Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 775

SEISMIC DESIGN/CONSTRUCTION OF LIFELINE BOX GIRDER BRIDGE – SAN FRANCISCO BAY BRIDGE APPROACH SPAN REPLACEMENT

Xiaoyun Wu[1], Hasan El-Natur[2], Mohan Char[3]

ABSTRACT

San Francisco Bay Bridge is an emergency lifeline structure in the San Francisco Bay Area transportation system. The eastern span was damaged in the 1989 Loma Prieta earthquake, which resulted in a decision to replace the 60-year-old existing steel structure. The replacement project includes a transition trestle structure, a self-anchored suspension main span, a skyway segmental structure and the Oakland Touchdown Approach box girder bridge. State-of-the-art seismic engineering was used to design the new bridge with minimal environmental impact.

The box girder approach structure represents the most commonly used bridge structure type in the California Highway System and it is also popularly used worldwide. There are several papers and reports available regarding the Bay Bridge project, however this article will particularly focus on the box girder approach spans and discuss analysis, design, detailing, and construction coordination issues in more detail to share the experience and findings for the benefit of the future lifeline bridge projects.

Introduction

The San Francisco-Oakland Bay Bridge (SFOBB) is one of the emergency lifeline structures in the San Francisco Bay Area. It was decided that the eastern span of this bridge would be replaced after it partially collapsed during Loma Prieta Earthquake in 1989. The replacement project includes a pair of parallel transition trestle structure starting out of the Yerba Buena Island, followed by a self-anchored suspension main span, a skyway segmental structure and the Oakland Touchdown Box Girder Structures. The new East Span will include two parallel five-lane roadways, providing motorists with a sweeping view of the Bay Area, and will be constructed alongside the old span to minimize traffic interruptions. The design also includes provisions for future inclusion of rail. A bicycle/pedestrian path will provide additional access and recreational opportunities. The new lighting will transform the night panoramic view of the Bay Area with bridge as the primary subject.

[1]PhD, PE, Principal, IDC Consulting Engineers, Inc, Anaheim, CA
[2]PhD, PE, Senior Bridge Engineer, California Department of Transportation, Sacramento, CA
[3]PhD, PE, Vice President, AECOM, Lake Forest, CA

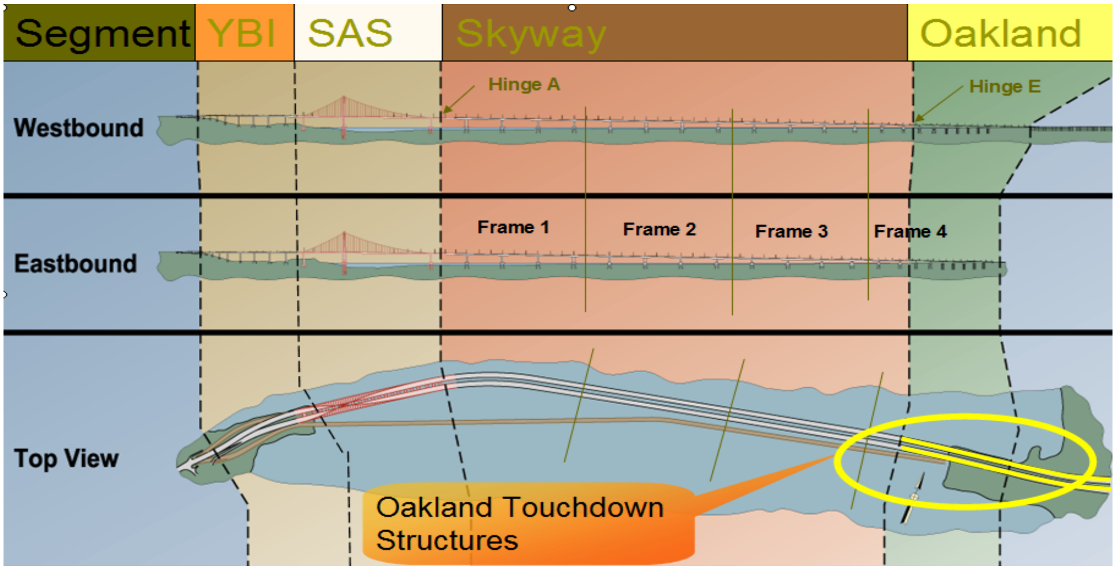


Figure 1 Segments of the San Francisco-Oakland Bay Bridge East Span Replacement Project

