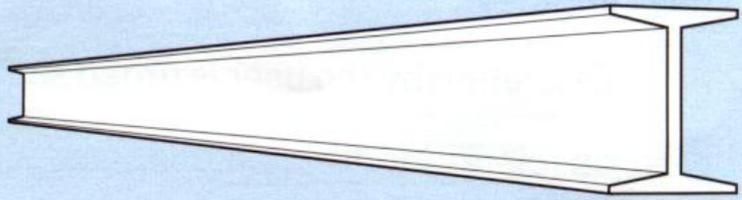


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TECHNICAL INFORMATION & PRODUCT SERVICE

June 2007

EARLY CALIFORNIA ACCELERATED STEEL BRIDGE CONSTRUCTION

By

**Jay P. Murphy
President
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In recent years the U.S. Federal Highway Administration in conjunction with state highway departments and universities has had a series of conferences on what they call accelerated bridge construction. This is in anticipation of federal funding throughout the country mandated by the Safe, Accountable, Flexible, and Efficient Transportation Equity Act (SAFETEA) of 2003 and will apply to the planned replacement or retrofit of bridges throughout the country originally constructed in the early 1950's and 1960's as part of the interstate highway system. A significant number of these are in California and Nevada. A review of the material presented at these conferences shows a strong similarity between the methods suggested for retrofit and replacement and the methods used to fabricate and erect massive west coast orthotropic box girder bridges in the 1960's and 1970's.

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ACKNOWLEDGMENTS

The publication of this report was made possible in part by the support of the Structural Steel Educational Council (SSEC). I would also pay tribute to my mentors who guided me through the construction of these bridges, in particular ironworker superintendent William Choate, also known as the “Big Okie,” who was the field superintendent on all of the structures discussed. Bill and his riveting gang, like many others in his craft, were known as boomers and traveled to wherever work was available. In our case it was on a small truss bridge across the Sacramento River in Northern California. When told that we had sufficient help he offered to work one day for free, and if his gang drove more rivets than ours, we would hire them. He did and we did. I would also pay tribute to my father, J. Philip Murphy, who upon his death in 1970 was awarded the title of Pontifex Maximus – Master Bridge Builder – by the American Institute of Steel Construction (AISC).

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**PRESIDENT
MURPHY PACIFIC CORPORATION**

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1. Introduction

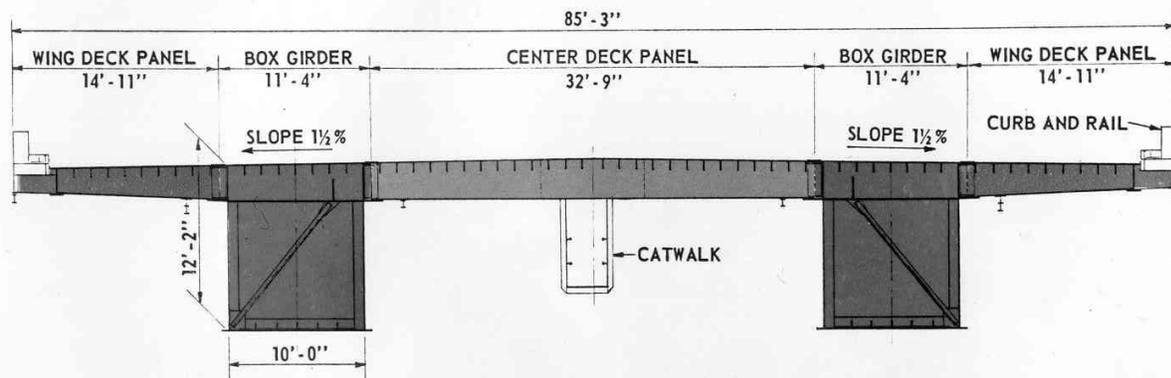
In the early 1950's the U. S. steel mills began producing a wide range of higher strength weldable steels. This coupled with the rapid development of automatic welding processes gave bridge designers wider latitude in type selection and configuration of their structures for incorporation into the recently enacted Federal Interstate Highway Program.

Many state highway departments elected to transform the weight and dollar savings obtained by these developments into more aesthetically pleasing structures. In California particularly the emphasis was on longer-span steel plate girder bridges rather than on classic truss bridges.

