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SEISMIC DESIGN STRATEGY OF THE NEW EAST BAY BRIDGE SUSPENSION SPAN

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SUMMARY

The San Francisco Oakland Bay Bridge was closed in 1989 when a 15-m portion of the East Span collapsed due to the 7.1M Loma Preita Earthquake. Seismic evaluation of the bridge performed by the California Department of Transportation concluded that replacing the East Bay Bridge is more cost effective than a seismic upgrade of the 50 year old bridge. T.Y.Lin International and Moffatt & Nichol, a joint venture, were hired in 1998 to design a new bridge consisting of a skyway portion and a signature main span. This paper focuses on the seismic design strategy that was employed in the type selection process of the signature span and throughout the design phase.

The single tower asymmetric self-anchored suspension bridge was selected from a total of four design alternatives that were developed for the signature main span; these included two cable stayed bridges and two self-anchored suspension bridges (each bridge type included single tower and dual portal tower alternatives). Each design alternative was evaluated based on its seismic response, construction cost and aesthetic properties. Because this is a seismic safety replacement project, the seismic behavior of each design alternative was the most important factor in this design selection process.

The Bay Bridge lies between the Hayward and the San Andreas faults which are capable of producing magnitude 7.5M and 8M earthquakes, respectively. Design criteria for the Bay Bridge require that the bridge must be operational almost immediately after a major earthquake. The seismic behavior was evaluated based on dynamic analysis, push-over analysis and various parametric studies targeted to evaluate the structural lateral system.

In order to provide a seismically reliable structure, a limited-ductility design approach was followed. This required that the bridge should have a clearly defined plastic mechanism for response to lateral loads. This was achieved by the following: (1) limiting plastic hinging to the east and west piers, (2) providing shear links between the tower shafts which would yield in the event of a major earthquake (these links will be replaced afterwards), (3) providing a tie-down/counter weight at the west pier to ensure stability after the west pier forms plastic hinge and (4) improving the seismic response of this bridge by using seismic devices between the tower and the deck and at the expansion joints.

INTRODUCTION

The San Francisco-Oakland Bay Bridge was constructed in 1936. At that time, it was one of the longest high-level bridges in the world. Today, it carries 280,000 vehicles a day and is the busiest bridge in the world. Figure 1 shows the bridge's geographical location. The 1989 Loma Prieta Earthquake seriously damaged the East Span of the bridge when a 15-m portion above Pier E-9 collapsed onto the lower deck and the bridge was closed for repairs for a period of four weeks. Post-earthquake inspections of the bridge revealed that another portion above Pier E-23 nearly experienced a similar collapse. Seismic evaluation of the bridge performed by the California Transportation Department concluded that replacing the East Span of the Bay Bridge is more cost effective than

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a seismic upgrade of the 50 year old bridge. T.Y.Lin International and Moffatt & Nichol, a joint venture, were hired in 1998 to design a new bridge which consisted of a skyway portion and a signature main span. The Self-Anchored Suspension bridge and its twin 2.4-km concrete viaducts will replace the existing and seismically unsafe eastern crossing. This design was selected by the Metropolitan Transportation Commission out of four other competing designs (Figure 2). The span lengths planned for this bridge make it the longest self-anchored suspension bridge in the world.