Oakland Touchdown (OTD) structures are the gateway to Oakland and are at the very end of the East Span of SFOBB. It consists of a pair of parallel west bound and east bound approach structures. Both the structures measure approximately 330m in length and 27m in width. The structures consist of seven (7) spans with two hinges Hinge E and Hinge F. At Hinge E, the tie-in between segmental skyway structure and cast-in-place (CIP) approach structure occurs. The Hinge F separates marine and dry land frames. The eastbound structure has a bike path cantilevering approximately 6m out on the south side. The superstructure consists of uniquely shaped CIP post-tensioned prestressed concrete box girder with sloping soffits and is supported on substructure piers. The rigorous seismic design criteria for lifeline structure and other features pertinent to this signature bridge impose special challenges to the project teams. Since the box girder structure represents one of the most commonly used bridge structure type in the California Highway System as well as worldwide, this article will discuss the analysis, design, detailing, and construction issues from the OTD project and share the experience and findings.

Seismic Design Criteria

For seismic design of the standard bridges Caltrans uses rock motion from Maximum Credible Earthquake (MCE). The MCE is estimated by a deterministic method without considering the earthquake return period. Under MCE the standard structure is designed to have non-collapse criteria. As a Caltrans designated emergency lifeline bridge San Francisco-Oakland Bay Bridge needs to provide post-quake transportation service for emergency response and to provide support for the region's economic development after a quake. Therefore it is critical for the bridge to remain functional following an earthquake. A two levels design approach was used for the East Span Replacement Project. Probabilistic Seismic Hazard Analysis method was used to evaluate the basic seismic rock motions for the site. The 1500-year return period and 92-year return period were adopted for the Safety Evaluation Earthquake (SEE) and the Functional Evaluation Earthquake (FEE) respectively. The dominant seismic sources are found to be San Andreas Fault and Hayward Fault which essentially straddles the bridge in-between. Total six sets time histories of ground motions and ARS design curves were derived for the seismic analysis of the bridge. The seismic excitations include fault perpendicular, fault parallel and vertical components.

The bridge was designed to provide a high seismic performance under both an SEE and FEE. When subject to an FEE, the bridge will be able to provide full service without interruption and structure response will be essentially elastic. The bridge's damage under an FEE is limited to minor cracking in concrete and no apparent permanent deformation. However damage to expansion joints is allowed. After an SEE, the bridge will be able to provide full service immediately, with only repairable damage to the structure. Repairable damage is defined as:

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

For more detail seismic hazard evaluation & criteria, see Ref.1

Box Girder Approach Span Analysis

Except the strict seismic design criteria for the lifeline bridges the OTD approach structures have the following features (referring to Fig.2 and Fig.3):

- Depth of the CIP concrete superstructure box girder varies in both longitudinal and transverse directions due to aesthetic design. The superstructure depth starts at 2.5m at Hinge E and gradually tapers down to 1.89m at Abutment E23. The architectural enhancement required sloping soffit towards edge of deck on either side which created a situation of progressively shallower girders on the outside.
- OTD CIP post-tensioned P/S box girder structure connecting to the Skyway Bridge with a mid span hinge (Hinge-E) due to balance requirement of the skyway segmental construction. Hinge-E is consists of two 1.0m diameter steel pipe beams bearing on two diaphragms on Skyway side and two diaphragms on OTD side.
- The OTD CIP box girder structure was constructed four year later after the Skyway segmental bridges were in place.
- Dumbbell shape substructure pier wall with two pentagon columns at both ends of the pier cross section. The faux precast closure wall is architectural and expected to crack or fail during FEE or SEE events, and it is easily replaceable.
- The embankment fills at the abutment use light weight concrete fill in lieu of traditional soil for easy construction and less lateral pressure.

