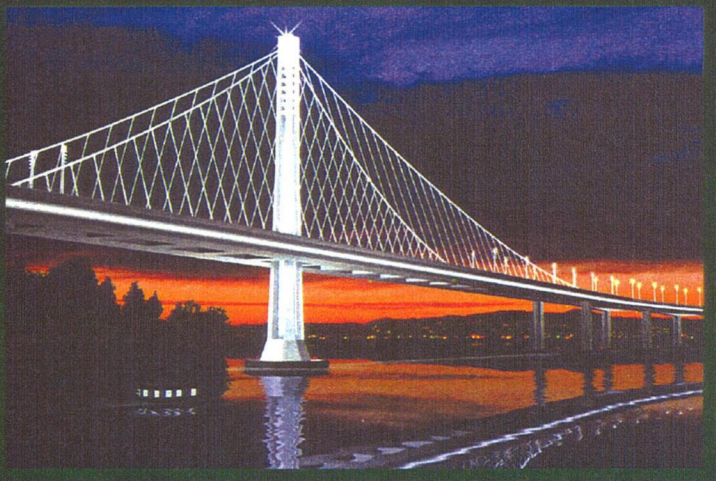
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SAN FRANCISCO – OAKLAND



BAY BRIDGE

T.Y. LIN INTERNATIONAL
MOFFATT & NICHOL
a joint venture

Design of the New San Francisco – Oakland Bay Bridge – Phase I

ABSTRACT

The seismically vulnerable East Span of the San Francisco-Oakland Bay Bridge will be replaced with a dual east bound and west bound 3.6-km-long parallel structure. The cost of the replacement bridge is estimated at $1.5 billion and will be constructed by the year 2004. The Bay Bridge lies between the Hayward and the San Andreas faults, which can generate magnitude 7.5 M and 8.1 M earthquakes, respectively. Performance criteria require that almost immediately after a 1500-year return period earthquake from either of these two faults the bridge must be operational. The new crossing will accommodate five lanes of traffic in each direction and is designed to carry a light rail transit (LRT) system on each of the 25.0-m-wide decks. In addition, the east-bound structure will support a 4.8-m-wide pedestrian/bicycle path. Four distinct structures will make up the bridge crossing: a low rise post-tensioned concrete box girder near the Oakland shore; a 2.4-km-long segmental concrete box girder; a self-anchored suspension signature span; and a post-tensioned concrete box girder that connects to the east portal of the Yerba Buena Island tunnel. This paper focuses on the design parameters and constraints; the design evolution; the bridge type selection process; the selected design's major structural and architectural characteristics; and the structural details.

INTRODUCTION

The San Francisco-Oakland Bay Bridge was constructed in the mid-1930s. At 8 miles long, it was one of the longest high-level bridges in the world at the time. Today, the bridge is a primary route between the San Francisco Peninsula and the East Bay, carrying nearly 280,000 vehicles daily.

A variety of bridges span the waterways of the San Francisco Bay. There are suspension bridges such as the Golden Gate Bridge, long skyway bridges such as the San Mateo-Hayward Bridge, and truss bridges such as the Richmond-San Rafael Bridge. Figure 1 is a map of the San Francisco Bay Area showing these bridges.

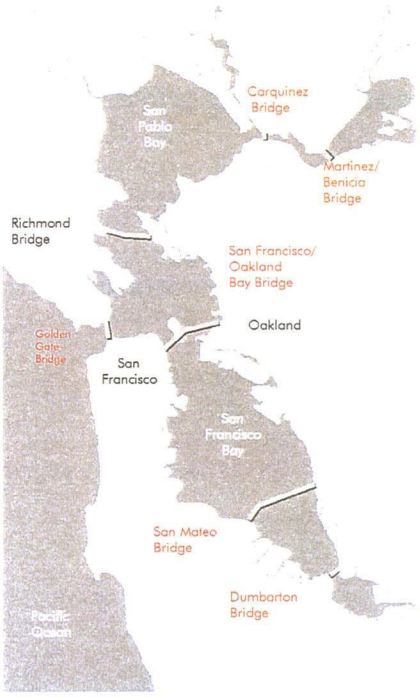


Figure 1 – Location Map

Comparable to the levels specified in the 1930 Uniform Building Code for buildings, the Bay Bridge was designed for only 10 percent gravity (g) earthquake accelerations. The new structure is being designed for seismic demand of up to 100 percent gravity at some locations.

In 1989, an earthquake measuring 7.1 on the Ritcher scale centered 70 miles to the south of San Francisco, caused a 15-m section of the upper deck of the East Span of the San Francisco-Oakland Bay Bridge to collapse as shown in Figure 2.

Figure 2 – Damage to Bay Bridge



A more severe earthquake has been forecast to occur in the next 50 years and will cause significant damage to the existing East Span's truss structures and possible collapse. Retrofitting the structure was not only determined to be expensive and unreliable but also very difficult to implement under traffic.

Senate Bill 60 authorized funding for the design and construction of a replacement structure for the East Span of the Bay Bridge. The new structure will be located about 150 m north of the existing east span. The structure will traverse the Bay from the east end at the Oakland Mole to the west end of the signature span with side-by-side parallel decks that are separated 17 m. The structure will continue west with a double-deck structure at Yerba Buena Island (YBI) tunnel's east portal.

The westbound (WB) deck is 25 m wide and will carry five lanes of traffic with two exterior 3 m wide shoulders. The eastbound deck is 25 m wide, carrying the same number of lanes and shoulders as the WB, but in addition this will accommodate a 4.8m wide pedestrian/bike path on the south side.

The bridge is located between two highly active seismic faults; the San Andreas and the Hayward. The four types of structures, comprising the crossing, are presently being designed by the T. Y. Lin International and Moffat and Nichol Joint Venture team. From east to west a description of each type of structure selected for final design and construction follows: (See Figure 3 in the next page for bridge plan and elevation.):

- A low-rise of post-tensioned concrete box girder to be constructed on falsework at the Oakland touchdown.
- A segmental concrete "skyway" with varying span lengths from 120 m to 160 m to be constructed using the balanced cantilever precast segmental construction method. The length of this structure is 2.4 km. The skyway will be founded on large diameter steel piles, 90 m to 100 m long, arranged in groups of 6 and 4, and partially infilled with reinforced concrete. The piers will be hollow box reinforced concrete.
- A single steel tower located just off YBI's eastern shore, asymmetrically self-anchored suspension span over the navigational channel. The main span will be 385 m with a back span of 180 m. When built, this will be the largest self-anchored suspension span in the world. The tower will be 160 m tall and will consist of four steel shafts connected with intermittent steel shear links along its height. The decks will be orthotropic steel box girders connected transversely together with cross beams and supported with rope hangers at the outer edges of the girders. The tower will be supported on steel pipe pile socketed to rock with concrete. The east pier will be supported on steel pipe piles founded on the Alameda Formation and the west pier (Anchored Pier) will be supported on cast-in drilled hole concrete
- A double deck post-tensioned concrete box girder with span lengths up to 80 m at the YBI transition structure, to be built on false work.

The selected bridge for the East Bay Crossing is shown below in Figure 4.

