UNIQUE STEEL CURVED ORTHO-TROPIC BRIDGE FOR THE I-880 REPLACE-MENT PROJECT

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SUMMARY

This paper discusses the Maritime Off-Ramp a "horseshoe" shaped steel orthotropic cellular box girder with trapezoidal ribs bridge. The site's soils are liquifiable clay due to earthquake ground movement. This bridge’s foundations are concrete filled 42-inch diameter steel pipe piles. It is a new bridge built after the Cypress Viaduct collapsed in 1989. Thirteen sections were fabricated, with maximum weight of 700,000 pounds. Three sections were lifted, one per night over I-80, using the truck-pulled hydraulic platforms, and field welded to the bearings. These 13 sections were bolted together creating an instant superstructure. The orthotropic section is 2356 long and 36.2 feet wide.

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written by:

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INTRODUCTION

This paper discusses the 2,356-foot long "Maritime Off-Ramp" (Bridge Number 33-6235) which is also known as the Horseshoe Line or "HS" Line because of its 250-foot radius horseshoe shape. The "HS" Line a $22 million bridge is one of eleven bridges for Contract E worth $10 million dollars, as shown in Figure 1. Contract E is located at the intersection of I-880 with I-80.

Figure 1 Two trucks start across the "Horseshoe" bend of the Maritime Off-Ramp

The I-880 Replacement Project consists of seven bridge contracts A through G with about $500 million dollars worth of bridges. The "HS" Line is an orthotropic steel bridge while the remainder of the bridges are steel plate girder bridges with concrete decks or post-tensioned cast-in-place concrete. The "HS" Line becomes California's sixth orthotropic bridge. The other bridges are Hayward-San Mateo, (completed 1967); North 680 to West 580 Connector Bridge, Br. No. 33-0371G (completed 1969); Queensway Bridge, City of Long Beach, Br. No. 53C-0551 (completed 1971); San Diego-Coronado Bay Bridge, Br. No. 57-0857 (completed 1969); and the redecking of the Golden Gate Bridge (completed 1985). The North 680 to West Connector Bridge was built as an experimental bridge to check the accuracy of design calculations and actual capacities various trapezoidal ribs and deck plate thickness (40). The word orthotropic is a shortening or acronym of two words, orthogonal and isotropic. This deck system is an anisotropic element about its orthogonal axis's. When the bridge is entirely steel, the jargon "orthotropic" bridge is used, but if only the deck is orthotropic then the term "orthotropic deck" is used to describe the bridge. The jargon "tub" girder is used by many individual when a reinforced concrete deck is placed on top of steel components consisting of two webs and bottom plate. This bridge was more difficult to fabricate than a tub girder because the welders and painters must work inside a closed cellular structure.

The I-880 Replacement Project has many names also because it replaced a concrete structure on Cypress Street. It has also been called the Cypress Replacement Project, Cypress Project, I-880 Cypress Replacement, I-880 Reconstruction Project and Cypress Viaduct Replacement Project. Cypress Street was renamed the Nelson Mandela Parkway.

This paper was written because the bridge was successful and the authors expect a similar type structure to be built in the future. The Carquinez suspension bridge with orthotropic superstructure is currently under design. The San Francisco-Oakland Bay Bridge is reviewing the possibility of using an orthotropic deck. This project is part of the State of California, Department of Transportation (Caltrans) system. I-880 is also known as the Nimitz Freeway.

LOMA PRIETA EARTHQUAKE

The Loma Prieta 7.1 magnitude earthquake of October 17, 1989 occurred near Oakland and San Francisco, California. The I-880 Cypress Street Viaduct, a two-level reinforced concrete structure completed in 1957, carried I-880 freeway traffic through downtown Oakland. The upper roadway collapsed resulting in 41 deaths and 108 injuries (1). The portion between 7th and 34th streets was removed. Since the earthquake and the removal of this section of I-880, the remaining freeway network consisted of I-980 and portions of I-580. I-880 is a critical interstate freeway connecting San Jose and the East Bay Area to San Francisco, Sacramento and the Sierra Nevada. The I-880 Freeway Viaduct was a section of I-880 that carried between 140,000 and 160,000 vehicles per day and provided for eight lanes of mixed-flow traffic. Cypress Street was renamed the Nelson Mandela Parkway by the City of Oakland.

This site is seven miles from the business district of the City of San Francisco. After the earthquake damage, the I-880 Viaduct was demolished by Caltrans to the top of the footing elevation and covered up with backfill. A portion of the viaduct that was not damaged was saved for destructive testing (1). The concrete filled steel pipe piles were successfully installed and survived undamaged during Loma Prieta earthquake. An independent panel of experts made recommendations to the Governor and the people of California (1).

Caltrans initiated its own internal review of what happened and how to improve on the durability of its infrastructure. Much of the research done for the I-880 Replacement Project has been utilized on other Caltrans projects.

THE I-880 REPLACEMENT PROJECT

The I-880 Replacement Project was designed and then built in seven separate construction contracts to avoid impacting traffic in this important transportation corridor (Figure 1). Caltrans was assigned to deliver the bridge plans, construction specifications and construction cost estimate. The bridge design staff consisted of two Caltrans engineering teams and five teams of consulting engineers (consisting of about 30 firms as sub-consultants). Caltrans geotechnical engineers and geologists performed geotechnical investigation and recommendations.

"The Maritime Off-Ramp" is a new unique curved steel orthotropic bridge that provides access to the Port of Oakland through a U-turn from Westbound I-80 (Figure 2). The remainder of the new bridges were replacement bridges for the collapsed I-880 Street Viaduct.

The project's $900 million budget is consisting mainly of federal emergency relief funds from FEMA, is divided into approximately $550 million for construction and $350 million for right-of-way acquisition, railroad and utility relocation, traffic management, transit enhancement, and a number of commitments to the City of Oakland.

EXISTING STEEL PIPE PILES EXTRACTED FOR CORROSION STUDY

The new bridge foundations had been planned using steel pipe piles filled with concrete similar to the old I-880 Viaduct foundation built in the 1950's (Figure 1). The new piles are of a much larger diameter than standard pile details. New site borings revealed that corrosion protection should be considered, due to the presence of high levels of chloride, sulfates and low values of minimum soil resistivities. It was estimated that corrosion protection costs could be in the millions of dollars.

The goals of the pile exhumation were as follows: 1.) To examine pile condition to rationalize an economical solution for corrosion protection of new bridges; and 2.) seismic tension uplift test piles to compare against calculations by bridge design engineers. This Raid on the Lost Viaduct was important because very limited information is published on the extent of material deterioration for bare steel piles. Most detailed corrosion reports are very site specific.

Caltrans District 4, San Leandro Maintenance (SLM) provided personnel and equipment to exhume the piling. ICF Kaiser provided Caltrans with a 125-ton crane with vibratory hammer(s) and drilling rig, plus engineering recommendations. Three sites along Nelson Mandela Parkway (Bents 61, 75 and 91 of the original viaduct) were selected, based on profile maps of the area which represented the varying soil strata similar to the I-880 Replacement site. During the week of June 22, 1992, Caltrans performed soil borings at these bent locations to verify that soils were indeed similar to the I-880 Replacement site locations. At Bents 75 and 91, the liquifiable bay mud were present and at Bent 61 the dense Merritt Sand was present. It was expected that corrosion of the steel piles would be more severe at the bay mud sites due to the corrosive potential of the fine, dense clay, which had high chloride content and low minimum resistivities. The Corrosion Branch reviewed soil borings taken in the 1950's for the Cypress Viaduct.

