THE DESIGN AND CONSTRUCTION OF THE NEW SAN FRANCISCO-OAKLAND BAY BRIDGE (SFOBB) EAST SPAN

Tom Ho, Ph.D., P.E.[1]

Abstract

The new San Francisco-Oakland Bay Bridge East Span consists of four major components; the signature 385 m single-tower, self-anchored suspension span, the 1.6 mile dual box girder skyway, the 467 m Yerba Buena Island transition structures, and the 406 m Oakland touchdown approach. Complex and varied geological conditions at the bridge site, hazardous seismic conditions, and state-mandated design criteria contributed to the challenges of designing and constructing this landmark lifeline structure.

Bridge History

On October 17, 1989, at 5:05 p.m., a Richter scale 7.1 earthquake struck the San Francisco Bay Area. The epicenter was located in a remote hilly area of the Santa Cruz Mountains called Loma Prieta, which is about 97 km from San Francisco. The earthquake lasted approximately 15 seconds, killed 62 people, injured thousands and brought widespread damage to buildings, utilities, communications and transportation systems. One of the most significant transportation structures damaged during this earthquake was the East Span of the San Francisco-Oakland Bay Bridge.

The East Span of the Bay Bridge, designed in 1933 and completed in 1936, consists of four shallow simple span trusses on Yerba Buena Island (YBI), a long cantilever truss structure, five deep simple-span trusses, fourteen shallow simple-span trusses, and a number of simple-span deck systems that use steel and concrete stringers supported on transverse concrete bents. The total length of the East Span is 3.4 km.

The principal earthquake damage to the bridges was the failure of the upper and lower closure spans at Pier E9 (Figure 1). The bridge was closed for one month for repair. These 15.2 m-long upper and lower closure spans fell when the bolts failed that connected the pier and the 88.4 m truss to the east. This collapse was caused by a 13 cm movement of the truss span which pulled the closure spans off their seat support. Another span at Pier E23, close to the eastern edge of the bridge, was near failure of a comparable type. Connections at Pier E18-E23 also failed.

FIGURE 1: PIER E9 DAMAGE FROM THE LOMA PRIETA EARTHQUAKE

[1] Vice President, T.Y. Lin International, San Francisco, California, USA

The existing Bay Bridge East Span was designed for 10%g earthquake acceleration which was consistent with the 1930 Uniform Building Code recommendations. A modern seismology study indicated that the Hayward and San Andreas faults, which the bridge is situated in between, can generate magnitude 7.5 and 8.1 earthquakes respectively. The acceleration from these earthquakes will be many folds higher than that of the designed value. After long and extensive studies, it was determined that retrofitting the structure, in particular the aged short timber piles, was not only too expensive but also unreliable. Engineering and economic analyses suggested that a simple replacement bridge would cost only marginally more than a retrofit and the replacement would have a far longer useful life. Subsequently, it was decided by the California State authorities that the East Span would be replaced (California Department of Transportation Vulnerability Reports).

Design Constraints and Criteria

The California Transportation Department (Caltrans) coordinated the design process with input from the Metropolitan Transportation Commission (MTC) of the Bay Area, the Federal Highway Administration (FHWA), and the San Francisco Bay Conservation and Development Commission (BCDC). The MTC, at the request of the California Governor and Bay Area legislative leaders, become involved to ensure regional interests were appropriately addressed during the project development process.

In July 1997, the MTC-appointed Engineering and Design Advisory Panel (EDAP) and the MTC Bay Bridge Design Task Force created a 17-point criteria list for the design of the new bridge. MTC recommended that two cable-supported design alternatives be taken to the 30% design level before a final selection was made. The possibility of incorporating a bicycle and pedestrian path on the structure was also to be evaluated. Additional MTC recommendations outlined that the new eastern crossing should:

- Provide post-earthquake “lifeline” service
- Include 10 traffic lanes, five in each direction, with two standard 10-foot shoulders in each direction
- Accommodate the possibility of future light rail service
- Be built on the northern adjacent alignment
- Include a cable-supported main span with a single vertical tower, with single or multiple legs in the transverse direction and single to multiple planes of supporting cables
- Incorporate two parallel separated decks on the causeway section and either parallel separated decks or a single deck on the cable-supported span, instead of a double-deck like its western counterpart
- Utilize structural elements that are visually consistent
- Be comprised of long, equal span lengths in the causeway section, although shorter span lengths might be necessary adjacent to the Oakland shore

- Incorporate a cable or suspension tower that is no taller than the suspension towers on the existing western span
- Provide a vertical clearance above mean sea level of 42 meters.

