Recorded Seismic Response of the Samoa Channel Bridge-Foundation System and Adjacent Downhole Array

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Abstract

A large set of earthquake records from the highly instrumented Samoa Channel bridge-ground system has been compiled and made available by the California Geological Survey. During six seismic events, more than 30 data channels have been documenting the seismic response of the bridge, abutments, and adjacent ground surface. Of special interest is the response of one of the bridge piers with records at the deck level, pile cap and within the underlying pile foundation. Response of this pile foundation is compared to that of the ground as documented by the nearby Eureka geotechnical downhole array. In this paper, records from the strongest to date 2010 Ferndale earthquake (PGA of about 0.15g), along with other available low-amplitude events (2007-2014) are employed to evaluate the ground, pile foundation, and overall bridge seismic response. Spatial variation of the recorded motions is examined. Linear and nonlinear response of the ground and the bridge are assessed using system identification techniques. During the strong shaking phase of the 2010 Ferndale Earthquake, a clear and significant stiffness reduction was observed in the column as well as in the foundation of the instrumented pier.

Keywords: Highway Bridge; Earthquake Response; Soil-Structure-Interaction; SSI; Seismic response

Introduction

For over two decades, strong motion sensors have been deployed on California bridges and nearby free-field/downhole sites [1] through the joint efforts of the California Department of Transportation (Caltrans) and the California Geological Survey (CGS). As of 2013, 65 highway bridges, 13 toll bridge structures and 38 downhole geotechnical arrays have been instrumented in California [2-4].

The recorded data sets along with system identification techniques are permitting the evolution of further insights into the involved response mechanisms [5-8]. For instance, the seismic characteristics of the heavily instrumented Meloland Road Overpass have been evaluated using a number of different assessment methodologies [7, 9-11]. Response of the I-10/215 Northwest Connector during the Landers and Big Bear earthquakes was evaluated [12, 13] based on an extensively deployed strong motion instrumentation network. With such invaluable recorded data sets, relative bridge displacements, modal frequencies, and mode shapes can be identified [14-17]; and our understanding of the involved soil-structure interaction (SSI) effects has been further advanced [18-26].

More recently, accelerometers within bridge pile foundations are being deployed. As of 2014, eight bridges include such instrumentation, along with a nearby geotechnical downhole array (the Benicia-Martinez East Bridge, the Benicia-Martinez West Bridge, the Carquinez Bridge West Span, the Oakland - SF Bay Bridge/East, the San Francisco - Bay Bridge/West, the San Rafael Bridge, the Samoa Channel Bridge and the Eureka Channel Bridge). Among those, only the Samoa Channel Bridge and the Eureka Channel Bridge have multiple earthquake records available, varying in peak ground acceleration, epicentral distance, shaking duration, and frequency content bandwidth. The recorded bridge, deep foundation, and adjacent downhole-

array seismic motions provide a unique opportunity for documenting and analyzing the associated salient ground-foundation-structure response mechanisms.

In this paper, seismic response of the instrumented Samoa Channel Bridge and the closely neighboring Eureka geotechnical array is investigated. Time history response along the bridge deck and the underlying ground is presented. Performance of the bridge structure and the effects of local site geology during this earthquake are discussed. Response of the instrumented Pier (S-8) and its foundation is closely studied. Focus is placed on the bridge seismic response during the Ferndale Earthquake of January 9, 2010 which resulted in a moderate but substantial level of earthquake excitation (PGA of about 0.15g). As a reference, comparisons are made with the other available low amplitude seismic motions. Spatial variation of the recorded motions is examined. Resonance parameters are identified to describe the behavior of the downhole array site, the bridge super-structure, and the overall soil-foundation-structure system. The salient characteristics of linear and nonlinear response of the site, the bridge, and the foundation are highlighted and analyzed.

Instrumentation at the Samoa Channel Bridge and Adjacent Downhole Array

Bridge Configuration

The Samoa Channel Bridge connecting the Samoa Peninsula and Indian Island (Figure 1) is one of three bridges crossing the Humboldt Bay in Eureka, California. It was designed in 1968, constructed in 1971, and since has undergone a seismic retrofit that was completed in 2006. Retrofit details along with soil boring data at the site are presented in Caltrans [27, 28].