The piles at Bents 75 and 91 were much longer than at Bent 61, because of the poor structural properties of the bay mud. These longer piles extend through several soil layers of varying corrosion properties. Exhumation of these piles would allow for more accurate method determination of the propensity for corrosion cell activity to occur at the soil transition zones at various depths. Due to the vast scope of the project the use of only published site specific data was not deemed economically rational.

The excavation of the three selected concrete footings began the week of Monday, June 29, 1992.

Figure 2 Hand-held jackhammers were used to remove corners of the footings

SLM crew cut the only reinforcement, the bottom mat, with torches. One pile from each bent was exposed. The piles were all 12-inch diameter spiral welded pipes with concrete fill. Four No. 6 rebars connected the pile to the concrete footing. This was one of the standard pile foundation types used in the 1950's. A similar design is still in use as part of Caltrans Standard Plans. SLM crew welded a six foot long extension pipe with two slots at the top to attach to the tension test beam. The crew worked overtime to keep the test on schedule. Tension test beams weighing 17,000 lb. were trucked from the Office of Materials Engineering and Testing Services (OME&TS) lab in Sacramento to the site in Oakland. On Wednesday, July 29, 1992, the vibratory hammer was attached to a pile to be extracted. This pile failed five feet below ground surface at a butt weld splice.

Caltrans decided to drill a series of holes full pile length alongside of a pile to be exhumed. These holes would reduce the amount of skin friction. It was realized that the vibratory hammer (Tomen 6000) supplied was too large and ordered that a smaller vibratory hammer (Tomen 5000) be utilized. The smaller hammer worked more effectively putting smaller forces on the pile and not overloading it. Even though holes were drilled around the perimeter of the pile, tremendous suction forces were still present. These holes in the soil adjacent to the pile significantly reduced the soil skin friction against this pile. Eventually, the pile was completely removed, as shown in Figures 3, 4 and 5.



Figure 3 Figure 4 Figure 5

Figure 3 Welding tab plate to existing pile
Figure 4 Vibratory hammer clamps onto welded tab plate to pull pile
Figure 5 A quick visual inspection prior to shipping to Caltrans lab

It was apparent from a quick visual inspection that corrosion was at a minimum after 35 years of service. The pile surface appeared as though it was brand new. A second pile was removed Thursday, July 30, 1992 and the final third pile was removed on Monday, August 3, 1992. The piles were cut up into ten foot segments and trucked to the OME&TS lab in Sacramento for detailed inspection. The exterior surface of each pile segment was pressure cleaned with water/steam to remove any clay, sand or gravel remaining after transport. The location, type, and amount of corrosion was then identified for each pile. Ultrasound thickness measurements were taken to determine how much metal was lost due to corrosion. The original thickness of each pile was obtained by taking ultrasound thickness measurements from non-corroded regions.

Results of the corrosion testing and inspection were as follows:

Old I-880 Viaduct - Bent 91

- Total pile length was 49.5 feet. Tip elevation was at approximately at 46 feet. The average original pipe wall thickness was 0.189 inches, about 3/16 of an inch.
- This pile exhibited the most severe corrosion damage of the three piles examined, as was evident by the presence of shallow surface pitting corrosion which occurred along the pile from one foot below top of pile down 29 feet. The minor surface pitting corrosion was not concentrated in any localized area but was distributed along this 28 foot region of the pile, with approximately 30 percent of the surface area affected within that region. The maximum pit depth was 0.019 inches (19 mils) or about 1/64 of an inch.
- Soil in the region of the pitting is broadly defined as loose to dense bedded sands, silts and gravel. This soil contained high levels of chlorides (up to 5100 PPM). The remaining portion of the pile (below 29 feet) appeared to be in very good condition with no evidence of corrosion.

Old I-880 Viaduct - Bent 75

- Total pile length was 59.5 feet. Tip elevation was at approximately at 55.5 feet. The average original pipe wall thickness was 0.181 inches, about 3/16 of an inch.
- There was no evidence of corrosion. Two locations on the pile corresponding to sand lenses (as evident by material stuck onto the pile) exhibited a small amount of uniform scaling (removal of the adherent mill scale), however, no appreciable loss of metal was evident.

Old I-880 Viaduct - Bent 61

- Total pile length was 15 feet. Tip elevation was at approximately at 13.5 feet. The average original pipe wall thickness was 0.184-inches, about 3/16 of an inch.
- This pile was located in a Merritt Sand formation along its entire length.
- Minor surface pitting was found along the top five (5) feet of the pile, generally above the level of the water table. Less than 30 percent of the surface area was affected in this region. Maximum pit depth was 0.016 inches (16 mils), about 1/64 of an inch.
- Chloride levels of the soil at all depths for this bent location were negligible.

STEEL PIPE PILE CORROSION PROTECTION FOR THE I-880 PROJECT

1. The steel pipe wall thickness of each pile used at the realignment was increased by a minimum of 0.125-inch (1/8 of an inch) over the structural requirement for the pipe, except as noted in the following recommendation. This will allow for a small amount of sacrificial metal loss in the form of surface pitting and/or uniform corrosion during the proposed design life. Since each bridge engineering team wanted to use a different size of pile they were directed to use to the nearest 0.125-inch thickness.

2. Most of the steel pipe piles used for this bridge were within 1,000 feet of salt water. This location could be more corrosive due to its close proximity to San Francisco bay, which may result in sea water encroachment into the soils. The wall thickness was increased by a minimum of 0.250-inch over the structural requirement for the pipe. Piles within 500 feet of salt water will be electrically connected for future monitoring and/or future cathodic protection. Caltrans electrical engineering staff designed the system to connect the piles electrically. Caltrans Bridge Design Specifications (section 8.22.1) discusses extra cover requirements for reinforced concrete.

At this time no steel pilings older that 50 years have been exhumed and examined. There is no scientific evidence whether long term corrosion remains constant or varies with time. For simplicity many people assume the rate is constant. Assuming corrosion is constant, the additional steel will resist corrosion for over 250 years.

Caltrans professionals economically utilized taxpayer dollars, because over $30 million in steel pipe piling was saved by performing corrosion inspection of the 35-year old piles. Site-specific information is better (as with Geotechnical) than utilizing research reports based on piles from another location.

With coating costs of $2 to $4 per square foot, between $6.7 to $13.4 million (USA dollars) was saved in the foundation design by Caltrans Corrosion Technology Section by avoiding the use of corrosion inhibiting coatings. With an additional sacrificial steel thickness of only 0.125-inch Caltrans expects the new piles life span to match or exceed that of the superstructure. The price of steel piling was approximately $0.33 per pound. Therefore, about $5.7 million was spent for additional steel wall thickness. Another advantage of increasing wall thickness to combat corrosion is that the foundation system will be stronger than calculated. Paint and other coatings have no structural strength, and may modify the soils coefficient of skin friction. Caltrans bridge policy is that the foundation system will be the most durable component and receive no damage in an earthquake. Obviously, it is not practicable to find pile damage easily or repair it.

Table 1 - I-880 Replacement Project Steel Pile Summary

CONTRACTS	PIPE PILE OUTER DIAMETER	LENGTH (FEET)	SURFACE AREA (SQUARE FEET)
A	16-inch	3,570	14,953
A	24-inch	128,850	809,588
C	16-inch	9,775	40,843
C	24-inch	29,419	184,845
D	24-inch	64,966	408,212
E	24-inch	1,081	11,886
E	42-inch	93,755	1,030,890
F	24-inch	63,685	400,144
F	42-inch	17,033	187,288
G	16-inch	10,140	42,474
G	24-inch	32,500	204,203
Total		**454,773**	**3,335,326**

The "HS" Line has 118,692 square feet of 42 inch pipe piles.