Geotechnical Conditions

The site geology varies dramatically along the alignment of the bridge (Figure 2) and represented a major design challenge. For the YBI structure on the western end of the bridge, all the piers will be on land and founded on rock. As the bridge alignment progresses east towards Oakland, the bedrock/Franciscan formation dips abruptly and the remaining piers overlay deep Bay mud, followed by the inter-layered clay and sands of the Alameda formation. The bedrock elevation along the majority of the bridge alignment is over 100 m below the sea level. Only the main span tower structure is sited on relatively shallow, sloped bedrock. The remainder of the skyway section is foundered on thick sediment layer (Fugro/EMI Geotechnical & Seismic Report, 1999).

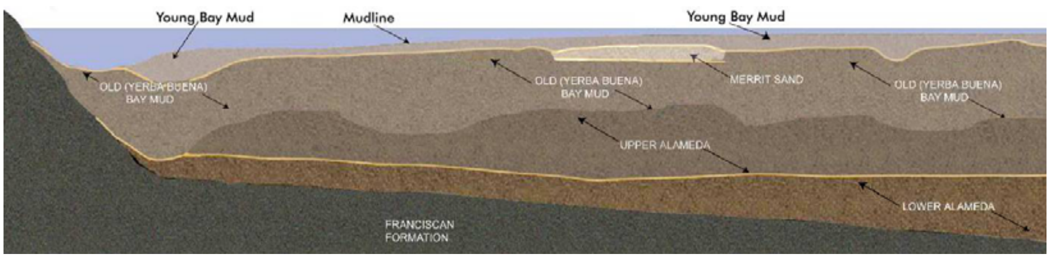

FIGURE 2: SITE GEOLOGY

Seismic Hazards

Seismic hazard evaluations were performed to define the safety evaluation earthquake (SEE) and the functional evaluation earthquake (FEE). The SEE corresponds to an earthquake with a return period of 1,500 years and FEE corresponds to an earthquake with a return period of 450 years. Both deterministic and probabilistic approaches were used to set the appropriate seismic criteria for the site. The seismic risk is dominated by the San Andreas and Hayward Faults. Six seismic ground motions were developed and used to determine the structural deformation, strength demands, and drifts.

Seismic Performance Criteria

During the eight years after the Loma Prieta Earthquake, several studies were performed by researchers, engineers, and governmental agencies regarding the damage that major infrastructures suffered and the impact to the public. It later became government policy that a lifeline structure such as the Bay Bridge must be functional after a major earthquake in order for emergency vehicles and personnel to reach population centers to perform post-earthquake relief work.

According to this policy, the new Bay Bridge East Span was designed to provide a high level of seismic performance and to resist both an SEE and FEE. After an FEE, the bridge will provide full service almost immediately, with only minimal damage to the structure. Minimal damage implies essentially elastic performance of most structure components, and is characterized by:

- Minor inelastic response
- Narrow cracking in concrete
- No apparent permanent deformation
- Damage to expansion joints.

After an SEE, the bridge will provide full service immediately, with no more than repairable damage to structure. Repairable damage is damage that can be repaired with a minimum risk, such as:

- Minimum damage to superstructure and tower shaft
- Limited damage to piers (including yielding of reinforcement and spalling of concrete cover) and tower shaft links
- Minimal damage to piles and pile caps
- Small permanent deformations, not interfering with serviceability of the bridge
- Damage to expansion joints that can be temporarily bridges with steel plates.

The bridge was designed as a limited ductility structure with stable response to ground motions equivalent to the safety evaluation earthquake. The stability of the structure was demonstrated by means of pushover analysis or an equivalent method of structural evaluation. This means:

- The bridge shall have a clearly defined inelastic mechanism for response to lateral loads
- Inelastic behavior was restricted to piers, tower shear links, and hinge pipe beam fuses
- The detailing and proportioning requirement for full-ductility structures as defined in ATC-32 was met.