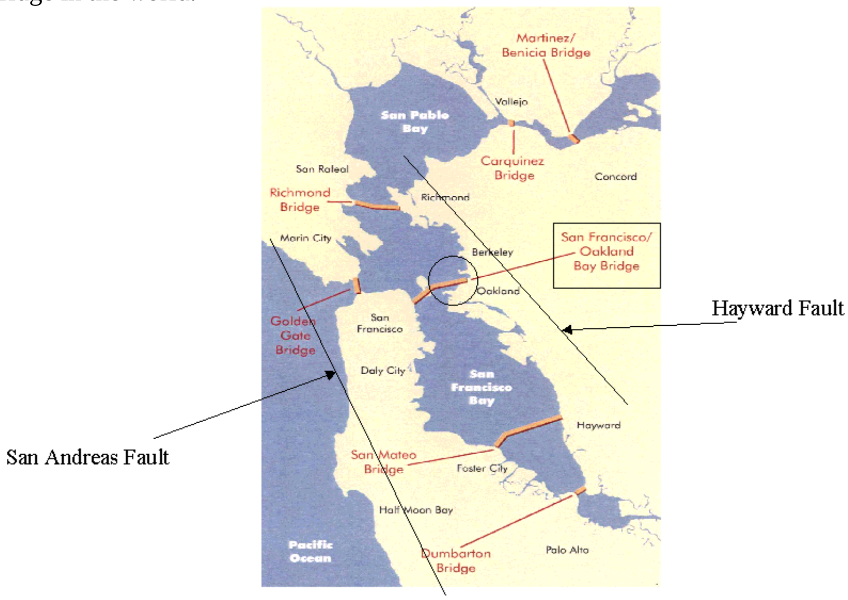

Figure 1. Geographical Location of Bridge

Suspension Bridge Alternatives

Cable Stayed Bridge Alternatives

Figure 2. Design Alternatives for the Main Span

DESCRIPTION OF THE BRIDGE

The self-anchored suspension bridge consists of a 385m main span and a 180m back span (Figure 3). As illustrated in Figure 3, the 0.78m diameter cable is anchored to the deck at the east bent and is looped around the west bent through deviation saddles. Unlike traditional suspension bridges, these deviation saddles are fixed to the west bent and the cable force on either side is balanced during construction using a jacking saddle (Figure 4). These saddles are supported by a pre-stressed cap beam which is designed to carry the differential stresses arising during service and seismic loads. The weight of this cap beam is designed to balance the dead load uplift at the west bent arising from the asymmetry of the bridge. The cables at the tower do not cross and are secured in a single saddle. The saddle at the east pier is supported by the box girders and is designed to move in order to balance the cable forces on either side. The suspenders are splayed to the exterior sides of the box girders and are spaced at 10m. The superstructure consists of dual hollow ASTM Grade 50 orthotropic steel boxes (Figure 5). These boxes are in compression (supporting the cable tension forces) and are a part of the gravity load system. Diaphragms spaced at 5m support the orthotropic deck and distribute the suspender loads to the box. The box girders are connected together by 10m wide x 5.5m deep crossbeams spaced at 30m. These cross beams carry the transverse loads between the suspenders (span of 72m) and ensure that the dual boxes act composite during wind and seismic loads.