Then by adapting the orthotropic deck plate theory first developed for autobahns built in Germany prior to World War II, box girder bridges such as the San Mateo-Hayward Bridge, recipient in 1968 of both the American Society of Civil Engineers' Outstanding Civil Engineering Achievement Award and the American Institute of Steel Construction's Most Beautiful Long Span Bridge Award, were fabricated and erected. In orthotropic bridge design a longitudinally stiffened steel deck plate is used as a structural member in addition to carrying the traffic load as contrasted to a floor beam and stiffener system supporting a concrete deck. This results in a significant dead load reduction. Subsequently the San Diego-Coronado Bridge, Poplar Street in St Louis, Queensway in Long Beach and the Fremont Bridge in Portland, Oregon were constructed. At that time these were the only five long-span orthotropic bridges existent in the United States and four of them were fabricated and erected by Murphy Pacific Corporation.

2.) The San Mateo-Hayward Bridge

This bridge is located across a lower arm of San Francisco Bay between the cities of San Mateo and Hayward. The length of the steel superstructure is 9,650 feet and consists of 37 spans grouped symmetrically around the navigation channel. The central group of spans is referred to as the channel span and consists of a 750-foot main span and two 375-foot back spans. At the time of construction the main span was the longest girder span in the western hemisphere and the third longest in the world. The vertical clearance at the navigation channel is 135 feet. On either side of the channel spans are seven 292-foot approach spans. The 292-foot and channel spans constitute that portion of the bridge having an orthotropic deck, a total length of 5542 feet. All of these spans are erected upon steel towers.



TYPICAL CROSS SECTION - 292 FOOT SPANS

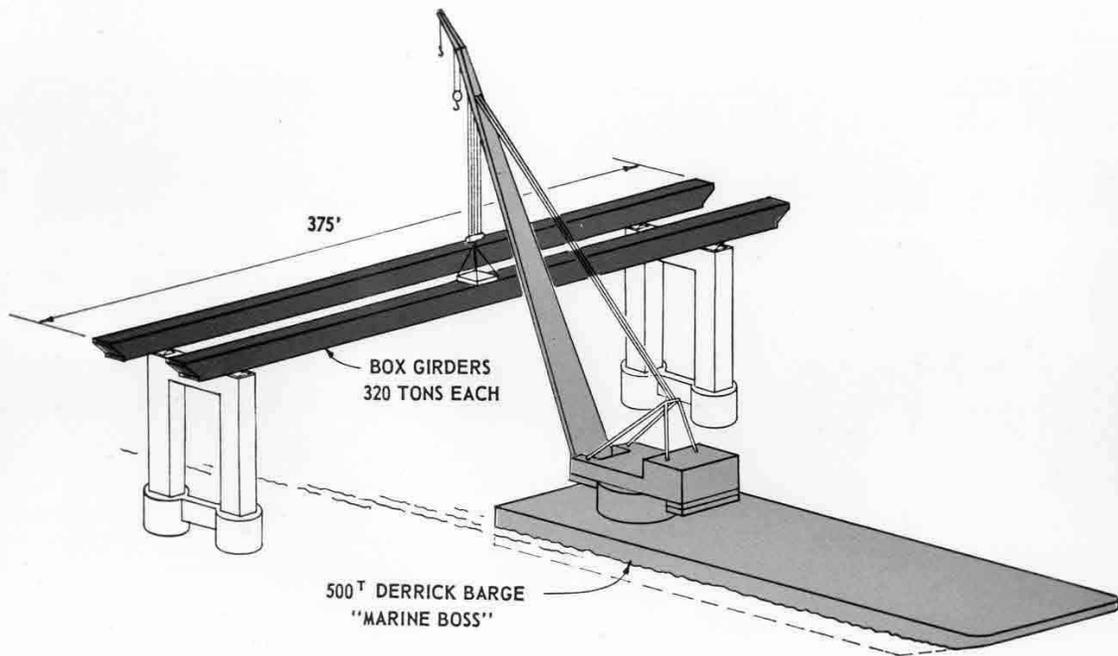
Figure 1 Typical Cross Section, San Mateo-Hayward Bridge

Figure 1 shows a typical cross section of the 292-foot spans. The essential elements are parallel box girders approximately 12 feet deep and 10 feet wide; a central orthotropic deck panel to which is attached a catwalk and maintenance platform; and cantilevered wing deck panels on either side of the bridge with steel curbs and railings. Total width of the bridge is approximately 85 feet. The same configuration remains true for the cross-section of the channel spans except that the depths of the box girders increases to 30 feet over the main piers. The box girders although appearing continuous, are divided by hinge connections into anchor spans that are supported by, and cantilever out at both ends over, two adjacent towers; and the suspended spans are supported by the ends of the anchor spans. As a matter of record post-Loma Prieta earthquake analysis indicated that the structure performed quite well.

