International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

2010 - Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

28 May 2010, 2:00 pm - 3:30 pm

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Fifth International Conference on
Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
and Symposium in Honor of Professor I.M. Idriss
May 24-29, 2010 • San Diego, California

GROUND MOTION STUDY ON DUMBARTON TOLL BRIDGE

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ABSTRACT

The existing Dumbarton Toll Bridge was built in 1982, connecting the cities of Newark and East Palo Alto in the San Francisco Bay Area. The initial vulnerability studies conducted by California Department of Transportation (Caltrans) in 2004 indicated that the performance of the bridge during a maximum credible earthquake was uncertain. Earth Mechanics, Inc. (EMI) has carried out the necessary study for the seismic evaluation of the bridge. An extensive field investigation was undertaken both on-land and over-water at the site to develop the idealized subsurface profile along the bridge alignment. According to the probabilistic and deterministic seismic hazard analyses incorporated with new seismic source model and Next Generation Attenuation (NGA) models, a 1,000-year return period spectrum was adopted for the Safety Evaluation Earthquake (SEE) event and a 100-year return period spectrum for the Function Evaluation Earthquake (FEE) event. SHAKE and KIPS programs were used to conduct the seismic response analysis and kinematic soil-pile interaction analysis were carried out at selected piers. From this study, two sets of Acceleration Response Spectrum (ARS) curves were generated for the seismic retrofit of this bridge: one for the Main Channel piers and another for the West and East Approach structures. Other seismic retrofit-related issues are also addressed.

INTRODUCTION

The Dumbarton Toll Bridge crosses the southern San Francisco Bay via California State Highway 84, and connects the cities of East Palo Alto, San Mateo County (southwest) and Newark, Alameda County (northeast). This 1.63 mile-long bridge is the southernmost of the highway bridges spanning the San Francisco Bay. The location of the Dumbarton Bridge and the aerial photograph of the structure are shown in Figure 1.

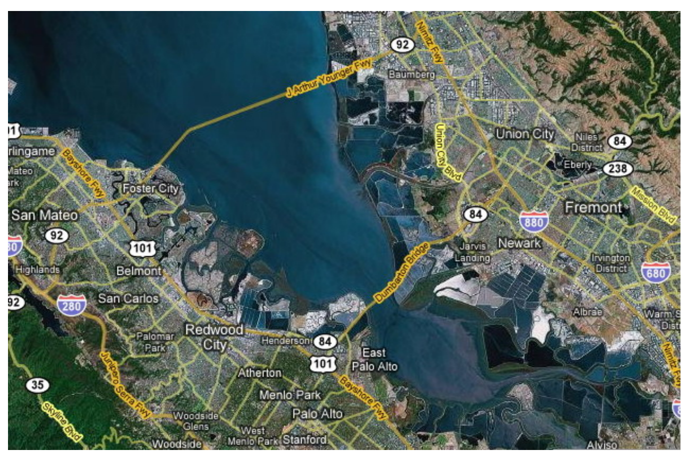

Fig. 1. Location of Dumbarton Toll Bridge

The existing Dumbarton Toll Bridge was designed in the late 1970's and the construction was completed in 1982, replacing the original 1927-era structure located 90 feet south of the existing bridge; portions of the earlier structure exist today as fishing piers. The bridge consists of the Main Channel crossing, East and West Approaches, and a Trestle structure at the end of each Approach structure. The bridge has three lanes in each direction and a separated bike/pedestrian lane on the eastbound side. A 340 foot center span provides 85 feet of vertical clearance for shipping. The approach spans on both sides of the Bay are of pre-stressed lightweight concrete girders supporting a lightweight concrete deck. The center spans are twin steel trapezoidal girders which also support a lightweight concrete deck.

While the Dumbarton Toll Bridge construction incorporated seismic resistant features as required by the post-1971 San Fernando earthquake codes, recent significant changes in seismic design practice prompted Caltrans to set off vulnerability studies in 2004 (Caltrans, 2005). The studies were unable to conclusively determine the performance of the bridge during a maximum credible earthquake. As a result, a comprehensive geotechnical study was required to determine the neccessity and extent of the seismic retrofit work for the bridge.

Earth Mechanics, Inc. (EMI) has been retained by the Bay Area Toll Authority and Caltrans to conduct geotechnical investigation to assist seismic evaluation and retrofit design of this bridge. Structural analyses and retrofit designs of the bridge are conducted by structural engineers from the Engineering Service Center within Caltrans. The geotechnical investigation program conducted by EMI consists of the following three major elements for the Dumbarton Toll Bridge: geotechnical site characterization, ground motion study and foundation analysis. This paper mainly describes the outcomes of our ground motion study, including the seismic hazard analyses, development of reference rock motion criteria for the SEE and FEE events, and kinematic soil-structure interaction analyses to develop ARS design curves. Detail can be found in an EMI report (2009) entitled “Ground Motion Study for Dumbarton Bridge Seismic Retrofit Project.”

SEISMIC HAZARD STUDY

Previous Seismic Hazard Studies.

At the onset of the study, EMI conducted a task to gather information from relevant seismic hazard studies at the vicinity of the project site. The following lists some of these studies that were performed by others:

- A URS Corporation report entitled “Draft Seismicity, Bay Division Pipelines Reliability Upgrade, Bay Tunnel Project” dated January 5, 2007 (URS 2007). This study was conducted as part of the SFPUC new pipeline located about 1,200 feet from the Dumbarton Toll Bridge.
- A report from Norm Abrahamson entitled “Peak Velocities and Peak Accelerations for Bay Division Pipe Line” dated November 6, 2003 (Abrahamson 2003). This study was conducted for seismic retrofit of the existing BDPL 1 and 2 that are partially buried and partially supported on bridge.
- A Geomatrix Corporation report entitled “Seismic Ground Motion Study for Dumbarton Bridge, Report for Caltrans” (Geomatrix 1993). This report was produced as part of the overall ground motion studies for the toll bridge program.

However, significant advances have been made in seismology and geotechnical engineering in recent years, particularly in ground motion attenuation relationships and seismic source modeling. The EMI team conducted a very comprehensive seismic hazard study for Dumbarton Bridge, embracing the latest technology and tools available at the time.

Seismic Hazard Updates for Dumbarton Bridge.

A number of developments (from about mid 1990s to current time) led to the need to update prior seismic hazard studies.

