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CURRENT SEISMIC SOIL-FOUNDATION-STRUCTURE INTERACTION STATE OF THE ART AND PRACTICE ON CALIFORNIA TOLL BRIDGE PROGRAM

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ABSTRACT

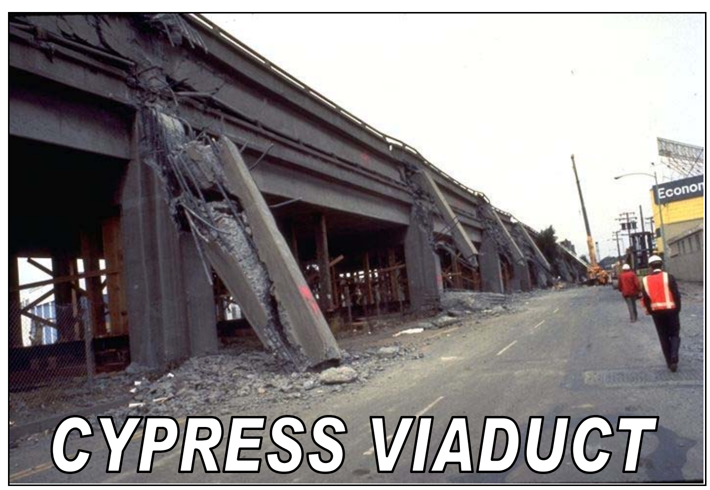
California Department of Transportation (CALTRANS) initiated a seismic retrofit program in early 1990 following the Loma Prieta and Northridge Earthquakes to strengthen existing toll bridges and many regular bridges in California. Prior to the CALTRANS' seismic retrofit program, there were very little guidelines and criteria available to undertake seismic retrofit of existing bridges and design of new structures to withstand potentially large magnitude future earthquakes. Significant advancements have been made since the beginning of the seismic retrofit program.

This paper will discuss the author's experience from the seismic retrofits of many existing toll bridges and designs of new toll bridges. The lessons learnt from the seismic retrofit program paved the way for the designs of new major bridges, including East Span San Francisco – Oakland Bay Bridge, New Carquinez Bridge, New Benicia – Martinez Bridge, and New Gerald Desmond Bridge.

In the areas of seismology, many advances have been made following the measurements of strong motion earthquakes in Turkey and Taiwan, which have significant impact on establishment of ground motion criteria for recent major bridge projects. Site response and soil-foundation-structure interaction (SFSI) analyses have been improved over the last decade since the first seismic retrofit was undertaken.

INTRODUCTION

The Loma Prieta earthquake of October 17, 1989, a Magnitude 7.1 earthquake caused damage related costs of over $6 billion and collapses of many transportation systems. The collapses of the Cypress Viaduct of Interstate 880 and the Bay Bridge span reminded engineers of deficiencies of many bridges that had been designed to the standards at the time they were built. The Cypress Viaduct was designed and constructed to CALTRANS seismic requirements for reinforced concrete when it was built in 1950, and the Bay Bridge was designed for 0.1 g static equivalent loading, comparable to the level specified for earthquakes in 1930 Uniform Building Code for buildings (The Governor's Board of Inquiry, 1990). Following the 1989 Loma Prieta earthquake, CALTRANS appointed a Seismic Advisory Board to provide an evaluation of CALTRANS seismic policy and technical procedures. At the same time, CALTRANS began to implement a seismic retrofit program of state-owned bridges.



The Northridge earthquake of January 17, 1994 (Mw=6.7) in the Los Angeles area provided opportunities for the Seismic Advisory Board and CALTRANS to evaluate the newly implemented retrofit program. There were 24 retrofitted bridges in the region of very strong ground shaking and a total of 60 in the region having peak accelerations of 0.25 g or greater. The retrofitted structures resisted the earthquake motions much better than the unretrofitted structures. The Seismic Advisory Board concluded that if seven collapsed bridges had been retrofitted, they would have survived the earthquake with little damage (Seismic Advisory Board, 1994).

In 1995, CALTRANS started the Toll Bridge Seismic Safety Program that included the seismic retrofit of California's toll bridges and eventual replacement of some older bridges. Due to the age and complexity of these long-span bridge structures, the retrofit program has presented unique engineering challenges. During the period between 1995 and 1998, CALTRANS completed the seismic retrofit design of seven toll bridges: Benicia-Martinez Bridge, Carquinez Bridge, Richmond-San Rafael Bridge, West Span Bay Bridge, San Mateo-Hayward Bridge, Vincent Thomas Bridge, and San Diego-Coronado Bridge.

The Dumbarton Bridge and Antioch Bridge are two California toll bridges that were based on design criteria developed after the 1971 San Fernando Earthquake, and thus these two structures had the seismic resistant features required by the post 1971 codes. In 2006, a detailed seismic evaluation of the Dumbarton and Antioch Bridges was initiated to assess earthquake performance, identify potential vulnerability, and develop a retrofit strategy.

In addition to the seismic retrofit program, three new toll bridges are built by CALTRANS as part of the Toll Bridge Seismic Safety Program to replace some of the old bridge structures, and the Port of Long Beach plans to replace the existing Gerald Desmond Bridge with a cable stay bridge.

CALIFORNIA TOLL BRIDGE PROGRAM

The following briefly describes some of the toll bridges that are seismically retrofitted:

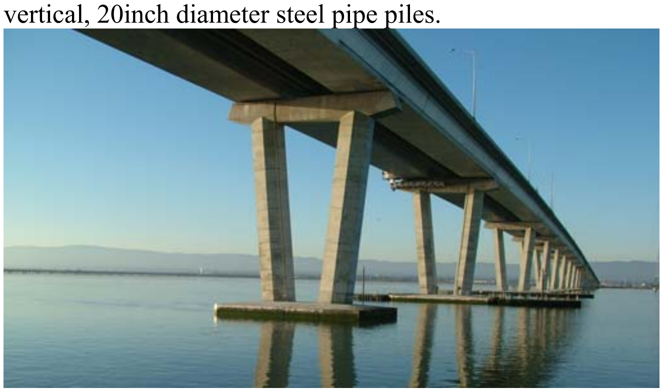
San Diego-Coronado Bay Bridge: a 7,400-ft long concrete box girder bridge with a 90-degree-turn alignment in San Diego, California. Foundations consist of 54-in diameter hollow concrete piles driven in dense sand. Typical foundation consists of 12 to 35 piles.

Vincent Thomas Bridge: a 2,500-ft long suspension bridge in Long Beach, California. The anchorages supporting the suspension cables are founded on 188 steel or reinforced concrete piles; most piles are battered. The towers are supported on 165 piles

Existing Carquinez Bridge: a 3,300-ft long twin steel-truss bridge in Vallejo, California. Some piers are supported on gravity caissons resting on bedrock and other on steel or timber piles driven in bay mud.

Richmond – San Rafael Bridge: a 15,000-ft long steel truss bridge in Richmond, California. Most piers are founded on steel HP piles arranged in circular or rectangular patterns. Bay mud is the predominant soil type in this area. There are over 70 piers supporting the structure.