On Friday, July 17, 1992, the first tension test was performed at Bent 91. The Consultant Contract Management Branch (CCMB) had requested that piles be tested until a significant movement occurred to verify that the proper crane would be rented and arrive at the site to extract each pile. At 160 tons (320 kips) the pile extension had a buckling failure without significant pile movement (Figure 8). On Saturday, July 18, 1992, a pile at Bent 75 failed suddenly below the ground surface at test loading on only 90 tons (180 kips). On Monday, July 20, 1992, the final pile at Bent 61 was tested and at 30 tons (60 kips) significant movement of the pile occurred. Tension test data confirmed that seismic induced pile pullout that could cause collapse of the structure would not occur.

Geotechnical tests were performed and showed that the "soil set up factor" around piles was significant. Therefore, Caltrans with FHWA funding performed a comprehensive Indicator Pile Load Program from October 1993 to January 1994.

PILE INDICATOR PROGRAM FOR THE I-880 REPLACEMENT PROJECT

In order to gain a better understanding of construction and design issues associated with the installation of large diameter steel pipe piles, an Indicator Pile Test Program was developed prior to award of production contracts. Various pile installation methods were tested. Pipe piles were installed with both vibratory and impact hammers.

The Indicator Pile Test Program reports were made available to all bidders for the I-880 Replacement Project. This was an attempt to eliminate major pile driving claims plus make for a more competitive bidding process.

SEISMIC CRITERIA AND STUDY

Caltrans has entered into more than $7 million in research contracts with some of the nation's foremost engineering and seismic experts since the 7.1 Loma Prieta earthquake rocked the San Francisco Bay Area on October 17, 1989.

This ambitious seismic research program is beginning to pay big dividends in helping engineers better understand and address the impacts of large tremors.

After the San Fernando earthquake, a number of improvements were made in the design criteria including the use of spiral steel reinforcing which holds the vertical reinforcing steel together, thus building ductility into the columns. This makes the structures less vulnerable to damage and large displacements in a strong tremor (10).

Some of this research was:

- Two contracts at the University of Southern California (USC) and the University of California, Davis (UCD) focusing on soil interaction with structures when subjected to seismic forces. Researchers looked for ways to help bridge designers improve ways of using fill material at bridge abutments to dissipate energy generated in an earthquake. The $100,000 research contract at USC was conducted under the auspices of professor Dr. Geoffrey R. Martin. Dr. Karl M. Romstad directed the $350,000 research effort at UCD.
- A contract with UCD to produce experimental measurements of bridge abutment stiffness and strength characteristics. This research, conducted by Dr. Romstad, was used to develop design criteria to determine how abutments can better be used to help dissipate seismic forces.

Tests were conducted on the response of pile supported bridge elements in the event soft soil liquefies. These tests, at the request of Caltrans, were conducted by The National Cooperative Highway Research Program, under a grant from the National Science Foundation and the Transportation Research Board. Researchers built test piles in soft soil and shake the material until liquefaction occurred. This work is significant because the six structures suffering major damage in the Loma Prieta earthquake were constructed on deep soft soils (10).

BRIDGE DESIGN CRITERIA

- Load factor design was used with Bridge Design Specifications (BDS) and 1983 AASHTO with interims and revisions by Caltrans.
- Live loading: HS20-44 and alternative and Caltrans permit design load.
- Seismic Design: I-880 Reconstruction Project Design Package (revised 6-22-92) with a site specific response curve unique to contract E.
- Reinforced Concrete: fy= 66,000 psi, f' c = 3,250 psi, n = 9.
- Structural Steel: ASTM A709 Grade 50T2 was specified for the tension components of the orthotropic box girders. The "T2" is specified to ensure adequate toughness (CVN Impacts) for improved fatigue performance.
- ASTM A709 Grade 50 For Box Girders, Flat Plate Stiffeners, Bearing Stiffeners and Shear Keys.
- ASTM A709 Grade 36 for Bearing Shim Plates, Manhole Door Covers including Stiffeners and Hinges and Latch Components and Ladders.
- ASTM A500 Grade B for Structural Tubing.
- ASTM A668 Class G with Supplemental Charpy Requirement for heavy wall cylindrical forging for shear keys.
- ASTM A252 Grade 3 (minimum yield strength of 45 ksi) for steel Piles.
- ASTM A-325 for Structural Steel Connections. All bolt heads shall be located on the exterior face of all exterior steel plates including the top deck flange plate, bottom flange plate, exterior inclined web plates and end diaphragm plates.
- ASTM A-449 for Masonry Plate Anchor Bolts embedded in concrete.
- ASTM A-307 for miscellaneous Anchor Bolts unless noted otherwise.

Hexagonal shaped reinforced concrete footings with reinforced concrete columns using the latest Caltrans seismic detailing methods were selected by Tudor’s engineers for all bridge footings on Contact E.

ORTHOTROPIC SUPERSTRUCTURE

Type Selection

Steel bridges were selected by Caltrans to cross over the busy I-80 freeway to minimize travel delays or lane closures during bridge erection to commuters and highway traffic. An orthotropic bridge with trapezoidal ribs was selected by the consulting engineering firms of Tudor Engineering and ICF Kaiser Engineers. This concept was approved by Caltrans and contract plans were developed as shown in

Figure 8. The unique 180-degree curve or horseshoe bend shape of the bridge made a closed cell structure the most economical shape to resist the torsional forces.

Figure 6 The horseshoe bend "Maritime Off-Ramp" provides truck access to the port of Oakland from west-bound Interstate I-80

The bridge utilized reinforced concrete "T" bents with a single column with spiral reinforcing ties. Two special bearings connect the superstructure to each "T" bent. New joint shear criteria for reinforced concrete design was developed for Caltrans by UCSD was first used on the project. The level of reinforcement was higher than previous projects.

SPECIAL ISSUES

Orthotropic Deck Stiffness

Caltrans is the owner of two very large orthotropic bridges, the Coronado-San Diego and the Hayward-San Mateo Toll Bridges. Caltrans provided technical assistance and construction inspection to the Golden Gate Toll Bridge Authority for the replacement of the original reinforced concrete deck with an orthotropic steel deck. Some of the details used on the "HS" Line were first utilized on the Golden Gate Bridge (12 & 17). Many welding details for the trapezoidal rib were repeated for the "HS" Line. The transverse flexural stiffness of the orthotropic steel deck is critical for the long term behavior of the asphaltic overlay. The weight-saving concerns, which guide the seismic design, dictate minimal overlay thickness. This leads to a compliant deck system which is very sensitive to temperature and to local deflections imposed by concentrated wheel loads. The cyclic nature of this loading only increases the problem of compliance between overlay and steel underlayment.

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

Fatigue

The over-all stress range amplitudes under global loads are very low for this type of multicell box girders. The longitudinal continuity and the 180°± in plane "curve" further minimize longitudinal tensile stress ranges. The torsional stress components are very low for this large "cell perimeters". The resultant equivalent stress intensity values (von Mieses yield criterion) are very low.

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

Fracture Control Plan

The optimization of the safety and of the performance of the structure under 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 is given to the danger of subcritical crack growth and unstable crack extension. Special attention, at the design stage and subsequently at the fabrication stage, is given to the factors affecting these fatigue induced micro cracks.