- Ground motion criteria from the Geomatrix (1993) study adopted deterministic seismic hazard procedure for formulating the design ground motion criteria. However, starting from the San Francisco-Oakland Bay Bridge East Span project around 1997, probabilistic hazard procedures have increasingly been adopted for design applications from the recommendations of the Peer Review Panel and also the State Seismic Advisory Panel. Such a procedure is also preferable for the Dumbarton Bridge in order to provide a flexible framework suitable for balancing risk versus cost for proposing the retrofit design to insurance companies and the bond holders.

- Another reason toward the need for the update is due to changes in source characterization from the USGS Working Group on Earthquake Probabilities in Northern California (WG 2003). The 2003 WG report gives the time dependent probabilities of large earthquakes during the next 30 years on seven fault systems in Northern California: San Andreas, Hayward/Rodgers Creek, Calaveras, San Gregorio, Concord, Greenville, and Mt. Diablo. All of the faults except for the Mt Diablo fault have segmentation alternatives.
- The most important reason for the update is due to development in attenuation models. As part of the PEER Next Generation Attenuation (NGA) program, empirical attenuation relationships were developed by five separate modeling teams in 2006-2007: Abrahamson and Silva, Boore and Atkinson, Campbell and Bozorgnia, Chiou and Youngs and Idriss. These newly developed NGA models represent improvements over prior vintage attenuation models (commonly referred as attenuation models dated back in the 1997s).

<u>Seismic Hazard Characterization.</u>

This subsection briefly describes the characterization of seismic parameters required by the probabilistic and deterministic seismic hazard analysis (PSHA and DSHA), including the source model, magnitude density function, rupture dimension relations, and attenuation relations.

1) Seismic Source
The location of the project site relative to the major faults in the San Francisco Bay Area is shown in Figure 2. The project site is located approximately equidistant from the Hayward and San Andreas faults. The Hayward fault is located approximately 13 km east of the east end of the bridge and the San Andreas is located approximately 14 km west of the west end of the bridge. For the major faults in the northern California, the seismic source characterization is based on the 2003 USGS Working Group on Earthquake Probabilities in Northern California (WG, 2003). The 2003 WG report gives the probabilities of large earthquakes during the next 30 years on seven fault systems in the northern California. The WG model includes Poisson and non-Poisson models of the earthquake recurrence. The 2003 WG model also allows for the fault segments to rupture separately or together. The 30-year probabilities given in the 2003 WG report are converted to equivalent recurrence intervals for use in the PHSA computer program.

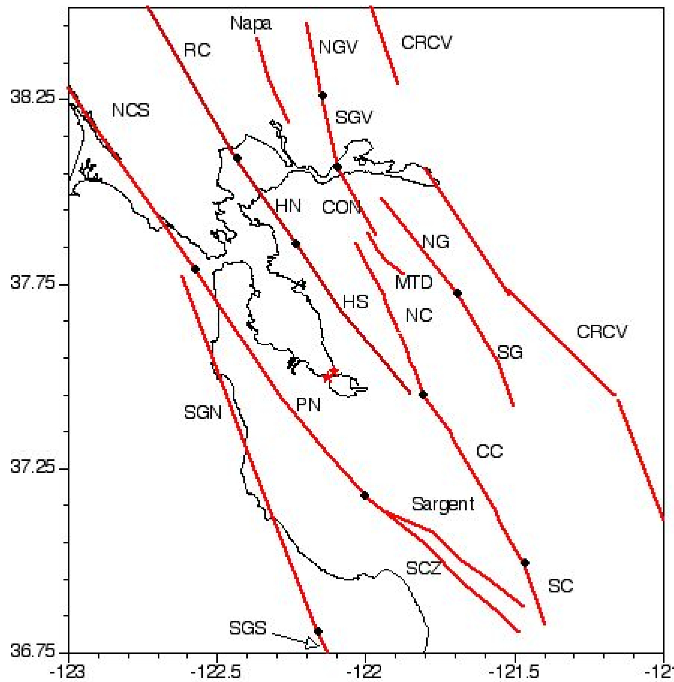

Fig. 2. Nearby faults and the site (red stars)

2) Magnitude Density Function
The magnitude density function describes how the fault slip-rate is distributed in different size earthquakes. In this study, the characteristic model developed by Youngs and Coppersmith (1985) is used. This model is very similar to the 2003 WG model; it has about 95% of the seismic moment in the characteristic part and about 5% of the seismic moment in the exponential tail, whereas the 2003 WG model has 6% of the moment in the exponential tail. The minimum magnitude used in the hazard calculation is magnitude 5.0.

3) Rupture Dimension Relations
Since the attenuation relations are based on the closest distance from the site to any point on the earthquake rupture, the dimensions of the rupture need to be specified for each magnitude. The rupture dimension is modeled using the relations for fault area and fault width developed by Wells and Coppersmith (1994) for all source types. The rupture length is computed by dividing the area by the width.

4) Attenuation Relations
The five NGA attenuation models as aforementioned were used at the later stage of this project. A key parameter used in four of the five NGA models is the used an average shear wave velocity in the top 30 m for the estimation of site effects on the predicted ground motion. The Idriss model is defined for a range of average shear wave velocity between 450 – 900 m/s rather than a specific shear wave value. More discussions will be made in the following section.

5) Directivity Effects

The five attenuation relations listed above describe the attenuation of the average of the two horizontal components of ground motion. These attenuation relations were adjusted to account for near-fault directivity effects using a modified form of the Somerville et al. (1997) fault-rupture directivity model from Abrahamson (2000). The Somerville et al. (1997) model comprises two period-dependent scaling factors that may be applied to any ground motion attenuation relationship. The two scaling factors depend on whether fault rupture is in the forward or backward direction, and also the length of fault rupturing toward the site. Rupture directivity is only applied to the major faults.

6) Vertical to Horizontal Spectra Ratios

Vertical spectra were developed by applying a vertical to horizontal spectral ratio (V/H) to the horizontal spectra. Updated vertical ground motion models have not been developed yet as part of the NGA project. Therefore, the V/H ratio was estimated based on the Abrahamson and Silva (1997) and Sadigh et al. (1997) attenuation models

Chronological Development in the 1,000-Year UHS.

