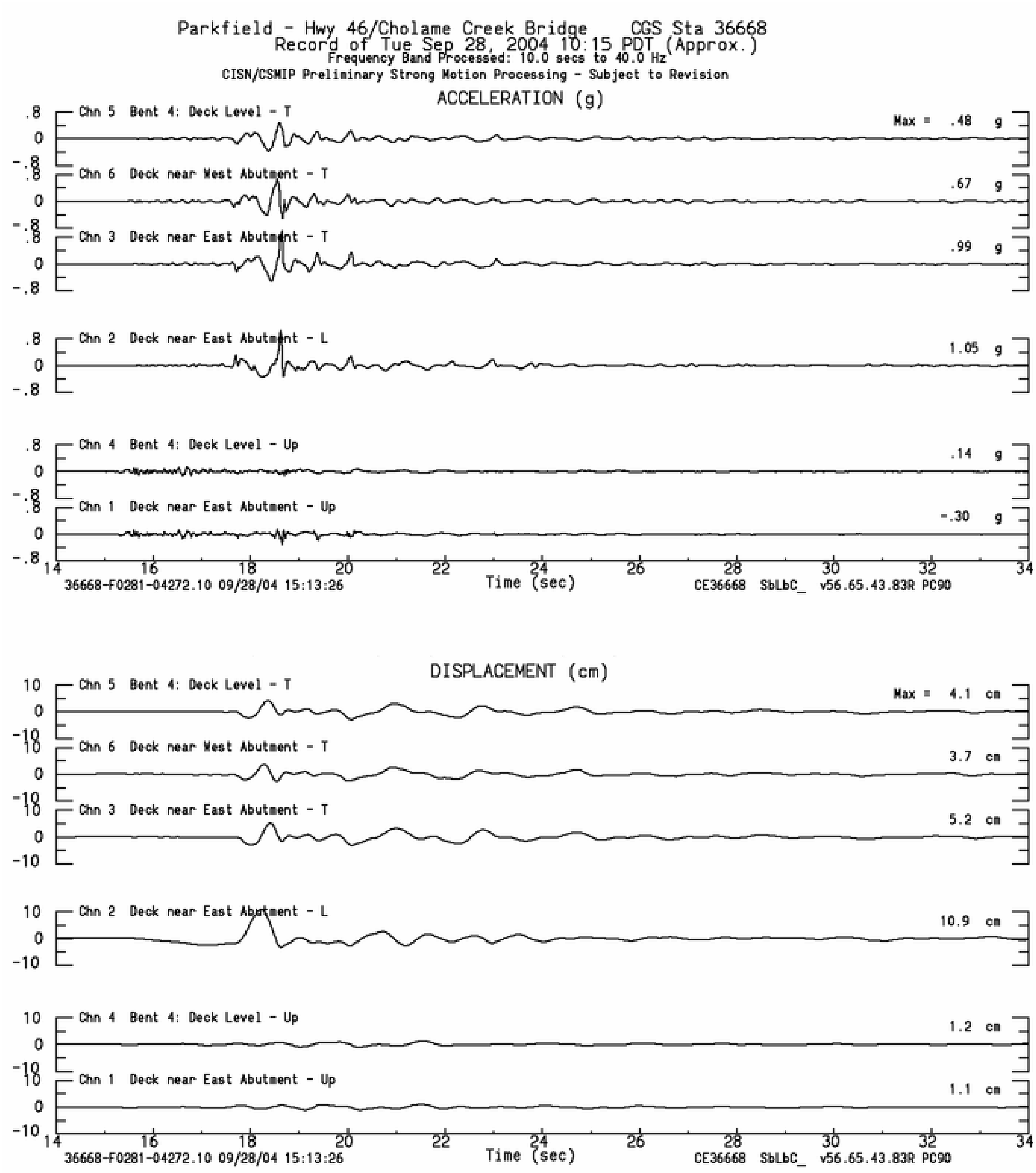

Figure 5 Hwy 46/Cholame Creek Bridge Time Histories Recorded by CSMIP

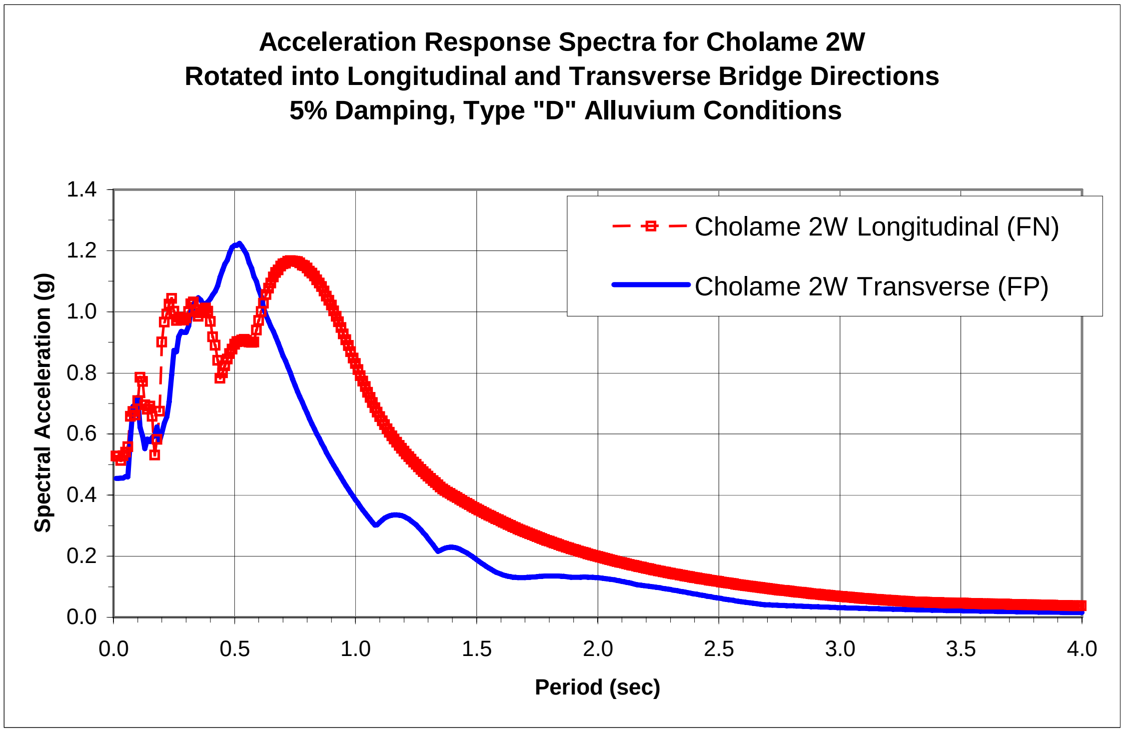

Figure 6. Cholame 2W Acceleration Response Spectra for 2004 M_w 6.0 Event

The free-field acceleration time histories from the Cholame 2W Station were rotated into the longitudinal (~ N63°E) and transverse (~ N27°W) directions of the bridge to develop site-specific acceleration response spectra for the analyses (Fig. 6). The time histories were downloaded off the CSMIP web page, and no further corrections were made. It is our understanding that the CSMIP time histories have already been frequency band processed and baseline corrected.

Bridge Displacement Analysis

Description of the Structural Model

A 3D dynamic finite element model was used to analyze the Hwy 46/Cholame Creek Bridge. A "spine" model was developed in the SAP2000 program with a single line of frame elements used at the cg of the superstructure. The bents were modeled with frame elements located at the cg of the cap, and frame elements for each column/pile. Effective section properties were used. The bridge profile, curved alignment and skewed bents and abutments were accounted for in the model.

The bent piles were first modeled with idealized boundary conditions - fixed at 5 pile diameters (6.7 ft) below the ground surface (bgs). Subsequently, the model was refined to include pile-soil springs to represent the soil stiffness. First linear soil springs were used to represent an equivalent secant stiffness, later non-linear springs were used. Soil springs were developed from p-y curves generated using the LPILE5 program.

The abutments were modeled with a pair of linear springs, longitudinal and transverse, at the cg of the bridge deck. We developed equivalent linear abutment springs by combining passive pressures against the abutment, and lateral forces from the piles. The passive pressures were based on the Caltrans recommended value of 20 kips/in per foot of wall width, modified for wall height.

The pile stiffness values (32 kips/in for 10-inch piles and 45 kips/in for 16-inch piles) were based on a force-displacement relationship developed using the software LPILE5 for the site-specific piles and soil conditions. The bridge model is shown on Fig. 7, 8 and 9. The abutment springs were confirmed to remain within their linear elastic range under loading from the 2004 M_w 6.0 event, by comparing the calculated lateral abutment forces to the assumed abutment capacity per typical Caltrans design guidelines.