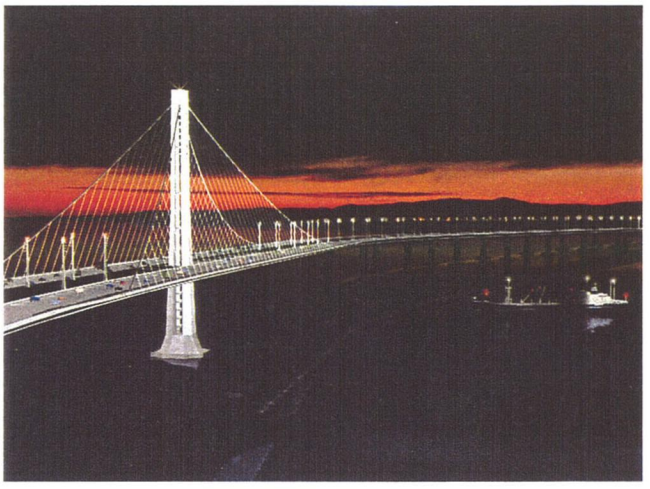


Figure 4 – Rendering of Selected Bridge

The project will also require the design and construction of two temporary detour steel structures at YBI for traffic routing during construction of the YBI transition structure. In addition, the existing bridge will be demolished.

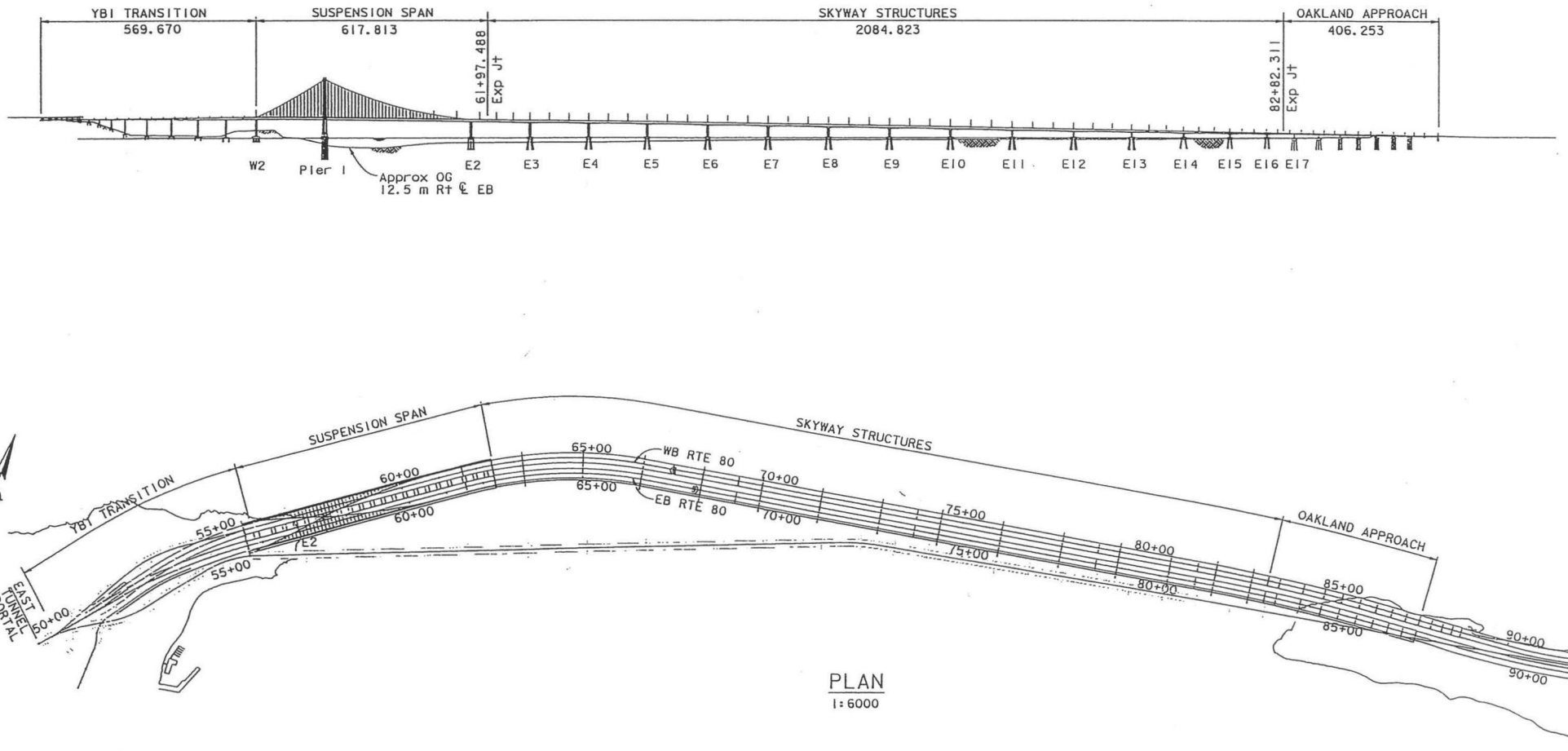


Figure 3 – Bridge Plan and Elevation

The design will be completed in May 2000, with construction to start in the fall of 2000.

Caltrans is coordinating the design process with input from the Metropolitan Transportation Commission (MTC), the Federal Highway Administration, and the San Francisco Bay Conservation and Development Commission. The MTC, at the request of the California Governor and Bay Area legislative leaders, became involved to ensure that region-wide interests are appropriately addressed in the project development process. The project is owned by Caltrans.

To foster public involvement and consensus in the selection of the design for a replacement bridge, MTC formed the Bay Bridge Design Task Force in February 1997. Its primary objective was to review preliminary bridge design concepts and select the bridge type and amenities. The Task Force assembled an Engineering and Design Advisory Panel (EDAP) of 35 members to conduct expert technical evaluation of the design options, alignments and amenities such as pedestrian/bikeway and Light Rail Transit (LRT) options. MTC recommended that two cable-supported design alternatives (a cable-stayed bridge and a self-anchored suspension bridge) be considered for the signature span over the navigation channel of the bridge and taken to the level of 30 percent design completion before a final selection was made. The Joint Venture design team produced designs of a sufficient level of detail at the 30 percent design phase to support reliable judgments concerning cost, seismic performance, visual appearance, construction risks, and schedule, and allow EDAP and MTC to select the bridge type and amenities.

ALIGNMENT CONSIDERATIONS

The limits of the potential bridge locations were defined by the alignments designated as N1 and N2 shown on Figure 5. The selected alignment, N6 (shown in white), was developed by considering these factors and constraints:

- The vertical profile of the proposed structure needed to match that of the existing YBI viaduct. This restriction was necessary to facilitate detour construction and maintain traffic flow during the construction of the permanent structure.
- The horizontal alignment of the proposed west bound structure needed to join the existing alignment approximately 110 m east of the YBI tunnel portal.
- It was desirable to locate the main span on a tangent alignment. This was intended to maintain construction economy and aesthetics.
- It was desirable to maintain the main span tower reasonably close to YBI. The bedrock is at a higher elevation near YBI, which provides more favorable foundation conditions for the main span tower.
- It was preferable to align the east end of the bridge adjacent to the existing structure since the thickness of the soft young Bay mud deposits typically increase as the alignment progresses northward from the existing structure.
- It was necessary to achieve highway geometrics that provide for a design speed of 100 km per hour.

The vertical profile of the proposed WB structure extending from the existing YBI viaduct, provides a long gradual vertical curve through the main span, then descends at a constant grade of approximately 2 percent to the Oakland shore. The vertical alignment of the proposed EB structure also extends from the existing YBI viaduct and emerges from beneath the proposed WB structure, rising to meet the WB vertical alignment at a point just west of the main span tower.

GEOTECHNICAL SUMMARY

Geological and geotechnical studies were conducted for the new San Francisco-Oakland Bay Bridge East Span Replacement. Activities included: 14 marine borings, three land borings on YBI, and three seismic cone penetration test soundings (CPT) on the Oakland Mole. For the 14 marine borings the following tests were performed: in situ testing, soil sampling, rock coring, and downhole geophysical logging.

