California's First Curved Orthotropic Bridge

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Introduction

In October 1989, the Loma Prieta earthquake (magnitude 7.1 M) caused the upper roadway of the I-880 Cypress Street Viaduct in Oakland CA to collapse, resulting in 41 deaths and 108 injuries [1]. The viaduct was a two-level reinforced concrete structure completed in 1957, and carried about 150 000 vehicles per day with eight lanes of mixed-flow traffic.

The project to replace the collapsed viaduct was divided into seven contracts. The Maritime Off-Ramp is one of the eleven bridges that make up one of these contracts, and is located at the intersection of Interstate highways I-880 and I-80. It is a curved steel orthotropic bridge [2] providing access to the Port of Oakland via a U-turn from the westbound carriageway of I-80. The bridge, which entered service in July 1997, is also known as the Horseshoe (HS) Line because of its 75-m-radius horseshoe shape (*Fig. 1*).

Corrosion of Steel Pipe Piles

The new bridge foundations had been planned as steel pipe piles filled with concrete, similar to those of the old I-880 Viaduct. The new piles have however a much larger diameter (1.07 m outer diameter) than standard piles. Site borings revealed that corrosion protection should be considered, due to the presence of high levels of chlorides and sulfates, and low values of minimum soil resistivity.

Fig. 1: View of the Maritime Off-Ramp (Photo courtesy of Lynn Harrison)

After the Loma Prieta earthquake, the I-880 Viaduct was demolished to the top of the footing elevation and covered with soil. A portion of the viaduct that was not damaged was retained for destructive testing of new seismic design methods. Three of the intact piles were exhumed in order to examine their condition and thus determine an economical solution for corrosion protection of the new bridge piles. The exhumed piles were also subjected to seismic-tension uplift tests, and the results were compared with those calculated theoretically for the piles.

A series of holes were drilled alongside the piles to be exhumed (*Fig. 2*). These holes reduced the amount of surface friction when removing the piles, but very high suction forces were still present. The piles were completely removed, cut into 3-m segments and transported to the laboratory for detailed inspection. Ultrasound measurements were taken to determine the thickness of the piles and thus how much metal had been lost due to corrosion. The results of the corrosion testing and inspection are given in *Table 1*.

Protection of Steel Pipe Piles

Most of the steel pipe piles used for the Maritime Off-Ramp were within 300 m of salt water. Therefore, the wall thickness was increased by a minimum of 6.35 mm compared with the structural requirement for the pipes. Electrical devices were installed in piles within 150 m of salt water to allow future monitoring and/or cathodic protection. Assuming a constant rate of corrosion, the additional steel will resist corrosion for over 250 years, and the expected life span of the new piles therefore matches or exceeds that of the superstructure. The cost of the additional steel wall thickness was offset by the savings made due to the lack of expensive corrosion-inhibiting coatings for the foundations.

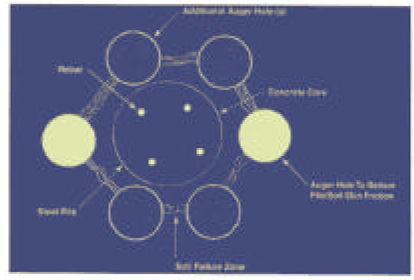

Fig. 2: Scheme for removal of piles of the old viaduct

Another advantage of increasing the wall thickness was that the foundation system was stronger than initially calculated. Furthermore, paint and other coatings have no structural strength, but may modify the soil coefficient of skin friction. The foundation system should be the most durable component of a bridge and should be designed to receive no damage in an earthquake, since it is not practicable to find and repair pile damage.

Geotechnical tests were performed and showed that the soil set up factor around the piles was significant. Therefore, a comprehensive study of pile loads was carried out from October 1993 to January 1994, prior to awarding the production contracts. The study program was intended to provide a better understanding of construction and design issues associated with the installation of large-diameter steel pipe piles. Various pile installation methods were tested, including the use of vibratory or impact hammers. The program reports were made available to all bidders for the project, which eliminated major pile-driving claims and resulted in a more-competitive bidding process.

Foundation and Column Design

Based on the results of a structural, geotechnical and economic optimization study, hexagonal foundations were designed with seven 1.07-m-diameter steel piles. The need for pile compliance at the interface between soft mud and the stiff underlay led to the use of steel piles. The deep overlying mud layer requires very long piles

Bent no.	Tip elevation (m)	Corrosion	Max. pit depth (mm)
91	14	shallow surface pitting from 0.3 m to 8.3 m below the pile top	0.483
75	16.9	no visible corrosion	0
61	4.1	minor surface pitting along the top 1.5 m (i.e. above the water table)	0.406

Table 1: Corrosion of three pile bents of the old viaduct

because of the high seismic loads. The hexagonal shape of the pile cap resulted from the isotropic nature of the seismic loading imparted by the soil-structure interaction with the plastic hinge load reaction at the pier base.