3.) Fabrication Yard Assembly Procedures

When making a bid analysis for this types of structure it becomes readily apparent that one has to analyze the fabrication and erection of massive box girders. Erection of these box girders can be

simple and safe if, as in this case, they are structurally capable of carrying their dead weight without participation of the orthotropic deck panels, and if one has lifting equipment of sufficient capacity. The procedure would be first to erect the anchor spans, second to hang the suspended units, and finally to fill in the deck units.



STAGE ONE. ERECT ANCHOR SPAN BOX GIRDERS

FINAL ERECTION SCHEME - 292 FOOT SPANS

Figure 2 Final Erection Scheme Showing the Marine Boss

However when bids were opened in October of 1964 available erection equipment was limited to 150-ton capacity tower or shear-leg derricks. This type of equipment would limit the erector to a longitudinal girder dimension of 122 feet to come within the allowable hook capacity of 108 tons. Obviously temporary support towers (falsework) would be required. This is not economical or safe erection. These bridges are located over navigable water with subsequent exposure to ocean going traffic which operates-and not always successfully-under adverse weather conditions such as wind and fog. I cite as examples the collision by a string of river barges with



the falsework for the Poplar Street Bridge, resulting in a year's delay in completion of the structure. Accidents in California to the existing Antioch and Rio Vista (center photo above) Bridges caused closures of up to six months. We as erectors therefore elected to construct the derrick barge known as the Marine Boss in order to assure that erection would proceed simply, safely and rapidly. The hull dimensions are 300 by 90 feet with a depth of 22 feet. The derrick was fully revolving at 500-ton capacity; straight up over the stern we could lift 700 tons. The barge also had cargo-carrying capability as shown above working at the San Mateo-Hayward Bridge (left and right photos above). At San Mateo the center deck sections stored on the back of the barge are doubling as a counterweight for the pick (figure 3).



Figure 3 the Marine Boss Shown Erecting a 320-Ton Box Girder at the San Mateo-Hayward Bridge

The next step when one has this type of equipment is of course to fabricate large erection components. Although most fabrication shops in the United States at that time were able to accomplish such work, dimensional and weight limitations on highways and by rail were such that massive components could not be shipped to the jobsite. Therefore development of an assembly area serviced by rail and truck, but with deep water access, wherein the erector can receive shop-fabricated pieces and assemble them into erection units for barge shipment to the site would significantly reduce the cost of this type of construction. The area so selected for this purpose was located in Richmond, California—a towing distance of 30 miles from the bridge site. As shown in Figure 4 it encompassed 73 acres with a complete underground utility system including electrical, natural gas, compressed air and liquid oxygen. Three mobile cranes of more than 100-ton capacity each were required to service the work. Fabrication flow was from both left and right along transfer tracks to the perpendicular load-out track where large floating equipment would place erectable pieces onto barges for shipment to the site. Painting facilities

are located to take advantage of natural fabrication sequence breaks for application of the prime coat.