The following geologic features influenced the subsurface conditions underlying the new East Span northern alignment:

- Sandstone bedrock of the Franciscan Formation on Yerba Buena Island. To the east and west the sandstone outcrops at the ground surface, while in the central portion of the alignment on YBI, the bedrock is overlain by up to 80 feet of dense sandy soil.
- Shallow, easterly sloping bedrock of the Franciscan formation offshore is found east of YBI.
- Age marine and alluvial sediments (sands, clays, and silts unconformably overlay the Franciscan formation bedrock at the skyway location).

The site geology varies dramatically along the length of the bridge. Figure 6 is a geological profile at the bridge alignment. At the western end of the bridge (YBI), the piers will be founded on rock. As the bridge alignment

- The cable or suspension tower on the eastern bridge should not be taller than the suspension towers on the existing western span.
- The "diamond" shape for the main span tower should not be employed in any cable or suspension tower.
- Other constraints in the design include navigational requirements. The minimum horizontal clearance (fender-to-fender) shall be 152 m, and the vertical clearance above mean sea level shall be 42 m.

DESIGN CRITERIA—The bridge is designed in accordance with Caltrans Bridge Design Specifications Manual (1984 and amendments), and was modified or augmented as described in the project specific design criteria document developed by the Design Team. In addition, pertinent sections of the following standards or codes were used in the document including;

- AASHTO LRFD Guide Specifications for Design of Segmental Concrete Bridges, S.I. Units, 2nd Edition, 1999
- Light Rail Transit Design Criteria specific for this project
- Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges, Vol. I: Final Report, February 1991
- AASHTO LRFD Specifications for Highway Bridge Design, 1994, with 1996 and 1997 Interims.

SEISMIC PERFORMANCE CRITERIA—The bridge is designed to resist two levels of earthquake:

- A functional evaluation earthquake (90-year return period)
- A safety evaluation earthquake (1500-year return period)

After a functional evaluation earthquake, the bridge will provide:

- Full service almost immediately
- Minimal damage to the structure

After a safety evaluation earthquake, the bridge will experience damage in only pre-determined areas where damage can be repaired without significant traffic interruptions. This structure is specially designed for:

- Controlled inelastic action of structure in pre-determined locations
- Spalling of cover concrete in designated plastic or hinge regions
- Small permanent deformations
- Controlled yielding of reinforcement in shear links of tower
- Damage to expansion joints

BRIDGE TYPE SELECTION (30 PERCENT DESIGN)

In the next 150 years, the new bridge is expected to experience four major earthquakes. Therefore, seismic performance was considered the most important criterion in selecting the new bridge.

The main design goal was to create a bridge that is safe and integrates well with its environment, one in which all the design elements related to each other. The crossing from Oakland to San Francisco should become a unified whole, rather than a series of unrelated structures.

MAIN SPANS EVALUATED:

The design team was charged to develop several cable-supported design alternatives for the main span and navigation of the entire crossing. Two separate design teams; one for the suspension span option and another for the cable-stayed option, evaluated several design types and combinations thereof for the crossing in search of the best solution, balancing the design constraints imposed by EDAP and criteria such as seismic performance, aesthetics, constructibility, cost, and construction schedule.

SUSPENSION SPAN DESIGN ALTERNATIVES

The Bay Area has a strong tradition of suspension bridges. A new suspension bridge would complete the "necklace" of suspension bridges and other catenary shaped structures around the Bay (Golden Gate Bridge, west spans of Bay Bridge, and the new Carquinez Bridge.)

The high rock profile on Yerba Buena Island to the west of the main span and the soft, seismically sensitive soil to the east limit the bridge types that can be considered at the signature span site. A self-anchored suspension bridge is well suited to the site because of the poor soil conditions at the east ends that prohibit a cable rock socketed anchorage and the shallow rock profile near the YBI shoreline where the tower will be placed. Its longer eastern span of 385 meters rests lightly on the east pier. It delivers small dead and live loads to this pier, placing small demand on the foundation.

The cable carries to the deck indirectly by suspenders and directly by anchorage at the girder ends. The main cable is connected to the top of the tower. Thus, with no earth anchorage and with a floating deck at the tower, the

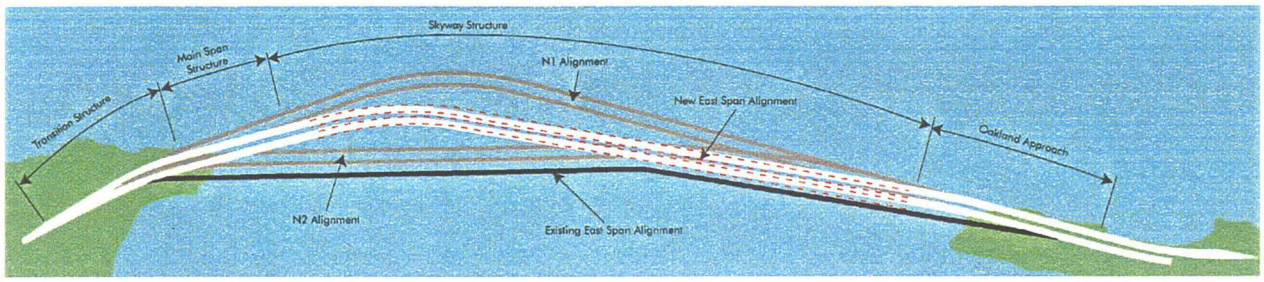


Figure 5 – Bridge Alignment

progresses east towards Oakland, the bedrock Franciscan Formation drops abruptly and the remaining piers overlie deep Bay muds, followed by the interlayered clays and sands of the Alameda Formation.

The main span tower structure will be sited on relatively shallow, sloping bedrock. The remainder of the skyway will be founded on a significant thickness of sediment.

Seismic hazard evaluations were performed to define the safety-evaluation earthquakes (SEE) and the functional-evaluation earthquakes (FEE). The SEE corresponds to an earthquake with a return period of 1,500 years, and the 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 Fault and the Hayward Fault. The bridge design is controlled by the SEE events. Six seismic ground motions were developed and used to determine the structural deformation, strength demands, and drifts.

- The new eastern span should be designed to provide post-earthquake "lifeline" service.
- The new eastern span should have 10 traffic lanes, five in each direction, with two standard 10-foot shoulders in each direction.
- The new eastern span should be designed to accommodate the possibility of future light rail service.
- The new eastern span should be built on the northern adjacent alignment (NI).
- The new eastern span should have 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.
- The new eastern span bridge should not be double-decked. It should have two parallel separated decks on the causeway (skyway) section and either parallel separated decks or a single deck on the cable-supported span.

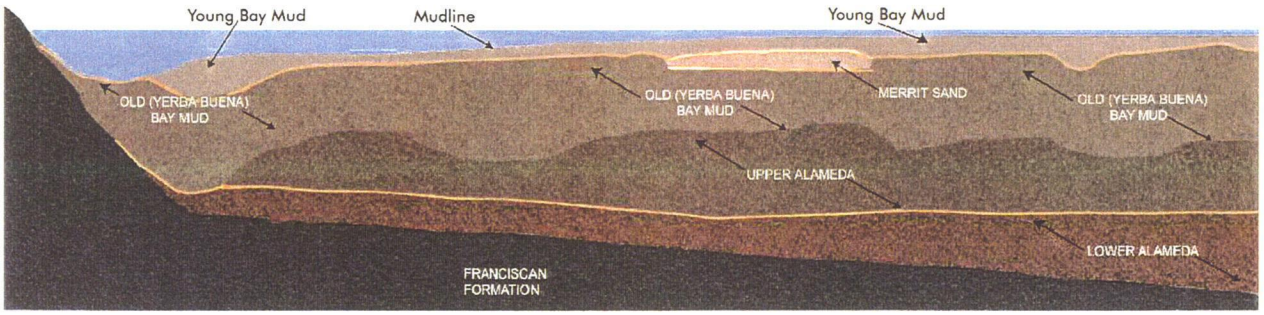


Figure 6 – Soil Profile

DESIGN CRITERIA AND CONSTRAINTS

In July 1997, EDAP and the MTC Bay Bridge Design Task Force made 17 recommendations to be considered for the new bridge design. MTC recommended that two cable-supported design alternatives be taken to the level of 30 percent design completion before a final selection is made, and that the possibility of incorporating a bicycle and pedestrian path on the structure be evaluated. The other MTC recommendations that have significant impact on the design include:

- The structural elements of the new eastern span should be visually consistent throughout.
- The causeway section should have long, equal span lengths, although shorter span lengths might be necessary just adjacent to the Oakland shore.
- For the causeway section, particular attention should be paid to the design of the supporting pier as it enters the water, including the possibility of submerging the pile cap below water.

bridge acts as three nearly uncoupled, long-period elements.