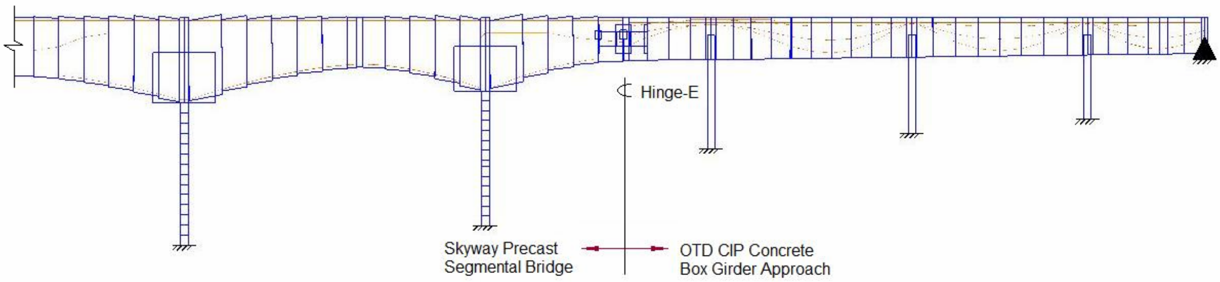


Figure 2 Computer Model & Girder Shape of Partial Skyway and OTD Frame-1

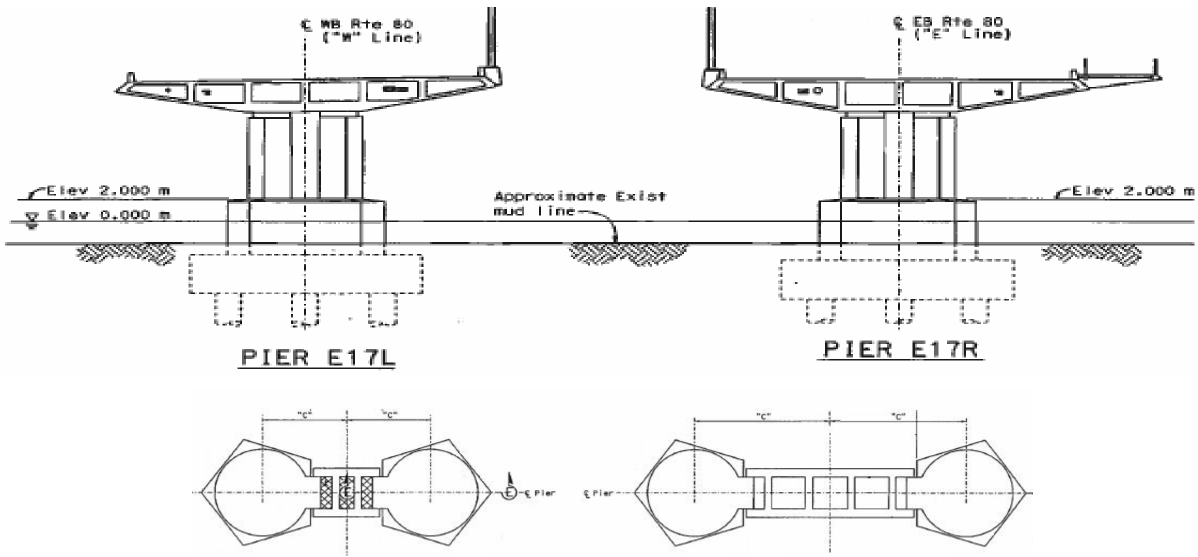


Figure 3 Typical OTD Bridge X-Section & Pier X-Section

Because of the uniquely shaped sloping soffit, both 3-D grillages computer analysis model and 3-D finite element with shell element analysis model were setup and explored to study the distribution of gravity load on to the girder webs. The study concluded that even though girder depth varies in transverse direction the plane section assumption is still valid for the structure. Therefore a 2-D model analysis is adequate for gravity load under service condition. The 3-D grillage model confirmed the need for intermediate diaphragms for girders' deflection compatibility.

To ensure the smooth installation of the mid span Hinge-E pipe beams and the adequate capacity of the pipe beam, it is critical to accurately predict the deflection of the OTD girder at Hinge-E for various days and stages. Due to the sensitivity of the deflection relative to the loading and construction sequences a stage by stage time dependent analysis is deemed to be necessary. By balancing the accuracy and simplicity of the analysis a 2-D stage by stage time dependent analysis computer model was chosen for the OTD CIP concrete box girder service load analysis and post tensioned P/S design. To increase the accuracy of the time dependent analysis a check analysis utilized the material properties from test data of field concrete mix was also performed during construction phase, the results were used to adjust the final camber values for the construction and to check capacities for the structures shown on the design plans. Because of the non-symmetric bike lane loading, a 3-D time dependent analysis was also carried out for the East Bound and twisting effect was incorporated in the predicted camber values at this stage.