For seismic loading during construction, the performance objective was to avoid collapse (Figure 3), (T.Y. Lin International/Moffatt & Nichol Design Criteria, 1999).

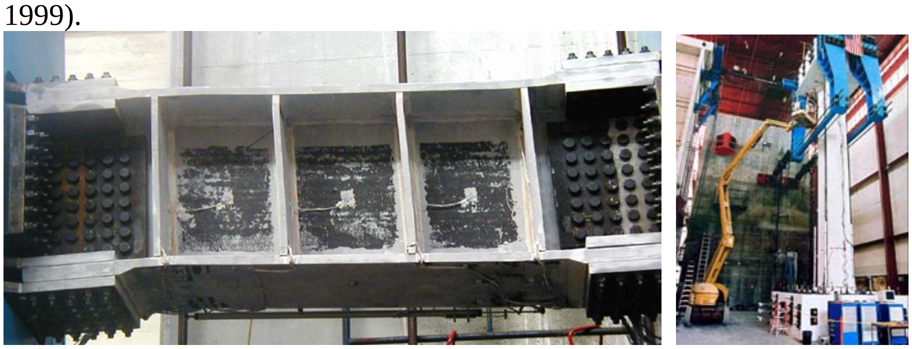

FIGURE 3: SHEAR LINK AND CONCRETE PIER LABORATORY TESTING

Seismic Analysis

The preliminary design of the skyway structure was performed using response spectrum analysis. Response spectrum analysis was used to establish target displacements for non-linear static (pushover) analysis. The self-anchored suspension bridge was designed using nonlinear time history analysis. Elastic time history was used in the preliminary design of this structure.

A displacement-based design approach was used for the final seismic design of the skyway structure. This approach includes direct design using non-linear time history analysis and design verification using pushover analysis. All displacement-based design utilized models that reflected the "best estimate" of the likely structure and soil conditions at the time of the safety evaluation earthquake. The maximum instantaneous demands for any of the six 3-dimensional input motions were used as the deformation demands for the non-linear time history analysis. When pushover analysis was used to calculate the deformation capacity of the structures, the deformation capacity, corresponding to the limiting strain in materials, exceeded or was equal to the calculated deformation demand.

Soil structure interaction effects were considered in all analyses. Foundation dynamic characteristics were incorporated into the analysis with discrete element representation piles and footings with appropriate representation of the effects of soil structure interaction (SSI). Depth varying free-field motions were applied to the analysis model along the buried length of the appropriately discretized piles (T.Y. Lin International/Moffat & Nichol 30% Design Report and Supplement to Final 30% Design Report, 1998).

Major Structural Components

Yerba Buena Island Transition Structure (YBI)

The transition structure at YBI is technically and logistically challenging because of roadway geometry, future on- and off-ramps, island topography, existing historical structures and temporary detours.

The eastbound and westbound transition structures connect the main suspension span structure to the existing viaduct section near the Yerba Buena tunnel east portal. The two structures are carried on separate single-column bents except near the viaduct end where they are supported on outrigger bents. The separate transition structures merge into the existing double-deck viaduct structure, just east of the YBI tunnel.

The length of each transition structure is approximately 467 m. The westbound structure has an overall deck width of 25 m and varies in depth from 1.6 m at the viaduct end to 4.5 m at the main suspension support. The height of both structures varies from 8 m at the viaduct end to 46 m above-grade at the main span end.

The superstructure of the transition structures consists of cast-in-place reinforced concrete box girders in the vicinity of the concrete outrigger type bents and cast-in-place prestressed concrete box girders over single column bents. The shape of the box girders has been determined primarily by aesthetics to match the streamlined girder shape of the main span superstructure and skyway. Most bents are located in over-burden sand and are supported on cast-in-drilled-hole piles.

As with all seismic retrofit projects, construction must take place in close proximity of moving traffic with an effort to minimize disruptions to traffic flow. To accomplish this task, eastbound and westbound traffic will be detoured onto temporary structures, which are supported by 60 m steel towers. The detour structures will parallel the existing lanes on YBI and will tie into the existing bridge and tunnel lanes. Once traffic has been safely detoured, the existing roadway will be removed and new lanes tying into the tunnel will be built (Manzanarez, 2007).