The SAP2000 model was used to perform three basic types of analyses:

1. Response Spectrum Analysis (Elastic Dynamic Analysis per Caltrans)
2. Linear Time History Analysis
3. Non-linear Time History Analysis

For the response spectrum analysis, displacements were multiplied by a modification factor of 0.70 to account for increased damping (15%) due to a high level of soil mobilization at the piles and abutments. This is consistent with Caltrans (1995) MTD 20-4 for seismic retrofits of short, stiff bridges with monolithic abutments. For the time history analysis runs were made both with 5% and 15% damping ratio.

The site-specific ground motions from the Cholame 2W Station were input into the model as the seismic loading. Both response spectra (Fig. 6) and acceleration time histories were used. The ground motions were rotated into the longitudinal and transverse directions of the bridge.

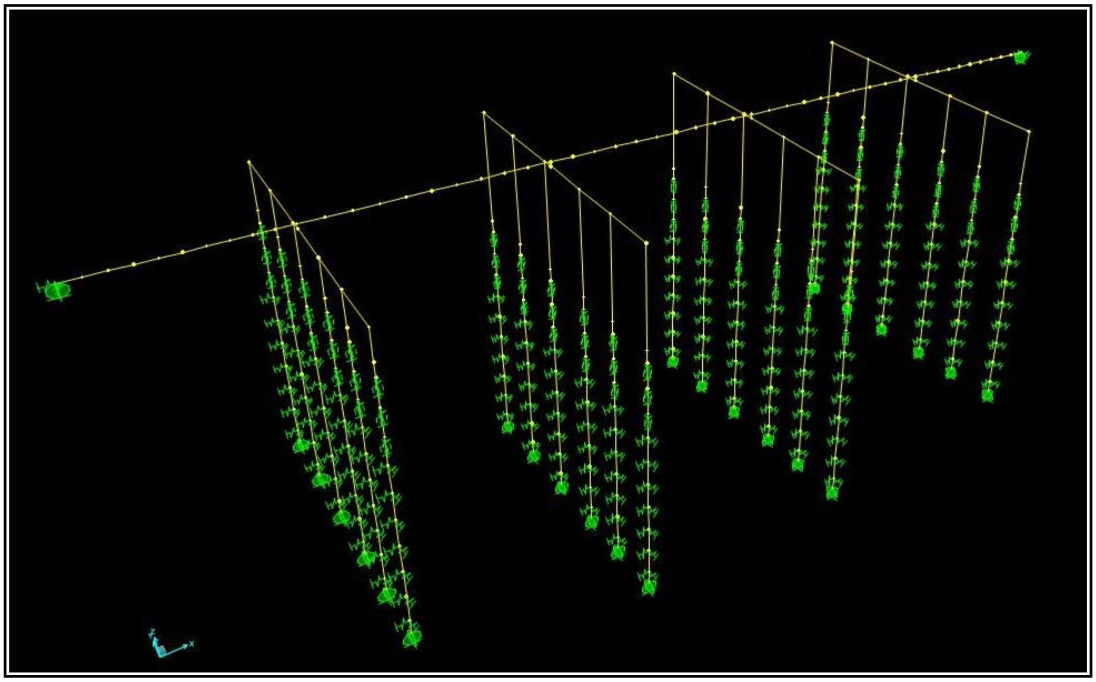

Figure 7. SAP2000 Model of Bridge Structure with Soil-Pile Springs

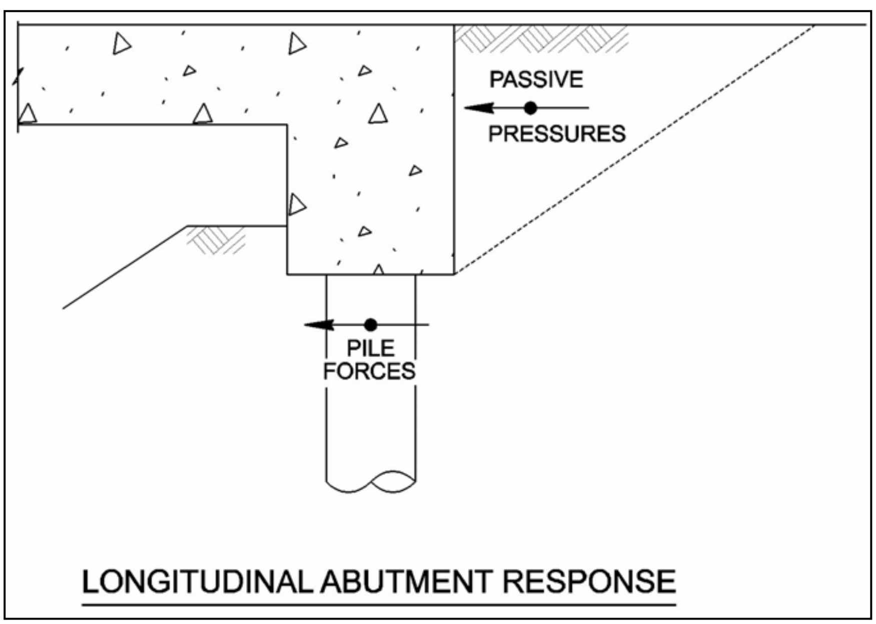