The Dumbarton Bridge seismic retrofit project was initiated back in 2006, when NGA models are still being finalized. In the course of the Dumbarton Bridge retrofit project, three rounds of 1,000-year UHS solutions were generated for SEE retrofit design as the team gained insights on the newly developed NGA models:

- Version-1 dated around April, 2007: This is the version of the PSHA study conducted back in late 2006 and early 2007 using preliminary versions of the NGA models. Those NGA models were later found to be poorly constrained at long period range (e.g. at 6-second period). Such a deficiency was later found to have an adversely impact for the design of the friction pendulum isolation bearings used for retrofitting the Dumbarton Bridge.
- Version-2 dated November, 2008: This study was prompted in the course of designing the isolation bearing adopted for retrofit which led to revisiting the issue of the questionable long-period displacement curve shape in preliminary NGA models. Dr. Norm Abrahamson informed the team that the NGA models were updated and some limited study commenced referred as the Series-2 PSHA. In this Series-2 PSHA, only the 1,000-Year UHS horizontal components were generated using the updated NGA models. Up to this point (including Version-1 and -2), all five NGA models were used (equal weight among each model).
- Version-3 PSHA dated February, 2009: Further investigations of the NGA models revealed potential problems in the Boore and Atkinson and the Campbell and Bozorgnia NGA models at long period range, while the other three models appeared better constrained at very long periods. Since long-period response from large magnitude earthquakes are important for the Dumbarton Bridge, the Campbell and Bozorgnia and the Boore and Atkinson models were excluded from this version-3 PSHA and only the remaining three models were used for the Series-3 of PSHA.

Figure 3 presents some comparisons of the 1,000-year UHS for the Fault Normal (FN) component motion among the above listed versions of PSHA studies to illustrate some of the discussion above regarding the chronological development of the Dumbarton Bridge ground motion criteria. As can be observed in Figure 3, the main difference among the various PSHA solutions relates to the long-period motion (above 2-second period) demand, especially issues regarding the preliminary version of NGA developed prior to April, 2007. It can further be observed that the displacement spectral curve shape for some of the NGA models (even the updated version) do not conform to common expectation that the displacement spectrum should be asymptotic to the constant displacement demand at long period.

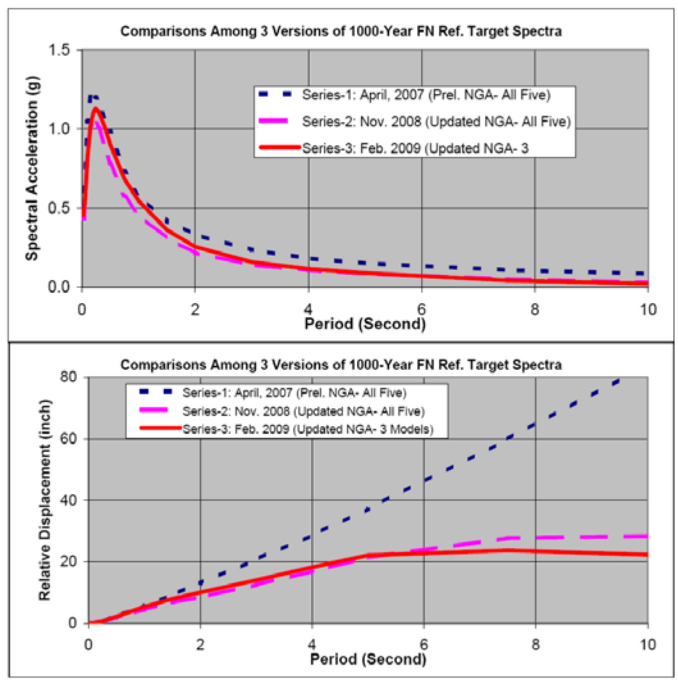

Fig. 3. Comparison among 3-series of 1,000-year FN UHS

UHS Hazard Results and Comparison to Deterministic Spectra.

This section provides a summary of the Uniform Hazard Spectra solutions and their comparison to the corresponding benchmark deterministic spectra. Hazards were originally computed for both the east end and west end of the bridge.

The hazard is similar at the two ends with the east end being slightly larger for shorter periods and larger annual probability levels (i.e., shorter return periods). The results from the west end site are slightly larger for the longer spectral periods and lower annual probability levels (i.e., longer return periods). For the average horizontal component, the largest difference between the east end and west site ground motions is less than 3% and less than 6% for the fault normal component. The design spectra are ultimately based on the spectra enveloping spectra over the east and west end sites.

The uniform hazard spectra (UHS) were computed for a suite of return periods between 100 and 2000 years. The FN spectra for these return periods are shown in Figure 4. The deterministic spectra for the average horizontal component without directivity effects are also shown in these figures. The median deterministic ground motions from the San Andreas faults correspond to return periods between 300 and 500 years. The 84% deterministic spectrum from the San Andreas faults corresponds to return period close to 2,000-year return period.

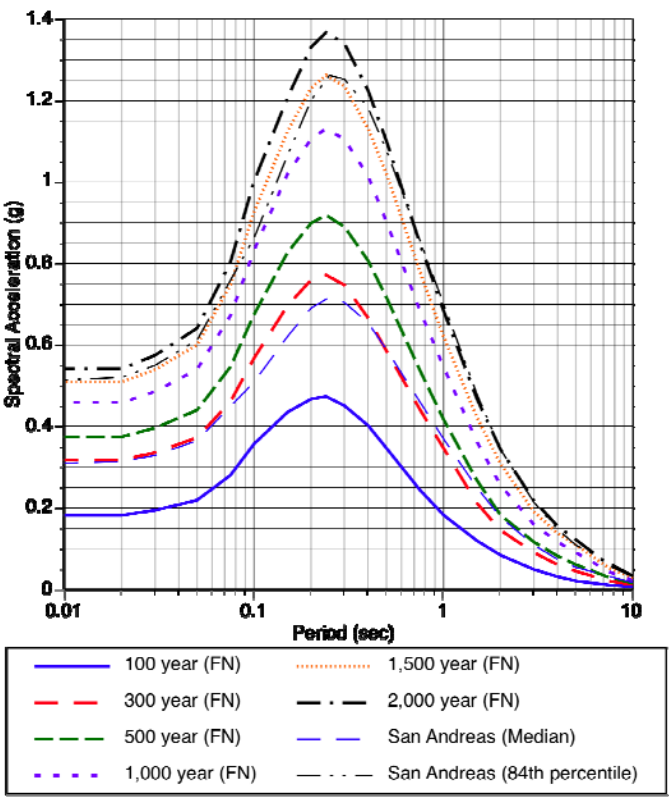

Figure 4. Uniform hazard spectra and MCE spectra

SEE and FEE Target Design Spectra.