Self-Anchored Suspension Bridge (SAS)

The SAS consists of two connected, continuous steel box girders with a 385 m main span and a 180 m back span (Figure 4). The single 0.78 m diameter cable is anchored within the steel box girder deck at the east and is looped around the west end through two large deviation saddles.

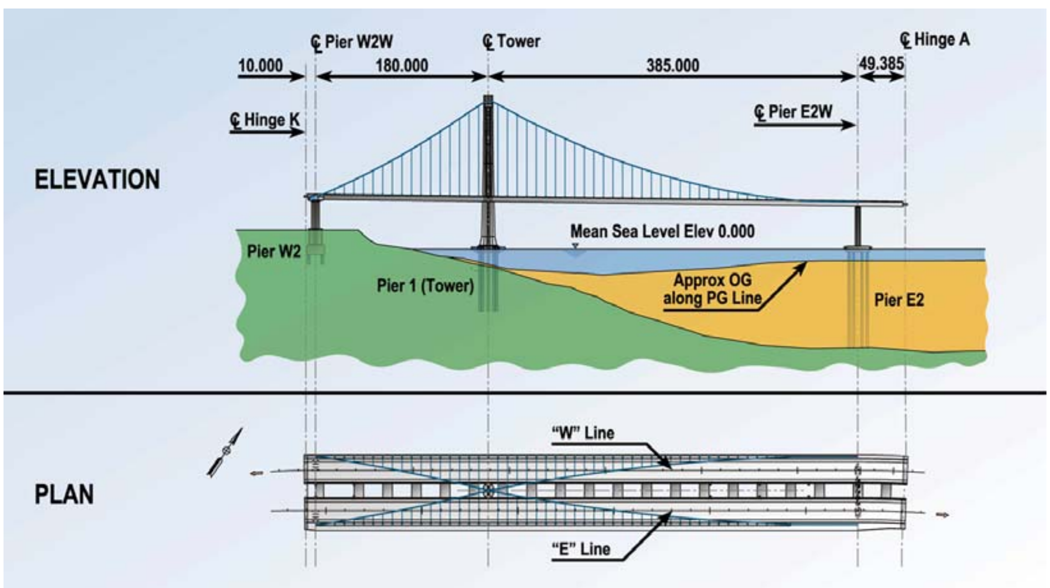

FIGURE 4: SAS PLAN AND ELEVATION

Unlike traditional suspension bridges, these deviation saddles are fixed to the pier cap and the cable force on either side of these saddles is balanced during construction using a jacking saddle. These saddles at W2 are supported by a pre-stressed concrete cap beam, which was designed to carry the differential cable forces arising during service and seismic loads. The weight of this cap beam was designed to balance the dead load uplift at the west bent arising from 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 (E2) is supported by the box girders and is designed to slide in order to eliminate unbalanced cable forces on either side of the saddle. The suspenders are splayed to the exterior sides of the box girders and are spaced at 10 m. The superstructure consists of dual hollow ASTM Grade 50 orthotropic steel boxes (Figure 5). These boxes are in compression (restraining the cable tension forces). Deck diaphragms spaced at 5 m support the orthotropic deck and distribute the suspender loads to the box. The two box girders are connected together by 10 m wide x 5.5 m deep crossbeams spaced at 30 m. These cross beams carry the transverse loads between the suspenders (span of 72 m) and ensure that the dual box (Verendeel truss system) act composite during wind and seismic loads.