This 20-span bridge is 764 m long and 10.4 m wide (Figure 2). The superstructure consists of a cast-in place reinforced concrete deck with four precast, prestressed concrete I-girders. As shown in Figure 2, there are eight separation joints along the bridge superstructure.

The bridge I-girders are supported on 19 concrete single column piers and seat-type abutments. The piers and abutments are numbered S-1 through S-21 from the Indian Island side to the Samoa Peninsula side (Figure 2). Significant soil profile variability can be seen as depicted in Figure 2 and Figure 3.

The abutments and piers were supported originally on pile groups consisting of driven pre-cast prestressed concrete piles. Referenced to the mean sea level (MSL), elevation of the underlying ground surface (mudline) varies from -15.8 m below Pier S-8 to +0.9 m at Pier S-20 (Figure 2). Eleven pile groups (from S-3 to S-13) have a pile cap located above the mudline with a maximum free span of 16.72 m (up to the pile cap base) at Pier S-8 (Figure 2 and Figure 4).

The seismic retrofit work completed in 2006 included (Figure 4): i) strengthening of the foundations by installing additional cast-in-steel shell (CISS) piles (e.g. 6 additional 1.52 m diameter, 19 mm shell thickness piles at Pier S-8), ii) enlarging the pile caps to cover the new piles, and iii) encasing the original bridge columns in reinforced concrete (RC) jackets.

Samoa Channel Bridge Instrumentation

The Bridge is heavily instrumented with over 30 accelerometers (deployed mostly in 1996). Side view and deck level plan view of the bridge instrumentation are shown in Figure 2, where sensor locations are marked with numbered arrows in the direction of measured motion.

As depicted in Figure 2, 24 sensors are placed on the bridge structure, including 16 on the deck, 3 at the abutments and 5 at the pile caps of Pier S-8 and Pier S-14. These sensors are oriented in the longitudinal (LONG) and transverse (TRAN) directions of the bridge. At the nearby ground surface station (Figure 2) denoted here as BGS (48.77 m west of Abutment S-21), three sensors are installed at an elevation of +2.5 m approximately, oriented in the north-south (NS), east-west (EW) and vertical (VERT) directions.

In addition, 6 accelerometers are embedded inside one of the Pier S-8 retrofit-deployed CISS piles (Figure 4). Through these sensors, it is possible to document the Pier S-8 foundation seismic response at the two elevations of -10.36 m and -16.46 m (BPF). As such, a rather unique opportunity is facilitated to visualize and study the response of this Pier (Figure 4) simultaneously at the bridge-deck level, the pile cap level, and the 2 locations along the pile foundation (down to the mudline).

Instrumentation at the Adjacent Eureka Geotechnical Array

The Eureka Geotechnical Downhole Array (denoted here as GDA, CSMIP Station 89734) is located approximately 198 m south-east of the Samoa Bridge Abutment S-1 (Figure 1 and Figure 2). A relatively deep soft alluvium geological profile with high water table is noted at this location (Figure 3). Subsurface data from a nearby boring shows 10 m of very soft clayey silt underlain by slightly compact gray materials (Figure 3). In this boring, the very dense soil layer is approximately 22 m below the ground surface.

The sensor layout as well as the site geophysical testing shear/pressure wave velocity profiles is shown in Figure 5. This GDA consists of 15 accelerometers (Figure 5) at ground surface (+1 m) and at the depths of -16 m, -30 m, -53 m and -133 m (referenced to MSL). The shear wave velocity (V_s) is about 158 m/s - 230 m/s in the upper 20 meters, and in the range of 210 m/s to 460 m/s at the depths of 20 - 60 meters. Bedrock is estimated to be at a depth of about 220 m where the shear wave velocity reaches 870 m/s. The calculated Poisson's ratio (ν) ranges from 0.43-0.49 [29], with the highest values generally indicating saturated soil conditions.

Recorded Earthquake Motions

The bridge is located in a seismically active area of Northern California. Records from a total of six earthquakes (Table 1) are currently available with Moment Magnitudes (M_w) in the

range of 4.6 to 6.8. Acceleration, velocity, displacement and acceleration response spectra are available. Varying in horizontal peak ground acceleration (from 0.01 g to 0.17 g), epicentral distance (from 40.2 km to 81.3 km), shaking duration, and frequency content bandwidth, these earthquakes provide valuable information on the soil-foundation-structure seismic behavior over a broad range of excitations.