The design team started the Dumbarton Bridge retrofit project at the end of 2007 under very tight schedule. Significant amount of design analyses were conducted using the out-dated Series-1 input motion in the course of the project, especially analyses of the vulnerability of the as-built structure and also analyses to evaluate various versions of retrofit strategies. Toward the end of 2007, use of isolation bearings for the main span became the preferred retrofit strategy and design analyses moved toward designing the isolation bearing. It was discovered at that stage that the friction-pendulum isolators (a component central to the overall retrofit strategy for the main span) have an apparent response period at about 6 second and the size of the isolator will be highly dependent on the displacement demand around 6 second. Feedback from Dr. Norm Abrahamson led to the Series-2 and Series-3 PSHA.

As can be observed from Figure 3, the issue of displacement spectral shape at long period was found to have a profound influence (cost implication) on the design of the isolators. It should be realized that this topic is clouded by problems regarding long-period correction problems of historic strong motion records and common practice toward conservatism for the acceleration spectra at long period because of the low acceleration level in force-based design practice. However, the long-period motion became a major issue in the current Dumbarton Bridge retrofit project. Realizing that the Series-1 PSHA was based on out-dated NGA models, the Series-1 PSHA was abandoned and design analyses for the Dumbarton Bridge was switched to the Series-2 and Series-3 PSHA solutions. The ultimately adopted SEE spectra are based on the Series-2 1000-year return period UHS and the FEE spectra are based on the Series-3 100-year return period UHS, as shown in Figure 5.

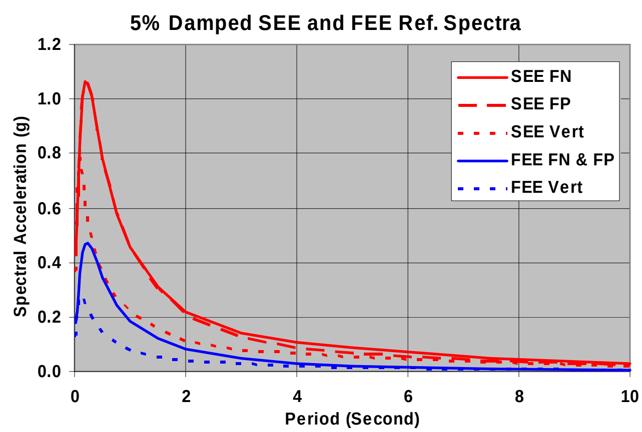

Figure 5. SEE and FEE acceleration reference spectra

Time History Records for Design Analyses.

The use of spectrum-compatible input time histories has been widely adopted for bridge design projects due to complexity in defining the predominant period for bridge structures and also in order to avoid the need for conducting design analyses using an excessive number of input time histories. Value Analysis Study of the Antioch and Dumbarton Bridges conducted in 2006 recommended generation of seven sets of spectrum-compatible time histories to establish adequate database for the seismic analyses of the bridge structure.

Seven sets of spectrum-compatible time histories were developed by modifying startup motions (usually actual earthquake records) so that the resultant spectra are similar to the reference rock spectra. 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 time domain method (Abrahamson 1998) is used.

As part of the spectral matching procedure, a baseline correction is applied to the ground motions. The baseline is computed by fitting the displacement time history to a high order polynomial (order 4 to 7) and excluding the constant and linear terms. The second derivative of this displacement baseline is computed and it is subtracted from the acceleration ground motion. The resulting ground motion is baseline corrected in acceleration, velocity, and displacement.

For the SEE event, the project team selected to use the same six sets of seed motions from the East Span San Francisco – Oakland Bay Bridge, and another seed motion from the 1999 Taiwan Earthquake. Figure 6 presents some statistics of the spectral demand from averaging these seven sets of spectrum-compatible motions. The average of 7 sets of motions have been shown in blue which can be compared to the intended target (implied by the FN and FP target spectra) shown in black. In contrast to individual motion set which can deviate from the intended target demand in rotated directions, especially at long-periods, the benefit of averaging seven sets of motion becomes evident from this figure. It can be observed from the figure that averaging 7 sets of input motions was very effective in arriving at the intended demand implied by the target fault normal and fault parallel spectra, not only in the principal axes of reference, but in all rotated directions.

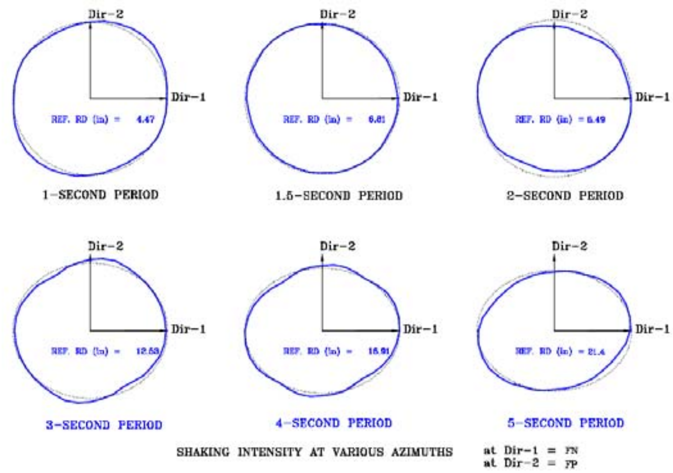

Figure 6. Average of 7 Sets of SEE spectrum-compatible motions

For the FEE event, the project team selected to use another seven sets of seed motions for representing the site seismic characteristics. Figure 7 presents some statistics of the spectral demand from averaging seven sets of FEE spectrum-compatible motions. The average of 7 sets of FEE motions have been shown in blue which can be compared to the intended target (implied by the FN and FP target spectra) shown in black. Again, the benefit of averaging seven sets of motion becomes obvious.

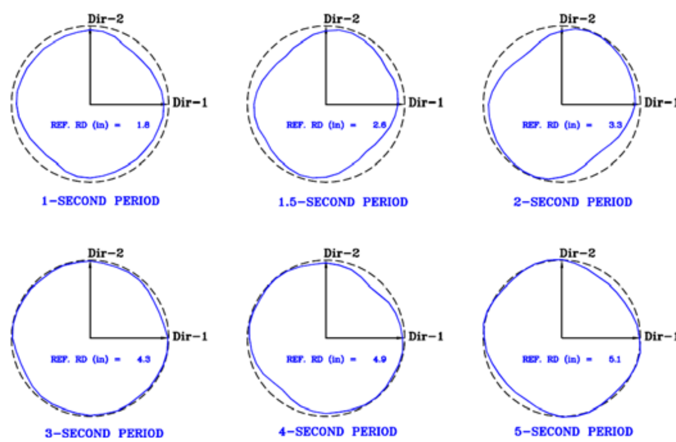

Figure 7. Average of 7 Sets of FEE spectrum-compatible motions

SITE RESPONSE AND SOIL-STRUCTURE INTERACTION

<u>Subsurface Conditions.</u>

As part of the geotechnical investigation program, EMI conducted extensive site exploration in the marine and on-land areas for the Dumbarton Toll Bridge. While most borings and CPT sounding investigated in this program penetrated depths of approximately 250 ft below the modern southern San Francisco Bay surface, some of the borings drilled by CALTRANS in 1995 were in excess of 600-feet depth. The following units have been identified and interpreted based on current and existing borehole data, geophysical information and previous work (Figure 8).