Self-anchored bridges are not new or uncommon. They were introduced about 75 years ago, and have recently been built in Japan (Konohana Bridge) and Korea (Jong Jong Bridge).

The suspension span designs evaluated include: a three-legged tower with three planes of cables, a double tower with four planes of cables and separate decks, a single tower design with a single large suspension cable and a single tower with splayed cable and inclined suspenders. In all the suspension span alternatives only the considered self-anchored suspension with the cable anchored in the box girder was considered. The design process was evolutionary after review comments were made by EDAP during regularly scheduled presentation meetings. Only two suspension span designs were carried out to the 30 percent design level, a single steel tower with splayed cable and the double steel tower design. These two designs were judged based on aesthetics, and constructibility and seismic performance. The winner of these two was then judged against the best cable-stayed design.

SINGLE TOWER SUSPENSION

In both the longitudinal and transverse directions the single tower is a contemporary, dynamic, and visionary design. It consists of a single steel tower with a double plane of splayed cables and inclined suspenders supporting both decks. See Figure 7.



Figure 7 – Single Tower Suspension Bridge

The main suspension tower consists of four trapezoidal tapered steel shafts connected by transverse link beams. Inclined suspenders connect to the outside edges of the decks. This creates a grand space, a majestic portal through which drivers pass between Oakland and San Francisco.

The single tower design, in general, is slender compared to the considerable size of the decks. By splitting the tower into four shafts, the suspension tower design is more in proportion to the decks and at the same time is seismically efficient. This design feature gives the tower transparency and a play of light and shadow that changes with the time of day and the viewer's location. "A majestic veiled room through a wire portal, with the tower prominently in the center," as the design team architect Donald MacDonald noted.

It is contextual in that it picks up the catenary shape of the other Bay Area bridges, and at the same time is unique in its expression and shape. It is state of the art in suspension bridge design, a recognizable landmark. Figure 8 shows an aerial view of the new east span single tower suspension bridge in relation to the West span of the Bay Bridge and the Golden Gate Bridge.

Suspension bridges with a single (or central) tower are not common, but many cable-stayed bridges have been constructed with a single central tower with splayed cables to the outside of the decks.



Figure 8 – Bay Area Suspension Bridges

DOUBLE TOWER SUSPENSION

The dual tower suspension option is a traditional and timeless design. Each deck is supported by an independent portal similar in size and proportion to the west Bay Bridge towers. Each portal has two legs, which are tapered and battered. This batter allows vertical suspenders. The legs are rectangular in cross section. The portals reinforce the sense of gateway. The main cables are aligned with the outside of the decks, and terminate in anchorage housing, which are integrated into the deck. Cable saddles are expressed as structural elements. Two completely independent bridges give more flexibility in alignment and provide structural provide redundancy in a seismic event.

The towers are made of steel and the deck is steel orthotropic box. The main span is 350 m with a back span of 215 m. Figure 9 shows a rendering view of this alternative.

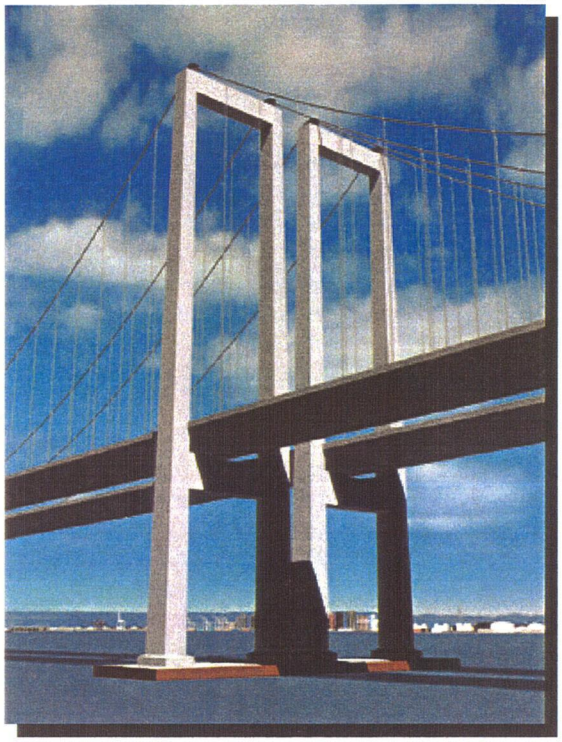


Figure 9 – Dual Tower Suspension Bridge

CABLE-STAYED DESIGN ALTERNATIVES

Five feasible alternatives were considered for the cable-stayed bridge over the navigation channel: two dual tower designs, two tri-pole tower designs, and a single tower design. For each, a gateway/portal effect was desired at the signature spans, achieved by a combination of tower elements and cable stays. At the 30 percent design level the cable-stayed design alternatives were narrowed down to two, the dual tower and single tower designs.

DUAL TOWER CABLE-STAYED

This scheme is a traditional twin tower design as shown in Figure 10. It is comprised of a twin two-legged tower merging into a three-legged form below the deck. The design exhibits a double symmetry with four vertical planes of cable. The twin towers are joined by cross beams at the top of the tower and below the deck, which serve an essential structural role and help to unify the composition of the design. The cables are arranged in a semi-fan configuration. Two deck options were evaluated; one the strutted box girder and the other steel box girder.

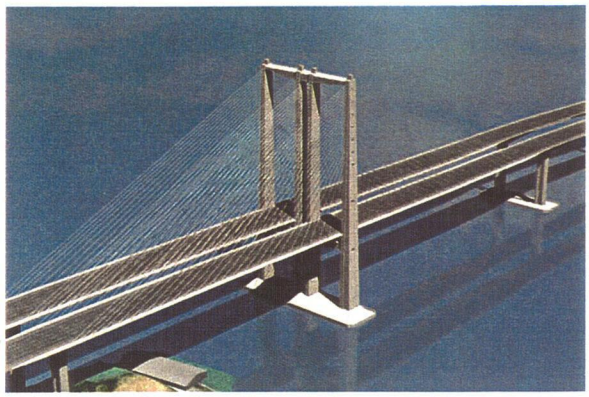


Figure 10 – Dual Tower Cable-Stayed

SINGLE TOWER CABLE-STAYED

This design is a single tower located between the two roadways. It was the finalist competing against the single-tower suspension design.

The tower, which gradually tapers towards its top, is comprised of two concrete shafts with an elliptical cross section. The two shafts are joined together by a rhythm of articulated transverse link beams that perform an essential seismic function and express the architectural form of the tower.

The design includes a symmetrical configuration with two planes of splayed cables from the central tower to the outside of the decks. This cable arrangement creates a spatial envelope and portal enclosure through which vehicular and pedestrian traffic passes.