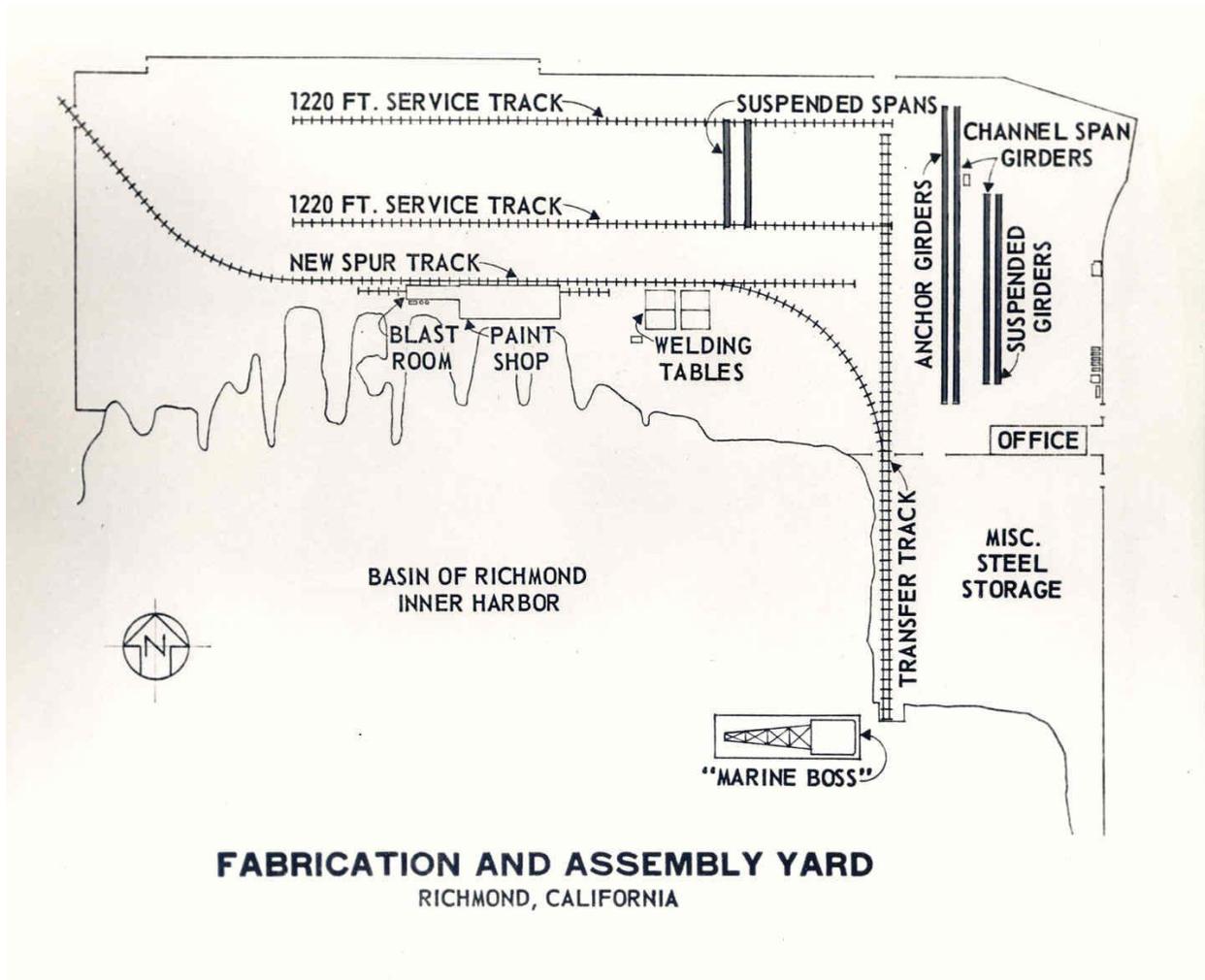


Figure 4 Fabrication Facilities in Richmond, California

The specifications called for a protective coating consisting of:

1. One coat of inorganic zinc paint.
2. One coat of vinyl wash primer.
3. Additional coats of red iron oxide vinyl primer.
4. One or more coats of a mixed vinyl paint containing 75% of red iron oxide and 25% of aluminum finish coat.

The total dry film thickness of the above system was specified to be not less than 6 mils and required blast cleaning so that a proper anchor pattern for the inorganic zinc would be obtained.

Figure 5 Interior of Grit Blast Room



Upon receipt of shop-fabricated subcomponents they were cleaned and painted. They would pass first through a drying room and then into an enclosed grit blast room (figure 5). Special trucks and trackage were provided for the movement of sections into this facility. The interior of the room shows the catwalks from which workers in protective clothing clean the sections. All equipment including the air supply for the workers is located in a passageway on the left and all personnel are in telephonic communication with the paint foreman. Excess grit will drop through an open grate floor to collection hoppers where by means of screw conveyors and elevators exterior to the blast room the grit is purified and recycled into the system. After cleaning the fabricated component receives the first coats of paint. Protective coating for all these structures, including the contact or faying surfaces of high strength bolted connections, consisted of a prime coat and required blast cleaning such that a proper anchor pattern for the inorganic zinc would be obtained. For safety reasons the interior surfaces of the box girders received the complete paint system prior to fabrication and erection. The finish coat on the exterior surfaces was applied after erection. After painting the sections they were stored in the yard prior to assembly.



Figure 6 Yard Assembly of the Orthotropic Boxes.

An all-welded splice of the flange and web plates along with the longitudinal stiffeners was made to join the boxes into erection lengths. In their proper erection sequence the boxes were placed upon timber cribbing (figure 6) with extreme care given to location. By proper sequencing of welding distortion was kept to allowable minimums. Welding shelters were erected for protection from the weather, and as people were of necessity working inside the girder adequate ventilation was provided. Various processes were used for this splice depending upon the geometry and frequency of the joint. After completion of the girder splices the top flanges were trimmed and brackets for the floor-beam connections were located at 10-foot, 5-inch centers along the axis of the bridge. Proper location of these pieces was extremely critical for success of the erection procedure.