To date, the highest levels of recorded acceleration are due to the 2010 $M_w = 6.5$ Ferndale Earthquake (hereafter, referred to as “the moderate event”) which occurred approximately 35 km away from Ferndale, CA in a deformation zone of the southernmost Gorda Plate (<http://earthquake.usgs.gov>, [30]). During this event, the recorded TRAN peak acceleration is 0.16 g at the ground surface near the bridge and 0.66 g at the bridge deck. The largest relative movement of the bridge structure is 9.6 cm longitudinally and 7.2 cm transversely referenced to the -16.46 m Pier S-8 near-mudline location (Figure 2 and Figure 4), with no reported significant damage [30].

Pattern Recognition Based on the Earthquake Records

Downhole Site Seismic Response

The GDA (Figure 5) recorded data allow for a closer investigation of the ground seismic response in which the bridge foundations are embedded. For the purpose of this study, these GDA records were re-oriented in the bridge TRAN and LONG directions (e.g., Figure 6).

Site properties between downhole stations along the depth (Figure 5) were evaluated using time and frequency domain analysis techniques [29]. Herein, results of interest are reported for the topmost stratum (elevation of +1 m to -16 m). Through cross-correlation analysis [31], average shear wave velocity (V_s) was estimated in the range of 162 m/s to 189 m/s from the low

amplitude shaking events (Table 2). The geophysical V_S measurements for this uppermost soil layer (Figure 5) are within this estimated range.

Indicative of the nonlinear site response, lower average shear wave velocity was evident during the moderate event (Table 2). In this event, average shear strain [32] during the strong motion phase (the 28 s - 33 s time window in Figure 6), reached a maximum of about 0.07%. In this time window, the estimated V_S decreased to a value of 113.3 m/s, corresponding to a shear modulus G of about 36% of the initial G_{max} . This change in modulus agrees generally with the reduction range of 34% - 44% as specified by the commonly used G/G_{max} curves for sands (e.g., [33, 34]).

Fourier spectral ratios [31] between the top two stations (elevation of +1 m to -16 m) allow for estimation of the first resonance (Figure 7) as shown in Table 2. To document the evidence of nonlinear response, similar Fourier spectral ratios were computed for the moderate event in individual 5-second time windows (Figure 7). During the early and late parts of the recorded motion, natural frequency reached as much as 2.60 Hz (Figure 7), similar to the estimates from the other low-amplitude shaking events (Table 2). During the strong motion phase (28 s - 33 s), a lower resonance was noted (as low as about 1.4 Hz). Generally, the identified first resonant frequency (f_l) is in agreement (Table 2) with that of the simple (constant shear modulus) site response shear beam formula of $f_l = V_s/4H$ where H is the vertical distance between the sensors ($H=17$ m).

It is noteworthy to mention that no such reduction in site stiffness was detected for any of the deeper soil layers (Figure 5) between the sensors at -16 m, -30 m, -53 m and -153 m [29]. Along with the markedly lower recorded shaking amplitudes at these depths [29], the higher shear stiffness at these greater depths contributed to a relatively linear response [29].

Finally, it may be observed that there was some difference in the moderate event V_s LONG and TRAN estimates (Table 2). This might be indicative of a site azimuthal anisotropy [35], a topic worthy of further investigation as more data becomes available.

Ground Response at the Bridge and Adjacent Downhole Array Sites

Some insights may be derived from comparison of the recorded ground motions at the bridge and the downhole array sites (Figure 2). Synchronization between the bridge and GDA records was done [29] based on the actual digital time stamp for each site (resulting for instance in a time shift of about 2 seconds for the moderate event). Further, in order to more rationally compare the GDA and bridge motions, the ground surface records at the bridge site (BGS) and the GDA records (Figure 5) were all re-oriented in the bridge TRAN and LONG directions.

On this basis, Figure 6 presents the moderate event records for: i) the adjacent geotechnical downhole array (GDA) at the elevations of +1 m, -16 m, and -30 m, ii) the bridge ground surface (BGS) station, and iii) the bridge pile foundation (BPF) near the mudline at the elevation of -16.46 m (after azimuthal orientation error correction as discussed in the Appendix). In general, arrival time of the ground surface seismic waves at the GDA and bridge sites [29] differed by about 0.1 second (estimated by cross-correlation analysis).