For seismic analysis a 3-D global stick computer model was setup. Section ductility analysis was carried out for all the superstructure and substructure components to obtain the effective section properties for dynamic analysis. Yielding moment and curvature surfaces of substructure piers were derived for forces and deformation capacities calculation. Acceleration Response Spectrum (ARS) dynamic analysis was performed to obtain OTD structure global seismic responses. Nonlinear static pushover analysis was carried out to see the OTD structure nonlinear behavior and to obtain the structure ultimate displacement capacities. For pier pile group foundations, which each consist eight CISS (Cast in Steel Shell) piles of 1.8m diameter and a reinforced concrete pile cap of 3.5m depth, a 3-D computer model with nonlinear soil springs along the full pile length was set up and a nonlinear time history dynamic analysis was carried out. The nonlinear soil springs were defined by P-Y curves in lateral directions and T-Z curves in vertical direction. The nonlinear time history analysis revealed that there is a big difference between the foundation responses of large diameter pile group in bay mud and small diameter pile group in competent soil. Pile bending stiffness plays an important role in the response of the former type foundation.

Besides global analysis many finite element model analysis were also carried out at the local zones for the design and detailing of P/S anchorage zones, bent caps, hinge diaphragms, column pinned keys, utility/access openings and large diameter steel pipe pile head anchorage zones etc.

Design Challenges/Solutions and Details

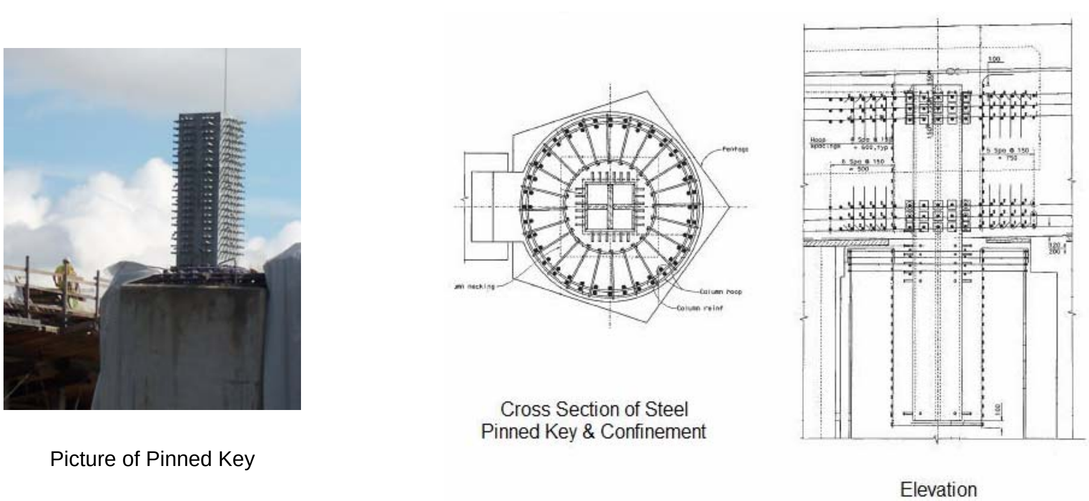
OTD Frame-1 concrete box girder has a 28m long cantilever which connected to Skyway. The long cantilever action creates very large demanding moment in the box girder section at the supporting end. Because of the bridge superstructure aesthetical stream line shape design the depth of the box girder can not be increased for structure capacities purpose. After numerical analysis/design iterations several measurements were adopted to achieve the required capacities and to maintain desired girder depth:

- Except P/S tendons running in the girder webs deck pre-stressing paths were provided for additional bending capacity. P/S jacking forces distribution among the tendons and tendon path profiles were optimized to achieve the balance between requirements for stresses at top of the section before P/S losses and at the bottom of the section under full service load condition.
- Higher concrete compressive strength (f’c=8.5ksi) was used to provide compatible compressive capacity for matching the large volume of the reinforcement. Soffit slab closed to supporting pier area was thickened to reduce the demanding stresses.
- Skyway and OTD Connection, Hinge-E, was designed to carry live load moment under service condition so that to reduce the demanding moment in supporting section at pier.