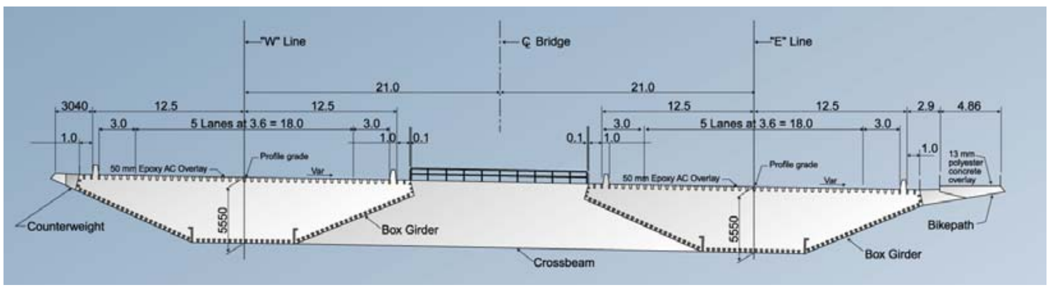

FIGURE 5: TYPICAL BOX GIRDER SECTION

The bridge carries a pedestrian path on the south side of the eastbound deck. The eccentric load is balanced by a counter-weight on the north side of the westbound deck. At the west bent, the steel box girders frame into the concrete cap beam which serves as an extension of the box girder, counterweight and transfer cable loop compression. The connection between the orthotropic steel box girders and concrete cap beam is subjected to the compressive forces of the cables. Additional prestress is added through post-tensioned strands connected at each deck rib to ensure that the steel box yields before the connection fails.

The 160 m single tower of the SAS is composed of four shafts interconnected with steel shear links along its height in both the longitudinal and transverse directions of the bridge. These shafts are rigidly connected at the top and bottom by tower saddle grillage and the tower base, respectively. The bridge deck is not connected to the tower in order to reduce the seismic demand. These shear links are one of the unique features of the East Span design. They supply the single tower with required stiffness for service conditions after a functional earthquake. They should remain elastic during a FEE but will plastify during an SEE, thus dissipating energy so that the tower shafts can maintain basically elastic. Therefore, the shear links are designed to supply the tower with the necessary ductility and toughness. These links are not a part of the gravity load carrying system and they are connected to the tower shafts with bolts. This means they can be replaced after an SEE event without any disturbance to traffic. The effectiveness of the shear links was confirmed by a full scale laboratory test.

The tower shafts are stiffened pentagonal box sections, which taper along the height of the tower (Figure 6). The shafts' ASTM A709 Grade 50 steel skin plates vary from 45 mm to 100 mm with diaphragms spaced at 3 m. The tower is fixed to the 6.5 m deep pile cap (consisting of a steel moment frame in-filled with concrete) and is supported on 13 - 2.5 m diameter steel shell pipe piles filled with concrete and socketed into rock. The rock slope is benched to give the piles equal lateral stiffness, thus avoiding torsional response; pile clear length is about 20 m. The east piers are reinforced concrete columns and are supported on 16 - 2.5 m steel shell pipe piles. These piles are 100 m long and are filled with concrete for the upper 55 m of the pile. The west piers are reinforced concrete columns which are monolithic with the prestressed cap beam (forming the west bent) and are supported on rock through a massive concrete foundation and 8 - 2.5 m cast-in-drilled holes 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 the footing. At the east piers, the box girders are supported on bearings. Shear keys and specially designed uplift bearings are provided to resist seismic demands. The box girders are supported at the east and west pier footing and are "floating" at the tower (Manzanarez, 2007).

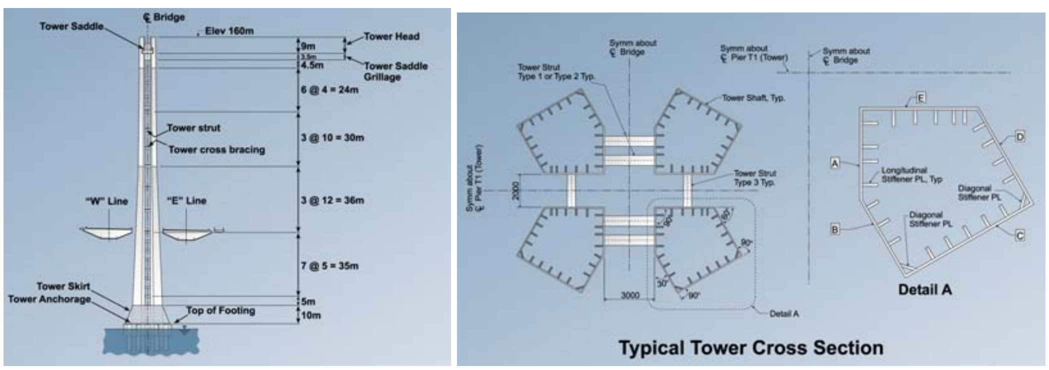

FIGURE 6: SAS TOWER DETAILS

Skyway

For the haunched concrete girder alternative, both balanced cantilever cast-in-place (CIP) and precast segmental construction erection were evaluated. Normal weight concrete is used for the superstructure except for the lightweight concrete side inclined panel. The concrete strength for both the lightweight and normal weight concrete is 50 MPa. The box girder is a single cell with two vertical webs. The width at the bottom slab (soffit) is 8.5 m at the piers. The 25 m width accommodates long deck overhangs of 8.3 m each. Inclined concrete struts are used to carry the deck overhangs and eccentric bike path loading, post-tensioned in both the longitudinal and transverse directions. Vertical post-tensioning is used to control shear stresses in the girder main webs.