Upon completion of this phase of the work the girders were then moved laterally along two parallel tracks by means of special trucks with built-in jacks to an area where the span was

completely assembled and the holes for field connections reamed out full size. Originally the contract's special provisions called for the deck sections to be field welded as the design authority could not find any wearing surface thick enough (2-inches) to cover the heads of a bolted connection. Shortly after contract award, a local firm known as Adhesive Engineering developed an epoxy asphalt coating. After field testing this material was found to be acceptable. From the erection stand point field bolting is preferable to welding due to the continual thermal contraction and expansion of the deck plates caused by San Francisco Bay's heavy morning fog and ensuing bright sun. A no-cost change order was issued to permit bolting the field splices. This bridge and all the aforementioned were 100% field bolted. In the case of the suspended units they were swung to their actual erection condition position from hanger frames. The anchor spans were set on the actual bearing assembly upon which they will rest in the field. To simulate field erection conditions the weight of one suspended unit was jacked into one end of an anchor girder. Concurrent with the proceeding operations the center and wing decks were in the process of shop fabrication and yard assembly.

Figure 7 shows a typical orthotropic center deck section as viewed from the underside of the bridge, consisting of the deck plate, longitudinal rib stiffeners, and transverse floor beams. Deck sections with ribs attached for any particularly center deck were shop-fabricated and sent to the yard for assembly. Three of these pieces were required for a full width of deck. Keyhole slots were shop punched in the floor beams. The pieces were then placed onto an assembly table and located with the help of jacks. After fit up, the floor beams were tack welded and the longitudinal seams welded from one side.



Figure 7 Deck Plate Assembly



Field Connection Drilling



Box Girder Load-Out

Ironworkers then positioned the table to take advantage of down hand welding of the floor beam to rib joint. These tables were capable of being rotated in either direction. They were then flipped completely over and the longitudinal seam weld completed. Completed center decks were then brought alongside the girders in the assembly area and special lifting lugs attached. They were then placed in the final position resting on temporary shoring. Using tape drilled splice plates as templates workers drilled the field connection bolt holes full size by using toggle bugs.

Upon completion the span was match marked, disassembled and moved laterally along the transverse trackage on trucks with built-in jacks to the load-out track and then to the load-out dock where the Marine Boss placed it on a barge for shipment to the site. A special lifting device was developed that relied upon the lifting lugs picking off the welded floor beam brackets. It also had provision for a quick release pin connection so the frame could immediately be attached to lifting lugs on the next girder in order to cut down derrick time.

4.) Erection Procedures

Erection of the 292-foot spans posed no particular problem due to the capacity of the Marine Boss and yard preassembly. Our erection procedure for all of these called for picking the south anchor arm of any one span first. Due to the proximity of the old bridge and a high-voltage transmission line it was necessary to do all erection from the north side. The girder would be picked off the barge, swung 90 degrees, and then lifted to clear the support towers. Utilizing deck winches and a six point anchor system, the derrick barge was then moved between the piers and lowered the girder onto the bearing assemblies. This was to avoid booming out and thereby limiting rig capacity. The derrick had three hooks: main load 700 tons, auxiliary 75 tons, and a runner line of 25-ton capacity. The main hook height at a 75-foot radius was 215 feet. A console on the barge controls the six point anchor system. The derrick barge itself has a crew of three: crane operator, deck winch operator, and an oiler. Raising gangs of structural iron workers hook on the pieces under the direction of the erection foremen. Power was supplied by two 525-horsepower diesel engines. After setting the south girder, workers erected the north girder in the same manner.

The Marine Boss was further utilized as a cargo barge carrying two suspended span box girders and all the deck sections for two spans. On the deck sections were placed the kegs of erection bolts, fuel for compressors, and the splice plates. After erection of one center deck piece to tie the north and south girders together the suspended girders could be placed. In order to assure equal loading on the permanent hanger bars they were landed on special jacking frames each with 200-ton capacity jacks bearing upon the top flange of the anchor arms. Dimensional adjustment was made to assure equal loading before the lower pin connection keeper plates were drilled and bolted. The remainder of the center and wing decks were then erected, and bolted up from special scaffolds hung from the permanent maintenance platform traveler rails. We were able to erect two such spans on the average of ten working days- a total of 1900 tons, or 600 lineal feet, of completed bridge.