Figure 8. Longitudinal Forces at Abutment

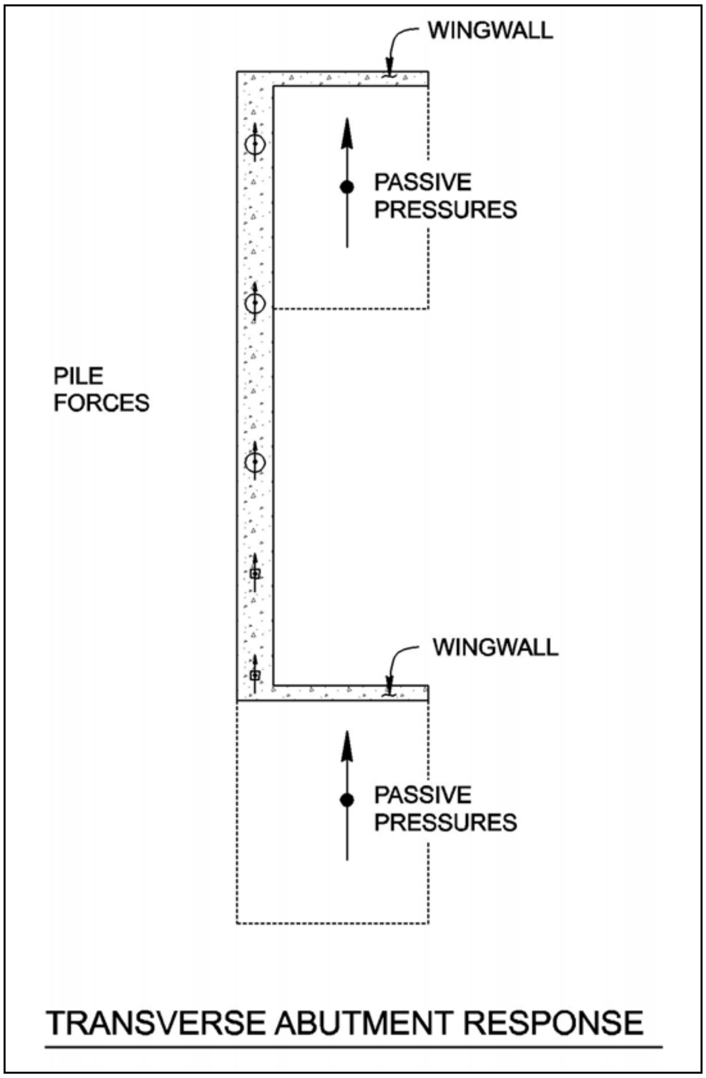

Figure 9. Transverse Forces at Abutment

Discussion of Results

The goal of the bridge analysis was to compare the measured bridge displacements with those calculated following typical Caltrans design guidelines with the Cholame 2W spectra. Accordingly, the first step was to estimate the relative bridge displacements of the bridge deck. The displacements listed in Table I are absolute values. The relative longitudinal and transverse bridge displacements are the absolute values minus the rotated free-field displacements measured at Cholame 2W. The peak relative bridge displacements at the east abutment (Abutment 6) are as follows:

- Longitudinal direction. 0.94 inches (2.38 cm) *at the same time as the peak longitudinal (fault normal) free-field displacement pulse* (t = 2.94 seconds). The relative displacement was only 0.35 inches (0.93 cm) at the time of the peak longitudinal bridge acceleration (t = 3.34 seconds).