The main elements of the fracture control plan considered are:

- the identification of the main tributary factors such as local loads, dynamic amplifications, stress risers, residual welding stresses two-dimensional states of "hydrostatic tensile stresses," etc.;
- the establishment of the relative importance and contribution of each of these tributary factors;
- the determination of the various strategies in design and fabrication to mitigate the most important "fracture-causing" elements;

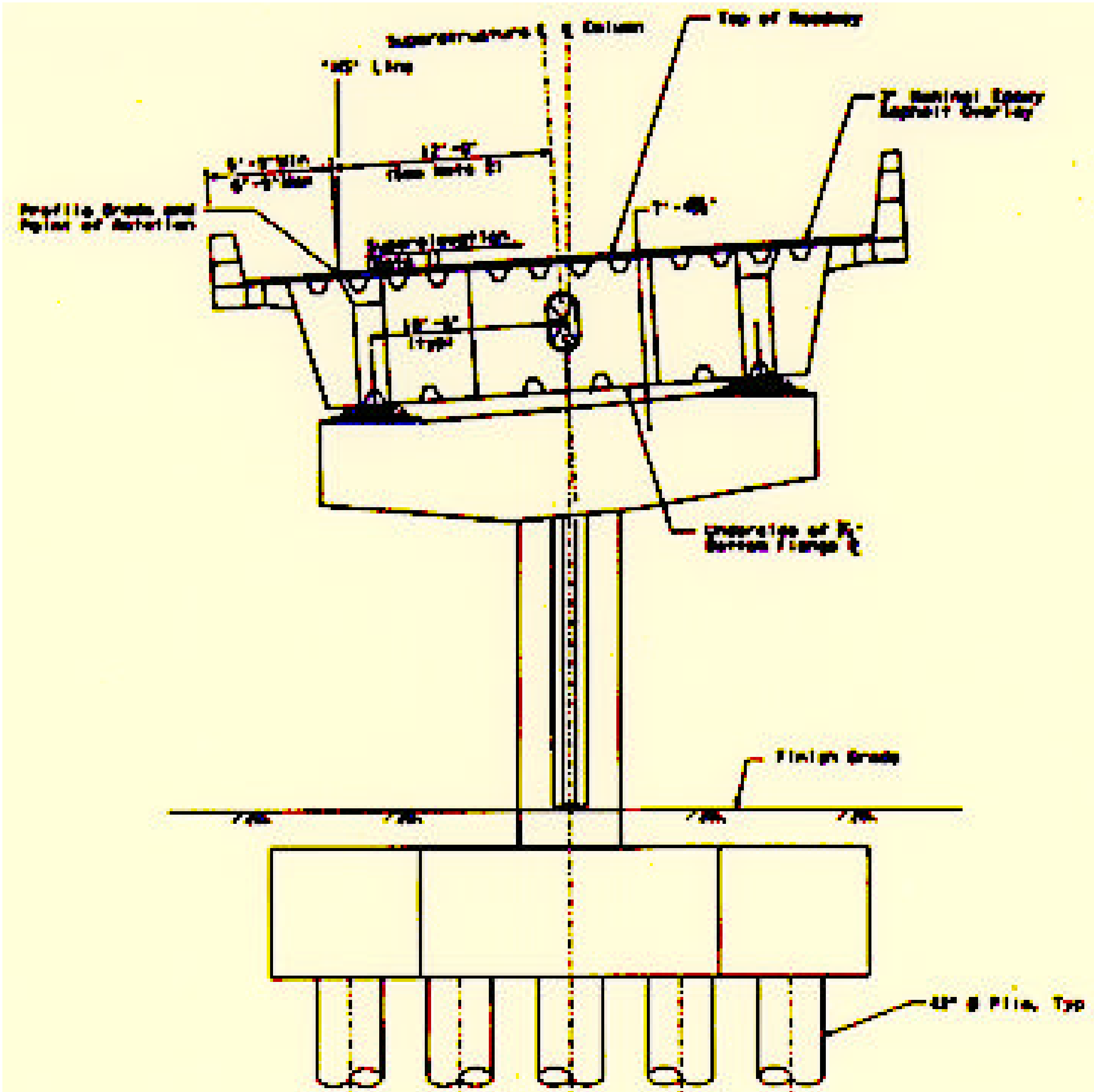

Figure 6 Typ. section - “T” bent with hexagonal footing - seven 42” pipe piles

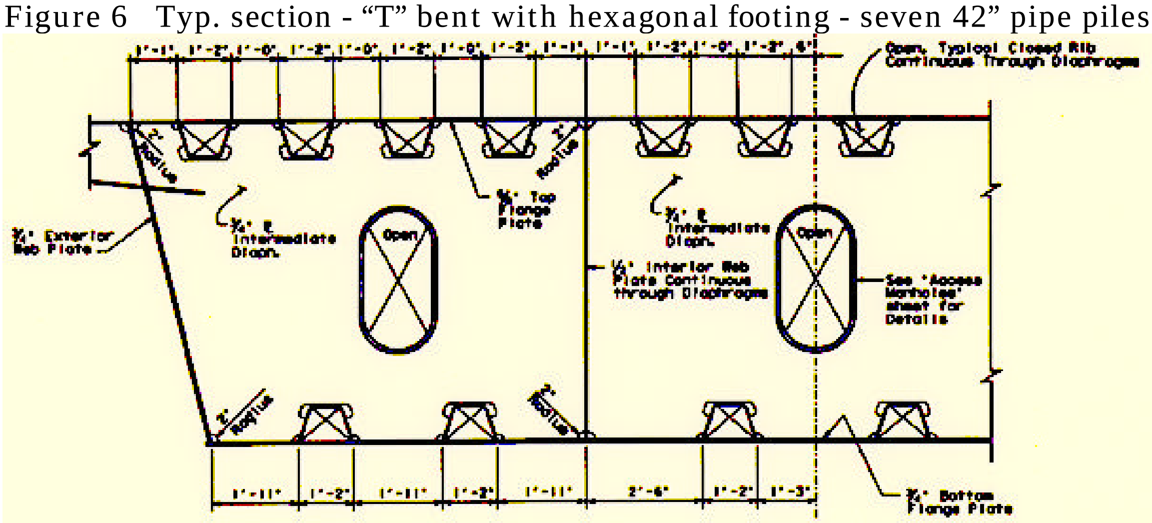

Figure 7 Half of orthotropic section (symmetrical - except for barriers)

- the recommendation of an optimal design and fabrication procedure, including choice of materials, quality control and inspection methodology.

The three primary factors affecting local failure and ensuing major damage and hence the lifespan of a structure undergoing fatigue loading are:

- the tensile stress range amplitude;
- the flaw size within material welds which means quality of fabrication and inspection;
- the material toughness properties which means choice of specific steel.

The design philosophy consists in providing for "minimal" stress raisers through a careful choice of details and weld types.

Redundancy is used with caution to minimize global and local stress range amplitudes and also to provide multiple load paths. Crack arresters are considered in various fracture propagation scenarios.

Buckling

Web and flange local buckling are considered in a "limit load analysis" approach with given transverse imperfections. Indeed, it is essential to include actual fabrication conditions simulating imperfect geometry and welding residual stresses.

Shear Lag

The web "slenderness" leads to some shear lag effect which needs to be considered in the local fatigue as well as in the local buckling capacity of the most critical web panels at midspan and near supports.

ANALYSIS

Various task-specific programs were used to simulate the static and dynamic behavior of the bridge structure Global Static Analysis.

The global static model was developed for the complete structure by means of SAP-90. Dedicated pre- and post- processors allowed the designers to produce the governing load combinations for service and for ultimate conditions for service and for ultimate conditions corresponding to the standard "LRFD" approach in accordance with AASHTO-NCHRP and CALTRANS-BDS regulations.

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

The global dynamic model was developed with the main requirement of capturing 90 percent of the total modal mass participation in all three orthogonal directions.

Special attention was given to the expansion joints and to the foundations. Upper and lower bounds for foundation impedance's were considered. The pier columns were also considered under two distinct state elastic uncracked and cracked as well.