To limit the earthquake damage within certain pre-defined locations, Hinge-E steel pipe beam was designed to have a fuse in the mid segment. The fuse was achieved by designing the wall thickness of the fuse segment to be smaller than rest of the beam and also using a lower strength steel for the segment.

Fig. 3 showed a typical substructure pier of the OTD approach bridge. It is seen that bent cap is supported by two closely spaced columns and resulted in two long cantilever bent cap portions. This architecture configuration plus relatively shallow depth of the bent cap ends up a very congested bent cap reinforcement design. To maximize the bent cap moment capacity with limited section depth, mild steel as well as post-tensioned P/S tendons were used in the bent cap design.

In order to improve substructure's ductility, pier wall was designed to consist of two well confined end columns with architecture panel in between. In the original design the OTD superstructure box girder was supported by substructure piers with monolithic connection between core columns and bent cap of which is embedded inside of the box girder. According to the seismic FEE & SEE criteria for the lifeline bridge, except high ductility capacities the columns shall also possess large flexural capacity. The over strength plastic moment of the end columns is much larger than the maximum nominal moment capacity of the bent cap, therefore the design of the column end connections were changed to be pinned connection at the top and fixed connection at the bottom. This change released moment connection problem for bent cap and box girder, however it did not reduce the column over strength plastic shears. To transfer such a big shear force between superstructure and columns the pinned key area became so big that it generated a large bending moment which could not be ignored. This additional moment resulted in increased additional shear force which made it impossible for a conventional reinforced concrete (R.C) pinned key to handle these large demand forces. To reduce the pinned key size, a built-up box shape structure steel key was designed (Fig.4). By changing the R.C pinned key to structure steel key the column over strength plastic shear transfer capacity is satisfied and the high reinforcement demanding area is moved from interface section of the superstructure and substructure into the superstructure and top of the column portions where more concrete volumes can be utilized.



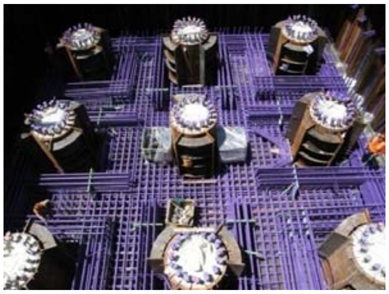
Picture of Pinned Key

Cross Section of Steel Pinned Key & Confinement

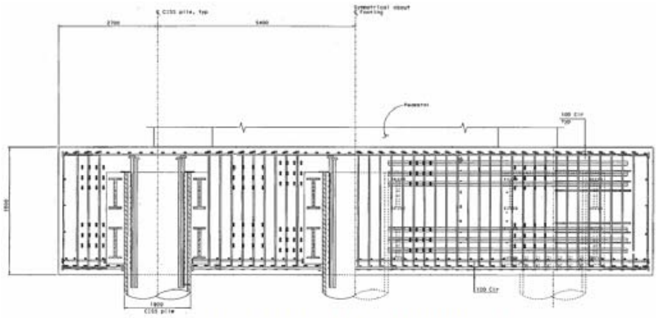
Elevation

Figure 4   Detail of Build-Up Structure Steel Pinned Key at Top of the Column

Fig.4 shows the column top pinned key details and reinforcement in superstructure and top of the column segment for the anchorage of the structure steel pinned key.



CISS Pile Head



Cross Section of Typical Pile Cap

Figure 5 Detail of CISS Pile Head and Pile Cap

Since well confined R.C. columns have relatively high ratio of longitudinal reinforcement due to the strength requirement and deformation control under FEE and SEE criteria, the demanding forces to the pier foundation is much larger than in a standard bridge. This high demand from the columns and low capacity of the deep bay mud combined with the rigorous seismic criteria resulted in a large deep foundation design. A typical pier foundation consists of nine 1.8m diameter CISS pile. The steel shell has a wall thickness of 65mm in up pile portion. To increase the foundation stiffness these large diameter CISS piles were embedded into the 3.5m depth R.C. pile cap and designed to have a fixed head condition. The concern of integrating the large structure steel component with reinforced concrete leaded the design team to perform a finite element model analysis for the pile head anchorage zone. The final design chose to weld large steel shear ring assemblies to the pile head to maximize the interface area with reinforced concrete. Fig.5 shows the design details and a picture of the pile head before pile cap was cast. Another major issue was the design of exterior pile head connection. Due to large demand forces involved, the bursting of the free edge had to be prevented. This was accomplished by providing a combination of loop bars and headed bars in the pile head region. A free body diagram of the forces involved is shown in Fig.6.