Dumbarton Bridge: The Dumbarton Toll Bridge is an 8,600-foot long bridge structure, connecting the cities of Newark and East Palo Alto. The structure is founded on pile foundations consisting of 54-inch diameter hollow concrete piles and vertical, 20inch diameter steel pipe piles.

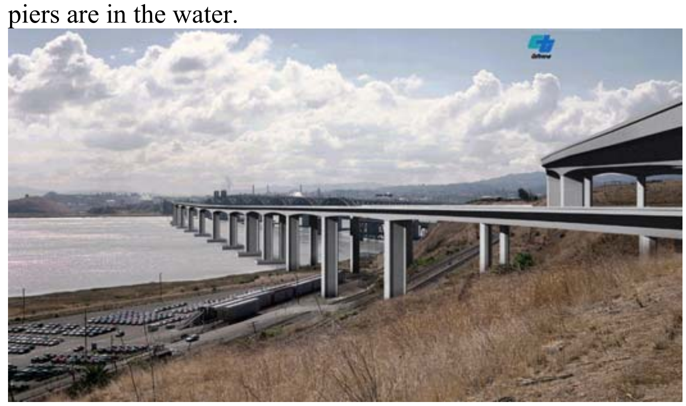


Antioch Bridge: It is a 9, 437 feet long bridge structure over San Joaquin River connecting the City of Oakley and the Sherman Island, CA. The main bridge is supported on 24-inch square driven concrete piles, 30-inch diameter steel piles, or 14-inch square concrete piles.

As part of the seismic safety program, CALTRANS plans to build three new toll bridges in the San Francisco Bay area. In southern California, Port of Long Beach, in association with CALTRANS, also plans to construct a major bridge across a shipping channel. Construction cost for each of these bridges ranges from $200 millions to $6 billions. The following provides a summary of these new bridges:

New Carquinez Bridge: a 3,500-ft long suspension bridge in Vallejo, CA. One of the anchorages is supported on a group of 380 steel pipe piles in soft bay mud, and the other is gravity caisson on rock. Two tower foundations are founded on 10 ft diameter piles.

New Benicia-Martinez Bridge: a 7,000-ft long box girder bridge in Benicia, CA. Typical foundations consist of about 9 piles, each is a 8.2 ft diameter concrete filled steel casing in sediment transitioning to 7.2 ft rock socket in bedrock. Ten piers are in the water.



New Gerald Desmond Bridge: a cable-stay bridge in Long Beach, CA. The main cable-stay span, consisting of two towers, is about 2,000 ft in length. The towers are supported on a group of large diameter piles driven in silty and sandy materials.

East Span San Francisco – Oakland Bay Bridge: a single tower suspension bridge, 1,800 ft in length, connected to a 6,900-ft long box girder viaduct located in the Bay Area, California.

Three supports of the main suspension bridge are in drastically different site conditions ranging from competent sandstone to bay mud. The viaduct, which is also known as Skyway in the contract plan, is on 8.2 ft diameter driven steel piles in bay mud and alluvium soil. Approach structures are on various foundation types, including pipe piles, HP piles, rock sockets and spread footings.

Prior to the CALTRANS seismic retrofit program, there were very few guidelines and criteria available to undertake the seismic retrofit of existing bridges and the design of new structures to resist strong earthquakes. The first generation of seismic retrofit was based on a textbook type of procedure along with some of the practices employed in a nuclear power industry and offshore oil platform designs. Since then, the retrofit design procedures have been modified regularly; various refinements were direct results of the experience learnt from the previous analyses and design processes, especially from the seismic retrofit of the toll bridges. The experience gained from the seismic retrofit program of the existing toll bridges contributed significantly to the development of seismic design criteria and analysis techniques used for the designs of new major bridges in California.

ROCK MOTION CRITERIA

Site-specific seismic hazard assessments were performed for the Toll Bridge Seismic Safety Program utilizing both deterministic and probabilistic approaches. A dual seismic design criterion was usually adopted consisting of a Safety Evaluation Earthquake (SEE) and a Functional Evaluation Earthquake (FEE). The SEE is defined as the most severe event that can reasonably be expected to occur at the site. For all the toll bridges, return periods between 1,000 and 2,000 years were selected as the basis for target spectra for the SEE scenario. The FEE is defined as one that has a relatively high probability of occurrence during the lifetime of the structure. Typical return periods of the FEE event ranged from 100 to 300 years. Figure 1 shows the rock motion criteria for the East Span San Francisco – Oakland Bay Bridge Replacement project showing three components; fault normal, fault parallel, and vertical.

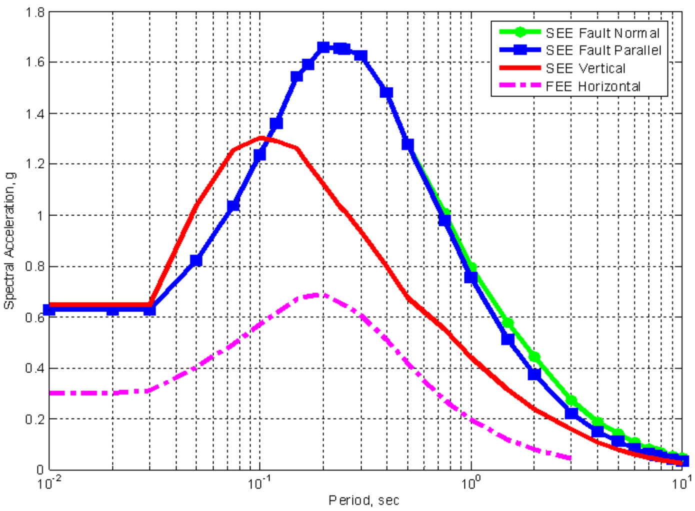


Figure 1 Rock Motion Criteria for the East Span San Francisco – Oakland Bay Bridge

The following are the seismic design criteria for some CALTRANS toll bridges:

- East Span San Francisco-Oakland Bay Bridge: A 1,500-year return period ground motion is adopted for the SEE, and a 92-year return period for the FEE.
- Benicia-Martinez Bridge: The shaking level for the SEE is an event with a 1,000 to 2,000-year return period. The FEE is defined as a 300-year return period.
- Richmond-San Rafael Bridge: At long periods, the SEE spectra fall between 1,000 to 2,000-year return periods. The FEE corresponds to a 300-year return period.
- San Mateo-Hayward Bridge: The SEE shaking level is close to a 1,000 to 2,000-year return period and the FEE is close to 300-years.
- Carquinez Bridge: The SEE event corresponds to return periods between 1,000 to 2,000-years at long periods. A 300-year return period is taken as the FEE event.
- Vincent Thomas Bridge: The SEE is an event with a 950-year return period. The FEE is defined as a 285-year return period.
- San Diego – Coronado Bay Bridge: The SEE and FEE return periods are 1,000 years and 300 years.
- Dumbarton Bridge: The SEE ground motion has a return period of 1,000 years. FEE ground motion 100-yr return period.
- Antioch Bridge: The SEE ground motion has a return period of 1,000 years.