The bridge utilized reinforced concrete T-bents with a single column and spiral reinforcing ties. Two special bearings connect the superstructure to each T-bent (*Fig. 3*). New criteria for joint shear in reinforced concrete structures resulted in a higher level of reinforcement than in previous projects.

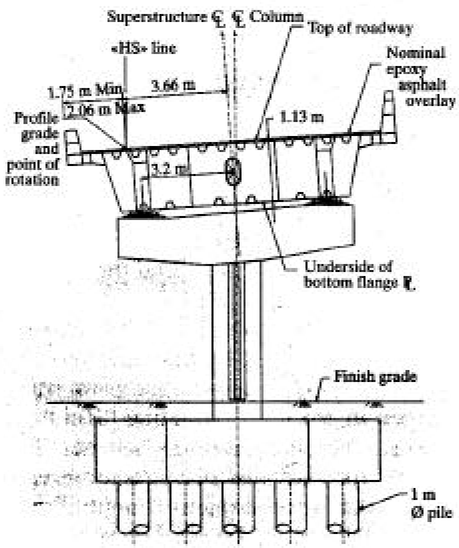

Fig. 3: Typical cross section of the Maritime Off-Ramp

Orthotropic Deck

The transverse flexural stiffness of the deck is critical for the long-term behavior of the asphalt overlay. The weight-saving considerations, which guided the seismic design, dictated a minimal overlay thickness. This led to a compliant deck system, which is very sensitive to temperature and to local deflections imposed by concentrated wheel loads. However, the cyclic nature of this loading only increases the problem of compliance between the overlay and the steel underlay.

The long-term risk of delamination of the overlay was greatly reduced by providing a uniform stiffening pattern with relatively closely spaced components and a relatively stiff top deck plate.

Fatigue

The stress amplitudes under global loads are very low for this type of multicell box girder. The longitudinal continuity and the 180° in-plane curve further minimize longitudinal tensile stresses. The torsional stress components are very low for cells with such large perimeters. The resultant equivalent stress intensity values (Von Mieses yield criterion) are thus very low.

The local deck stresses generated by localized dynamic wheel loading are more critical for fatigue, particularly at points of greater restraint and stiffness, such as at the intersections of the deck plating with the longitudinal webs and transverse diaphragms. Inclusion of the various stress risers and consideration of the weld types led to excellent long-term fatigue capacities.

Fracture Control Plan

The optimization of the safety and performance of the structure at minimal cost is the basic aim of a rational fracture control plan. While buckling and general yield are considered in the basic design approach, special consideration was given here to the danger of sub-critical crack growth and unstable crack extension. Special attention was given at the design and fabrication stages to the factors affecting these small fatigue-induced cracks.

The main elements of the fracture control plan are

- the identification of the main contributary factors, such as local loads, dynamic amplifications, stress risers, residual welding stresses, hydrostatic tensile stresses
- the establishment of the relative importance and contribution of each of these factors
- the determination of various strategies to mitigate the most important fracture-causing elements
- the recommendation of an optimal design and fabrication procedure, including choice of materials, quality control and inspection methodology.

The three primary factors that affect the local failure and ensuing damage (and hence the life span) of a structure undergoing fatigue loading are the tensile stress amplitude, the flaw size within material welds (i.e. the quality of fabrication and inspection), and the material toughness properties (i.e. the choice of steel type).

The design philosophy provides for minimal stress risers through a careful choice of details and weld types. Redundancy is used with caution to minimize global and local stress amplitudes and to provide multiple load paths. Crack arresters are considered in various fracture propagation scenarios. Local buckling of webs and flanges is considered in a limit-load analysis with given transverse imperfections. It is essential to include actual fabrication conditions that simulate imperfect geometry and welding residual stresses.

The web slenderness leads to shear lag effects, which need to be considered in the local fatigue and buckling capacity of the most critical web panels at midspan and near the supports.

Analysis

Various task-specific programs were used to simulate the static and dynamic behavior of the bridge structure. Dedicated pre- and post-processors allowed the designers to determine the governing load combinations for service, the ultimate conditions for service, and the ultimate conditions corresponding to the standard load and resistance factor design (LRFD) approach [2, 3].

The cross-sectional properties were derived from local sectional models. The use of sub-structuring by means of the super-element technique was contemplated but was subsequently abandoned because of the need for live loads applied to every span and the development of envelope curves for forces and displacements.