For the GDA records, there is a clear increase in displacement/acceleration amplitudes as the seismic waves approach the ground surface. Keeping the irregular ground surface topography and site stratification in mind (Figure 2 and Figure 3), response at BPF is seen to more closely align with that of the downhole -16 m station (more so than the closer BGS station). As such, this location, near the mudline within the pile, appears to be practically moving along with the ground, indicating a level of firm embedment (fixity) at this depth. Similar conclusions can be deduced based on comparison of data from all of the recorded earthquake events [29].

Figure 8 presents the Power spectral density (PSD) graphs at these three ground surface locations for two representative events. Only for the BGS station, a clear peak can be seen around the frequency range of 3.5 Hz-3.8 Hz, actually appearing for all shaking events [29]. This observed response at high frequency might be partially due to the stiffer upper soil strata at this location and/or the impact of ground surface topography (Figure 2 and Figure 3).

Seismic Response along the Bridge Deck

Figure 9 and Figure 10 display the bridge relative displacement referenced to the -16.46 m BPF pile motion (as discussed above, essentially the ground motion at this depth) for two representative earthquake events [29]. All along the deck, predominant in-phase response is noted in both directions. The bridge is seen to be noticeably more flexible in the mid-span zone (Channels 10-13), with the relatively compliant free span pile foundations at this location (Figure 2). For the moderate event, maximum relative displacement at Pier S-8 is larger in amplitude compared to that at the abutment (Chan 3) by about 60% (TRAN), and 80% (LONG).

The transverse displacement time histories (Figure 9 and Figure 10) clearly demonstrate the period of vibration due to the pronounced harmonic response of the structure in this direction. The observed resonance (Channels 10, 12) is particularly vivid during the low-amplitude shaking events with a period of about 1.1 s (0.91 Hz) for the entire vibration phase (Figure 9a), in agreement with the recently reported ambient vibration resonance of 1.0 s [26, 36]. During the moderate event (Figure 10), the bridge structure displays a period of about 1.57 s (0.64 Hz) during the strong shaking phase of this event (about 28 s – 33 s), which is then reduced to about 1.21 s (0.83 Hz) thereafter (Figure 10a).

This resonant response is further illustrated by the corresponding Transfer Function (TF) as shown in Figure 11a and Table 3 (where TF is ratio of cross power spectral density $P_{yx}(f)$ of

input x and output y , and power spectral density $P_{xx}(f)$ amplitudes). In the moderate event, the lower predominant frequency (Figure 11a) reflects the nonlinear behavior in the bridge including the column and pile group up to the reference mudline motion location. Figure 11b shows instants of resonant frequency variation (Short time TF intervals), with the lowest values coinciding with the strong motion shaking phase of the moderate event.

Compared to the transverse direction, the bridge structure is somewhat stiffer in the longitudinal direction with a lower resonant period [29]. As may be noted in Figure 9b and Figure 10b, longitudinal displacement displays a period of 0.75 s for the low-amplitude shaking events and 1.13 s – 1.55 s for the moderate event [29].

Nonlinear response during the moderate event was associated with sharp spikes in the deck acceleration time histories (Figure 12). The highest spikes reached about 2.23g in the longitudinal direction and 0.67g in the transverse direction (noting that the peak ground acceleration at the bridge site is only about 0.17g). Such spikes were likely caused by relative motion and impact at the separation joints between the different bridge deck segments [37, 38]. Near the S-8 separation joint (Figure 2), this relative motion is shown in Figure 12, reaching a peak of about 1.5 cm (LONG) and only 0.2 cm (TRAN). No such spikes were observed during any of the low-amplitude shaking events [29].

Pier S-8 Response

Transverse displacement along Pier S-8 at the four instrumented elevations (Figure 4) is shown in Figure 13a during two representative earthquake events. In-phase response with a dominant fundamental period is evident. Referenced to the mudline BPF base motion, the pile cap as well as the bridge deck displacements display significant amplification.

As shown in Figure 13b, much of the bridge Transverse displacements at Pier S-8 are due to deformation within the free-span pile group zone (Figure 4), with the bridge deck motions only slightly different from those of the pile cap. For instance, at peak Pier S-8 displacement (Figure 13b), the bridge column (between the deck and the pile cap) only contributes about 1.4% to the total deformation. This may be taken as an indication that the column above the pile cap is quite stiff in comparison to the lateral stiffness afforded by the underlying pile group.