Due to their massiveness it was necessary to build the four channel spans girders from the ground up in the assembly area. The 12-foot wide bottom flange of the box girder was supported

upon timber cribbing and pile bents. The deepest section of the girder which would rest upon the main pier is 30-feet. The far side web was placed, and then transverse stiffening plates and a bearing diaphragm were erected. The bottom flange itself consists of four 12-foot wide $\frac{3}{4}$ inch plates which are stitched together with high-strength bolts. All plates were cold formed in the shop and holes drilled $\frac{5}{16}$ of an inch undersize. This was required for good fit due to the vertical curvature of the bottom flange and necessitated a good deal of reaming. A typical web placement required the use of three 100-ton capacity cranes. Temporary shoring was placed until the other web was erected and the diagonal bracing bolted up. This is one of the few areas in this structure where rolled shapes were used, in this case as longitudinal stiffeners. Provision was made in this 600-foot, 1100-ton channel span anchor arm for two bolted splices to break it down into erection pieces. This girder was too big to be handled in one piece. Further, due to the size of these members it was determined that preassembly with the deck plates in the yard would not be feasible. The unit therefore was broken down into three pieces and moved laterally on portable trackage to the loading track with the same system of trucks with built-in jacks.



Figure 8 San Mateo-Hayward Bridge Erection Photos

One bent of falsework was required for the erection of each anchor span. This consisted of a 130-foot steel tower section resting on steel pipe piles 18 inches in diameter. Using a tubular frame as a template, pipe pile and follower combinations 200 feet long were driven to suitable strata. The steel tower was capped with a 42 inch deep plate girder upon which were placed four 200-ton capacity jacks for adjusting the girder. The first piece to be placed was the haunch girder extending shoreward from the main pier to the falsework bent. After its matching girder was placed the two were tied together with center deck sections. The same procedure at the site of lifting, swinging, and moving into position with the south girder first was followed. The second piece weighs 246 tons and extends from the falsework tower to the last pier of the 292-foot spans. This piece remained on the load for $4\frac{1}{2}$ hours until 25% of the splice was bolted up; which amounted to 500 high-strength bolts. The last 292-foot suspended span could then be erected.

Prior to erection of the third or cantilevered section of the anchor span, all center and wing decks had to be erected and 50% of the bolted connections made. Sequencing of this work was quite critical in that unlike the 292-foot spans, the channel span girders are not capable of carrying the dead load of the deck sections without their structural participation. Again using the splice plates

as templates all holes were drilled full size. It was also necessary to support the cantilevered wing decks while drilling and bolting. This was accomplished by the use of 17 ½ foot high tie back masts of 5 inch diameter pipes (figure 8). Erection of some of the wing decks on the south side required the most critical lifts on the job in terms of derrick capacity. Due to the higher elevation of the channel span it was not possible to use the main load block of the Marine Boss. These 15-ton sections were set with the 25-ton auxiliary hook reaching out over the width of the bridge itself. The third girder section to be erected was the 272-ton segment cantilevering 187 feet out into the shipping channel. Counterweight was placed on the girder to compensate for the vertical curve of the bridge. Our erection engineering consultants, Earl and Wright of San Francisco, specified that this piece had to be held in place until 50% of the bolts in the splice were installed. For this connection that amounted to 1416 bolts. The girder section had to be held at constant elevation for 6 ½ hours in order to install that many bolts. The crane operator had to make adjustments as the tide changed. These adjustments had to be made through the 24 parts of main load line. Upon completion of the work all deck panels were then completely bolted utilizing special scaffolds. As this work was being accomplished a 150-ton uplift force was applied to the cantilevered section by the Marine Boss to relieve dead load stresses in the box girder until the deck sections could assume their portion of the load. The same procedure was then repeated on the other side of the channel.

In erecting the two 375-foot, 550-ton channel span girders, tie-back towers were required to avoid overstress in the girders while erecting the deck panels. The tower sections of the falsework were used as masts for this system. Pad eyes were welded to the top flanges of the box girder and pinned to jacking frames, each containing two jacks. The four frames exerted a combined upward pull of 890 tons on the center decks. Utilization of this erection scheme was highly approved by the design authority as it enabled them, by means of comparing deck elevations before and after center and wing deck erection, to verify the participation of the orthotropic deck system in the load-carrying capability of the finished structure. The suspended unit was fabricated from the ground up in the assembly yard in the same manner as the channel span anchor arms. It was possible to assemble the complete span eliminating the need for field drilling of the transverse and longitudinal deck splices. Field erection basically followed the same sequence except that the derrick barge did not swing with load due to the weight of these pieces. An alternate procedure was utilized in that the girder was picked off the cargo barges, the barges were moved out of the way by tugs, and the Marine Boss was then winched into position. When the girder was positioned it was then lifted and placed upon hanger frames in the same manner as the 292-foot suspended units. Both girders complete with one center deck were erected in an eight-hour working day. This totaled slightly less than 1300 tons. Upon completion

of deck erection the tie back system was dismantled. The deck was then cleaned and the epoxy asphalt wearing surface applied.