Special attention was given to the expansion joints and to the foundations. Upper and lower bounds for foundation impedances were considered. The pier columns were also considered under two distinct elastic states, namely uncracked and cracked. The seismic analysis consisted of the spectral response approach based on complete-quadratic-combination modal superposition. These linear analyses were performed for a site-specific mud-site spectrum corresponding to the maximum credible earthquakes occurring on the Hayward and San Andreas faults, with magnitudes of 8.0 M and 8.5 M, respectively.

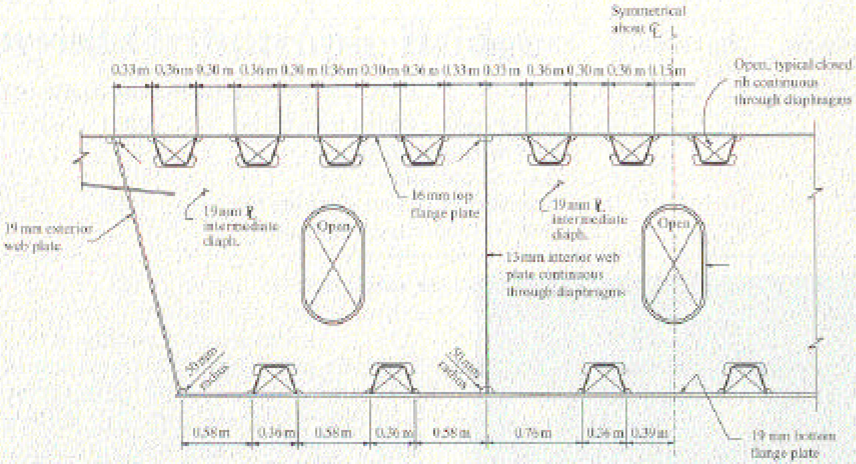

Fig. 4: Detail of a typical diaphragm

The seismic design philosophy was driven by a displacement-ductility approach, which permits controlled plastic straining to occur at the bases of the pier foundations. Soil-structure interactions were analyzed by means of a finite-difference program. The very deep Bay Mud layers were simulated under different material assumptions to obtain meaningful sensitivity curves and to verify the compliant pile behavior. Dedicated non-linear finite-element models were developed for local web-buckling and diaphragm-buckling analyses, and were used to study wheel loading on the orthotropic deck.

The details of the bridge deck (*Fig. 4*) were developed in order to reduce the number of locations where opposite face welding would occur, since laminar tearing could result. The ribs were fabricated in tangent chords to accommodate the sharp radius of the superstructure, and were welded to the top deck plate. The specifications required this to be an 80% partial penetration groove weld. To ensure that welding would be of the highest quality, the specifications required that the rib welds be made using the automatic submerged arc process.

Deck plates were oriented such that the grain or rolling direction of the steel plates was centered on the longitudinal axis of the bridge. Manhole openings and covers were added to the bottom flanges and placed in the interior diaphragms and webs to allow access for inspection and maintenance. The interior and exterior of the deck were painted with inorganic zinc-rich primer for maximum corrosion protection. Drain openings were added to the lowest point of the bottom flanges to allow drainage of water.

To maximize fatigue life, intersections of rib welds with diaphragm welds were avoided by means of large copings at the weld intersections. These copings were sealed by injecting polystyrene foam into the rib compartment to prevent entrance of moisture or other corrosive agents. The weld details and the use of the polystyrene foam sealant were the same as those used during the fabrication of the orthotropic deck sections used for the rehabilitation of the Golden Gate Bridge in 1984. Furthermore, the welding specifications and details for the rib-to-deck welds were very similar to those used in this earlier project.

An orthotropic barrier system with structural steel was designed. To reduce deadweight and simplify composite action, welded steel was selected. The barrier was designed to be higher than usual highway barriers because of the heavy truck traffic using the sharp horseshoe bend. The barriers were fabricated in 6.10-m-long components that were bolted to an internal W-flange system welded to the orthotropic superstructure. A series of access panels on the exterior allow the removal of damaged barriers after vehicle collision.

Seismic Detailing

The bridge has several unique seismic detailing features, including the use of rubber dock fenders as seismic-shock absorbers (*Fig. 5*). The delta-shaped fenders reduce the kinetic energy that occurs during a seismic event at the hinges in the superstructure. Rubber fenders were also used as bumpers to reduce forces transfixed in a compression shock wave along the longitudinal axis of the bridge. Larger fenders were used as part of a system to restrain the cables if seismic forces would try to