The seismic analysis consists of the spectral response approach based on the CQC modal superposition. These linear analyses are performed for a site specific "mud site" spectrum corresponding to the maximum credible earthquake occurring on the Hayward fault with an 8.0 M magnitude and, alternatively, on the San Andreas fault with an 8.5 M magnitude.

The seismic design philosophy is a displacement-ductility driven approach which permits controlled plastic straining to occur at the various pier foundation bases.

Soil-structure interaction analyses were performed by means of the "GROUP" finite-difference program. The very deep Bay Mud layers were simulated under different material assumptions to detain meaningful sensitivity curves and to verify the compliant pile behavior.

Dedicated non-linear "ABAQUS" finite element models were developed for local web and diaphragm buckling analyses for local wheel loading studies on the orthotropic deck.

The steel impact barrier was studied by means of a dedicated non-linear "ABAQUS" model as well in order to simulate the ultimate energy absorbing capability and the impact load reaction applied to the main superstructure.

FOUNDATION DESIGN

The hexagonal foundations with the seven, "medium size" 42-inch diameter, steel piles were the result of a structural geotechnical and economical optimization study. The need for pile compliance at the interface of soft mud with the stiff underlayment leads to steel piles. The deep overlying mud layer requires very long piles because of the high seismic loads. Cost minimizing is based on minimizing the number of steel piles while keeping the pile cap at a reasonable size. Also, the need for pile ductility and for elastic pile cap behavior dictate the use of 42-inch pile diameters with the pile cap dimensions chosen. The hexagonal pile cap shape results from the "isotropic" nature of the seismic loading imparted by the soil structure interaction with the "plastic hinge" load reaction at the pier base.

Current research papers discussing fatigue problems on European bridges and recommended detailing practices were reviewed by Caltrans and Tudor’s engineers

(13 to 20). No welds were allowed to cross over another weld and plates were coped to avoid the occurrence. The details were developed to reduce the number of locations where opposite face welding would occur since laminar tearing could result. The trapezoidal rib is the most popular system used in about 95 percent of all orthotropic deck bridges built world wide. The ribs were fabricated in tangent chords to accommodate the sharp radius of the superstructure. It was unfeasible to form the rib sections in the same curved configuration of the bridge. The ribs were therefore welded as tangent chords to the top deck plate to approximate the radius of the girders. An important detail of the orthotropic design is the welding of the closed rib stiffener to the top deck plate. The specifications required this to be an 80 percent 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. Special requirements in addition to those of the AWS Bridge Welding Code were specified for weld procedure and production control tests for this weld. Deck plates were oriented so the grain or rolling direction of the steel plates was centered on the longitudinal axis of the bridge. Personnel openings were provided throughout the structure for bridge maintenance inspections. Manhole openings and covers were added to the bottom flanges and manhole openings were placed in the interior diaphragms and webs to allow inspection and maintenance personnel future access. Both the interior and exterior were painted with inorganic zinc rich primer for maximum corrosion protection. Drain openings were also added to the lowest point of the bottom flanges to allow drainage of water intrusion. Expanded polystyrene foam was installed in the ribs to prevent corrosion. To maximize fatigue life and avoid Category "E" details, intersection of rib welds at their juncture with the diaphragm welds were avoided by large copes at the weld intersections. These copes were sealed by injecting polystyrene foam into the rib compartment to prevent entrance of moisture or other corrosion contaminants. This detail was also used on the Golden Gate Bridge. The weld details and the use of the polystyrene foam sealant were the same as used in the fabrication of the orthotropic replacement deck sections for the rehabilitation of the Golden Gate Bridge in 1984. Caltrans provided the construction management and laboratory testing inspection support for the Golden Gate Bridge District during their deck replacement construction. The welding specifications and details for the rib-to-deck welds were essentially the same as used on the Golden Gate Bridge redecking in 1984. Caltrans was responsible for the supervision of QA testing, as well as shop inspection and field welding inspection for that project. Caltrans incorporated these same specifications in the Maritime Off-Ramp during my Plans, Specifications and Estimates (PS&E) review of the contract special provisions and plans.

An orthotropic steel barrier system using structural steel fabricated in the approved FHWA barrier configuration was designed. To reduce deadweight and simplify composite action welded steel was selected over the reinforced concrete Caltrans "K" rail system. The system was designed to be higher because of the heavy truck traffic using the sharp horseshoe bend on I-80 traffic (Figure 6). The barrier was fabricated in 20 ft. long components that bolt to an internal "W" flange system welded to the orthotropic superstructure. A series of access panels on the exterior allow the removal of damaged barrier after a vehicle collision.

SEISMIC DETAILING FEATURES

The bridge has several unique seismic detailing features including the use of rubber dock fenders as seismic shock absorbers reducing forces between completed bridge sections. The rubber delta shaped "dock fenders" were used to reduce kinetic energy occurring during a seismic event at the hinges in the superstructure. The systems were developed for this bridge and are unique to Caltrans. Rubber fenders were used as bumpers to reduce forces transfixed in a compression shock wave in the longitudinal axis of the bridge and this detail is shown in Figure 7.

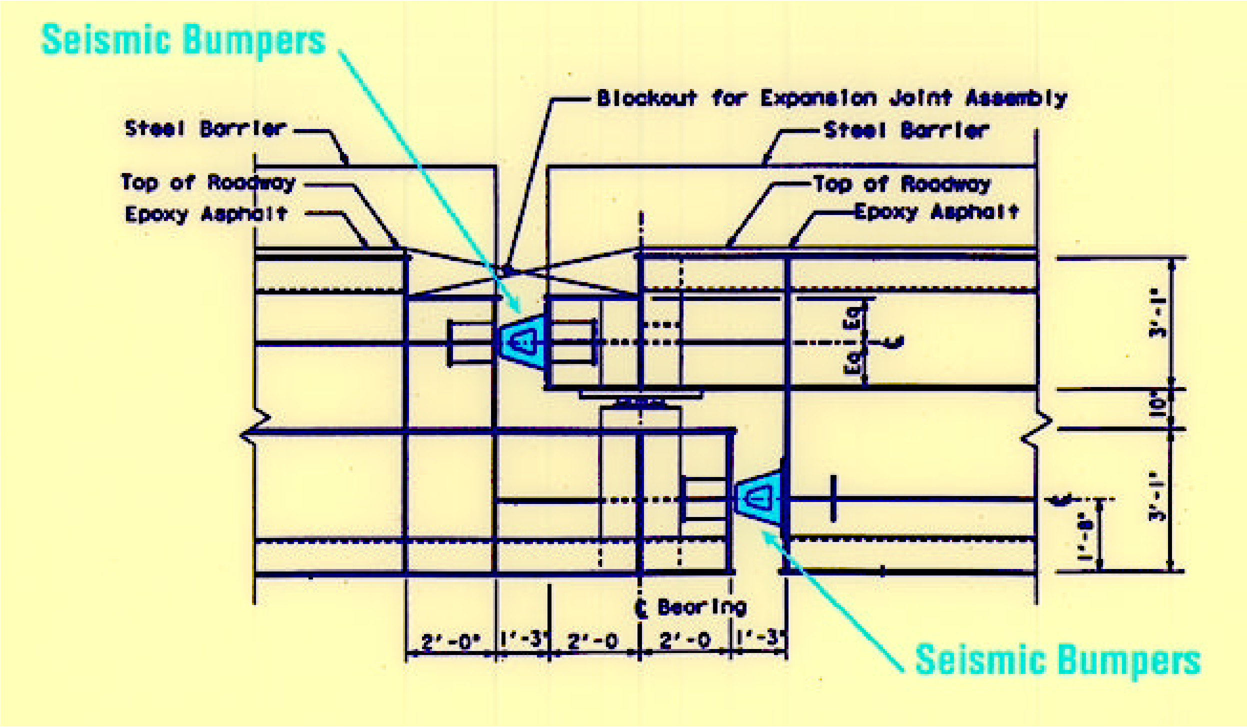

Figure 7 Rubber (dock fender) seismic bumper detail at hinge

Larger delta shaped rubber fenders were used as part of cable restrainer system when the seismic forces would try to pull apart the bridge segments at the two hinge factions. The lower mass of the steel orthotropic superstructure plus the energy damping system reduce seismic forces on the concrete columns and substructure.