- Transverse direction. 0.84 inches (2.14 cm) *at the same time as the peak transverse (fault parallel) bridge acceleration* at the east abutment (t = 3.36 seconds). The relative displacement was essentially zero inches (0.012 cm) at the time of the peak transverse free-field displacement (t = 3.12 seconds).

Comparisons between the free-field and bridge deck time history displacements are shown on Fig. 10 and 11.

Response Spectrum Analysis Results

Results showed over 90% mass participation each for the fundamental longitudinal and transverse modes, with periods of 0.38 and 0.37 seconds respectively. Running 25 modes provided over 99% mass participation each in the two lateral directions. Modeling assumptions were varied in the form of foundation and abutment stiffness values in order to evaluate the effect on the calculated displacements. The design assumptions and peak east abutment displacement results are summarized in Table II.

Table II. Summary of Response Spectrum Analysis Results for Cholame 2W Ground Motions, 5% Damping

Design Assumption	Longitudinal Displacement (inches)	Transverse Displacement (inches)
Assumed depth to fixity of 5 pile diameters	1.44	1.52
Bent pile springs only in upper 8-feet of pile	1.32	1.41
Bent pile springs along full length of piles assuming “sand” subgrade	1.45	1.51
Bent pile springs along full length of piles assuming “stiff clay” overlying “sand” subgrade	1.32	1.45

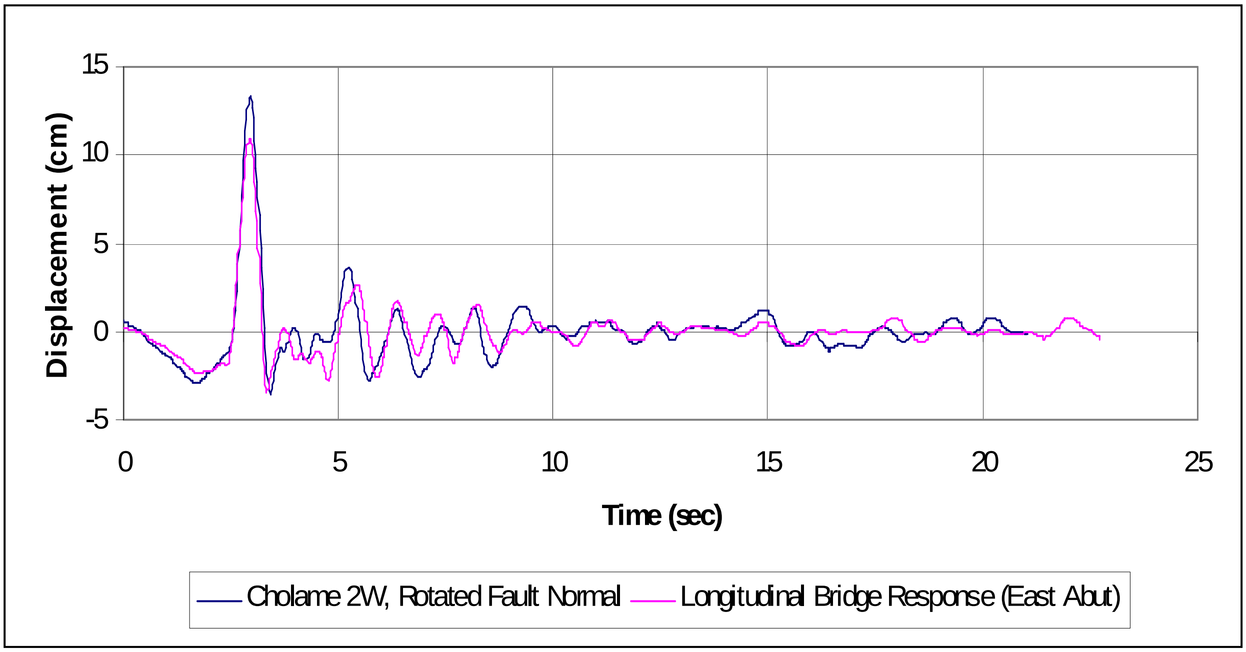

Figure 10. Measured Displacement Time History Comparison in Longitudinal Direction