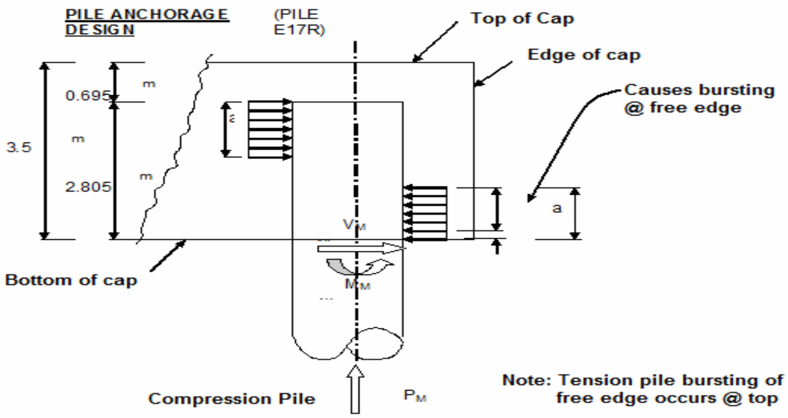


Figure 6 Pile Head Anchorage Design

Construction Coordination

Coordination on this part of the project is extremely important and includes many areas that may affect schedule and/or budget, most important aspects are listed below:

- Design-Construction coordination and oversight: This required continuous and intense coordination and communication among various parties including design, internal functional units and approving authorities within the department. Continuous daily design presence on site with construction for quick resolution of issues was necessary. This is particularly true for Hinge-E camber analysis and hinge pipe beam engagement installation.
- Seismic safety peer review panel: This panel is composed of world class, highly qualified experts in the field of earthquake engineering, design and construction. This panel reviews, evaluates and recommends on all important issues/elements pertinent to design and construction issues including but not limited to, bridge type selection, design loadings including seismic, and design criteria. Analysis models/method utilized for deep foundations of the Approach Structure was also reviewed by the panel before final analysis/design taking place.
- Integrated shop drawings (ISD's): Due to the high seismic demands, congestion areas of reinforcement and utility placements were present. It was necessary to produce ISD's to produce conflict free environment for construction.
- Partnering: Regularly scheduled monthly partnering meetings were scheduled with the contractor. These meetings were intended to build and maintain relationships based on trust, integrity, mutual respect and fair dealing through the accomplishment of open communication, shared goals and dispute resolution in a timely manner.
- Risk management: This is an important and critical element of project progress to ensure that construction is progressing on schedule and within budget. This process reflects a methodical approach of planning for, identifying, analyzing, responding to, and monitoring project risks. The goal is to help the project manager efficiently complete his project while mitigating risks that might impede that success.