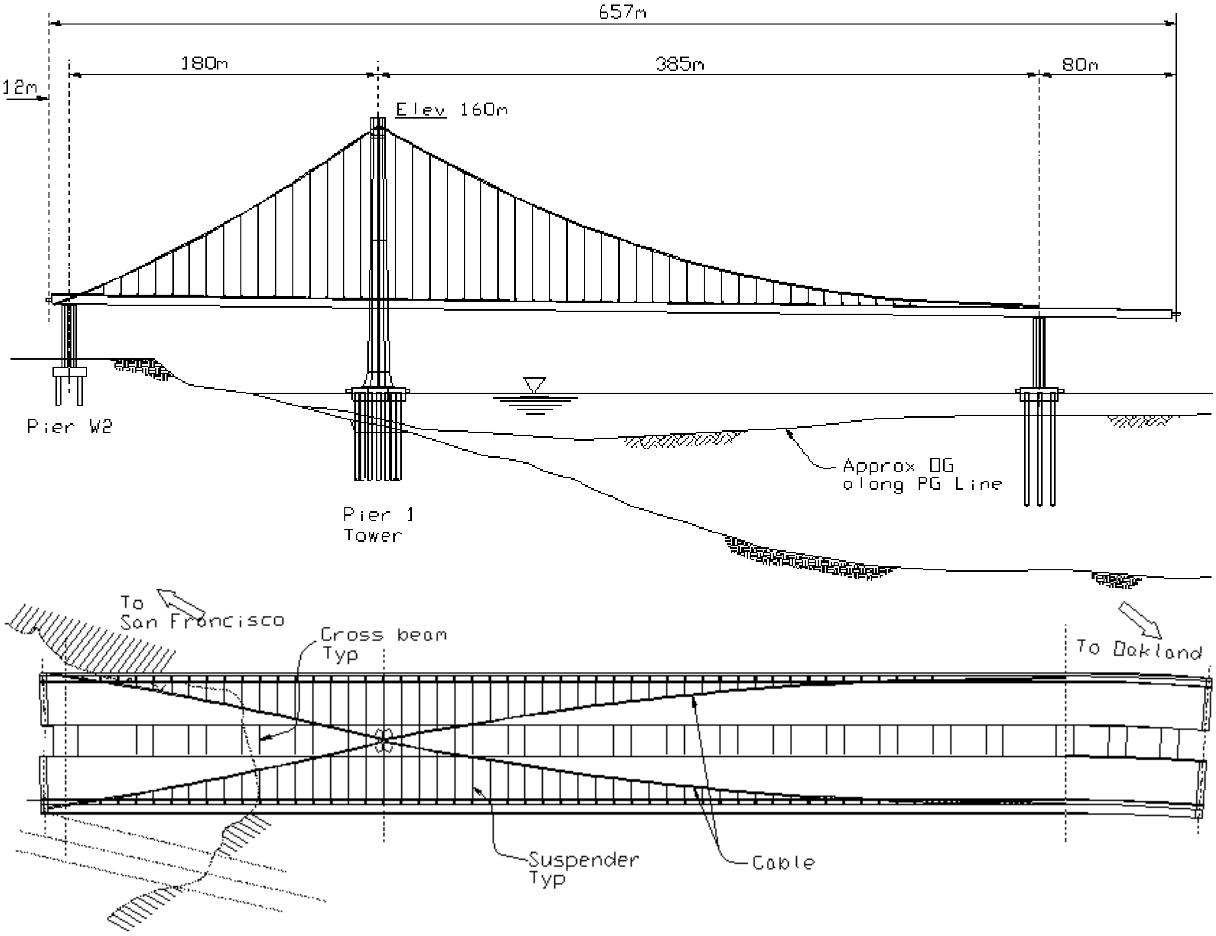

Figure 3. Elevation and Plan of the Self Anchored Suspension Bridge

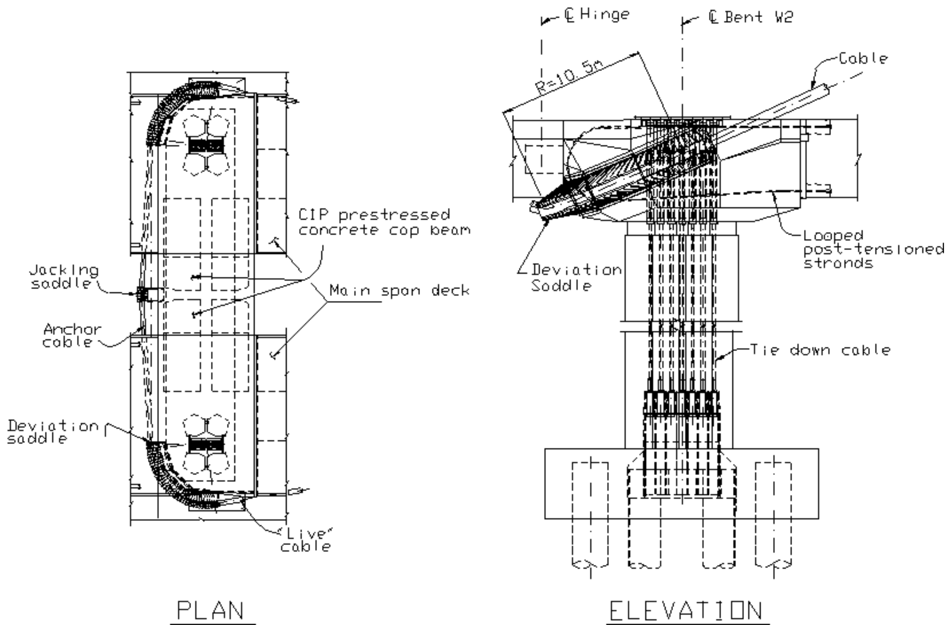

Figure 4. West Anchorage – Looped Cable throught two Deviation Saddles

As shown in Figure 5, the bridge carries a pedestrian path on the south side. This eccentric load is balanced by a counter-weight on the north side. At the west bent, the box girders frame into the cap beam (which serves as the box girder at that location). The connection between the orthotropic steel box girders and concrete cap beam is subjected to the compressive forces of the cables. Additional pre-stress is added through longitudinal post-tension strands at each rib (Figure 4).