In the longitudinal direction (Figure 14), displacements are more evenly accounted for by the column and the pile group. Compared to the TRAN direction, this might denote a less pronounced LONG stiffness contrast between the column and pile group. This can be explained by the clearly lower column Moment of Inertia (Figure 4) in the LONG direction, compared to that of the TRAN direction.

Finally, the Pier S-8 deck to pile-cap fundamental resonance is presented in Table 3. In the low amplitude events, a resonance of about 2.0 Hz is noted, with a lower 1.6 Hz for the moderate event. A closer picture of this frequency change may be seen in the short time TF results of Figure 15, reflecting the nonlinear response effects of this deck to pile-cap zone.

Abutment and Bridge Response along the ground surface

Figure 16 and Figure 17 compare the moderate event acceleration and displacement time histories at the bridge site along the ground surface (including motion at the abutments, mudline BPF, S-14 pile cap, and the nearby BGS). Particularly in the longitudinal direction (Figure 17), the deck-level acceleration spikes of Figure 12 are observed as well at the abutment and at the S-14 pile cap.

Nevertheless, it can be seen that the displacement pattern of variability is similar all along the ground (Figure 16 and Figure 17) with larger peak values at the abutment. This difference in

ground motion may be better appreciated in Figure 18 which depicts relative displacements referenced to the mudline S-8 BPF station. In this regard, the peak relative motion of about 5 cm (Figure 18) is of the same order of the actual peak motion at BPF (Figure 16 and Figure 17).

All these observations indicate that the motions recorded along ground surface including at S-8 and S-14 are: i) influenced by the irregular topography and soil profile stratification under the extended bridge domain, and ii) affected by the bridge response. As such, bridge modeling procedures using a ground surface response spectrum (or a travelling-wave multi-support ground excitation) may not fully reflect the actual input excitation complexities.

Discussion and Conclusions

The Samoa Channel bridge-ground seismic response was studied during a large number of earthquake events [29]. Specifically, the system dynamic properties were assessed by investigating the: i) ground response at the nearby Eureka Downhole array, ii) variability of motion along the bridge deck, iii) response of the instrumented Pier S-8 and its pile foundation down to the mudline, and iv) variation of seismic motion along the bridge ground surface. Representative responses were presented, with the study encompassing the entire set of available earthquake records [29]. Within this framework, salient response characteristics were highlighted, with evident nonlinear behavior during the 0.15g PGA 2010 Ferndale earthquake.

On this basis, and noting the relatively low peak ground accelerations to date, the main observations and conclusions include:

1) The downhole array motions revealed:

i) a shear wave velocity profile that is consistent with that documented earlier through in-situ investigations,

ii) with peak ground acceleration of about 0.15 g, G/G_{max} in the upper 17 m zone reached as low as 36 %, in agreement with the widely used such relationships, and

iii) below the depth of about 16 m, lower shaking amplitudes and higher soil stiffness precluded the appearance of any detectable nonlinear response.

2) Recorded motions at different locations along the bridge-ground interface revealed that:

i) significant differences are present in amplitude and frequency content, partially due to soil profile as well as topography variation (with as much as 19 m vertical offset between the highest and lowest ground surface locations). The nearby high elevation ground station, overlying a stiffer soil profile, showed the largest shaking amplitudes,

ii) the bridge inertial motion appeared to exert an influence on the immediate underlying soil domain and the embedded pile cap responses,

iii) the near mudline pile motion at -16.46 m was noticeably similar to that of the downhole array at comparable depth. This motion is significantly lower than that of the nearby ground surface. As such, this relatively deep mudline might be beneficial in reducing the input ground excitation. In essence, the -16.46 m BPF acceleration could be a candidate input motion for a numerical model that represents the bridge and upper soil strata, and

iv) with the similar soil and pile responses at -16.46 m, the pile group shows some evidence of a relatively firm level of fixity into the ground. Influence of the pile group in stiffening the surrounding soil, if any, remains a topic for further studies. In this regard, soil around the pile group (from -16.46 m downwards) was found to experience only low shear strains with virtually no reduction in stiffness.