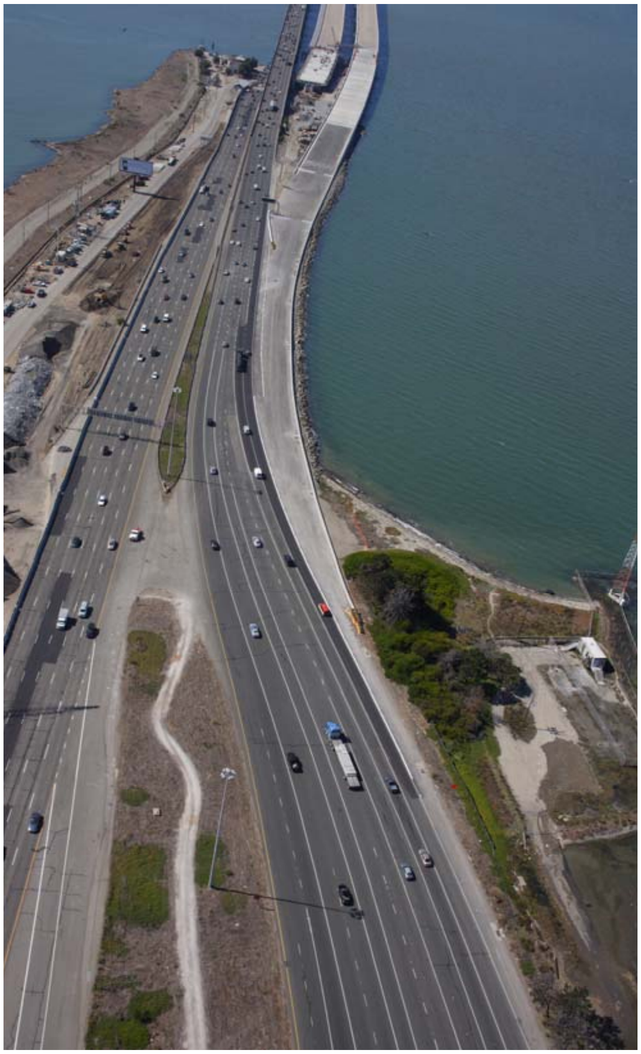
Column Cage Installation

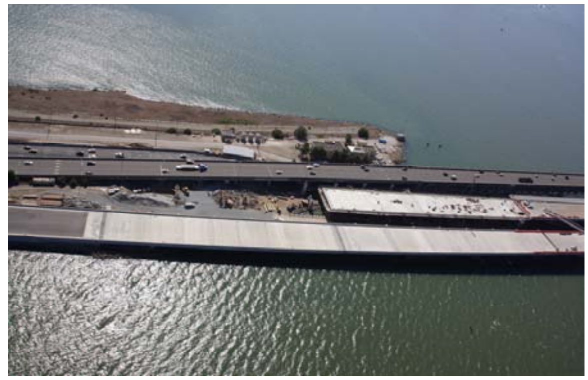
CISS Pile Installation

Rebar Work for Girder Webs

Reinforcement for Bent Cap

Figure 7 Construction of SFOBB East Span Approach Structure





Segment near East Abutment

Finished East Bound Deck with Bike Lane

Figure 8 SFOBB East Approach: Completed West Bound and In Construction East Bound

Conclusion

This paper presents the overall picture of the Oakland Touchdown from design phase through construction phase with emphasis on seismic engineering for lifeline structure. During the design and construction phase certain critical challenges were encountered and those challenges were resolved through designers' ingenuity, extensive coordination amongst the team which included the stakeholders and the contractor.

The paper comprehensively addresses design considerations for this lifeline box girder bridge and its key design elements. 1) Superstructure: The uniquely shaped sloping soffit superstructure required special consideration in the distribution of the prestressing forces among the girders. 2) Precast Segmental Skyway and Cast-In-Place Box Girder Interface: The two structures being significantly of different type and that precast segmental skyway was constructed at least four years before presented various degrees of difficulties in the formulation of time-dependent model. The results of the model correlated very well with the findings in the field, this serves as a great testament to solving extremely complicated issues through cooperation amongst all the parties involved. 3) Bent-Cap/Column Interface Built-Up Pin: Through innovative design concept, bending moment and shear forces transferred to the shallow superstructure was considerably reduced. 4) Pile-Head Interface with pile cap: due to pin-top condition of the columns, the bottom of the columns at the footing had to have a fixed connection. This further required the pile-head connection to the footing be fixed as well to further stiffen the structure. This resulted in large demand forces (axial, bending moment, shear) transferred into the pile cap during a SEE Earthquake event. Special pile-head connection details and confinement reinforcing details were developed to handle these large forces.

In summary, with thoughtful analysis/design/detail, careful planning/scheduling and seamless coordination, a reliable lifeline structure with multi-functions can be completed with great quality. The design elements with modifications from this project could be adopted to projects of similar nature. This project also gives insight into how even the most complicated challenges can be resolved through cooperation amongst the team which includes both the owner and the contractor.

References

Ho, Tom, 2007. The Design and Construction of the New San Francisco-Oakland Bay Bridge (SFOBB) East Span, *23rd US – Japan Bridge Engineering Workshop*, Tsukuba Science City, Japan

Law, Hubert, P. Lam, B. Maroney, S. Moham, 2004. Development of Seismic Ground Motion Criteria for New San Francisco-Oakland Bay Bridge, *Geotrans Conference*, Los Angeles, USA

T.Y.Lin International/Moffatt & Nichol Joint Venture, 1999. Design Criteria of the San Francisco-Oakland Bay Bridge East Span Seismic Safety Project, *Design Criteria*, San Francisco, USA