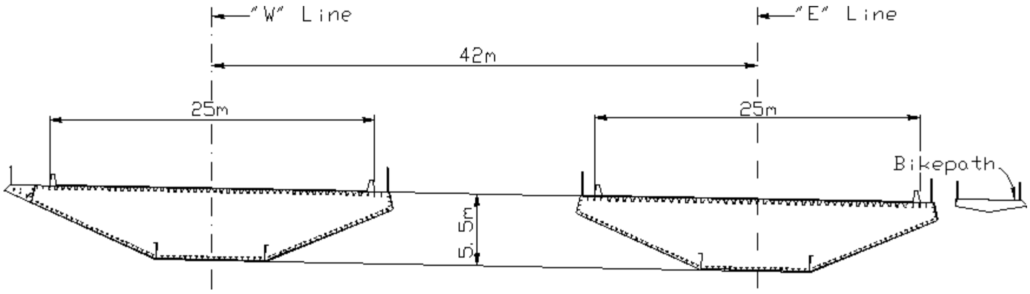

Figure 5. Typical Cross Section of Bridge – Dual Box Girder Connected with Cross-Beams

The single tower is 160m tall and is composed of four shafts connected with shear links along its height (Figure 6). The tower shafts are tapered stiffened steel (ASTM A709 Gr. 50) box members with diaphragms spaced at 3m. The tower is fixed to the 6.5m deep pile cap (consisting of steel moment frame encased with concrete) and is supported on 13 - 2.5m steel shell pipe piles (filled with concrete) which in turn are embedded and fixed into rock. The rock slope is benched to give the piles equal lateral stiffness and avoid torsional response (pile clear length is about 20m). The east piers are reinforced concrete columns (reinforced with additional pre-stress to avoid tension shear failure when subjected to seismic loading) supported on 16 – 2.5m steel pipe piles. These piles are 100 m long and are filled with concrete for the top 55m. The west piers are reinforced concrete columns which are monolithic with the pre-stressed cap beam (forming the west bent) and are supported on rock through 8 - 2.5m CIDH piles. At the west pier a tie-down system, designed to resist the seismic uplift, consists of 28 stay cables (61-15mm diameter strands each) which are anchored into rock through an additional 4-2.5 CIDH piles. At the east piers the box girders are supported on bearings. Shear keys and tie rods are provided to carry lateral loads and uplifts, respectively. The box girders are supported at the east and west pier for lateral loads and are "floating" at the tower. The transition spans between the skyway, suspension bridge and Yerba Buena Island structure each have a hinge. These hinges are designed to allow the structures to move relative to each other in the longitudinal direction only (Figure 7).