5.) The San Diego-Coronado and Queensway Bridges.

The design of the 1968 San Diego-Coronado Bridge has all the essential elements of orthotropic design but presented a different set of challenges for the fabricator. The bridge itself, having a total length of 7422 feet, utilizes orthotropic box girders only for the three channel spans, two of which have a horizontal clearance of 600 feet; the third is 500 feet. Vertical clearances were 200 and 180 feet, respectively. Unlike the San Mateo-Hayward Bridge the orthotropic span is continuous over its entire length. Another significant difference is the configuration of the cross section. A single box girder 25 feet by 34 feet in cross section was utilized with wing decks cantilevering 15 feet 4 inches on either side. This span weighs approximately 2½ tons per lineal foot. Due to the extreme width and depth of flanges and webs for these units all fabrication with the exception of internal bracing was accomplished in the assembly area. Mill-received material was trimmed and spliced in the flat position utilizing automatic submerged arc welding equipment.



Figure 9 Movable welding shelter building

For protection against inclement weather lightweight 50-foot by 100-foot buildings mounted on rails were positioned over the work during welding (figure 9). When welding was complete the buildings were moved clear and large crawler cranes moved the pieces to their proper assembly area. Four of these buildings were utilized in the course of fabrication. Assembly of the box girders followed the same methods utilized for the San Mateo-Hayward channel span with the exception that the wing decks were attached to the girder in the yard for drilling to full-size bolt holes before disassembly and barge shipment to the site. Due to higher elevation of this bridge it was necessary to extend the boom of the Marine Boss (figures 10 and 11).



Figure 10 Erecting San Diego-Coronado Bridge



Figure 11 Marine Boss erecting San Diego-Coronado Bridge

Due to the increased weight per foot of the individual units, main hook capacity was reduced to the point where the heaviest permissible lift was 220 tons, resulting in a longitudinal dimension

Early California Accelerated Steel Bridge Construction by Jay Murphy June 2007

of girder erection piece ranging from 60 to 90 feet. Obviously falsework was required, and a balanced system of cantilever erection was used. Due to the utilization of procedures and equipment previously described, erection of these pieces proved to be no problem. Each field splice required approximately 5000 high-strength bolts and they were installed and torqued up in less than six hours.



Figures 12 and 13 – The Queensway Bridge

Fabrication of the Queensway Bridge in Long Beach (figure 13) followed the aforementioned procedures with the exception that the entire 1200-foot superstructure was completely assembled in Richmond. It was then broken down into 30 sections for barge shipment; the largest weighed 617 tons. Six segments or 40% of the total were installed on the harbor side during the last ten days of January, 1970 (figure 12). An identical installation on the north side was completed during the first week of May. This left only the final two 290-foot girders. Each of these weighed 617 tons and they were transported on a separate cargo barge. By May 26th both spans were erected. Total erection time was 20 working days.

6.) The Fremont Bridge

In what was termed by state highway officials as one of the most ambitious projects in the history of bridge building, a 6000-ton bridge section was raised 170 feet above the Willamette River in downtown Portland, Oregon, in 40 hours. Known as the Fremont Bridge, at the time of construction it was the world's longest steel tied arch with a main span of 1255 feet and two side spans 488 feet long. Erection of the side spans was a major project in itself. Members were erected from the ground by 200-ton crawler cranes and from the river by the Marine Boss. Two falsework bents were required and a gantry mounted on the top flange of the girders for horizontal transport of steel members.

We were given a choice of three methods to erect the center span: falsework towers in the river, tie backs from the side spans, or lifting the pre-assembled span. Falsework was ruled out because of interference with river traffic. A cable tie back system would have required extensive strengthening of the side spans to take the construction loads. Travelers would have to be placed on the structure itself because the Marine Boss even with its 285-foot boom installed could not reach the top portion of the arch. Working with consulting engineers Earl and Wright, our field ironworkers recommended the use of a hydraulic jacking system. This would consist of eight 200-ton center hole jacks at each corner of the cantilevered arch sections. Four inch diameter threaded rods coupled together in 24-foot lengths extended through these jacks to frames in the span. Universal joints on both the upper and lower frames eliminated bending loads on the rods. Working in two-foot cycles, as a jack thrust up against a bearing nut, pulling the rod up, an ironworker screwed a nut down against the frame below the jack (figure 14). The lower nut took the load as the jack was lowered for the next thrust. The spherical nuts were split down the middle and held in place by collars so that they could be removed to permit rod couplings to pass up through them and the jack. Couplings were staggered so that they would pass through adjacent jacks at different times.