The cables are arranged in a semi-fan configuration. The deck is shown as an orthotropic steel box girder throughout the main span. The main span length is 275 m with a back span of 215 m.

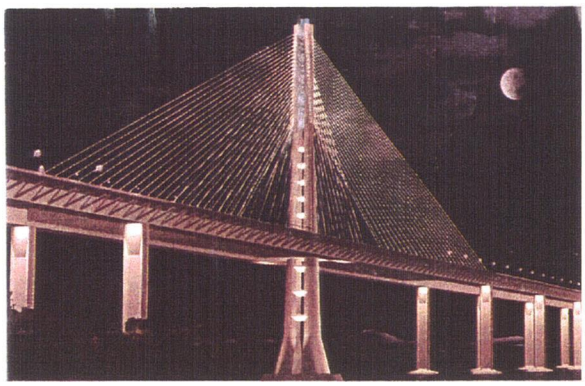


Figure 11 – Single Tower Cable-Stayed Design

SIGNATURE SPAN SELECTED

EDAP evaluated the four signature span designs described above and narrowed down the selection to two finalists, namely the single tower cable-stayed and the single tower self-anchored suspension span. See figure 12. At its May 28, 1998 meeting, EDAP made the final bridge selection; the single-tower, self-anchored, asymmetrical suspension span with a steel tower, and a steel orthotropic deck. The Panel also selected the segmental concrete haunched girder with spans of 160 m for the skyway. In addition, the Panel recommended that a single pedestrian and bicycle path be built on the south side of the EB structure and the bridge should be designed to carry LRT in the future.

The selection of the suspension signature span was made primarily on aesthetics because of visual compatibility of this design with the existing suspension bridges in the Bay Area. Seismic performance of the single-tower cable-stayed and self-anchored suspension was judged to be equal. The suspension span offered a more significant asymmetry as compared to the cable-stayed design. The larger span length, more asymmetry, and structural proportions similar to the west spans were judged by the Panel to be more in the tradition of other structures in the Bay Area. The estimated construction cost for the single-tower suspension signature spans at $370 million is about 20 percent higher than the single tower cable-stayed spans.

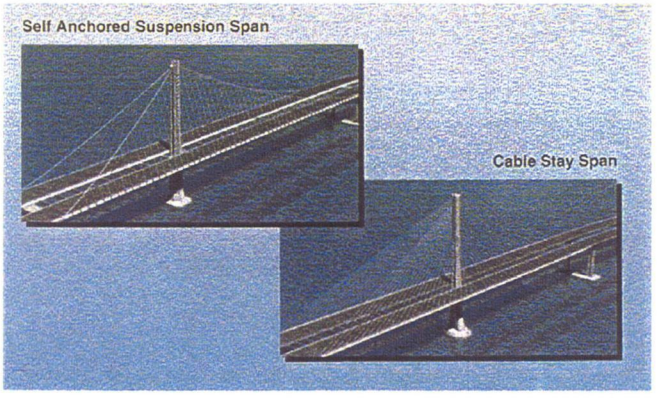


Figure 12 – Finalist: Single Tower Self-Anchored

BRIDGE DESCRIPTION

Suspension and Cable Stayed Spans

The self-anchored suspension bridge consists of a 385-m main span and a 180-m back span (Figure 13 and 14). As illustrated, the 0.75-m 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 16). 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 10 m.

The superstructure consists of dual ASTM Grade 50 orthotropic steel boxes (Figure 16). The steel box girder is designed for using an aerodynamic configuration for easy maintenance. All surfaces are smooth and easily painted. The compressive force in the deck of the self-anchored suspension bridge (15ksi) greatly reduces fatigue sensitivity. The wearing surface on the orthotropic plate will be 50-mm epoxy asphalt. Diaphragms spaced at 5 m support the orthotropic deck and distribute the suspender loads to the box. The 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 boxes act composite during wind and seismic loads.

CABLE SYSTEMS

MAIN CABLES

Cable wire will be 0.54-cm-diameter galvanized wire having a 1,550-Mpa minimum tensile strength. Soft wrapping wire will be used rather than a plastic cover to ensure that the cables remain tightly squeezed together. For erection, the cable is bunched into strands. Both prefabricated wire strands (PWS) and aerial spinning wire strands in the cable will be permitted for construction.

SUSPENDERS

The suspenders will loop over split casting strand shoes and squeeze over the cable by bolts. The ropes are constructed of six 37-wire strands around an independent wire rope center. The suspenders will resist fatigue, and have a rough surface to preclude rain-induced vibration. They will be installed with neoprene collar lateral restraints a short distance above the socket.

The east and west piers were designed to form plastic hinges at loads that are safely below those at which the piles either (a) form plastic hinges, (b) reach their axial capacities, or (c) exceed the strengths of the pilecap connections.

TOWER FOUNDATION

The tower foundation uses a single pilecap located at water level. It is shaped to support the central tower(s) and to encase the tops of the supporting piles. At this location the water depth is about 20 m (MSL). The rock surface is at an average depth of about 10 m below the mud surface and it slopes downward in a generally easterly direction. The rock surface is to be excavated to a level surface at a depth of 36 m to provide for a rock bench for uniform and seismic response pile lengths. This provision avoids undesirable torsional response of the tower foundation to earthquake shaking.

The supporting piles consist of reinforced concrete cast in a 2,500-mm-diameter, 60-mm-thick, tubular steel shell that extends from the pilecap to 5 m below the rock surface. This embedment provides moment fixity for the pile. A reinforced concrete pile extends a further 17 m into sockets drilled into the underlying Franciscan bedrock.

The piles are vertical. This simplifies their construction and provides lateral flexibility for the tower. The steel shell is fitted with a welded shear key system within the pilecap, which provides good bond strength between concrete and the otherwise smooth steel.

AESTHETIC LIGHTING SUSPENSION BRIDGE

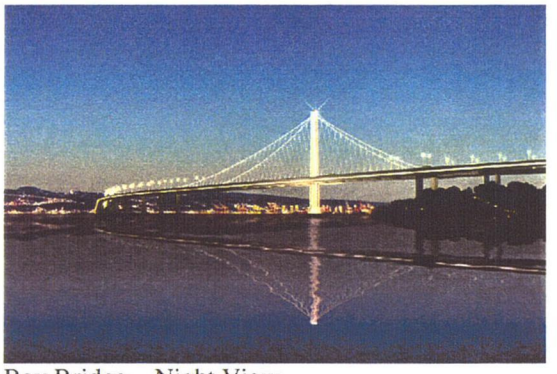
Aesthetic lighting of the bridge is an important consideration. The Golden Gate Bridge and the west span of the Bay Bridge have aesthetic lighting, in the form of "lace lighting."

Lighting schemes for both of the cable-supported alternatives were designed to enhance the structures as well as provide utilitarian lighting.

Lighting design for the suspension bridge alternative reinforces the practical and aesthetic objectives for the new bridge. It uses state-of-the-art lighting equipment, illuminating the bridge in an exciting aesthetic as well as energy efficient manner (See Figure 18). Features of the system include:

- The suspension towers are lighted from the base to the top in a graded wash that increases in brightness with the height.
- The tower is lighted as a gateway
- The suspender cables will be lighted from their attachment at the girder and from the main suspension cable.
- The roadway at the girder of the viaduct will be lighted from medium height mast poles located between the two decks.
- A pipe lighting system at the girder for the entire bridge length that will reflect the image of the bridge on the waters of the Bay.

The main objective of the lighting design was to provide an uncomplicated yet elegant revelation of the structure that would fit in with the other bridges on the Bay.