PTFE (poly-tetra-fluoro-ethylene) spherical bearings were used to allow for rotation and expansion of members. These bearings can resist higher lateral forces including seismic forces. A central shear key pipe was added for additional lateral capacity as shown in Figure 8.

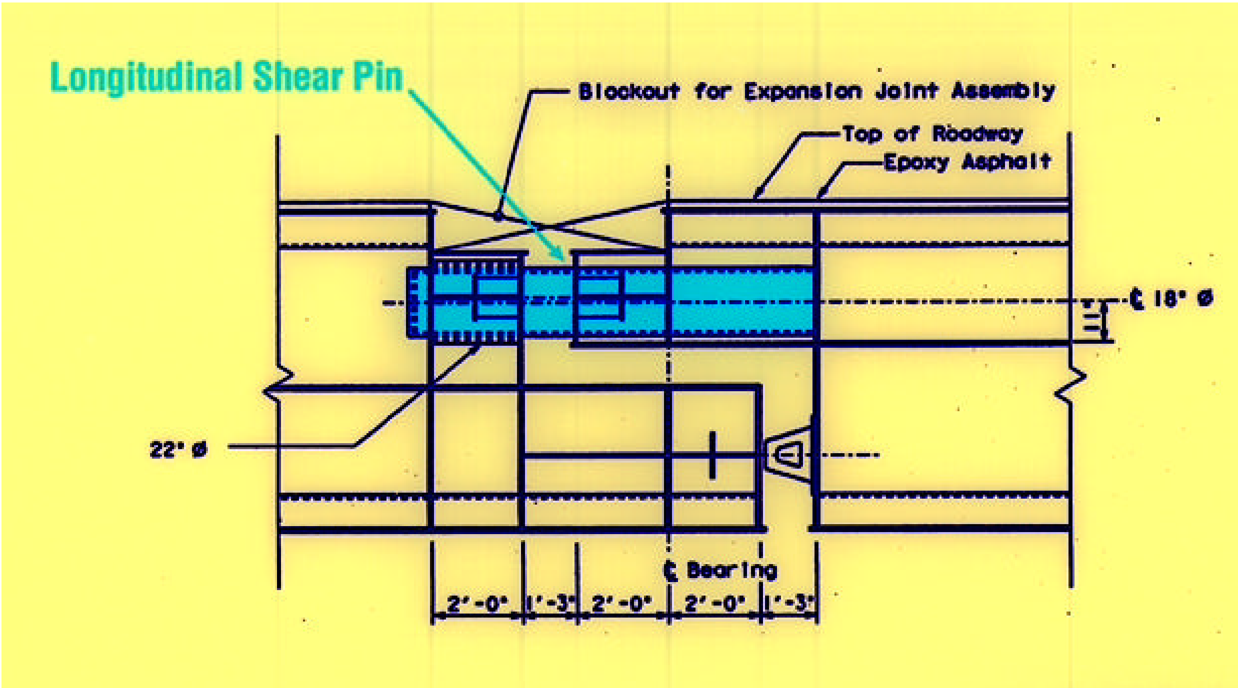

Figure 8 A central shear key pipe was provided in same location as Figure 7

FABRICATION OF ORTHOTROPIC BRIDGE SECTIONS

Steel shop abilities vary fabrication size of pieces. Caltrans consulted American Institute of Steel Construction (AISC) to allow for flexible section size. Caltrans met with AISC representatives during the course of the design of the orthotropic bridges. The technical representatives of these associations wanted to make sure that all their members would be able to bid on the project. Consequently, Caltrans directed the consulting engineers to provide detailing for maximum piece sizing. Longitudinal splicing near the centerline of the bridge was provided in the plans to allow a smaller shop to fabricate the superstructure in two halves. The successful bidder Kiewit-Marmolejo (Joint Venture) minimized piece size in conjunction with their fabricator, USI (Universal Structural, Inc.).

The unusually large dimensions and weight of the orthotropic box girders would limit their fabrication to only the largest fabricators. Both transverse and longitudinal bolted splice alternatives were included in the design to allow the fabrication of the girders in the smallest sections possible. The complexities of the welding and fitup of component pieces required experienced steel bridge fabricators. Fabrication was subcontracted to Universal Structural, Inc. who is an established steel bridge fabricator and a certified AISC Category III Shop. They had previously supplied many bridges for Caltrans as well as for other DOTs.



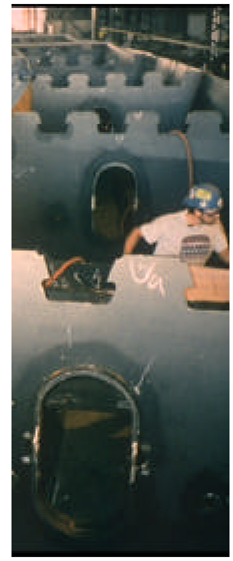

Figure 9 Figure 10 Figure 11

Figure 9 Trapezoidal rib fabrication on a brake press at USI fabrication facilities
Figure 10 Welding of ends of ribs fabricated in straight segments tangent to radius
Figure 11 Welder gives scale to large welded sections (prior to top deck installation)

The large size and weight of individual girder components created some difficult handling and fitup problems. This was further complicated due to the curved configuration of the girders. The top deck plates with the welded rib sections proved difficult to handle. Once all sub-assemblies were fitted together this required that most welding be performed in the vertical and horizontal positions within the closed box sections. The girder sections were too large to turn to allow welding in the optimum flat and horizontal welding positions. Both shop and inspection personnel had to work in the interior confinement of the box sections using artificial lighting and use the various manhole openings for access.

The fabricator provided their own in-house quality control inspection and testing. This was augmented by Caltrans QA shop inspectors who provided continuous oversight during the period of fabrication and painting. All inspections and tests were performed in accordance with the requirements of Caltrans Standard Specifications and added special provision requirements. With the exception of the added welding requirements for the rib-to-deck welds, welding was performed in accordance with the requirements of the AWS D1.5-88 Bridge Welding Code. Non-destructive testing was performed on all critical weld joints.

The contractor fabricated 13 full bridge width orthotropic sections 7'-0" deep by 35'-6" to 37'-6" wide, with lengths varying from 123 feet to 219 feet per section. The sections ranged in weight from 250 tons to a maximum of 459 tons. All sections were shipped with a steel orthotropic deck and with the installed steel barrier rails. The total weight of all fabricated steel equaled 5014 tons (see Table 2 below).