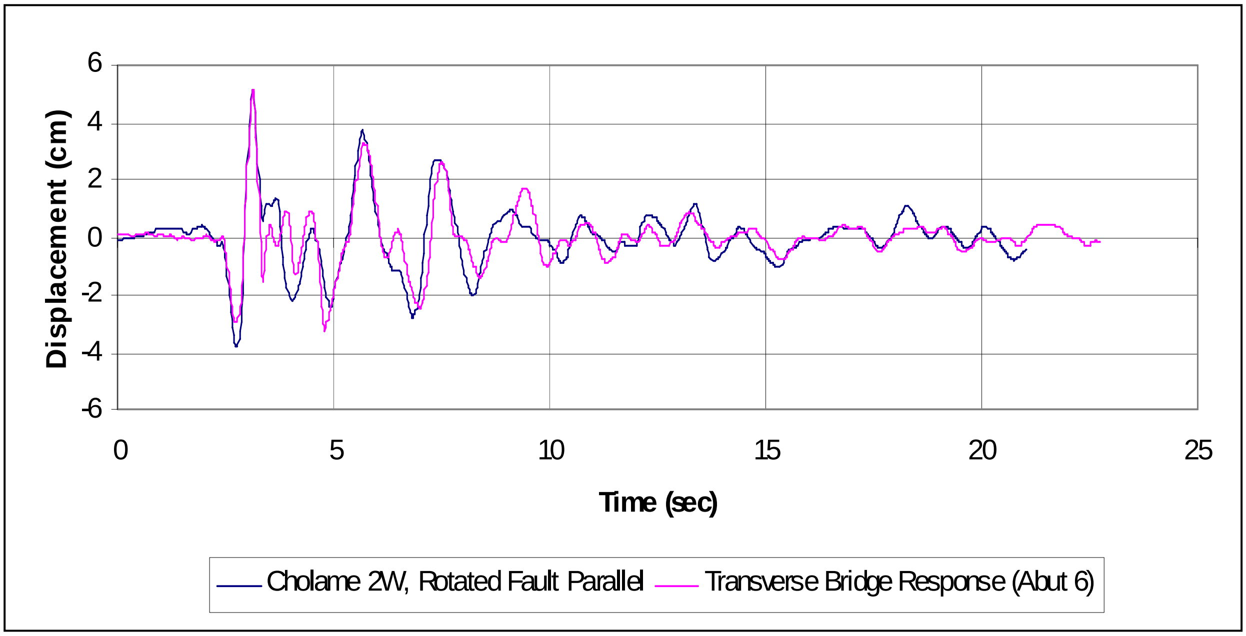

Figure 11. Measured Displacement Time History Comparison in Transverse Direction

Using a displacement reduction factor of 0.70 to account for the assumed higher damping of 15% for the short, stiff bridge (Caltrans 1995) results in longitudinal and transverse displacements on the order of 1 inch for each of the design assumptions. The estimated value of 1 inch is reasonably close to the measured relative displacements values of 0.94 inch (longitudinal) and 0.84 inch (transverse).

Time History Analysis Results

The free-field ground motions from Cholame 2W were rotated into longitudinal (~ N63°E) and transverse (~ N27°W) directions and applied to the bridge model simultaneously. Linear abutment springs were assumed for each model as the ultimate capacity of the abutments was not exceeded during the response spectrum analyses. However, both linear and nonlinear soil-pile springs were used to evaluate the effect on the results. Nonlinear soil-pile springs were modeled in SAP2000 using nonlinear links in the upper portion of the piles. The design assumptions and peak east abutment displacement results are summarized in Table III.

Table III. Summary of Time History Analysis Results for Cholame 2W Ground Motions, 5% Damping (except as noted)

Design Assumption	Longitudinal Displacement (inches)	Transverse Displacement (inches)
Bent pile springs only in upper 8-feet of pile	1.23	1.56
Bent pile springs along full length of piles assuming “stiff clay” overlying “sand” subgrade	1.09	1.43
Bent pile springs along full length of piles assuming “sand” subgrade	1.44	1.53
Bent pile springs along full length of piles assuming “sand” subgrade, *15% damping*	0.93	1.19
Bent pile springs along full length of piles assuming “sand” subgrade, nonlinear springs	1.26	1.53
Bent pile springs along full length of piles assuming “sand” subgrade, nonlinear springs and *15% damping*	0.92	1.15

The use of nonlinear soil-pile springs in the upper portion of the pile did not have a significant effect on the results. However, using 15% damping (Caltrans 1995) in the time history analysis reduced the calculated deck displacements significantly. These reduced values are reasonably close to the measured relative displacements values of 0.94 inch (longitudinal) and 0.84 inch (transverse).