Bay Bridge – Night View

SKYWAY ALTERNATIVES

STRUCTURAL DESIGN CONCEPT—Three structural alternatives were considered for the skyway. Two alternatives in concrete box girders and one in steel box girder. The three options are summarized below:

Type	Girder Depth	Max. span (m)
Haunched Concrete Box Girder	Variable (5.50m to 9.00m)	160
Uniform Depth Concrete Box Girder	Constant (5.00m)	120
Uniform Depth Steel Box Girder	Constant (6m)	160

Parametric studies determined an optimum span length and girder depth. For the concrete haunched girder, the optimum span length was found to be 160 m with a depth of 5.50 m at midspan and 9.00 m at the pier. For a constant depth concrete girder, the optimum span length was about 120 m with a depth of 5.00 m. For the steel alternative, parametric studies indicate that the optimum

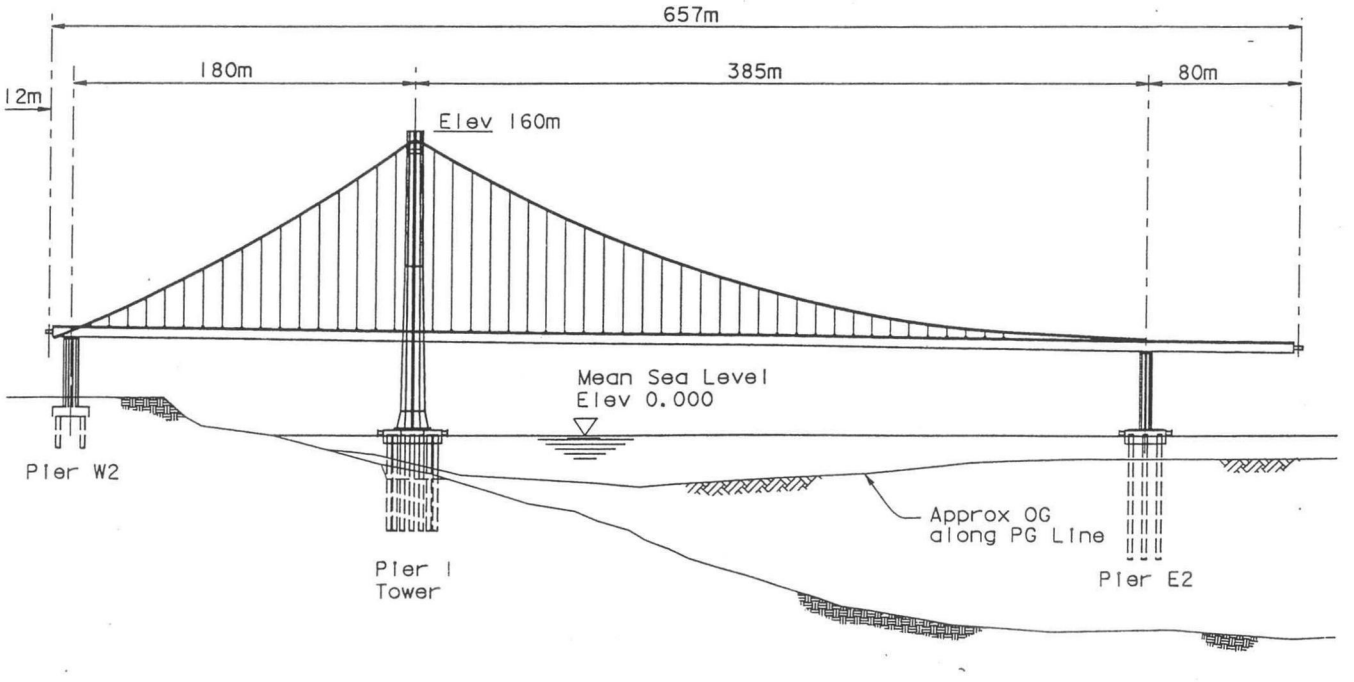


Figure 13 – Elevation of Self-Anchored Suspension Span

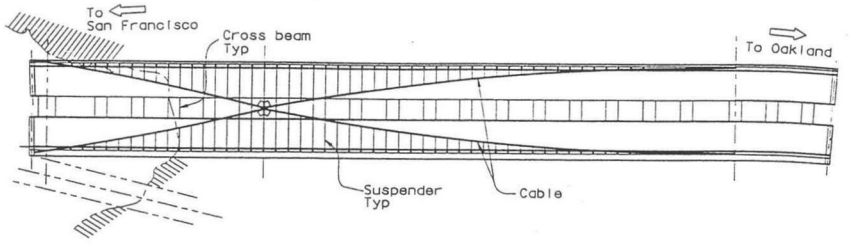


Figure 14 – Plan of Self-Anchored Suspension Span

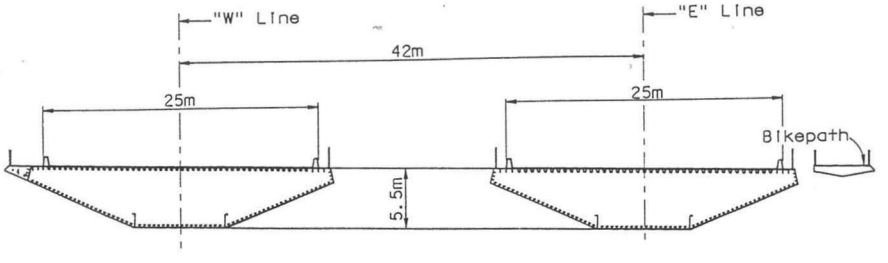
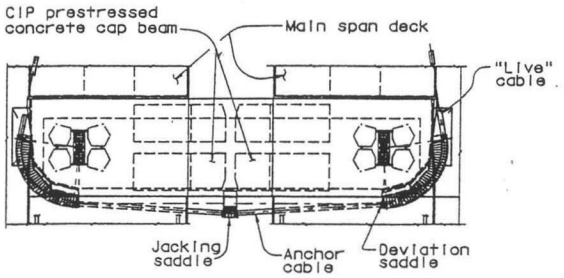
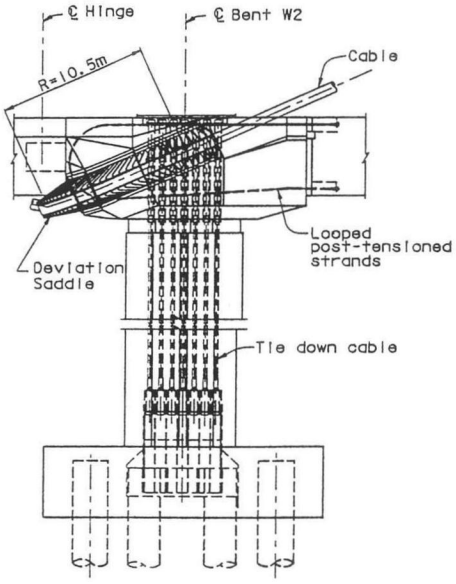


Figure 15 – Typical Cross Section Bridge – Dual Box Girder connected with Cross-Beams



PLAN



ELEVATION

Figure 16 – West Loop Cable Anchorage

SUPPORT CONDITIONS

The support conditions and interconnections of the various bridge parts are governed by seismic considerations. There are no direct connections between the tower and the deck during normal operation or during the SEE events although the deck will react (without impact) against the tower during earthquakes greater than the SEE. The deck is restrained to both the east and west piers for vertical, longitudinal, and transverse loads, and it cantilevers past the East Pier a short distance to provide girder and anchorage length for the cable and to join the skyway and YBI transition structure.

SINGLE STEEL TOWER

The tower is visually and functionally striking. It is made of four slender, non-rectangular, cross section tubes tied together by link beams. The link beams, which are the basis of the tower's excellent seismic behavior, provide damping. This damping absorbs energy while the main tower legs remain elastic and undamaged during the design earthquake.