The nominal 160 m spans are arranged in frame units of three or four piers per frame with a girder depth of 5.5 m at midspan and 9 m at the pier. Midspan hinges between the frames allow longitudinal expansion and contraction due to creep, shrinkage and temperature changes. The Skyway plan and elevation can be seen in Figure 7.

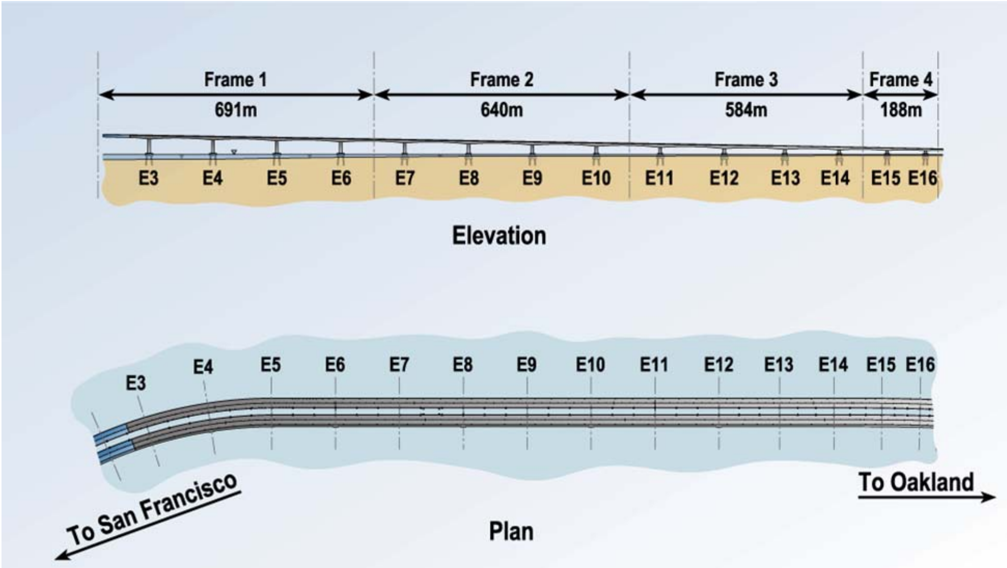

FIGURE 7: SKYWAY PLAN AND ELEVATION

One of the unique features of the East Span project is the internal steel pipe (1.90 m in diameter) beam assembly at the hinge, which provides shear transfer and moment resistance in addition to controlling deflections at the cantilever end of each frame. These beams (Figure 8) are in contact with the box through circular bearings. The hinge pipe beams are designed to have a fuse in the middle. When a strong earthquake hits the structure, this fuse is designed to yield which in turn will minimize the damage to the rest of the structure. This fusing action is achieved by using a thinner and lower grade steel at the mid-section of the pipe beam. The fuse portion can be replaced if yielding occurs.