Table 2 - Orthotropic Bridge Section and Barrier Statistics					
	A1	**A2**	**A3**	**A4**	
Section Length	156'-2 1/16	200'11	218'-9 11/16	146'11 7/16	
Section Weight	726,288 lb.	829,471 lb.	917,363 lb.	628.958 lb.	
Barrier Weights	125,040 lb.	138,701 lb.	151,464 lb.	101,163 lb.	
Total Weight	**851,328 lb.**	**968,172 lb.**	**1,068,827 lb.**	**730,121 lb.**	
	A5	**A6**	**A7**	**A8**	
Section Length	146'-11 3/8	122'-9 15/16	130-11 7/8	190'-7 9/16	
Section Weight	697,025 lb.	538,133 lb.	499,351 lb.	723,307 lb.	
Barrier Weights	103,455 lb.	83,623 lb.	117,649 lb.	93,535 lb.	
Total Weight	**800,480 lb.**	**621,756 lb.**	**617,000 lb.**	**816,815 lb.**	
	A9	**A10**	**A11**	**A12**	**A13**
Section Length	151'-8 3/8	137'4 1/8	148'-1 15/16	181'-4 1/16	132'-2 7/8
Section Weight	653,549 lb.	571,061 lb.	570,110 lb.	707,155 lb.	545,865 lb.
Barrier Weights	76,996 lb.	68,549 lb.	71,085 lb.	89,247 lb.	67,010 lb.
Total Weight	**730,545 lb.**	**639,610 lb.**	**641,195 lb.**	**796,402 lb.**	**612,875 lb.**

The Bethlehem Steel Corporation supplied all the steel plate for the superstructure. The combination of their 110" and 160" plate mills were capable of the full range of plate sizes required for this bridge.

The bridge sections were fabricated by USI and barged down in thirteen sections to Oakland, California.

<u>MIDNIGHT ERECTION PROCESS IS VERY DRAMATIC</u>

The bidders were given the option to fabricate and erect the bridge in sections that would allow the maximum number of bidders and fabricators. A minimum section size was specified on the contract plans. The smaller the sections the more field bolting that would be required. The smaller pieces, however, allowed smaller steel fabricators to bid on the project. The successful low-bidder was the general contracting joint venture of Kiewit/Marmolejo. Shaughnessy and Company's steel subcontract made them responsible to supply the equipment and trained operators to lift and position all the steel sections. Kiewit/Marmolejo's assignment was to assist with the jacking, do the traffic control coordination with Caltrans, and perform all of the work associated with making the splices and preparing the bearings. A creative solution for the installation was conceived by Bob Murphy, Heavy Haul Manager using a "SHLHP" (Special Heavy-Lift Hydraulic Platforms) which consists of two self-propelled hydraulic platforms braced in tandem with a strut beam. Shaughnessy and Company (steel erectors) owned a construction division of Crowley Marine Services, Inc., formerly an arm of Oakland-based Crowley Maritime Corp. during the time of the construction. The steel fabricator Universal Structural, Inc. had the capacity to built the 13 full bridge width (35.5 foot) sections as one welded unit "arc" shaped pieces as itemized in table one. A scale model was built to try out the various methods to pre-plan the most effective use of the "SHLHP".

The steel plate was ordered on April 4, 1995 and fabrication was from June 6, 1995 through May 24, 1996 (10). On November 30, 1995, loading began from the manufacturing site on the north banks of Columbia River adjacent to Portland, Oregon. The first three of 13 bridge sections were loaded on a 20-line Scheuerle platform trailer. A transition ramp was utilized between the barge and the USI dock (10). Three barge trips were conducted using a 400-foot long by 100-foot wide barge, one of the largest available in the United States. Crowley shipped the sections on this barge from the Universal Structural, Inc. fabrication facilities Vancouver, Washington to Oakland, California.

A 20-line, two-file Scheuerle hydraulic trailer with a custom-designed spacer frame was used to off-load the girders at Oakland. The frame between the axles distributed the load over a wider path to meet structural requirements on the dock. The Scheuerle hydraulic trailers have two unique features. First, each pair of tires can be individually steered. Second, each pair has a hydraulic cylinder that allows the pair of tires to move up and down (over uneven surfaces) to a maximum of 18 inches, while maintaining uniform loads across every pair of tires.

The girders were set on temporary cribbing in the staging area. The off-load sequence and staging locations required careful planning since the girders could not pass each other in the staging area once they were off-loaded. Therefore, each section was moved three times: first from USI fabrication facility to the barge, second from the barge to the staging area beside the I-80 freeway and finally erected. Each section was step jacked to a height on cribbing very close to final bridge elevation.

The final transport vehicle for the girders, the "SHLHP" consisted of 24 lines of Scheuerle hydraulic trailers which were assembled into two six-line, four-file platforms. Two hydraulic support towers on an adjustable frame mounted on hydraulic powered sliding turntables. The "SHLHP's" ability to make various motions allowed the girder to move on six axes.

Hydraulic telescoping draw-bars were designed and fabricated to give fine control of the fore and aft motion of the platform trailers and eliminate jerking of the equipment as the prime movers shifted from forward to reverse. Smooth adjustments for lining up the bolt holes was provided by these draw-bars.

The girder was jacked up then 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 side slope of up to 12 percent. The hydraulics of the "SHLHP" proved capable of making smooth adjustments of as little as three-sixteenths of an inch.

The center of gravity shifted as much as ten inches during the tilting operation which was taken into account when selecting the location of the transports. To verify the proper positioning, pressure readings were taken.

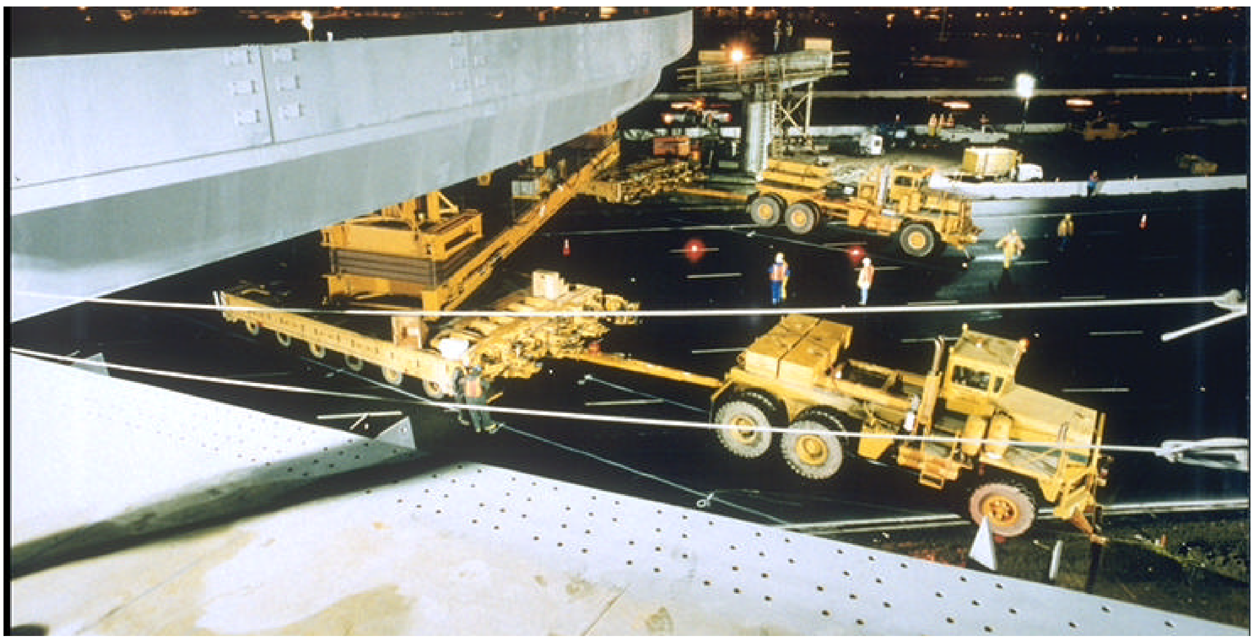

Figure 14 "SHLHP" (Special Heavy-Lift Hydraulic Platforms) pull a 450-ton all steel orthotropic section across I-80 (Note bolts holes in top deck of previously erected section)

Closure of I-80 adjacent to the Toll Plaza for the SFOBB (San Francisco-Oakland Bay Bridge) were required for setting three of the 13 sections. Approximately 500,000 vehicles cross over the SFOBB each day. The sections were staged on the east side of the freeway and crossed over during night erection. This could only take place in a ten-hour window beginning at midnight on a Saturday night, as shown in Figures 14, 15 and 16. A stiff financial fine was stipulated for each minute exceeding this time limit by the contractor. This fine would be paid to Caltrans. The first section was erected at the west abutment and allowed the team to practice for the erection over the freeway. This gave the contractor, Caltrans and the California Highway Patrol confidence that things would go according to plan.