3) Time and frequency domain analysis of motions along Pier S-8 (deck, pile cap, and pile near mudline) revealed that:

i) in the transverse direction, with the high column Moment of Inertia, deformation was mainly occurring below the pile cap within the pile group 16 m long free span,

ii) conversely, in the longitudinal direction (with a lower column Moment of Inertia), deformation was more evenly distributed between the column and the pile group below the pile cap,

iii) during the moderate event, evidence of nonlinear behavior was documented for the column as well as for the pile group (between the pile cap and the mudline), and,

iv) by studying the earthquake events chronologically, no clear evidence of permanent change in stiffness was revealed [29].

The results of this research are of significance to the current state of practice in seismic ground-foundation-structure analysis. With increased future availability of bridge-ground earthquake records, valuable new insights into the soil-foundation-structure response mechanisms can be derived (for the purposes of foundation design and retrofit strategies).

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Appendix

Figure A-1 shows a representation of recorded motion at selected time instants as depicted in Figure 13a and Figure 14a. The unexpected out-of-phase response of the sensors (consistently observed in all recorded events, [29]) within the pile at the 10.36 m depth station, necessitated an investigation of the reported azimuthal accuracy (as this sensor was lowered into the pile cavity with no direct control over its orientation). As such, cross-correlation analyses were performed [29] to assess this matter. Achieving peak cross correlation with the nearby BGS station, rotations of 103 degrees for the BPF and 144 degrees for the -10.36 m sensors were dictated (counterclockwise). After rotation, motions along Pier S-8 display in-phase behavior in the transverse and longitudinal directions (Figure 13a and Figure 14a). Similar re-orientation angles were obtained using a Fourier Spectra procedure in the studies of [25, 26].

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Table 1 Recorded earthquakes at the Bridge Site (arranged by order of peak acceleration)

Earthquake	Epicentral Distance (km)	Peak Acceleration (g)					
		TRAN		LONG		VERT	
		Ground	Bridge	Ground	Bridge	Ground	Bridge
Ferndale 2010* ($M_w=6.5$)	53.8	0.158	0.665** 0.216***	0.167	2.225** 0.660***	--	1.003
Trinidad 2007 ($M_L=5.1$)	63.8	0.025	0.063	0.015	0.021	0.008	0.072
Ferndale 2014 ($M_w=6.8$)	81.3	0.019	0.069	0.015	0.033	--	0.064
Trinidad 2008 ($M_w=4.6$)	40.2	0.016	0.026	0.018	0.043	--	0.056
Willow Creek 2008 ($M_w=5.4$)	56.6	0.014	0.032	0.010	0.019	--	0.025
Ferndale 2007 ($M_w=5.4$)	62.4	0.011	0.022	0.009	0.016	0.004	0.013

* the January 2010 Ferndale Earthquake will be referred to as “the moderate event” in this study

** large peak acceleration due to spikes from separation joints

*** estimated after removing spikes using a band pass filter

Table 2. Estimated average shear wave velocity (V_s) and resonant frequency (topmost stratum: +1 m to -16 m)

Earthquake	Epicentral Distance (km)	Horizontal PGA* (g)		V_s (m/s)		Shear Modulus G^{**} (MPa)		1 st resonant frequency (Hz)			
								$V_s/4H$		Identified	
		TRAN	LONG	TRAN	LONG	TRAN	LONG	TRAN	LONG	TRAN	LONG
Ferndale 2010 ($M_w=6.5$)	54.0	0.203	0.185	121.4 113.3 ^{***}	154.5 141.7 ^{***}	23.6 20.5 ^{***}	38.2 32.1 ^{***}	1.79 1.67 ^{***}	2.27 2.08 ^{***}	1.47 1.40 ^{***}	1.90 1.80 ^{***}
Weitchpec 2012 ($M_w=5.6$)	47.9	0.034	0.027	161.9	170.0	41.9	46.2	2.38	2.50	2.77	2.80
Bluelake 2012 ($M_L=3.5$)	22.2	0.012	0.009	161.9	161.9	41.9	41.9	2.38	2.38	2.70	2.70
Eureka Offshore 2000 ($M_L=4.4$)	24.4	0.006	0.011	170.0	170.0	46.2	46.2	2.50	2.50	2.47	2.73
Ferndale 2007 ($M_L=5.4$)	62.6	0.008	0.009	188.9	188.9	57.1	57.1	2.78	2.78	3.00	3.03
Ferndale Offshore 2000 ($M_w=4.1$)	46.4	0.004	0.008	170.0	188.9	46.2	57.1	2.50	2.78	2.77	2.73