The seismic performance expectations of the toll bridges varied for different structures. In general, they were designed to meet the 'no-collapse' criteria under the Safety Evaluation Earthquake, and to ensure prescribed levels of services for the Functional Evaluation Earthquake. In addition, the Vincent Thomas and the San Diego-Coronado Bridges were required a third design level, namely for Fault Rupture offset, in conjunction with the 'no-collapse' performance requirement for the SEE event. This requirement was due to crossing potentially active faults with the chance of ground surface fault offset within the bridge alignment.

Near-fault ground motions are essential elements of the seismic hazard evaluation. Rupture directivity is strongest in the fault normal direction and affects long period response (0.6 sec and longer). The near-fault effects are known to cause severe damage to long-span bridge structures, and were implemented in the toll bridge program. As seen in Figure 1, spectral acceleration values for the fault normal direction are higher than those of the fault parallel direction.

TIME HISTORY DEVELOPMENT

Non-linear time history analyses of the bridge structure were required due to the long span nature of the bridge, and thus the development of strong motion time histories was an essential part of the definition of seismic hazard. The earthquake time histories were constructed by modifying actual earthquake records in terms of the amplitude of their frequency content to match the entire design spectrum adopted for the design. The resultant time histories, denoted as spectrum-compatible motions, were used for all the toll bridges instead of scaling actual past earthquake records.

Other than matching of the response spectrum for each of the ground motion components developed for structural designs, correlation between the two horizontal orthogonal directions must be checked. In order to ensure that all structural components are adequately excited, the two horizontal components must be statistically uncorrelated. This requirement was recommended by the Seismic Advisory Board during the seismic retrofit program (Seismic Advisory Board, 1996), and the relationship between two orthogonal components was examined by a cross-correlation coefficient. The idea behind this requirement is to guarantee that the input motions to the structural model should have a minimum acceptable shaking intensity even after the ground motion time histories are rotated to any structural orientation of interest.

The earlier seismic retrofit work was based on one set of earthquake time histories consisting of three orthogonal components, while three sets of time histories were adopted for the seismic designs of the new toll bridges except the San Francisco – Oakland Bay Bridge, which utilized six sets of earthquake time histories. Figure 2 depicts a fault normal rock motion for the Dumbarton Bridge. From the project experience, the structural designers reported that earthquake demand quantities, derived from three different sets of time histories, sometimes varied as much as 200% or more. When this occurred, the engineers were faced with a dilemma of how to design for such a wide range of results, i.e., to average or envelop the demand quantities from the different time history analyses. This became a serious issue and raised the question to seismologists: “Can a single spectrum-compatible time history ever be developed that will lead to the largest structural demand?” Clearly, the seismologists could not predict which set of earthquake time histories would yield the greatest structural response and the subject was discussed among the design teams and the CALTRANS Peer Review Panels on numerous occasions during the course of Seismic Safety Program.

It is generally recognized that the time history analysis is a form of stochastic process, and that using a single earthquake time history does not yield a statistical mean. One needs to examine a suite of earthquakes to obtain statistically stable mean response to form the basis for design decision. The consensus on this subject is that peak structural response should be used when three sets of time histories are used in the analyses. To use an average of structural responses, a minimum of seven sets of time histories that are matched to a same spectrum must be considered. So far, the peak response from three sets of time histories has been the design basis for the new toll bridges except the East Span San Francisco – Oakland Bay bridge which adopted the peak response from 6 sets of time histories. The recent retrofit program for the Dumbarton and Antioch Toll Bridges employ seven sets to earthquake time histories.

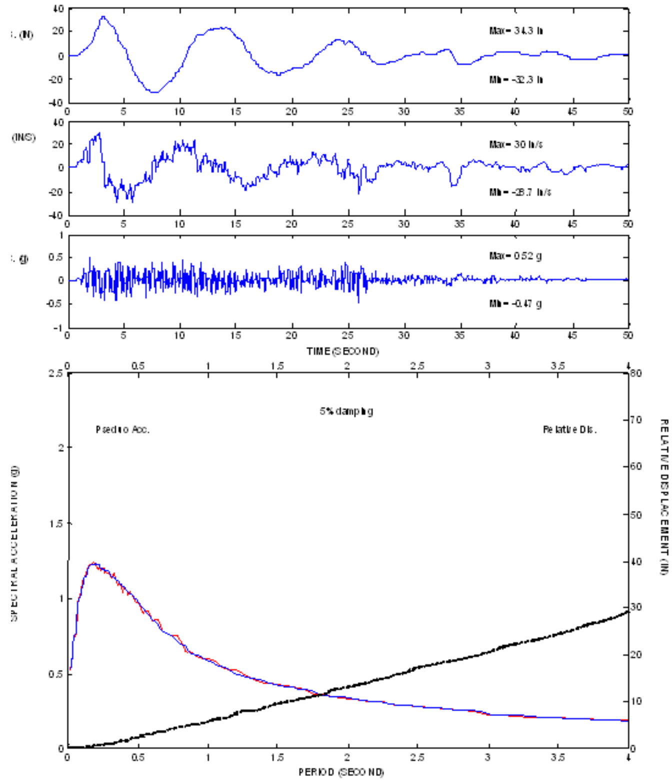


Figure 2. Set 1 Fault Normal Ground Motion for Dumbarton Bridge Seismic Retrofit Project

INCOHERENCY

Ground motions can vary spatially along the bridge alignment due to scattering and complex wave propagation. In all the long-span bridge projects, incoherent ground motions were considered as part of the multiple support time history analyses (Figure 3).

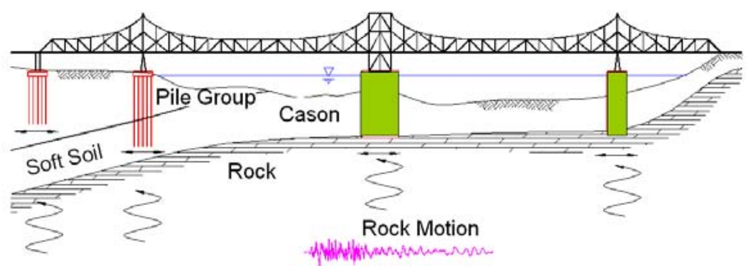


Figure 3 Propagation of Seismic Waves

This required exciting each bridge support with pier-specific motion, and three orthogonal components were used simultaneously. The spatial variability or incoherence is caused by a number of factors, such as:

- Wave Passage Effect: nonvertical waves reach different positions on the ground surface at different times causing a time shift between the motions at those locations
- Extended Source Effect: mixing of wave types and source directions due to differences in the relative geometry of the source and the site

- Ray Path Effect: scattering of seismic wave by heterogeneity of earth along the travel path causing different waves to arrive at different locations at different times
- Attenuation Effect: variable distance from the different locations to the seismic source
- Site Response Effect: variable soil conditions produce different motions at the ground surface

From the toll bridge experience, incoherency arising from the wave passage, extended source, and ray path effects tends to alter high frequency components which do not influence the seismic response of long span bridges, characterized as long period vibration. However the site response effect profoundly influences the ground motion characteristics. Because local site conditions can vary significantly over the length of the bridge, and the effect often overshadows the other sources of incoherency. For practical purposes, it is adequate to just consider site response effect for incoherency; other incoherency effects, such as wave passage, extended source, and ray path effects, can be ignored.