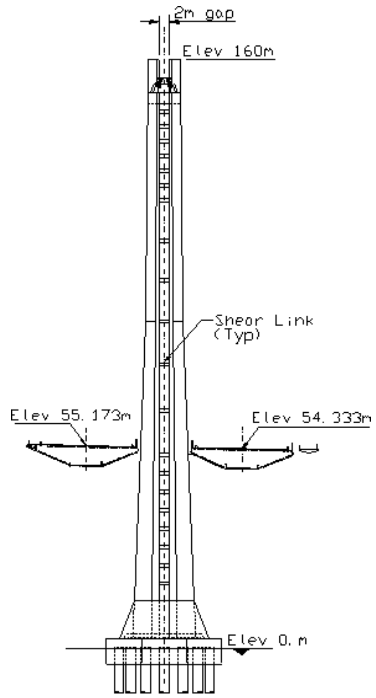

Figure 6. Single Tower

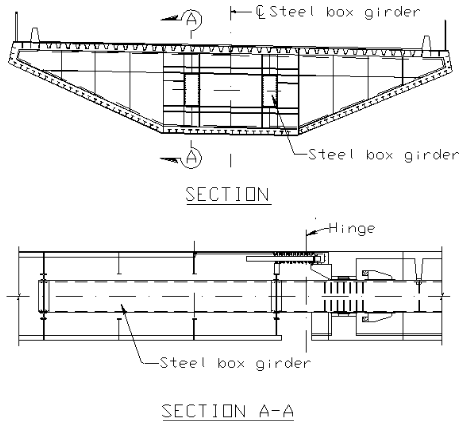

Figure 7. Expansion Joint

SEISMIC DESIGN PHILOSOPHY

SEISMIC PERFORMANCE CRITERIA – The bridge is designed to provide a high level of seismic performance. It is designed to resist two levels of earthquake, a functional evaluation earthquake (FEE) and a safety evaluation earthquake (SEE). After a functional evaluation earthquake, the bridge will provide full service almost immediately and there will be minimal damage to the structure. Minimal damage implies essentially elastic performance and is characterized by minor inelastic response, narrow cracking in concrete, no apparent permanent deformations, and damage to expansion joints. After a safety evaluation earthquake, the bridge will provide full service almost immediately and will sustain repairable damage to the structure. Repairable damage is damage that can be repaired with minimum risk of losing functionality; it is characterized by yielding of reinforcement, spalling of concrete cover and limited yielding of structural steel.

The bridge shall be designed as a limited-ductility structure. This implies that (1) the bridge shall have a clearly defined plastic mechanism for response to lateral loads, (2) inelastic behavior should be limited to tower shear links, piers and piles with a clear failure sequence, and (3) the detailing and proportioning requirements for full-ductility structures shall be met.

ANALYSIS METHODOLOGY - Seismic analysis was performed using the ADINA general-purpose finite element program. Three forms of analysis were employed: time history analysis (global model), push-over analysis and local detailed analysis. Time history analysis was used as the primary means of analysis for several reasons. Foremost among these is that the bridge foundations are subjected to different excitations. The tower and west pier of the bridge are founded on rock while the east pier is supported in deep soil. The ground motions at these supports are completely different in character and intensity. This was reflected in the analysis by applying different time histories of ground displacement at the supports. A large displacement analysis and the use of nonlinear material where necessary allow the designers to capture the true behavior of the bridge (geometric stiffness of bridge, P-delta effects, slacking of suspenders, plastic hinging of piers, tower shear links, etc.). The model was "built" in a single step, in the dead load state. Initial strains in the deck, cables and suspenders were applied in this single step rather than simulating the construction sequence of the bridge. Time history analyses were done as restart analyses from the dead load state. Push-Over analysis was primarily used to evaluate ductility of critical elements and to establish failure mode sequence. Local detailed analysis was used to establish local strain/stress demands and to evaluate the modeling used for the global model.

SITE CONDITIONS AND SEISMICITY - The Bay Bridge lies between the Hayward and the San Andreas faults which are capable of producing magnitude 7.5M and 8M earthquakes, respectively. The proposed bridge alignment is underlain by variable subsurface conditions. While the west pier sits on Yerba Buena Island and the tower is located on relatively shallow sloping bedrock, the east piers and the remainder of the skyway will be founded in deep soils. Spectrum compatible ground motions were generated for this site (three for Hayward and three for San Andreas). Figure 8 illustrates the fault normal spectral accelerations for San Andreas ground motion number 3. As noted, the character of these motions is very distinct.