TOWER DESIGN

The main tower will be 161.5 m high, which is the maximum permitted height (See Figure 17). The shafts will be made of ASTM A709 Grade 50 steel to reduce weight and provide high ductility. Tower shafts and shear links are stiffened to ensure that full plastification occurs with no buckling in the shafts.

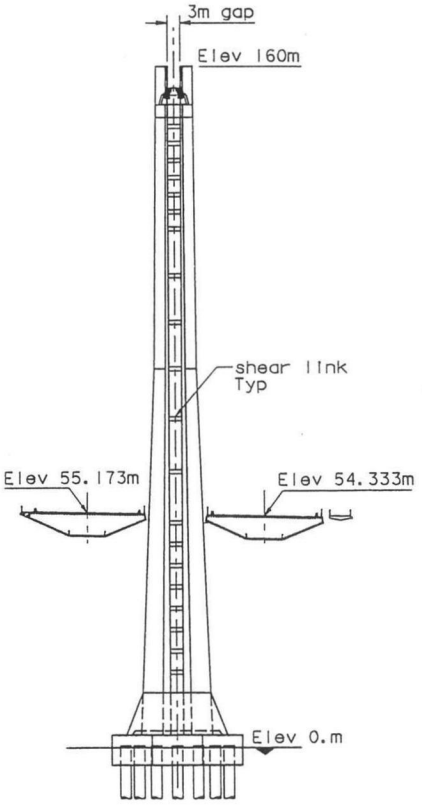


Figure 17 – Tower Elevation

MAIN SPAN SUBSTRUCTURE

The foundations are designed for loads which result from the bridge foundation system's earthquake response, dead and live loads, and the loads resulting from accidental vessel impact.

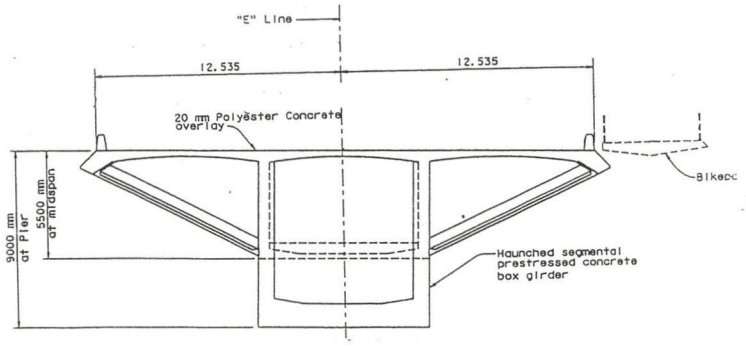


Figure 19 – Skyway Cross Section

depth was 6.00 m with a 160-m span. Figure 19 shows a cross section of the skyway with the haunched concrete girder, which was the selected design.

Spans closer to the Oakland shoreline become progressively shorter in length with reduced girder depth down to 3.00 m.

For the haunched concrete girder alternative, either balanced cantilever cast-in-place (CIP) or precast segmental construction erection were evaluated and selected. For the steel box girder, heavy lift erection was the preferred method.

HAUNCHED BOX GIRDER

Normal weight concrete will be used for the superstructure except for the side inclined panel or webs where lightweight concrete will be used. The 28-day concrete strength for both the lightweight and normal weight concrete is 50 Mpa (about 6,500 psi). The box girder is a single cell, with two vertical main webs. The width at the bottom slab (soffit) is 8.5 m, a dimension that is optimum from the seismic standpoint of pier column design. The 25.1-m width accommodates long deck overhangs of 8.3 m each. Inclined girder panels or webs were considered the most effective design to carry eccentric bike path loading, post-tensioned in both the longitudinal and transverse directions. Vertical post tnsioning 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. Midspan hinges between the frames allow longitudinal expansion and contraction due to creep, shrinkage, and temperature changes. An internal steel beam assembly at the hinge provides shear transfer and moment resistance in addition to controlling deflections at the cantilever end of each frame. These beams are rigidly connected to the box girder at one end and slides on bearings at the box girder at other end of the cantilevent.

STEEL BOX GIRDERS

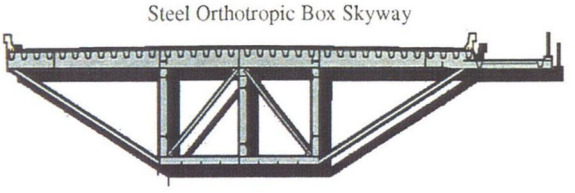


Figure 20 – Steel Orthotropic Box

The steel girder alternative competing against the haunched concrete was a welded steel orthotropic box girder (See Figure 20). Both a single box and twin box options were considered. ASTM A709 Grade 345 (F_y = 50 ksi) steel was used in the bottom flange and web plates of the central span, and Grade 485 (F_y = 70 ksi) was used over the pier locations where negative moments induce large compressive stresses in the bottom flange.

The steel skyway girder consists of continuous span units, with span lengths of 160 m each. A 130-m end span

transitions the structure depth from 6 to 4 m as the skyway approaches the Oakland touchdown. A five-span continuous unit, with span lengths of 88 m each, completes the remainder of the steel skyway viaduct.

Parametric studies were performed to determine (a) an optimum box girder depth, (b) benefits of a haunched steel girder, and (c) benefits of using Grade 485 steel for the full length of the girder. Results show that 6 m constant depth was the optimum steel girder depth.

The effect of using Grade 485 steel for the full length (instead of Grade 345 in positive moment regions) with 6.00-meter constant depth girders, reflected a decrease in cost of about 3 percent.

FOUNDATIONS

Seismic loading and drift control were the most significant criterion controlling the design of skyway foundations. Any damage to the structure from a seismic event should be limited to accessible areas; the damage below the water line should be minimal. Therefore, the foundations are designed to respond essentially elastically at the safety evaluation earthquakes. The potential for permanent offset at the pilecap level after a seismic event was fully evaluated. In order to meet serviceability requirements, the permanent offset of the pilecap was limited to about 0.15 m.

A further critical loading condition is the potential for ship impact. The foundations near the main shipping channel (east of main channel) are designed to withstand impact from a ship or a powered barge. All remaining skyway foundations will be able to resist the impact from a drifting barge with minimal damage.

The foundation and piers are designed to resist loads due to elastic and plastic deformations of the superstructure; these include 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 found best suited for skyway foundations because of their strength and ductility as well as the potential for ease of installation. These piles will be driven to the lower Alameda Formation and are 9 to 100 m in length.

Several different types of analyses were performed to select the appropriate pile diameter and thickness and penetration depth:

- Linear elastic response spectrum analysis
- Pushover analyses
- Inelastic time history analysis

The piers in Frame 1 of the skyway are about 50 m in height and stand in water depths of 15 m. This, added to the presence of about 15 m of young bay mud, establishes a very flexible structure. The design approach for such piers was to adopt a stiff foundation system thereby controlling pilecap elastic displacements to acceptable levels, and hence 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 steel piles filled with concrete battered piles.