Fill – Silty clay and silty sand present from elevation +10 ft to -10 ft, underlying the Trestles at both ends of the bridge;

Young Bay Mud (YBM) – Marine clay underlies the fills, generally found underneath both Approaches and Trestles from elevation 0 to -40 ft;

Posey Sand (PS) – River sand can be found throughout the bridge alignment from elevation -40 ft to -80 ft;

San Antonio Formation (SAF) – Stiff to very stiff clay can be found from elevation -70 ft to -140 ft;

Old Bay Mud (OBM) – Very stiff to hard marine clay, found from elevation -120 ft to -190 ft;

Alameda Formation – Very dense sand and gravel, and very hard clay can be found below elevation -190 ft;

Franciscan Formation – Sedimentary bedrock, expected at elevation -600 feet.

Figure 8 presents the idealized subsurface soil profile developed along the bridge. Within the limit of bridge alignment, the subsurface conditions underlying the site are relatively uniform. A separate more detailed site characterization report has been prepared for the project documenting geology, site investigation, field and lab testing, and interpretation of geotechnical conditions along the bridge (EMI 2007).

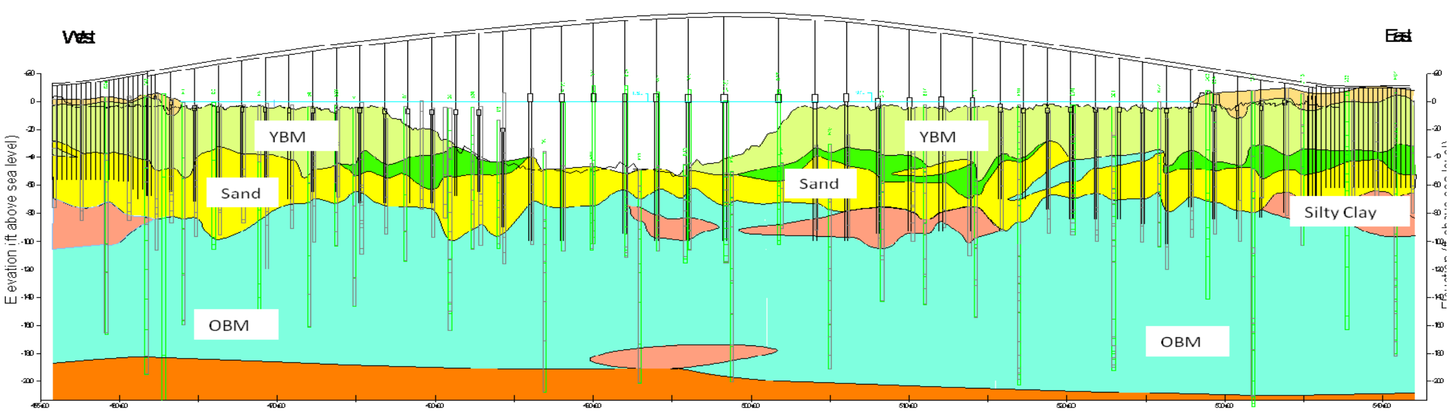

Fig. 8. Idealized soil profile along the bridge alignment

Description of the Bridge.

The entire existing bridge, consisting of the Main Channel crossing, East and West Approaches, and a Trestle structure at the end of each Approach structure, was supported on pile foundations. The West Approach is composed of Abutment 1 through Pier 16, each founded on 20-inch diameter steel pipe piles with a buried pile cap. For the Main-Crossing Channel, Piers 17 to 26 are supported on 54-inch diameter concrete hollow piles with a long cantilever pile length (18 ~ 48 feet) above mud-line, while Piers 27 to 30 are founded on 20-inch diameter steel pipe piles with a buried pile cap. The East Approach is composed of Piers 31 to 43 and Abutment 44, each carried by 20-inch diameter steel pipe piles with a buried pile cap. All the 20-inch diameter steel pipe piles had been completely in-filled with concrete from top to tip, while the 54-inch diameter concrete hollow piles had been partially in-filled with concrete from top to a certain elevation below the mudline. The design capacity of a single 20-inch diameter pipe pile is 80 tons, and that of a single 54-inch diameter hollow pile is 250 tons.

The elevation of pile top is between -22 and +1 feet, while that of pile tip is between -100 and -65 feet. The elevation of mudline varies from -48 to +10 feet, with the lowest values near Piers 21 and 22. The Trestles at two ends of the Approaches are supported on pile extensions of 20"×20" cross-section; each trestle has 20 trestle bents with both the north and south faces supported by pre-cast retaining walls. Among these bents, East Bents 1, 20, and West Bents 1, 20 have narrow pile-caps (cap beam) at the mudline. The design capacity of a single 20"×20" concrete pile is 90 tons.

Among all the bridge piers, seven representative piers were selected for development of ARS design criteria involving site response and kinematic soil-pile interaction analyses. The seven representative piers are:

- Piers 2, 9, and 15: within West Approach supported on 20 inch diameter steel pipe piles with a buried pile cap
- Piers 17 and 23: within Main Span founded on 54 inch diameter concrete pipe piles with a long cantilever pile length above mudline (see Figure 9 for the soil conditions and foundation data of Pier 23)
- Piers 30 and 43: within East Approach supported 20 inch diameter steel pipe piles with a buried pile cap.