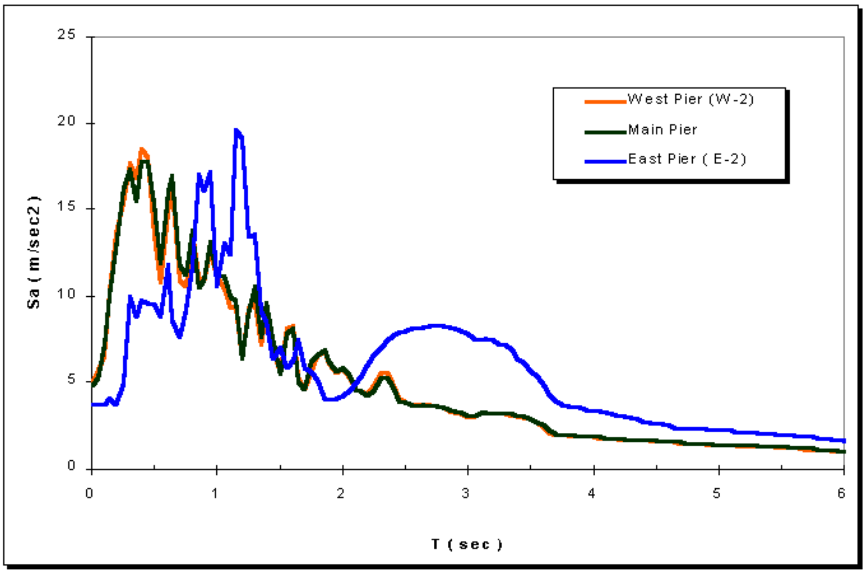

Figure 8. Fault Normal Response Spectrum of the San Andreas Ground Motion No. 3

ANALYTICAL MODEL

DESCRIPTION OF THE ANALYTICAL MODEL - The ADINA global model of the Self-Anchored Suspension Bridge is shown in Figure 9. In addition to the main span structure, the model includes boundary frames representing the skyway and the transition structure on Yerba Buena Island. The model is largely inelastic and consists of nonlinear truss and beam elements. The bridge deck is modeled with two parallel spines of beam elements representing the axial, bending, and torsional properties of the suspended structure. Stiff beam elements extending to the edge of the bridge deck provided nodes for connection of the suspenders (these elements captured the deck stiffness for vertical deformations and were rigid for transverse deformations). The suspenders were modeled with several truss elements in series. This was done to allow the suspenders to go slack during the time history analysis (large displacement formulation).

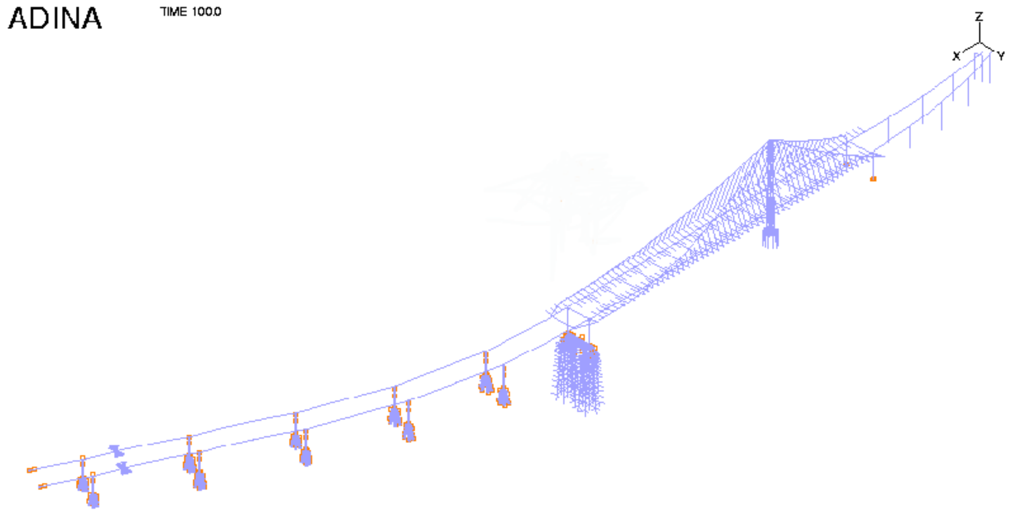

Figure 9. Adina Model of the Suspension Bridge

The dead load stress in the main cables and suspenders was modeled by means of an initial strain applied when the model was built. The tower shafts were modeled with nonlinear beam elements. The shear links between the shafts were also modeled with inelastic moment-curvature beam elements. The yield moment of these elements was set to obtain the desired plastic shear capacity of the links. The rotation of the beam plastic hinges serves as a measure of the shear deformation of the links. Each pile in the tower foundations was modeled directly with several nonlinear beam elements from the bottom of the pile cap to the rock surface. The ground motion was applied directly to the bottoms of the piles assuming the piles to be fixed at the rock surface. The mass of the foundation was lumped at a node at the center of gravity of the pile cap. The east and west piers were modeled with nonlinear beam elements. The west pier was assumed to be founded on rock and the ground motions were applied directly to the bottom of the pier.