Foundation Types on Skyway

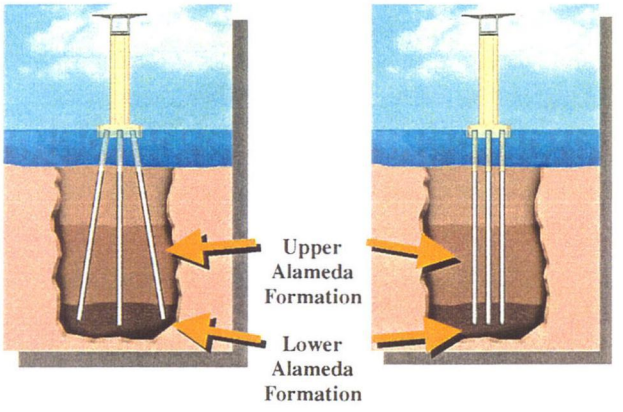


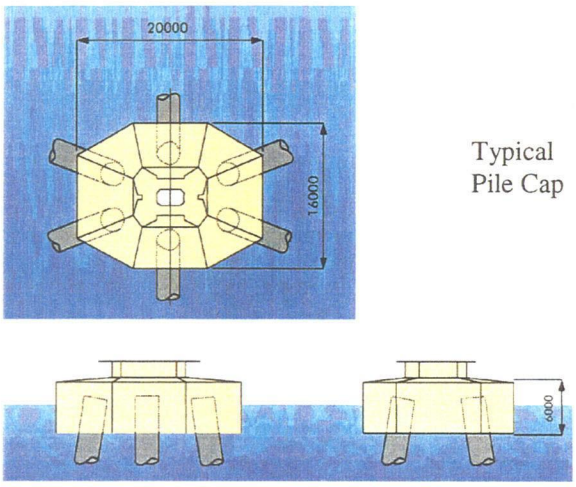
Figure 21 – Foundation Types Investigated

In contrast to the tall flexible western skyway piers, the eastern piers have heights as short as 10 m in water depths of only a couple of meters and hence are very stiff. In order to keep the inelastic response limited to piers, the frames were made shorter and the piers were made taller by lowering the pile caps.

Pile shell thickness was chosen based on required flexural capacity of the pile, ductility capacity, corrosion allowances, and driveability. 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 shell thickness (40 mm minimum).

The 2.5-m piles will be filled only in the upper 2/3 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 shell below the composite pile section. The pile shell thickness varies along the length of the pile from 70 mm to 40 mm. To ensure proper transfer of loads from the concrete fill to the steel shell, shear ring plates will be welded to the inside of the steel shell in the concrete filled section.

The pile cap will be approximately 6 m deep with 2 m exposed above the mean sea level. The pile cap structural system will be a moment resisting steel frame interconnected together with the steel piles and the pier reinforcement.



Typical Pile Cap

Figure 22 – Typical Pile Cap

YBI TRANSITION STRUCTURE

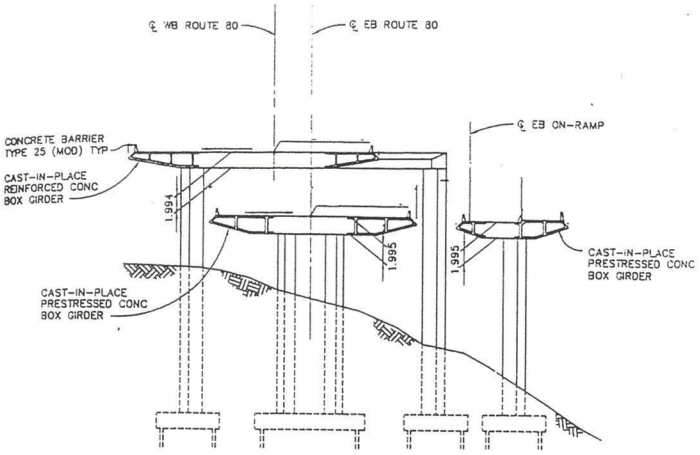


Figure 23 – YBI Transition Structure Cross Section

The transition structure 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 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.

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 span hinge support. The height of both structures varies from 8 m at the viaduct end to 46 m above-grade at the main span end.

Most bents are located in over-burden sand and are supported on cast-in-drilled-hole piles.

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.

OAKLAND SHORE APPROACH

The Oakland shore approach westbound structure extends from the tip of the Oakland Mole to the point where the proposed WB Route 80 joins with the existing Route 80, connecting the Oakland shore to the skyway. The total length of the westbound approach structure is about 660 m, which is subdivided into two sections, an elevated structure and an essentially at-grade structure. The eastbound approach structure is an elevated two-span frame 105 m long. The EB 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 pipe piles. For the WB at-grade structure, the flat-slab-on-piles option was selected.

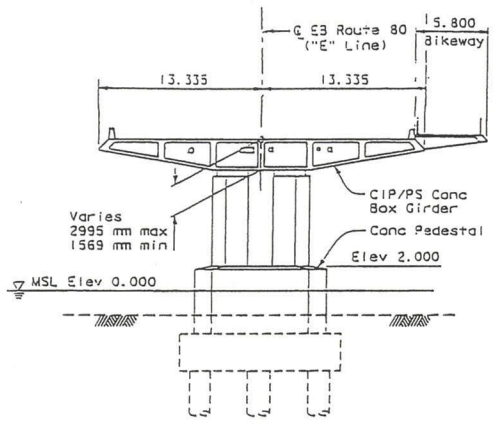


Figure 24 – Oakland Approach Structure Cross Section

DETOUR STRUCTURES AT YERBA BUENA ISLAND

Three temporary detour options are considered for the detour structures:

- North/north option
- North/south option
- South/south option

The length of each detour (westbound or eastbound) is approximately 500 m. The overall width is 20.1 m and the height of the deck from the ground varies from zero to 55 m.

The first two options use single-level decks for the eastbound and westbound detours; the third uses a double-deck detour structure. All options use tubular steel towers supporting a structural steel deck system.
The north/south option was selected for final design. The eastbound detour structure merges into the existing bridge at the lower deck.

BRIDGE DEMOLITION

The demolition will be accomplished from the south side of the existing bridge. The steel superstructure will be removed in large sections onto barges. Removal of Pier E-1 will be done by conventional demolition methods on land. Removal of Piers E-2, E-3 and E-5 will require a more complex demolition operation. The substructure will be removed to about 1 foot below the mudline, using underwater demolition equipment and/or cofferdams. The superstructure removal will essentially reverse the erection process used when it was built.

Spans from Pier E-4 to E-9 are simple span double-deck, steel trusses supported on concrete footings. The caisson at Pier E-5 is supported on a large diameter caisson while Piers E-6 through E-9 are supported on timber piles. Removal of the superstructure is accomplished by either floating out the entire span or disassembling the spans onto working barges. Removal of the piers to 1 foot below mudline can be done by underwater demolition methods or by cofferdam. The timber piles will be cut off and not extracted from ground. Spans between Piers E-10 through E23 are double-deck steel truss supported on concrete piers with timber piles. Removal of the superstructure will likely be done span by span or dissembled in parts onto working barges, using special shallow water barges and/or dredging near the Oakland shore.

AERODYNAMIC ANALYSIS

A 50-year recurrence basic design wind speed of 31 m/s was used for the site of the Bay Bridge east span.

Furthermore, our wind experts recommended that a 100-year recurrence wind speed is used for the service load design, and a 10,000-year recurrence wind speed is used for a lower end, critical flutter velocity threshold.

Recurrence Period (yrs)	U60 (mph)	U600 (mph)	U3600 (mph)
100	107	98	94
10,000	153	140	134

Where:
U60 60-second (1-minute) average wind speed
U600 600-second (10-minute) average wind speed
U3600 3600-second (1-hour) average wind speed.

For static equivalent pressures for the preliminary service load design, 100-year recurrence 1-hour average wind speed is recommended. Also, an approximate gust response factor of 1.90 and drag coefficients (Cd = 1.6 for towers and Cd = 1.2 for the bridge decks) are recommended.

For aero-elastic flutter, the 10,000 year recurrence, 10-minute averaged wind speed at the bridge deck (z = 60 m) is 62.5 m/s mph (as determined above) and is recommended for design.

WIND TUNNEL TESTS-Wind tunnel tests were performed on a 1:50 scale model.

OTHER STUDIES

Additional studies were performed including:

- Corrosion protection
- Pile drivebility study
- Future light rail
- Ship collision study
- Scour analysis

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