SITE RESPONSE

As seismic waves propagate though a soil deposit, the ground motion characteristics change when they arrive at the bridge supports. The ground motions would impart loading to the structure in a form of depth varying motions along the depth of the foundation. For a long span bridge, the effects of local site conditions virtually contribute all the spatial variation.

For the most part, the site response analyses were conducted using one-dimensional equivalent linear programs. However for the East Span Bay Bridge, for example, two dimensional site response analyses were conducted near the main span foundation area because of steeply sloping soil layering and the bedrock contact is non-horizontal as shown in Figure 4

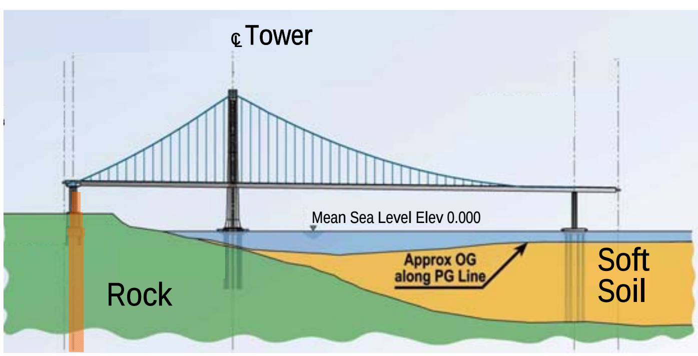


Figure 4 Bay Bridge Main Span Soil Profile

Also, nonlinear site response analyses were conducted to appreciate effects of permanent ground displacement in some instances. The shear wave velocity of soil is the one of the most important parameters for conducting the site response analyses. In the past, the shear wave velocity measurement was made with a crosshole geophysical sounding, requiring two boreholes for each measurement. Today, suspension P-S logging, a relatively new method, is used for measuring the seismic velocity profile in the borehole, eliminating the need to drill two boreholes. For example, 6 boreholes out of 20 boreholes drilled at the Vincent Thomas Bridge were logged with the downhole P-S logging to measure shear wave velocities of the soil (Figure 5).

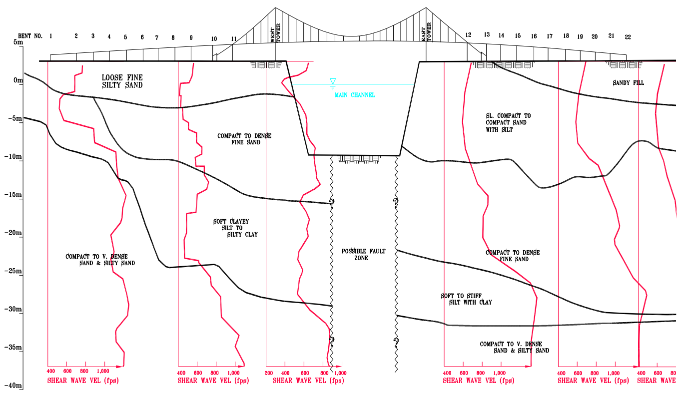


Figure 5 Soil Profile along the Vincent Thomas

The seismic retrofit analyses of the Vincent Thomas Bridge involved many engineers from different firms. Unintended mistakes had been made by others due to miscommunication during the early stage to provide the ground motions for multiple support time history analyses of the global bridge model which has a total of 30 supports. Since only 6 shear wave velocity measurements were made, each of the shear wave velocity profiles was assigned to a group of bridge piers; for example, the velocity profile (V1) was assigned to piers 1 through 5, and the velocity profile (V2) was assigned to pier 6 through 10. At a first glance, it appeared reasonable. When the site response analysis was conducted for each pier, no differential movement was observed among the piers that utilized the same shear wave velocity profile. However, at the boundary of the two groups of piers (for example between Pier 5 and Pier 6), significant differential movement as much as several inches were observed. When this set of multiple support time histories were first applied in the structural analyses, shear failure was reported at every boundary between adjacent groups of piers.

This was quickly recognized to be artificial and was later corrected by careful interpretation of the soil properties along the bridge length prior to conducting the pier-specific site response analyses. The interpretation included gradual transition of shear wave velocities consistent with the soil stratigraphy developed from soil borings, lab testing, and SPT and CPT results.

Another mistake that the writers pointed out to the design team was related to where the rock outcrop motion was assigned in the site response analyses. In any site investigation program, soil boring depths sometimes differed by as much as 100 feet between two adjacent boreholes. When one dimensional soil columns were constructed strictly following the boring specific soil data for the purpose of the site response study, the heights of two soil columns were different resulting in the bottoms of soil column where rock outcrop motions were prescribed at different elevations. Seismic waves in the long soil column had to travel longer than in the short soil column. If the column height is different by 100 feet, seismic waves would have to travel 0.1 second longer in soil media with an average shear wave velocity of 1000 feet per second. Such a delay time could readily lead to differential displacement as much as 8 inches between two piers for an earthquake time history having PGV of 80 inches per second. This is significant enough to tear the bridge apart if such differential movement occurs between two adjacent piers. Therefore it is essential to fix a reference rock motion elevation for any site response analyses to avoid any artificial differential movement.

KINEMATIC SOIL PILE INTERACTION

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. 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 as shown in Figure 6.

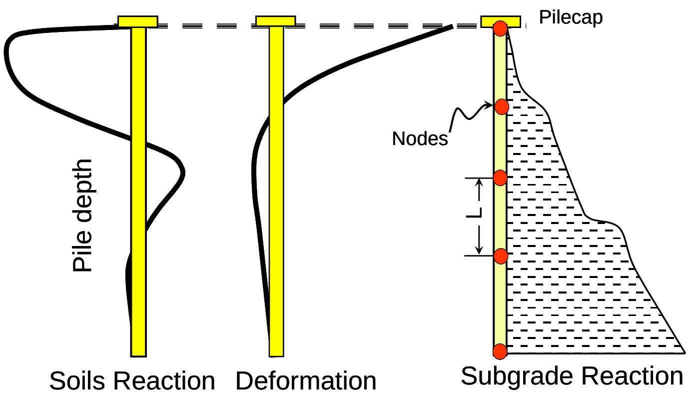


Figure 6: Soil-Pile Interaction

The pile is pushed at the top for the anticipated foundation displacement. The soils reactions along the soil profile are divided by the deformation along the pile length to calculate the linear subgrade reactions. To model the soil pile interaction effect, the stiffness of the soil surrounding each pile is modeled using lumped generalized spring elements attached to pile at discrete nodal points located along the centerline of the pile. The combination of the linear springs at the nodal points and the stiffness of the piles are condensed to a full 6x6 matrix at the pile caps.

The substructuring technique using static condensation is used to develop the foundation stiffness matrices for the each pile group at the bent and at the each abutment. Substructure configuration of the soil-foundation at the bent is shown in Figure 7.

A typical bridge problem can be considered as a multiple-input, multiple-degree-of-freedom (MDOF) system; it consists of superstructure (deck and column), pile-cap, and pile-foundation.