Two approaches were used to model the east pier piles: a hybrid model and a detailed model (Figure 10). The hybrid model consisted of beam elements to model each pile from the bottom of the pile cap to the mud-line. Below the mud-line each pile was modeled with a 12 degree of freedom stiffness and damping matrices (spanning between two six degree of freedom nodes). These impedance matrices were used in a local coordinate system at each pile, oriented along the pile axis, so that battering of the piles was rigorously modeled. The coefficients of the impedance matrices were varied with design iterations, according to the mud-line displacement of the piles. The ground motion was applied at the “bottom” nodes of the pile springs. A zero axial stiffness term was included in the stiffness matrix along with a nonlinear spring placed in “parallel’ with impedance matrices in order to account for the axial non-linearity of the pile support. The mass of the foundation was lumped at a node at the center of gravity of the pile cap. The detailed model consisted of nonlinear beam elements which modeled each pile from the pile cap to the pile tip. Each pile was supported with nonlinear p-y and t-z springs along its height.

The model also included t-z dampers to account for viscous damping. The east pier motion for the hybrid model is not the mud-line motion, but the motion at a firm soil layer below the Young Bay mud. Because the structure is supported on relatively large diameter piles, this was assumed to be the “effective” input into the structure, rather than the mud-line motion. This assumption was confirmed by kinematic motion studies of the east pier foundation and soil profile. The detailed model was driven with depth varying ground motions which were applied to the p-y and t-z springs. The model includes the first frame of the skyway structure as a boundary frame. This model accounted for the nonlinear behavior of the skyway piers and made use of the hybrid model to model the foundations. The connection of the main span structure and the skyway is in the transverse and vertical directions only. The two structures were free to move independently in the longitudinal direction. The skyway structure was driven with its site specific ground motion. A simple beam element model of the transition structure was included in the model also. This frame was connected to the main span transversely and vertically; longitudinally it was independent of the main span. The Yerba Buana transition structure was driven with its representative ground motion.

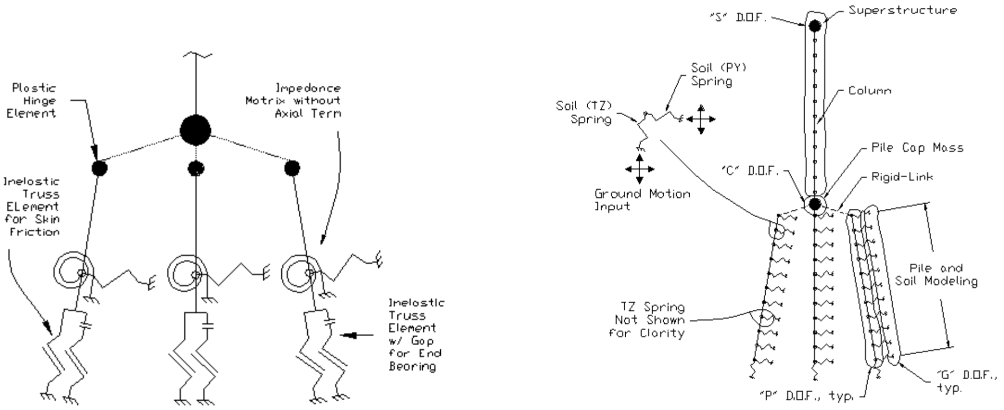

Figure 10. Hybrid and Detailed Model of the East Pier Foundations

SEIMIC RESPONSE

SEISMIC RESPONSE – As discussed earlier the bridge is designed based on a limited ductility design in which plastic deformations are clearly defined and predetermined. Figure 11 summarizes the seismic response of the bridge. As noted the bridge is designed to remain largely elastic with the exception of the east and west piers which are designed to form plastic hinges. The plastic strain in these piers is limited to 2/3 the ultimate strains based on Manders equation for confined concrete columns. The shear links between the tower shafts are also designed to yield in shear during the SEE earthquake. The maximum rotation demand on these links is 0.03 radians compared with an ultimate rotation of 0.09 radians. The piles were designed to sustain minimal damage (strains less than 0.01 for concrete and 0.02 for steel) when subjected to the SEE displacement demands. The tie down at the west pier was designed with a factor a safety of two. Figure 12 summarizes the displacement demands on the bridge. The bridge is long period structure and is mainly in the region of constant displacement demand. This improves the reliability of the structural response.