The hydraulics of the "SHLHP" worked smoothly to position the section and lower it onto the steel bearings anchored to the top of the concrete pier. The section over I-80 westbound was erected about midnight Saturday, February 3, 1996 (see Figures No. 14, 15 and 16). The section over eastbound I-80 was erected the following Saturday night.

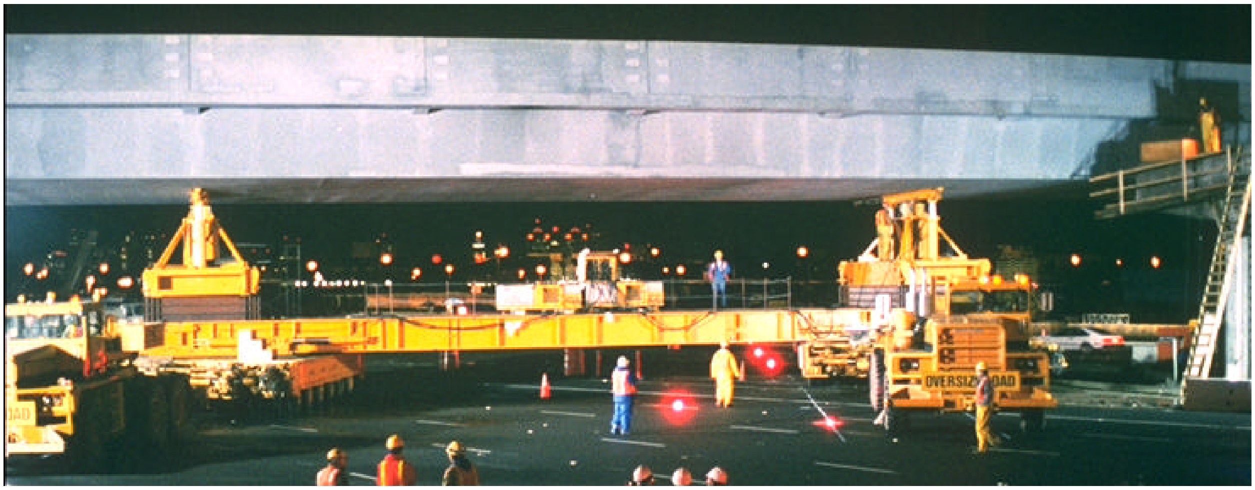

Figure 15 "SHLHP" (Special Heavy-Lift Hydraulic Platforms) pull a 450-ton all steel orthotropic section across I-80 (Note worker preheating bearings for field welding---right side of figure) (Note painted white line to guide truck drivers)

Figure 16 Ironworkers wait in position to install about 2,000 bolts to connect the second and third sections together. Before the "SHLHP" could be released some 500 of these bolts had to be torqued (A minimum of 25 percent)

Surveyors called out offsets as the "SHLHP's" six degrees of hydraulic-activated controls aligned the bolt holes and set the other end for grade and station. Before the first bolt could be secured, all holes had to be aligned. The bolting process began less than two hours from the start of the operation and was completed in another three hours. The girder placement and bolting procedure totaled six hours which was well under the ten hour minimum. Once 25 percent of the bolts were installed plus the field welding of the section to the bearings, then the dead weight of the section was transferred to the splice and bearings. Finally the "SHLHP" was driven away. The "SHLHP" have been used to hoist 11 story structures into place at Alaskan oil fields. "This is not a test," Simpson said. "It's actually something we've done many times." (22). The sections were field bolted together. "Some pieces weigh up to 500 tons," said Crowley spokesman Dick Simpson (22). More erection information is available in references and the remaining ten sections were not located over I-80 freeway lanes and were erected during the daytime. After the 13 sections were erected, the joint seal assemblies were installed and an epoxy asphalt overlay wearing was placed prior to opening the bridge to traffic.

COMPLETED BRIDGE - PARTIALLY COMPLETED I-880 PROJECT

Harry Yahata, Caltrans District Director for the Bay Area, said on July 23, 1997, "I'm sensitive to the next-of-kin of the 42 people who lost their lives on I-880. A ribbon-cutting is a celebration. I don't know that a celebration is appropriate." (24). The five mile freeway link, connecting the Nimitz Freeway directly to the Bay Bridge Toll Plaza, opened for business by 5 a.m. Wednesday, July 23, 1997, in time to ease the pressure of the morning commute, Caltrans said Tuesday (25). The section of freeway opening Wednesday was only the first link of the I-880 Replacement Project (25). When completed, the freeway section is expected to carry 118,000 cars a day. KCBS traffic anchor George Rask said "This is major, this is a big deal. Ideally, what it should do is eliminate a lot of the backup on westbound I-580 to the Bay Bridge and improve traffic flow coming from the northbound Nimitz Freeway." (25). By the summer of 1998, Caltrans is expected to finish the final link of the I-880 Project, connecting I-880 with eastbound I-80. Ramps at Union and Adeline streets are scheduled to open by January 1999 (26).

LESSONS LEARNED

- Steel piling corrosion did not occur after 35 years though some minor pitting did occur.
- Pile "load indicator program" eliminated piling driving claims and established pile tip elevations for seismic loading.
- Largest pipe piling used was 42-inch diameter. 12-foot diameter pipe piles are planned for the Richmond-San Rafael project.
- Although the steel orthotropic steel bridge system is complex, it is a buildable system. An orthotropic superstructure is under final design for the Carquinez

Suspension Bridge to replace the 1924 Carquinez Cantilever Truss Bridge. The orthotropic superstructure is one of eight currently under study by the San Francisco-Oakland Bay Bridge design team.

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Figure 17 ^^Port of Oakland^^ ^^San Francisco Bay^^ ^^"HS" Line^^

ORTHORTOPIC BRIDGE STATISTICS

Official Name: Maritime Off-Ramp CALTRANS Bridge Number 33-0623S
Nickname "HS" Line or "Horseshoe-Line"
Width: 36.2 feet Length: 2356 feet Deck Area: 85,287 square feet
Cost: $22.3 million Pounds of Steel: 10 million
Installed cost steel per pound: $1.34 lbs of steel / sf of deck: 117 psf
bridge cost per square foot: $262 steel cost was 60 percent of the total

CREDITS:

Owner: Caltrans (California Department of Transportation), Sacramento, CA
Engineer: ICF Kaiser Engineers, Inc. and Construction Group, Oakland, CA
Fabricator: Universal Structural, Inc., Vancouver, WA
Erector: Shaughnessy and Company, Auburn, WA
Lifting Specialist: Crowley Maritime Services, Inc.
Detailor: Candraft Detailing, Inc., Port Coquitlam, British Columbia
General Contractor: Kiewit-Marmolejo (A Joint Venture)
Structural Steel Plate: Bethlehem Steel Corporation, Burns Harbor Division, IN

Figure 18 At centerline I-80 to NE

Figure 19 At centerline I-80 to SW