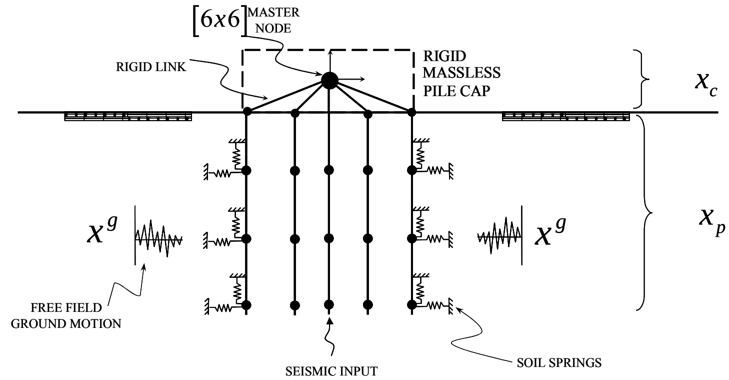


Figure 7 Substructure system

For the purpose of discussion, the damping term is ignored and the mass matrix is based on a lumped mass system. The equations of motion for this MDOF system due to multiple-input ground motion $\{x_g(t)\}$ are written as

$$\begin{bmatrix} [m_s] & & \\ & [m_c] & \\ & & [m_p] \end{bmatrix} \begin{Bmatrix} \{\ddot{x}_s\} \\ \{\ddot{x}_c\} \\ \{\ddot{x}_p\} \end{Bmatrix} + \begin{bmatrix} [K_{ss}] & [K_{sc}] & \\ [K_{cs}] & [K_{cc}] & [K_{cp}] \\ & [K_{pc}] & [K_{pp}] \end{bmatrix} \begin{Bmatrix} \{x_s\} \\ \{x_c\} \\ \{x_p\} \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ [K_g]\{x_g\} \end{Bmatrix} \tag{1}$$

where the subscript "s", "c" and "p" denote the superstructure, pile-cap and the pile degrees of freedom, respectively. The mass and stiffness are represented by the matrices [m] and [K]. The vectors $\{x_s\}$, $\{x_c\}$ and $\{x_p\}$ represent displacement of the superstructure, pile-cap, and pile degrees of freedom relative to the earth inertia reference frame (i.e, total displacements), respectively. The displacement vector $\{x_g\}$ represents the depth-varying free-field motions which can be computed from an appropriate site response study, and $[K_g]$ is the soil stiffness surrounding the piles. The pile-cap node would have 6 degrees of freedom (three translation, and three rotation). It is assumed that mass of the pile is small and can be ignored, i.e, $[m_p]=[0]$. Typically this is the case because inertial forces of the piles are relatively small compared to the more massive superstructure, and therefore neglecting the foundation inertial effect does not normally cause significant error. Then the system of equations becomes:

$$\begin{bmatrix} [m_s] & & \\ & [m_c] & \\ & & [0] \end{bmatrix} \begin{Bmatrix} \{\ddot{x}_s\} \\ \{\ddot{x}_c\} \\ \{\ddot{x}_p\} \end{Bmatrix} + \begin{bmatrix} [K_{ss}] & [K_{sc}] & \\ [K_{cs}] & [K_{cc}] & [K_{cp}] \\ & [K_{pc}] & [K_{pp}] \end{bmatrix} \begin{Bmatrix} \{x_s\} \\ \{x_c\} \\ \{x_p\} \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ [K_g]\{x_g\} \end{Bmatrix} \tag{2}$$

With the pile mass assumed to be zero, static condensation can be performed to eliminate the pile degrees of freedom. After eliminating the pile degrees of freedom, dynamic response analysis can be performed using the following set of equations involving the superstructure $\{x_s\}$ and the pile cap nodes $\{x_c\}$;

$$\begin{bmatrix} [m_s] & \\ & [m_c] \end{bmatrix} \begin{Bmatrix} \{\ddot{x}_s\} \\ \{\ddot{x}_c\} \end{Bmatrix} + \begin{bmatrix} [K_{ss}] & [K_{sc}] \\ [K_{cs}] & [K] \end{bmatrix} \begin{Bmatrix} \{x_s\} \\ \{x_c\} \end{Bmatrix} = \begin{Bmatrix} 0 \\ \{f\} \end{Bmatrix} \tag{3}$$

where [K] and {f} in the above equation are defined as

$$\{f\} = -[K_{cp}][K_{pp}]^{-1}[K_g]\{x_g\} \tag{4}$$

$$[K] = [K_{cc}] - [K_{cp}][K_{pp}]^{-1}[K_{pc}] \tag{5}$$

It is noted that Equation 3 contains only the pile-cap and the superstructure degrees of freedom, and in fact the solutions to this set of equations are what the structural engineers are seeking in order to determine demands for the superstructure and the pile-cap. The transformation of the original problem into a substructured system consisting only of the superstructure and pile-cap is entirely based on classical static condensation. The stiffness of the pile foundation system is now represented by the 6x6 condensed stiffness matrix [K], given by Equation 4 and the equivalent force applied at the pile-cap represented by the 6x1 forcing function {f}, given by Equation 5. Both the stiffness matrix [K] (which is constant with time) and the forcing function {f} (which is time dependent and related to the time histories of the depth-varying free-field input ground motion) can be pre-calculated without knowledge of the response of the superstructure or the pile cap. Instead of using the forcing function {f} explicitly on the right hand side of Equation 3, a 6x1 vector {X} is introduced, such that

$$\{f\} = [K]\{X\} \tag{6}$$

Then Equation 3 becomes

$$\begin{bmatrix} [m_s] & \\ & [m_c] \end{bmatrix} \begin{Bmatrix} \{\ddot{x}_s\} \\ \{\ddot{x}_c\} \end{Bmatrix} + \begin{bmatrix} [K_{ss}] & [K_{sc}] \\ [K_{cs}] & [K] \end{bmatrix} \begin{Bmatrix} \{x_s\} \\ \{x_c\} \end{Bmatrix} = \begin{Bmatrix} 0 \\ [K]\{X\} \end{Bmatrix} \tag{7}$$

It can be noted that loading from ground excitation is introduced into the overall structure as a right hand vector in a form of a displacement vector {X} times the foundation stiffness [K], and therefore {X} represent some form of rigid-base motion derived from the depth-varying free-field ground motion. This six-component motion {X} is termed "kinematic motion". This kinematic motion {X}, calculated at a single point on the pile cap implicitly contains the statically condensed forces transmitted from the ground to the superstructure along the entire embedded pile length due to both the depth-varying shaking intensity in soil motion as well as the depth-varying soil stiffnesses. The kinematic motion is derived using a massless pile-group model, and therefore maintains the frequency contents of the original ground motion.

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 stiffness, and the pile properties are included in the solution.

The shape of response spectra as obtained from the kinematic soil-pile interaction analyses sometimes contains multiple peaks and valleys. For practical use and simplicity for the design process, the final ARS recommendations are constrained by a well-behaved ARS curve shape for both spectral acceleration and displacement (i.e., the final ARS curves should be smooth-out).