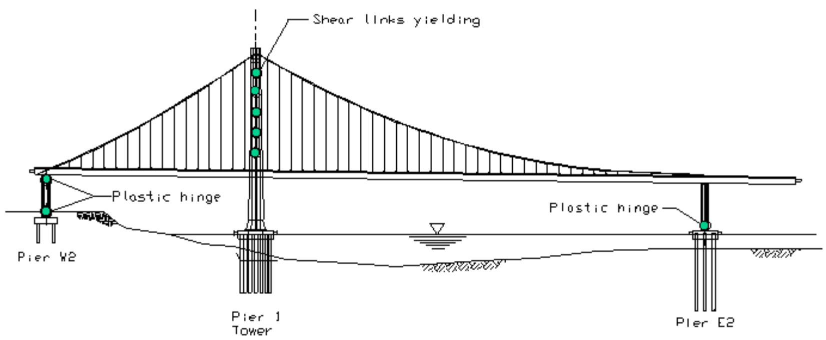

Figure 11. Seismic Response of the Bridge

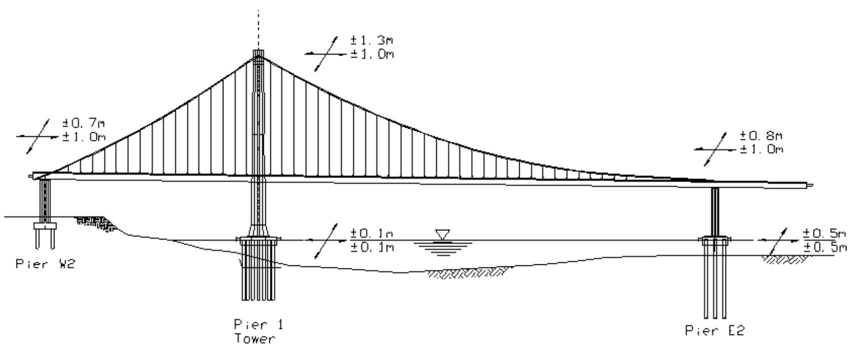

Figure 12. Seismic Displacement Demands

A pushover analysis of tower was also performed. This analysis was performed to evaluate the base shear versus top of tower displacement relationship, to optimize the design of the tower shear link and shaft, to evaluate the lateral ductility of the tower before collapse and to evaluate the ductility demands on the shear links and tower shafts at various levels of displacement demand During an earthquake the inertia load vector on a structure varies continuously, while in a pushover the load vector remains constant. Hence, it is necessary to consider more than one load pattern in order to bound the expected lateral behavior. Up to three load patterns were used. These are: (i) shape of the first mode, (ii) deformed shape of the tower at a time step corresponding to maximum drift in the global time history analysis and (iii) a uniform load vector. Figure 13 shows the results of the transverse push-over analysis. The base shear is normalized with respect to the total gravity load at the tower base. Figure 13 shows the effects of using shear links on the lateral behavior of the tower. As noted the tower has stable behavior for displacements much larger the SEE displacement demands.

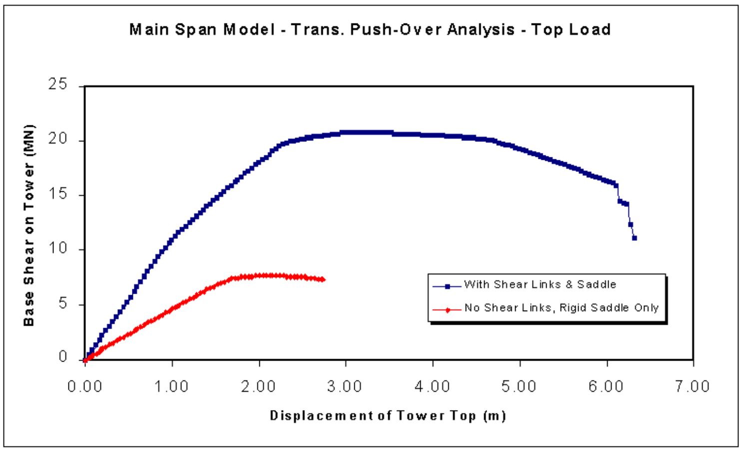

Figure 13. Pushover Analysis of the Single Tower

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