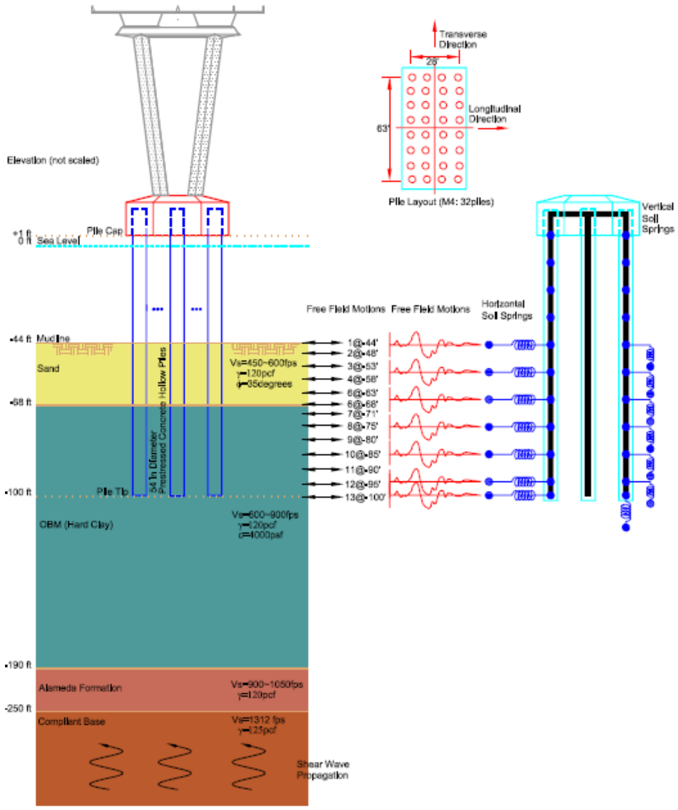

Fig. 9. Soil conditions and foundation data at Pier 23

Dynamic Soil Properties.

Small-strain shear modulus G_{max} is best estimated from shear wave velocity values that are measured in the field and by using the relationship of $G_{max}=\rho V_s^2$. In the equation, ρ is the soil density and V_s is the measured shear wave velocity. Down-hole seismic suspension logging (P-S logging) was conducted to supplement stratigraphic information and establish shear wave velocity profiles of the subsurface. Detailed descriptions of the instrumentation, procedures results and analysis of the seismic suspension logging are presented in the site characterization report (EMI 2007).

Soil dynamic properties in terms of normalized shear modulus and damping curves have been studied by many researchers. For the site response analyses, we adopted the Seed et al. (1986) relationship for sand and the Vucetic and Dobry (1991) for clays. To deal with potential variations in the determination of in situ soil properties, the following parametric studies were considered: 1) the best-estimate case established from the down-hole shear wave velocity measurements; 2) a lower bound case; and 3) an upper bound case. Scaling factors of 0.75 and 1.25 were used as multiplication factors on the best-estimate shear-wave velocity for lower and upper bound scenarios.

Free Field Site Response Analyses.

Site response analyses were conducted using the computer program SHAKE91 (Idriss and Sun, 1992), an equivalent linear analysis. The program SHAKE91 has been used for solving one-dimensional shear wave propagation problems for three decades. Engineers have accumulated knowledge from the performance of SHAKE in predicting ground response during earthquakes. To avoid unrealistic prediction of free-field motion for a long soil column, a relatively short soil column was used for our site response analyses. A transmitting boundary was selected near Elevation (El.) -250 feet where an average shear wave velocity is 400 m/sec which was a controlling parameter in the NGA attenuation models.

Free field site response analyses were conducted at each pier for the fault normal and fault parallel ground motions. However, no site response analysis was preformed for the vertical ground motion which is a practice adopted in all toll bridge programs. Horizontal free-field motions at different depths along the pile length for Pier 23 are shown in Figure 10. The uniform shear strain (defined as 65% of the maximum shear strains) profile from the response analysis for this pier is presented in Figure 11 from all seven input motions. Generally the uniform shear strains are less than 1.5% for all piers and all sets of input accelerations.