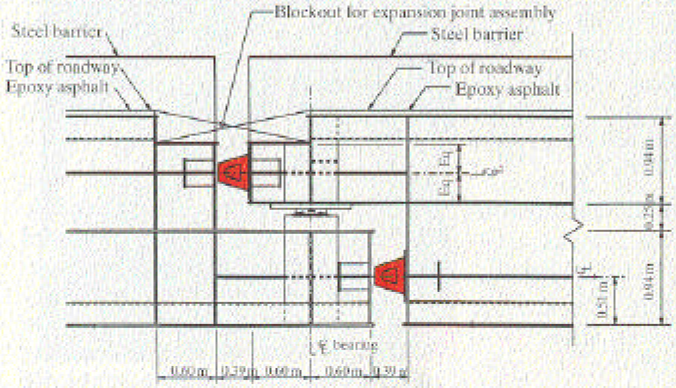

Fig. 5: Seismic bumpers (in red) in the steel hinge section

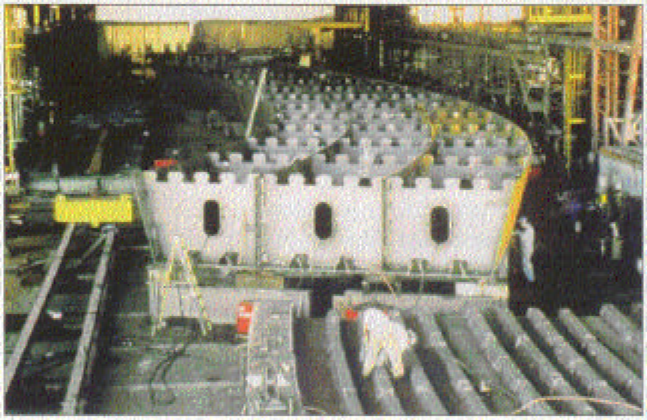
Fig. 6: Fabrication of a steel plate (Photo courtesy of Larry Lowe)

Fig. 7: Transportation of a bridge section into place (Photo courtesy of Robert Colin)

Fig. 8: Placing a bridge section into its final position (Photo courtesy of Robert Colin)

pull the bridge segments apart at the two hinges. The lower weight of the steel orthotropic superstructure and the energy-damping system will reduce the seismic forces on the concrete columns and substructure. Spherical polytetrafluoroethylene (PTFE) bearings were used to allow for rotation and expansion of members. These bearings can resist high lateral forces, including seismic forces. A central shear key pipe was added for additional lateral capacity.

Erection Process

The steel plate was fabricated between June 1995 and May 1996 (*Fig. 6*). Thirteen full-width orthotropic sections, 2.13 m deep and 10.8–11.4 m wide, were fabricated in lengths from 37.5 m to 66.8 m. These sections ranged in weight from 227 t to 416 t. All sections were transported with the steel orthotropic deck and steel barrier rails pre-installed.

Installation of the sections was carried out using special heavy-lift hydraulic platforms (SHLHP), which consist of two self-propelled platforms braced in tandem by a strut beam [4, 5]. A scale model was built to determine the most effective use of the SHLHP.

The girders were set on temporary timber formwork in the staging area. They were then jacked up, and the SHLHP was maneuvered under the girder and positioned at the calculated center-of-gravity location. Then the girder was lifted and tilted to the correct slope of up to 12%.

Closure of I-80 was required to position three of the 13 sections. Approximately 500 000 vehicles pass along this highway every day, and therefore it was essential to minimize any disruptions to the traffic flow. Therefore, sections were staged on the east side of the freeway and erected during the night (*Figs. 7* and *8*). The first section was erected at the west abutment, allowing the team to practice before erection over the freeway. By means of the SHLHP, the sections were positioned and lowered onto the steel bearings anchored to the top of the concrete pier. The section over I-80 westbound was erected on a Saturday night, and the section over the eastbound carriageway of the I-80 was erected the following Saturday night.

Once 25% of the bolts had been installed and the section welded to the bearings, then the dead weight of the section was transferred to the splice and bearings, and the SHLHP was driven away. After the 13 sections had been erected, the joint seal assemblies were installed, and an epoxy asphalt overlay was placed prior to opening the bridge to traffic.

Conclusions

Studies of the steel piles of the old I-880 Cypress Street Viaduct revealed that corrosion had not occurred after 35 years, although some minor pitting was noticeable.

Extensive studies of the pile loads eliminated piling-driving claims and established the elevations of the pile tips needed for seismic loading.

Pipe piling with an outer diameter of 1.07 m was used. Even larger pipe piles (outer diameter 3.65 m) are planned for the seismic retrofitting of the Richmond-San Rafael bridge.

Although the steel orthotropic steel bridge system is complex, it is a buildable system. California's ninth orthotropic superstructure has been designed for the 762-m-span Carquinez Suspension Bridge to replace the cantilever truss bridge built in 1924. California's tenth orthotropic superstructure is under design for the 385-m-span suspension bridge to replace the east cantilever truss span of the San Francisco Oakland Bay Bridge [2].

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