Figure 14 Jacking frame with threaded rods and split nuts



Figure 15 Fremont Bridge arch span being floated on barges

For purposes of assembling the erection pieces a 22-acre site with 2000 feet of river frontage one mile downstream from the bridge site was leased. Components that had been shipped by barge from Richmond were assembled into erection units. In the case of the center span this required driving nine bents of pipe piling spaced 110 feet apart. After assembly the arch span was transferred from the temporary support bents to the Marine Boss and another derrick barge, “*Elizabeth*”, which mounted a 4600 Manitowoc (figure 15). Timber cribbing was installed on each barge deck to distribute the loads.

Approximately 5000 tons of water ballast was added to each barge so that they could float under the span. Ironworkers then pumped the water out until the barges were supporting the full weight of the span. This required keeping track of the receding water depth in the ballast tanks with sounding lines so that the load was assumed evenly by the barges. They then moved to the center of the river by hauling on their own anchor lines. At that point six tugboats with total hauling capacity of 7600 horsepower took over to push to barges upstream to the site. The number of tugboats required was determined by an analysis of wind speeds for the previous ten

years. Earl and Wright determined that the towing and anchoring system should be planned to operate against a 40 mile per hour wind. The trip upstream was made with the arch parallel to the banks of the river. One tug pulled on the span, two pushed the forward barge and the other three pushed on the rear barge. As the span passed through the approach structures, anchors and mooring lines were dropped off. When the rear barge was in position, the tugs backed off and allowed the current to swing the lead barge into position near the opposite shore. Final alignment was attained with the aid of the tugs and by hauling on a total of 1¼-inch anchor cables.

After erection crews lowered the rods and attached them lifting began. A console on top of each side span operated the hydraulic power unit for the jacking cylinders. Various safety devices were in place to control hydraulic oil temperature and power surges. The jacks worked in two-foot strokes at a rate of approximately four feet an hour without a break for 40 hours until the pre-drilled connection holes were lined up and field bolting accomplished. This event was listed in the Guinness World Records Book for many years as the world's heaviest lift.



Figure 16 Erecting the Fremont Bridge

7.) Modern Accelerated Bridge Construction Examples

The Alfred P. Zampa Memorial Bridge across the Carquinez Strait of the Sacramento River was opened to traffic in the year 2003. The suspension bridge design, with an orthotropic deck system, eliminated the need for a center pier, thereby providing obstruction-free travel for the oceangoing vessels utilizing this channel. The bridge is 95 feet wide and 3465 feet in length. The deck girders were fabricated in 24 full-width sections and transported to the site. A detailed description can be found in the August 2006 Steel Tips by Alfred Mangus and Sarah Picker.

8.) Conclusions

All of the aforementioned projects have the same mantra coined by the Federal Highway Administration of “Get In, Get Out, and Stay Out.” Another common thread is the exposure to maritime traffic. The latter will not be the case in the massive amount of highway rehabilitation planned under SAFETEA 2003. The emphasis will rather be on existing and heavily travelled highway overcrossings. In California this is particularly true due to the recent analysis by the Caltrans Seismic Advisory Board relative to the ability of our structures to withstand earthquakes. For the convenience and safety of the motoring public the same philosophy of prefabrication to minimize on-site time is essential.

It would therefore behoove the owners of these structures to avail themselves of the experience gained by designers at the national level. Additionally local steel fabricators/erectors would be willing to make suggestions and budget analysis for the structure type selection process. The National Steel Bridge Alliance has knowledge of recent successful projects.

As this report was being finalized a fire-induced girder failure at the MacArthur Maze of the San Francisco-Oakland Bay Bridge occurred. I was called the next day by the National Steel Bridge Alliance, which advised that Caltrans did in fact immediately call them for assistance in locating steel fabricators who could rapidly furnish the replacement steel plate girders. Members of the SSEC will be monitoring this project as the reconstruction proceeds.

About the Author:



Shown here on the right Jay Murphy is a Life Member of ASCE and a Life Industry Member of SEAONC, and he holds a personal California Class “A” Heavy Engineering Contractor’s License. He spent 30-years working in all departments of his family-owned company and was elected president in 1970. The company ceased fabrication and erection operations in 1983. He now serves as chairperson of dispute review boards on Caltrans projects and also consults in the area of contract claims

Others shown left to right are Mike Foley, chief engineer of the Division of Bay Toll Crossings; Jim Moe, Caltrans director; and Howard Schirmer, regional engineer for AISC.

List of Published Steel TIPS Reports

- April 07: Progressive Collapse Prevention of Steel Frames with Shear Connections, by Abolhassan Astaneh-Asl.
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