Displacement Demand Assessment

A displacement demand assessment was performed on the Hwy 46/Cholome Creek Bridge to put the measured displacements and damage patterns resulting from the 2004 M_w 6.0 Parkfield seismic event into perspective. The assessment followed current Caltrans seismic design and retrofit guidelines. The design level spectrum (ARS) was based on a magnitude M_w 7.5 earthquake on the San Andreas Fault with a peak ground acceleration of 0.7g for Type D soil conditions (Mualchin, 1996). Displacement demands for the M_w 7.5 design event were determined using the standard 5% damped ARS curve above. An elastic displacement reduction factor of 0.70 was then applied to account for an increased damping ratio of 15% for the short, stiff bridge in accordance with Caltrans Memo to Designers (MTD) 20-4 (1995). This resulted in a longitudinal displacement demand of 1.8" and transverse displacement demands varying from 1.6" at Bent 2 to 2.0" at Bent 5.

Capacity Assessment

A capacity assessment was performed utilizing moment-curvature analysis. The short piles at Bent 2 were found to be the most critical for the bridge due to their reduced displacement capacities and increased shear demand. However, the results of the assessment indicate that the piles have adequate displacement capacity and shear strength to withstand the Caltrans design level seismic event. The results are summarized below:

Table IV Transverse Pile Displacement Assessment at Critical Bent 2

Displacement Demand reduced by factor of 0.7, Δ_{dem}	Yield Displacement, Δ_y	Ductility Demand, μ_Δ (Δ_{dem}/Δ_y)	Displacement Capacity, Δ_u	Displacement Demand/Capacity, Δ_{dem}/Δ_u
1.8"	0.58"	3.1	8.7"	0.21

Table V Pile Shear Assessment at Critical Bent 2

Plastic Shear Demand, V_p	Design Shear Demand, $V_o = 1.3\ V_p$	Factor1*Factor 2 at $\mu_\Delta = 3.1$	Shear Capacity, $\phi\ V_n$	Shear Demand/Capacity, $V_o / \phi V_n$
18 kips	24 kips	2.7*1.05 = 2.8	46 kips	0.51

Furthermore, results of the pile moment-curvature analysis show that under the displacement demands of about 1" measured during the 2004 event, cracking of the cover concrete and yield of the reinforcement is expected. This correlates well with the damage documented in the Caltrans inspection reports. The table below summarizes the results of the moment-curvature analysis for a typical 16" diameter pile. Displacements were calculated by assuming a 10-ft effective column/pile length, which is appropriate for the critical Bent 2 piles.

Table VI Results of the Pile Moment-Curvature Analysis at Critical Bent 2

Pile Damage Description	Criteria	M (k-in.)	ϕ (10^{-3}/in.)	Δ (in.)
1. Cracking of cover concrete	f_c = 530 psi (tension)	324	0.025	0.17
2. First yield of reinforcement	f_y = 68,000 psi	976	0.278	0.51
3. Idealized yield	$M_p/M_y * \phi'_y$	1100	0.313	0.58
4. Concrete spalling	$\varepsilon_c = 0.005$	1150	1.30	2.2
5. Plastic hinge failure	$\varepsilon_{cc} = 0.016$	1082	5.33	8.7

CONCLUSION

The seismic response of the Hwy 46/Cholame Creek Bridge was studied under a grant from the CSMIP. The goal of the evaluation was to compare the measured bridge deck displacements with those calculated using typical Caltrans design guidelines. The free-field acceleration time histories from the Cholame 2W Station were rotated into the longitudinal and transverse bridge directions to develop the site-specific acceleration response spectra.

Both response spectra and time history analyses were performed using a variety of design assumptions for the bridge foundation. Based on our displacement analyses, we found that using the current Caltrans bridge design guidelines with the Cholame 2W spectra resulted in a close match with the measured bridge displacements during the 2004 M_w 6.0 seismic event. As the measured bridge displacements were much lower than the estimated pile capacity, the minor pile top cracking noted after the recent earthquake is consistent with the capacity assessment for the bridge.

Acknowledgements

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