MODELING OF GRAVITY CAISSONS

In early 1990 California Department of Transportation was commissioned to conduct a seismic vulnerability assessment of the existing Carquinez bridge. An analytical technique based on an elastic-dynamic approach was used to model the caissons which requires everything to be linearly elastic, i.e., the caisson is perfectly 'glued' to the ground without allowance for separation and the surrounding soils have unlimited shear strength. This resulted in enormous shear forces within the caissons when used in the global bridge model for seismic loading such that the shear forces would have crushed the caissons. The conclusions from this vulnerability study led to the development of a retrofit strategy for the caisson foundations of the bridge. The estimated cost of retrofitting the caissons exceeded the budget allocated for the bridge retrofit program.

California Department of Transportation then engaged the writers for a second opinion of the recommended retrofit strategy. After evaluating the previous study it was determined that the magnitude of the shear forces was not sustainable as the caisson would have toppled under this kind of shear load. To develop a more correct deformation mechanism nonlinear modeling was adopted to evaluate the performance of the gravity caissons. The nonlinear approach allowed for geometric nonlinearity due to uplifting at the base of the caisson and material nonlinearity due to yielding of the soil. The analyses indicated that the maximum shear force that can develop in the caisson was limited to the overturning moment associated with the deadweight and the half width of the caisson, divided by the height of the center of gravity. This more correct modeling of the caissons also contributed to realistic and successful modeling of the global bridge model for the seismic analysis. The conclusion that the existing caissons of the bridge were not vulnerable to earthquake damage resulted in significant savings in the retrofit cost for the bridge. The retrofit was then successfully completed marking the first completed seismic retrofit of all the California long-span bridges.

The caisson models of the Carquinez bridge that were used in the final PS&E stage were represented using lumped nonlinear moment-versus-rotation and base shear load-versus-lateral displacement relationships, as illustrated in Figure 8.

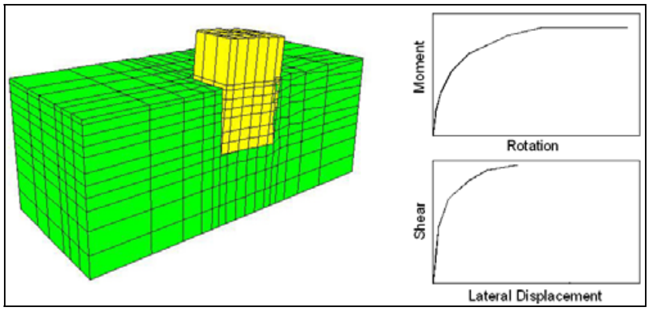


Figure 8. Pushover Analysis of Gravity Caisson

This nonlinear lumped foundation behavior was established by performing pushover analyses to capture essential elements of soil-structure interaction phenomena and to consider the limiting force and moment. Because of the nonlinear nature of foundation behavior, uncoupled springs must be used; i.e., the load-versus-displacement relationship and moment-versus-rotation relationship were operated independently. To evaluate whether the uncoupled springs performed appropriately, the shear load and overturning moment of the caisson computed from the global bridge model were checked to ensure that the assumptions made during the pushover analyses were valid.

A refined caisson model can be made to reconcile the importance of coupling between shear and moment loads. This model entails Winkler springs distributed over the bottom surface of the caisson to represent the soil continuum underlying the foundation, and another sets of soil springs attached to the vertical sides of caisson walls to model passive soil pressure acting on the concrete. The soil springs may be nonlinear for consideration of yielding of localized soil. In addition, gapping elements can also be implemented in series with the soil springs to engage a full contact between the soil and the caisson during compression and to allow separation under tension. Figure 9 illustrates a distributed soil spring model used in the seismic analyses for the Second Tacoma Narrows Bridge Project.

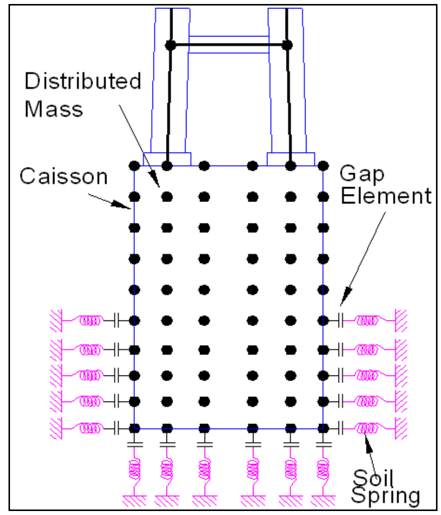


Figure 9. Distributed Spring Model

This modeling approach would address the two significant features; nonlinear behavior and coupling between lateral loading and overturning moment, and hence exhibits significant improvements over the lumped spring models. Establishment of the proper distributed soil springs is a key to successful modeling of the caisson.

MODELING OF PILE GROUPS

In many long span bridge projects, substructuring was used to reduce the size of the problem. The substructuring technique involves modeling the pile foundation to a convenient interface with the superstructure, e.g., at the base of the pile cap (see Figure 7 and Figure 11). Static condensation was then used to derive the appropriate foundation substructure stiffness and the effective ground motion transmitted to the superstructure; the resultant effective ground displacement is termed the kinemtaic ground motion (Lam and Law, 2000). The foundation stiffness matrix was used in conjunction with the kinematic ground displacement to represent the entire foundation system in the superstructure analysis.

For a vertical pile group shown in Figure 10, the form of pile group stiffness matrix is identical to that of a single pile, and has the following terms for a fixed head condition:

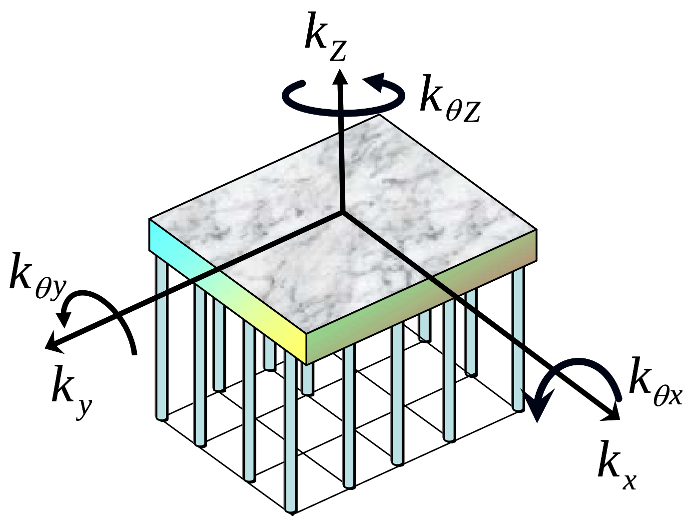


Figure 10 Typical Vertical Pile Group Foundation

$$[K] = \begin{bmatrix} k_x & 0 & 0 & 0 & -k_{x\theta y} & 0 \\ 0 & k_y & 0 & k_{y\theta x} & 0 & 0 \\ 0 & 0 & k_z & 0 & 0 & 0 \\ 0 & k_{y\theta x} & 0 & k_{\theta x} & 0 & 0 \\ -k_{x\theta y} & 0 & 0 & 0 & k_{\theta y} & 0 \\ 0 & 0 & 0 & 0 & 0 & k_{\theta z} \end{bmatrix}$$