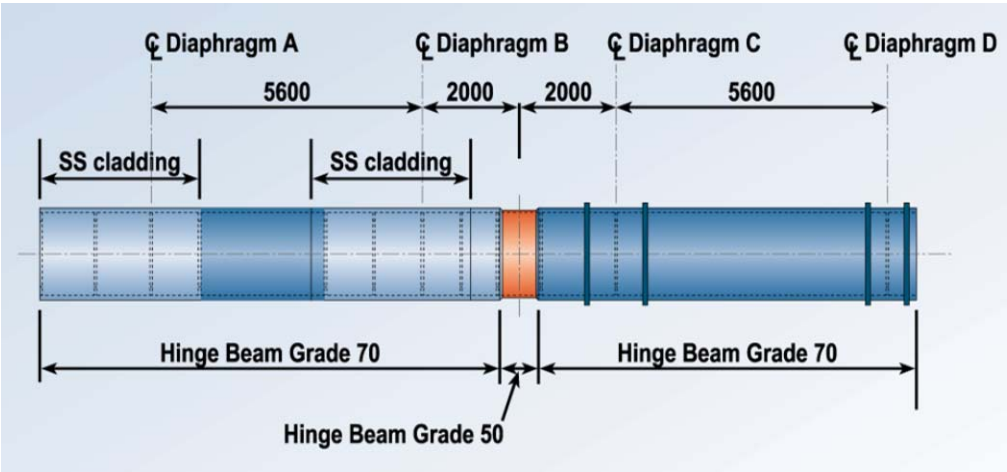

FIGURE 8: STEEL PIPE BEAM

The foundation and piers (Figure 9) were designed to resist loads due to elastic and plastic deformations of the superstructure including creep, shrinkage and temperature drop. This loading is critical for the proposed concrete superstructure for frames with short piers near the Oakland shore. Steel tubular piles were best suited for skyway foundations because of their strength and ductility as well as ease of installation. These piles were driven into the lower Alameda Formation and are 90 to 100 m in length.

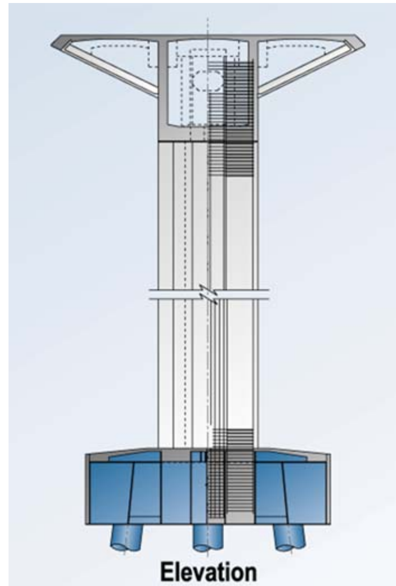

FIGURE 9: TYPICAL SKYWAY PIER

The piers in Frame 1 of the skyway are about 45 m in height and stand in water depths of 15 m. When combined with the presence of a 15 m layer of young Bay mud, this establishes a very flexible structure. The design approach for such piers was to adopt a stiff foundation system, thereby controlling pile cap elastic displacements to acceptable seismic demand levels and minimizing the potential for permanent pile cap offsets. A relatively stiff foundation system for the tall flexible piers was achieved through the use of large diameter battered steel piles filled with concrete.

Pile shell thickness was chosen based on required flexural capacity of the pile, ductility capacity, corrosion allowances and drivability. Pile flexural demands controlled the pile shell thickness at the upper region of the pile. As flexural demands decreased down the pile length, the shell thickness required for driving governed the design.

The 2.5 m piles were filled only in the upper two-thirds of the pile length with structural concrete to form a composite steel/concrete pile. A sufficiently thick shell allowed for the use of an unfilled pile below the composite pile section. To ensure proper transfer of loads from the concrete fill to the steel shell, shear ring plates were welded to the inside of the steel shell in the concrete filled section.

The pile cap was approximately 6 m deep with 2 m exposed above mean sea level. The pile cap structural system is a moment-resisting steel frame interconnected together with the steel piles and the pier reinforcement (Manzanarez, 2007).

Oakland Touchdown

The westbound approach structure extends from the tip of the Oakland shoreline to the point where the proposed westbound I-80 joins with the existing I-80, connecting the Oakland shore to the skyway. The total length of the westbound approach structure is about 406 m. The eastbound approach structure is an elevated two-span frame which is 105 m long. The eastbound structure and the westerly portion of the structure are elevated structures consisting of a cast-in-place prestressed concrete box girder superstructure supported on reinforced concrete piers, reinforced concrete footings, and prestressed concrete piles or small diameter steel pipe piles (Manzanarez, 2007).

Construction

The San Francisco-Oakland Bay Bridge East Span replacement is truly an international project. Elements of the bridge are supplied by vendors from all over the world:

- China – Main steel tower, steel orthotropic box girder, main cables
- Japan – Cable saddle, cable wrapping
- Korea – Shear key casting, suspender cables
- United Kingdom – High strength prestressing anchor rods, cable bands
- Canada – Structural detailing, some aggregates
- United States – Piles, pile caps, transition structure (steel box girder) and pipe beams.