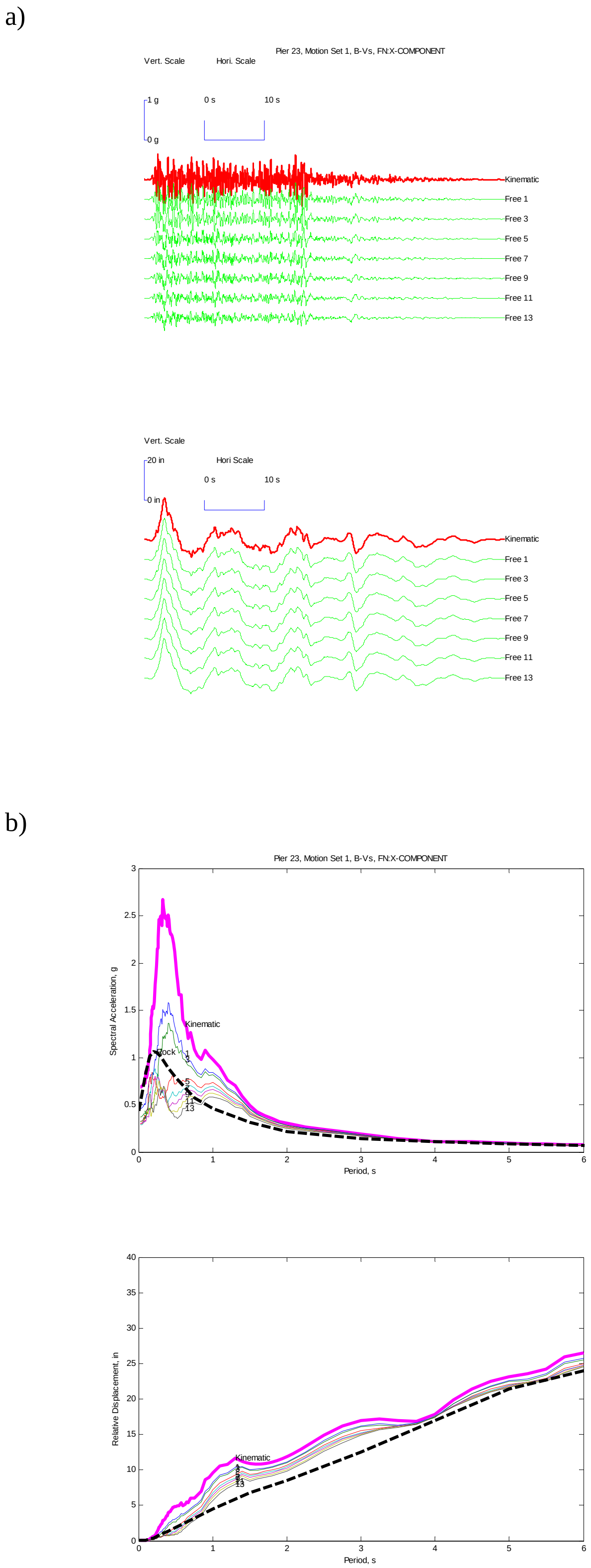

Fig. 10. Free-field site response for Pier 23: a) time histories; b) spectra

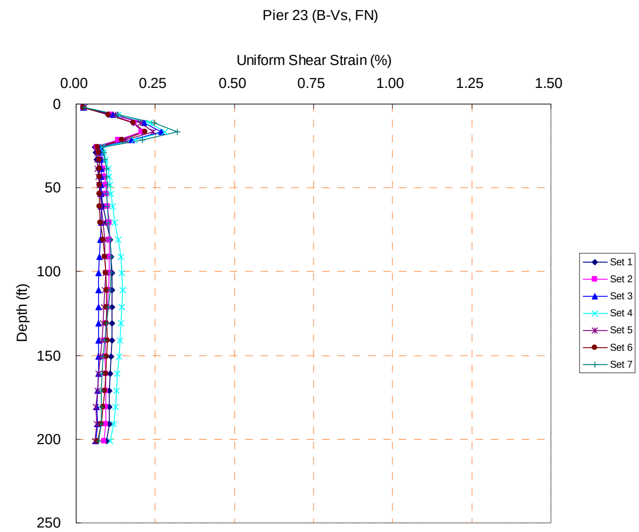

Fig. 11. Uniform shear strain profile for Pier 23

It is interesting to note the local site response effects. The approximate relations between PGA at ground and on rock for selected piers are present in Figure 12. The differences between these curves are small for pier number 02, 09, 15, 17, 23, 30 and 43. It is implied that PGAs at ground due to the free field seismic response for these piers are approximately uniform. As may be seen in the same figure, for PGA on rock less than 0.4g, the curves for these piers is considerably lower than the curve suggested by Idriss (1991), which was based on the data of 1985 Mexico City and 1989 Loma Pieta earthquakes. One reason may due to the low stiffness and nonlinearity of soft soils (Young Bay Mud) at these piers.

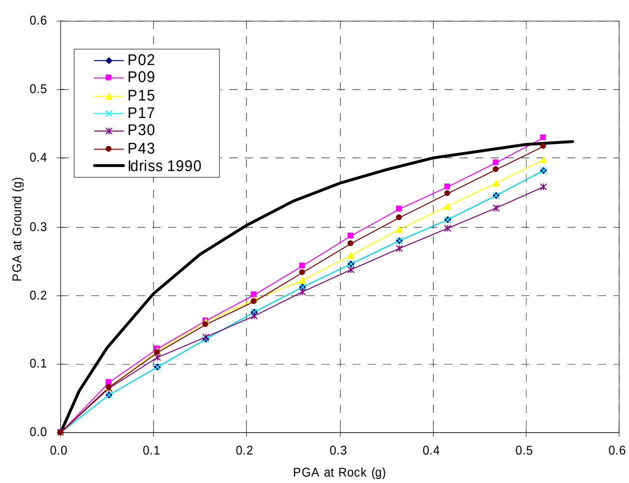

Fig. 12. Variation of acceleration at ground vs at rock

Kinematic Soil-Pile Interaction Analysis.

To rigorously develop the design response spectra for the pile-supported structure, soil-pile interaction was considered. The method is based on a linear theory making use of the sub-structuring procedure (see Lam and Law 2000 for detail). The first step involved linearization of *p-y* curves by performing lateral pushover analysis of a single pile to a representative displacement level expected during the earthquake. A pile foundation model was then created in which each pile was supported on elastic soil springs that were excited by depth-varying, free-field motions computed from the site response analyses. Sub-structuring was performed to compute resultant forces acting at the deck level. The resultant forces were divided by the foundation stiffness to result in so-called kinematic motions. The kinematic motions formed the basis for development of ARS design curves for the pile-supported structure. The kinematic motion is calculated at the pile cap level and implicitly contains the statically condensed forces transmitted from the ground to the superstructure along the entire embedded pile length. Therefore, the effects of the depth-varying shaking intensity in the soil column, the depth-varying soil stiffnesses, and the pile properties are included in the solution. The following pile properties were used in conducting kinematic soil-pile interaction analyses:

- 54-inch diameter concrete piles (Main Channel Piers 17 ~ 26)
  Effective pile EI = 8.35×10^{11} lb-in^2. (Infilled section)
  Effective pile EI = 5.83×10^{11} lb-in^2. (Hollow section)
- 20-inch diameter steel pipe piles (the remaining piers)
  Effective pile EI = 3.59×10^{10} lb-in^2.
- 20"×20" square concrete piles (Trestle bents)
  Effective pile EI = 2.40×10^{10} lb-in^2.

A fixed pile head condition was assumed in all the cases. The analyses were conducted with the in-house computer program KIPS, which is dedicated to performing kinematic soil-pile interaction analyses (Earth Mechanics, Inc., 1999). Considering seven motion sets and three levels of shear wave velocity, 21 kinematic spectra (spectral accelerations and displacements) for each selected pier were computed, from which their mean and mean plus one standard deviation spectra were then obtained.

Development of ARS Curves.

For development of ARS design curves, the kinematic motions of the seven selected piers were evaluated considering potential variations of dynamic soil properties. The variation consisted of three shear wave velocity scenarios: best estimated, lower bound, and upper bound. The selected piers (Piers 2, 9, 15, 17, 23, 30, and 43) should cover different pile types, different pile cantilever lengths, and subsurface conditions along the bridge. Spectral accelerations and displacements for Pier 23 is presented in Figure 13, showing all kinematic motions from seven input ground motions with three different shear wave velocity profiles (a total of 21 runs = 7×3). In this figure, mean and mean plus one standard deviation spectra computed from the 21 runs are also presented. The mean spectrum (thick black line) represents the 50th percentile confident level, while the mean plus one standard deviation spectrum (top thick dot line) represents the 84th percentile confident level accounting for variation in soil conditions and different ground motions.