in which k_x, k_y, k_z and $k_{\theta x}$, $k_{\theta y}$, $k_{\theta z}$ are stiffness coefficients corresponding to translational and rotational degrees of freedom associated with the X ,Y and Z axes, respectively. The X- and Y-coordinates are taken as horizontal axes, and the Z-coordinate is the vertical axis. For the case of vertical pile group with a pinned head condition, the off-diagonal terms become zero, and the stiffness matrix has the following form:

$$[K] = \begin{bmatrix} k_x & 0 & 0 & 0 & 0 & 0 \\ 0 & k_y & 0 & 0 & 0 & 0 \\ 0 & 0 & k_z & 0 & 0 & 0 \\ 0 & 0 & 0 & k_{\theta x} & 0 & 0 \\ 0 & 0 & 0 & 0 & k_{\theta y} & 0 \\ 0 & 0 & 0 & 0 & 0 & k_{\theta z} \end{bmatrix}$$

For the pile group foundations with the battered piles such as Richmond – San Rafael bridge, the pile groups represent an example of full stiffness matrix due to a large number of battered HP piles arranged in circular patterns. There are numerous cross-coupling terms among different degrees of freedom, and the stiffness matrix is represented as a full matrix as shown below:

$$[K] = \begin{bmatrix} X & X & X & X & X & X \\ X & X & X & X & X & X \\ X & X & X & X & X & X \\ X & X & X & X & X & X \\ X & X & X & X & X & X \\ X & X & X & X & X & X \end{bmatrix}$$

where ‘*X*’ signify non-zero coefficient in each of the entry in the stiffness matrix. The pile group model at Pier 48 is shown in Figure 11 consisting of 308 steel HP piles. The directions of strong axis for these HP piles are oriented tangential to the circular pile group arrangement. Since the bridge has over 60 piers, it was not feasible to employ the complete system to include every individual pile and soil support in the global bridge model.

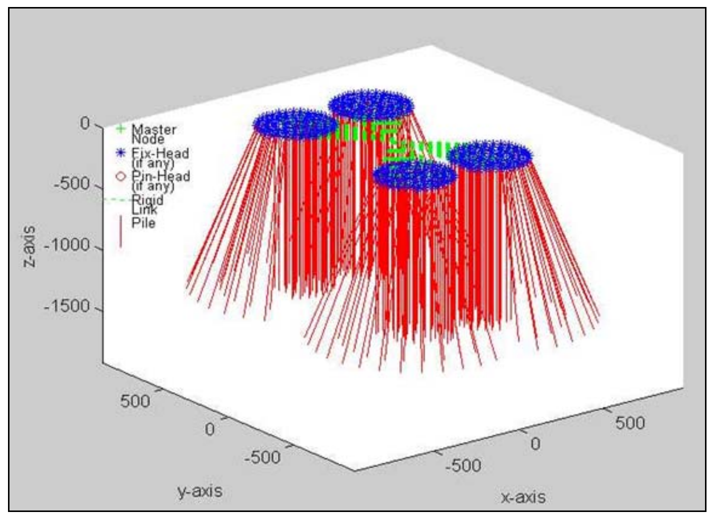


Figure 11 Richmond – San Rafael Bridge Pile

For this situation, the substructuring approach is highly suitable to formulate the problem into a manageable size.

Figure 12 shows the pile group layout at south anchorage of the new Carquinez Suspension Bridge. The anchorage block, which provides an anchoring point for the suspension cables, has a three-step bottom founded on 380 piles, each a 0.75 m diameter Cast-in-Steel-Shell (CISS) pile.

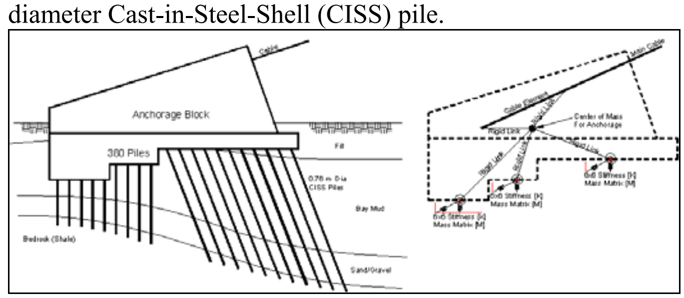


Figure 12 Anchorage Foundation for New Carquinez Bridge

These piles are driven to bedrock through soft bay mud. The bedrock is located at variable elevations; the difference in bedrock elevations is as much as 10 meters within the footprint of the foundation. Due to the large size of the anchorage footprint and the variability of subsurface conditions, the piles were divided into six groups. Each group was represented with a condensed mass matrix and a condensed stiffness matrix. In the global bridge analysis, these six sets of stiffness and mass matrices were rigidly linked together to form the foundation model. After displacement demands on the anchorage were obtained from the global analyses (six degrees of freedom displacement), they were then back-substituted into the foundation substructure to recover individual pile loads by a means of performing pushover analyses. This approach substantially reduced the total number of degree of freedom needed to model the suspension bridge structure.

As part of the seismic retrofit program for the San Diego – Coronado Bay Bridge, substructuring of individual piles has also been used for the time history analyses of the global bridge. The foundations for this bridge consist of prestressed concrete piles of a 1.4-meter diameter and a 122-mm wall thickness driven into dense sands. Each foundation is supported on 12 to 44 piles in a group, and the piles have substantial cantilever lengths above the mudline. Potential plastic hinging of the piles at the pile-cap connection point was of concern, and it needed to be addressed in the global time history analysis. Therefore, the pile segment above the mudline was included in the global bridge model; the portion of the piles below the mudline was represented by a 6x6 stiffness matrix and mass matrix, as shown in Figure 13. This modeling technique was found to be very effective in addressing plastic hinging in the piles, and is a compromise between the full substructure model and the complete model approaches.

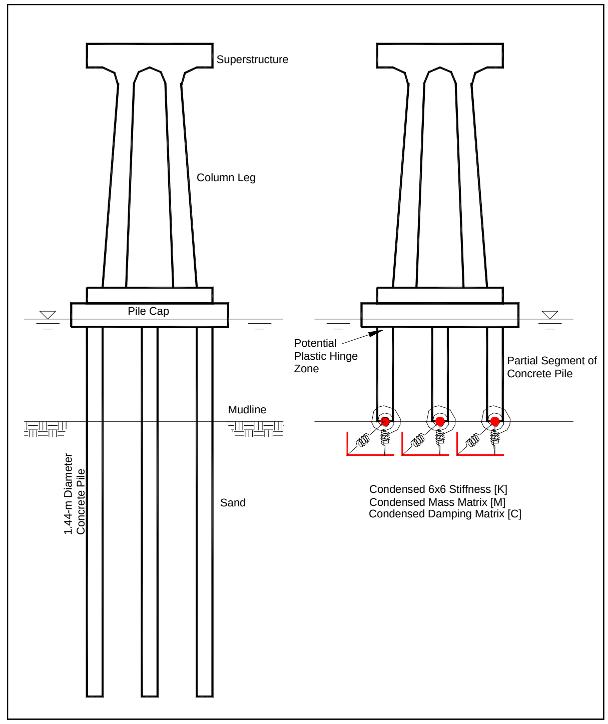


Figure 13 SFIS Model for Coronado Bridge, San Diego

At the onset of seismic retrofit strategy phase of the Dumbarton and Antioch Bridge project, the structural designers evaluated the most appropriate foundation model to be used for the primary design of the bridge. A special study was conducted on a stand-alone pier to check validity of the substructuring models.