The cranes, barges, temporary towers and many other construction related equipment also come from fabricators around the world.

Self-Anchored Suspension Bridge

The construction of the SAS structure started in the summer of 2006, with the construction of the W2 cap beam. At the time of this report, the falsework has been completed and the concreting will begin in the fall of 2007. The construction of the E2 columns will start soon.

The preparation work for the fabrication of the tower and orthotropic box girder is also underway; over 100,000 shop drawings will be produced to provide the fabricator detailed information for the construction of this complex steel structure. The first mark-up segment of the tower is under way in Shanghai. Two more tower mark-ups and one box girder mark-up section will be produced to make sure that all the fabrication details and issues are addressed and so construction workers have enough familiarity with the fabrication procedure. The first box girder segment is scheduled to arrive in San Francisco by the fall of 2008. It is estimated that this portion of the construction will be completed by 2012.

When building a conventional suspension bridge, one must follow a rigid erection sequence: First the tower, followed by the ground anchor, suspension cables, suspenders, and finally the deck. In the construction of a SAS bridge, the entire deck has to be in place and completed prior to the installation of the suspension cables. A series of temporary tower/trusses are needed to support the deck at this stage. After connecting all the deck segments, the cable can then be installed, followed by the load transfer and the completion of the structure.

Skyway

The skyway structure construction began in 2002. By fall of 2007, the entire skyway structure will be completed. The construction of the skyway structure started with pile driving First, 182 of the 2.5 meter diameter battered steel piles were driven deep into the Bay soil by one of the world's largest hydraulic hammers. The length of the piles ranges from 86 to 100 m and wall thickness ranges from 51 to 76 mm. After the piles were in place, barge mounted cranes lifted 32 steel pile caps in place and the welder connected the piles to the pile caps. The piers and pier tables were constructed with cast-in-place concrete.

The skyway deck is comprised of 452 precast, post-tensioned concrete segments. These segments, the largest of this kind ever cast in the U.S., were fabricated in Stockton, California and transported by barges to the project site. They were lifted into place by winches (Figure 10), which were custom made for the East Span project.

FIGURE 10: LIFTING CONCRETE SEGMENTS INTO PLACE

The Stockton Yard (Figure 11) is located on a deepwater shipping channel 125 km northeast of the construction site. To ensure the segments fit together precisely, they were match cast in a long line set up. After the concrete set, the segments were transported to a storing area for a curing period from two to 18 months based on the design. The time to transport the segments from the yard to the construction site typically took 10 to 12 hours. All segments that arrived the site were lifted and erected on the same day to avoid any accidental damage to the segments.


FIGURE 11: THE STOCKTON CASTING YARD

Oakland Touchdown

The first step in building the Oakland Touchdown was the completion of the geofill area in the winter of 2002. This created a roadbed for the land portion of the new bridge near the Toll Plaza.

The soils near the bridge in Oakland tend to be muddy. As part of the geofill project, bridge builders made the underlying soil stable for supporting a road by applying dirt fill. This job required placing 72,000 cubic yards of fill and 22,000 cubic yards of rock to make a roadway along the Bay shore where previously only water and mud existed. The work also included the installation of 6,000 plastic wicks and flexible plastic drain pipes, which allowed the water that percolates up to the road substrata during an earthquake to drain through a gravel layer, rather than pooling under the road.

The next phase of the Oakland Touchdown work will be to construct all marine and land-based foundations for both the westbound and eastbound bridges, both frames of the westbound structure, one frame of the eastbound structure, roadway approach for the westbound structure, including lightweight fill, etc.

Conclusion

The East Span of the San Francisco Oakland Bay Bridge is a critical link and lifeline structure for the Bay Area road system. The closure of the bridge after the 1989 Loma Prieta earthquake not only interrupted tens of thousand commuters' daily lives, but also wrought major damage to the Bay Area's economy.

Experts continue to try and predict when the next big earthquake is coming to the Bay Area. But engineers, builders, and researchers have been working diligently over the past 17 years to build an earthquake-proof structure that can withstand the next major earthquake. Once construction is completed, the new East Span of the San Francisco-Oakland Bay Bridge will be one of the engineering marvels of the 21st century and will serve Bay Area residents for the next 150 years.

Acknowledgements

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