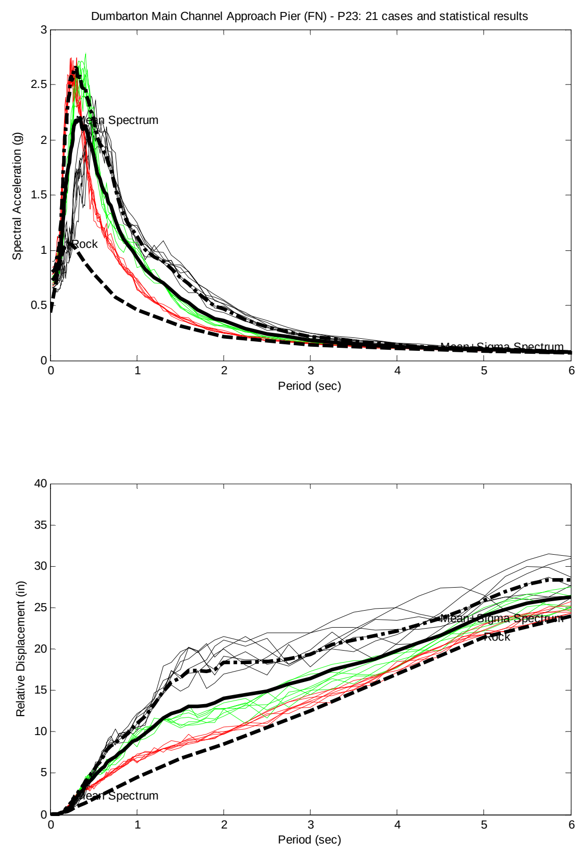

Fig. 13. Soil conditions and foundation data at Pier 23

The shape of response spectra as obtained from kinematic soil-pile interaction analyses sometimes contains multiple peaks and valleys. From past experience, this type of spectrum shape with multiple peaks and valleys often results in difficulty with the structural design process. For example, the lengthening of a structural period could lead to a higher spectral acceleration and a higher load demand. For this reason, it was decided that the final ARS recommendations should be constrained by a well-behaved ARS curve shape for both spectral acceleration and displacement (i.e., the final ARS curves should not contain multiple peaks and valleys). With this consideration, "smooth" spectra were developed. From a review of the kinematic spectra from the seven piers, it appears that two sets ARS curves would be adequate to cover horizontal loading for the entire bridge (see Figure 14):

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

The ARS Curve No.2 recommended for Pier 17 to 26 has higher shaking at the shorter period range (0 to 2-Second). At these piers, the foundation system consists of 54-inch concrete piles, with the pile cap cantilevered typically about 48-ft above the mudline. For such cantilevered pile cap foundations, there is a significant rotational motion of the pile at the mudline elevation. The pile rotation, when amplified by the long cantilever height of the pile cap, leads to a higher kinematic pile cap motion as compared to embedded pile cap kinematic motions. The ARS Curve No.1 is higher than the ARS Curve No.2 at the long period range (greater than 2-second), because the smaller piles associated with ARS Curve No. 1 would have shallower soil-structure interaction zones (i.e. point of fixity of the pile) where the free field motions would have a consistently higher long-period spectral displacements at shallower depths within the soft bay mud. For the vertical spectrum, we recommend using the reference vertical motion spectrum from PSHA without further site response analysis.

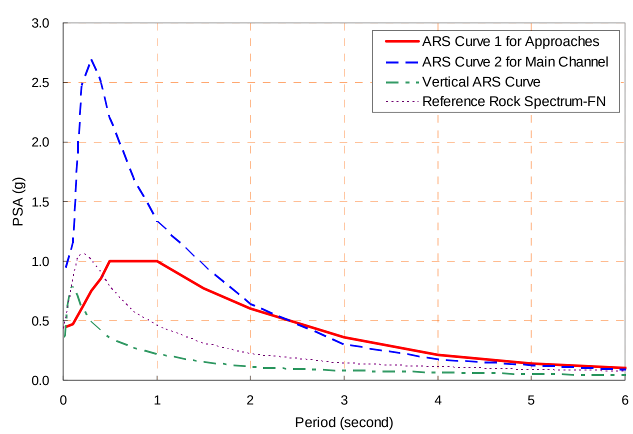

Fig. 14. ARS design curves for safety evaluation earthquake

SUMMARY

This paper presents our findings on probabilistic earthquake analyses for the Dumbarton Bridge Seismic Retrofit Project and the results are compared with the deterministic spectra. Site response and kinematic soil-pile interaction analyses were conducted to develop pier-specific kinematic time historiest and to derive the design ARS curves for the project. The following lists the major points from our conducted studies:

- Uniform hazard spectra were developed for the reference rock motion at six return periods: 100-, 300-, 475-, 1,000-, 1,500-, and 2,000-years.
- The 1,000-year return period spectra were adopted for the Safety Evaluation Earthquake (SEE), and the 100-year return period spectra were adopted for Functional Evaluation Earthquake (FEE).
- Seven sets of rock motion time histories have been developed for each of the SEE and FEE events. They are all spectrum compatible to the target spectrum; almost all the time histories were started from an actual earthquake record.
- Free-field site response analyses were conducted at seven selected piers. All seven time histories were used in the site response studies. Sensitivity site response analyses were conducted by lower bound, best estimated, and upper bound shear wave velocities.
- Kinematic soil-pile interaction analyses have been carried out using the pile properties and configurations of the selected piers to provide ARS design spectra for the bridge.
- Time histories of kinematic motion and depth-varying free-field motions were developed at all piers for non-linear time history analysis of the bridge.
- Two sets of ARS design curves are recommended for the horizontal loading for the Safety Evaluation Earthquake. The developed ARS criteria correspond to shaking from the stronger fault normal (FN) component motion.
- The ARS Curve No.1 is recommended for all piers which have 20-inch diameter piles and the pile caps typically embedded below the mudline. It is applicable to all approach and trestle piers, and some piers of the Main Span (i.e., Piers 1-16, Piers 27-44, Bents W1-W20, and Bents E1-E20)
- The ARS Curve No.2 is recommended for Piers 17 to 26 of the Main Span, supported on 54-inch diameter concrete piles with the pile cap cantilevered above the mudline.
- The single vertical ARS curve, as shown in Figure 14, is recommended all piers.

ACKNOWLEDGEMENT

The authors are very grateful to the following individuals who had provided significant contribution to this project:

- CALTRANS: Mo Pazooki, Robert Price, Mike Keever, Tariq Masroor, Yong-Phil Kim, Steve Mitchell, Anoosh Shamsabadi
- Peer Reviewers: I.M. Idriss, Frieder Seible, Joseph Nicoletti, Jerry Fox
- Consultants: Norm Abrahamson, Jennie Watson-Lamprey, Robert Pyke, Doug Hamilton

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