Figure 14 presents an example of the idealized subsurface soil profiles and the pile foundations the for the Antioch and Dumbarton bridge structures. Figure 15 shows two separate three-dimensional computer models of Antioch Pier 17. One model includes the complete piles and soil supports along the length of the pile which are excited by depth-varying free-field motions. The second model consists of substructured foundation model, represented by a 6x6 stiffness matrix and a kinematic motion exciting the foundation system. The solutions from the two models are compared in Figure 16 showing displacements of the deck in the transverse and longitudinal directions. The solutions are shown to be very comparable; the minor variations are believed to be attributed to non-linear springs used in some of the foundation models. The studies verified that all foundation models yield satisfactory solutions.

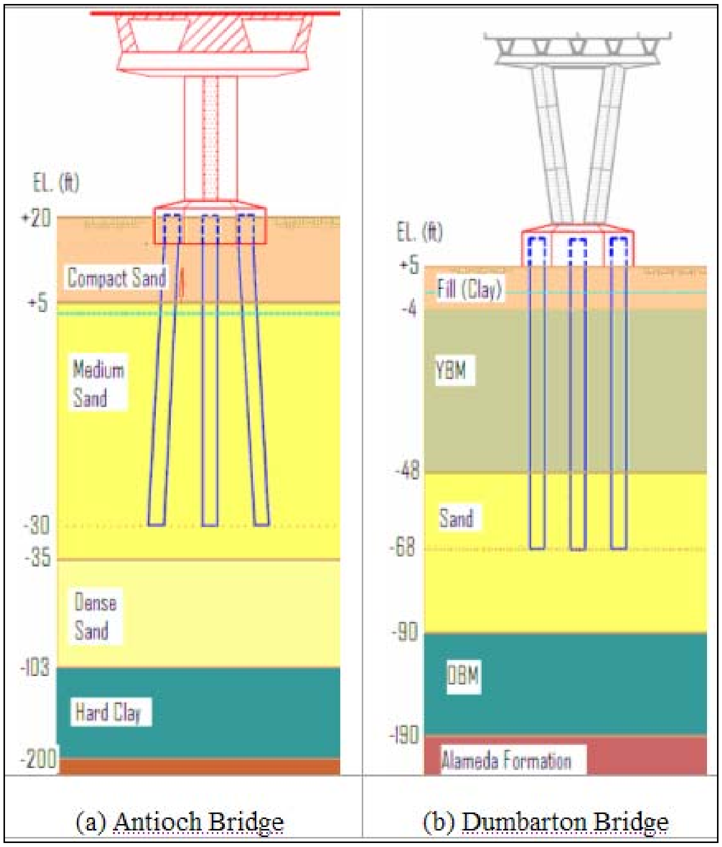


Figure 14 Excerpts from the Idealized Soil Profiles

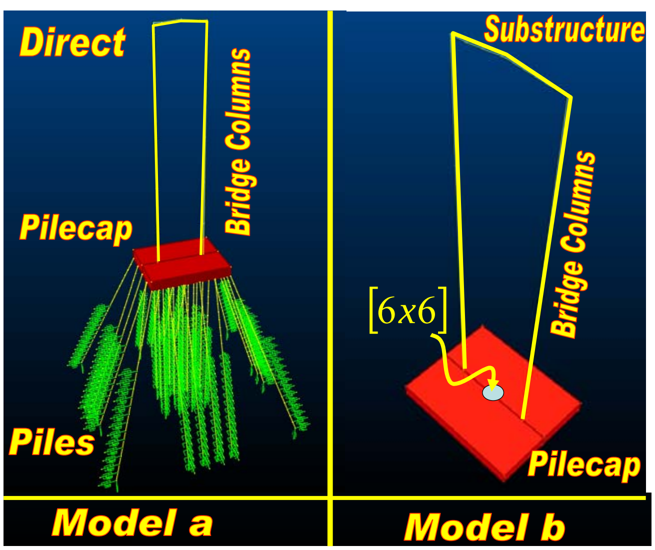


Figure 15 Foundation models for Antioch Pier 17

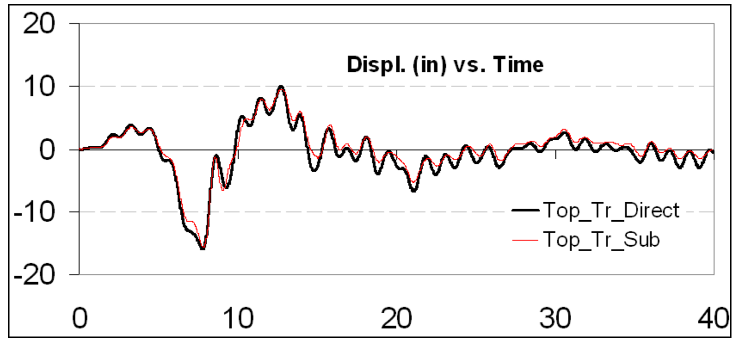


Figure 16a Superstructure response in the transverse direction

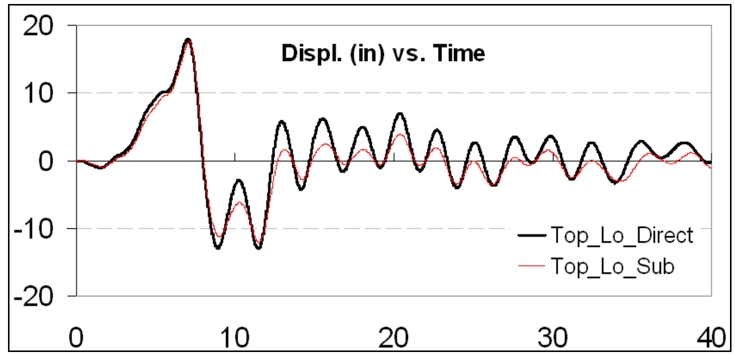


Figure 16b Superstructure response in the longitudinal direction

CONCLUSIONS

Following the 1989 Loma Prieta and 1994 Northridge Earthquake, California Department of Transportation, USA, initiated a seismic retrofit program to strengthen existing long-span bridges and many regular bridges in California. Prior to this seismic retrofit program, there were very few guidelines and criteria available to undertake seismic retrofit of existing bridges and design of new structures to withstand potentially large magnitude future earthquakes. Since then, the retrofit design procedures have been modified regularly to improve more robust designs specific to California bridge structures and California geological and seismological conditions. Various refinements were direct results of experience learnt from previous analyses and design processes.

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