* PGA = Peak Ground Acceleration

** assuming a soil mass density of 1600 kg/m³

*** during the strong shaking phase 28 s – 33 s.

Table 3. Identified Fundamental resonance of bridge system in the transverse direction

Earthquake	Fundamental Period (Frequency) based on TF analysis		
	S-8 Deck referenced to Pile at -16.46m	S-8 Deck referenced to BGS	S-8 Deck referenced to Pile Cap
Ferndale 2010	1.43 s (0.70 Hz)	1.49 s (0.67 Hz)	0.63 s (1.59 Hz)
Trinidad 2007	1.04 s (0.96 Hz)	1.12 s (0.89 Hz)	0.50 s (2.00 Hz)
Ferndale 2014	1.11 s (0.90 Hz)	1.15 s (0.87 Hz)	0.51 s (1.96 Hz)
Trinidad 2008	1.04 s (0.96 Hz)	1.04 s (0.96 Hz)	0.51 s (1.96 Hz)
Willow Creek 2008	1.00 s (1.00 Hz)	1.11 s (0.90 Hz)	0.52 s (1.92 Hz)
Ferndale 2007	1.04 s (0.96 Hz)	1.12 s (0.89 Hz)	0.52 s (1.92 Hz)

(a)

(b)

Figure 1. Bridge Configuration: (a) Samoa Channel Bridge, Geotechnical Downhole Array, Middle Channel Bridge and Eureka Channel Bridge (Map Data @ 2015 Google) and (b) photo of the Samoa Channel Bridge (<http://www.strongmotioncenter.org>)

Figure 3. Soil Profiles at Geotechnical Downhole Array and along the bridge site (defined based on [28])

Figure 4. Pier S-8 configuration and layout of instrumentation [28]

Figure 5. Plan view and elevation view of Eureka Geotechnical Array, measured wave velocities along the depth (<http://www.strongmotioncenter.org>), and estimates from the recorded downhole motions.

Figure 6. Recorded (absolute) time histories at the GDA, BGS, and BPF stations during the moderate event

Figure 7. GDA acceleration and Transfer Function (TF) of the topmost stratum during the moderate event

(a)

(b)

Figure 8. Power spectral density (PSD) of the BGS, BPF and GDA ground surface stations in the transverse direction during (a) the moderate event, and (b) the 2007 Ferndale Earthquake

Figure 9. Variation of relative displacement along the bridge deck level for the 2007 Ferndale Earthquake

Figure 10. Variation of relative displacement time histories along the bridge deck for the moderate event (no data recorded for the right abutment channels 22-24 during this event)

Figure 11. Transverse direction: (a) Acceleration Transfer function for deck (Chan. 10) referenced to pile response at -16.46 m, and (b) Time history and short-time acceleration Transfer Function (deck Chan. 10 referenced to pile -16.46 m)

Figure 12. Recorded acceleration and relative longitudinal/transverse displacement of deck across the separation joint near Pier 8 for the moderate event (a) before and (b) after high frequency response filtering (band-pass filter between 0.02 Hz and 8 Hz)

Figure 13. Transverse direction: (a) time histories and (b) displaced configuration of Pier S-8 for selected time instants during the moderate event (upper), and during the 2007 Ferndale Earthquake (lower)

Figure 14. Longitudinal direction: (a) time histories and (b) displaced configuration of Pier S-8 for selected time instants during the moderate event (upper), and during the 2007 Ferndale Earthquake (lower)

(a)

(b)

Figure 15. Transverse direction short-time acceleration transfer function (deck to pile cap) during: (a) the moderate event, and (b) the 2007 Ferndale Earthquake

Figure 16. Moderate event TRAN (a) acceleration and (b) displacement time histories along the ground surface

Figure 17. Moderate event LONG (a) acceleration and (b) displacement time histories along the ground surface

(a) TRAN

(b) LONG

Figure 18. Moderate event relative displacement time histories (referenced to BPF) along the ground surface

Moderate Event

2007 Ferndale Earthquake

TRAN

LONG

Figure A-1. Original recorded displaced configuration of Pier S-8 at the representative time instants of Figure 13 and Figure 14.