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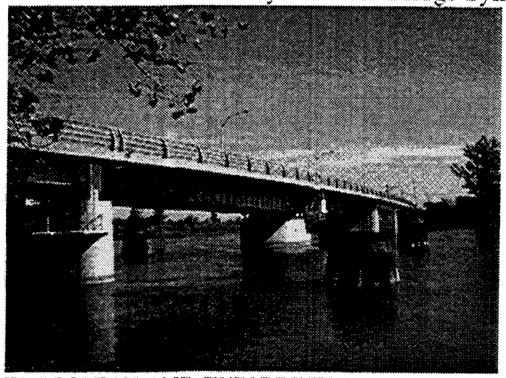
"Existing Movable Bridges Utilizing Orthotropic Bridge Decks"

by Alfred R. Mangus, P.E.
CalTrans

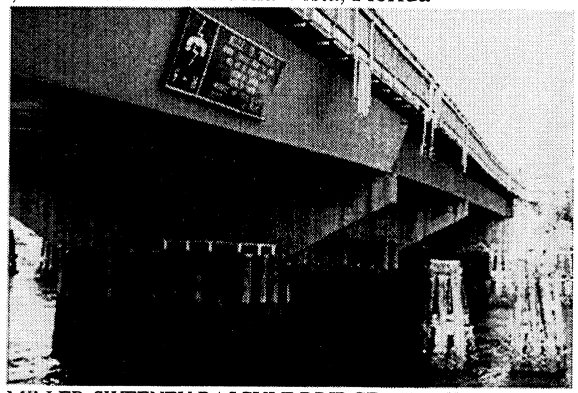
Existing Movable Bridges Utilizing Orthotropic Bridge Decks

Performance / Construction / Maintenance or Structural Elements

For Heavy Movable Bridge Symposium Nov 8,9 &10 1999 Lake Buena Vista, Florida



WALPOLE ISLAND SWING RIVER BRIDGE,CANADA (1970) Photo courtesy and by Alfred Ho PE Province of Ontario Ministry of Transportation [Photo # 1]



MILLER-SWEENEY BASCULE BRIDGE ALAMEDA ISLAND CALIFORNIA photo by Alfred R. Mangus PE [Photo # 2]

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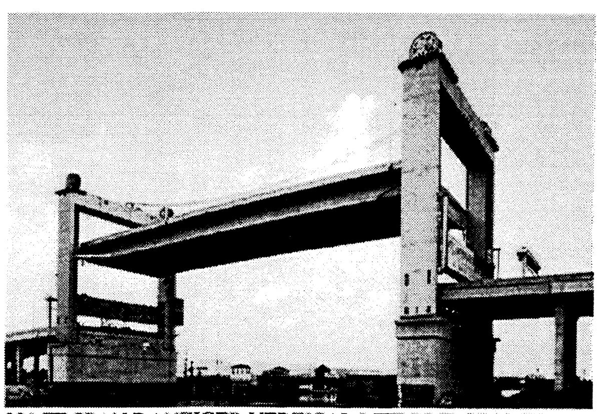
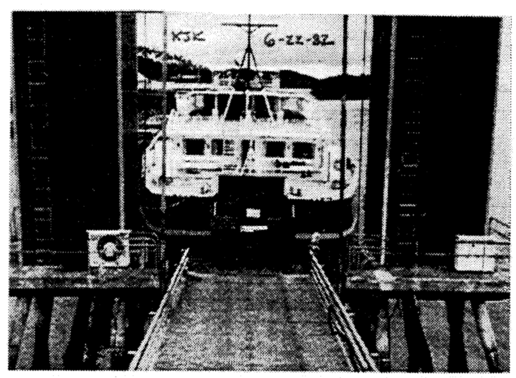
Abstract: Movable Bridges with Orthotropic steel decks in North America are very rare. This paper describes successful steel plate deck bridges built in North America. Many feel that orthotropic steel decks are too challenging to design and offer little benefit to the owners of movable bridges. The advantages will be summarized. State-of-the-art design procedures for orthotropic steel decks are briefly summarized with practicable references. The drawings of 7 featured bridges of North America include the Burlington-Bristol Bridge (1931) ; the Harlem River Bridge (1936) ; Danziger Vertical Lift Bridge New Orleans Louisiana (1988); Cordova, Alaska Ferry Terminal Bridge (1968);); removable bridge across the Sacramento River at Colusa, California (1985); the Miller-Sweeney bascule bridge on Alameda Island, California (1974); and the Walpole Island swing bridge of Ontario, Canada (1970). Other bridges such as the City of Valdez Alaska Floating Dock articulating transfer bridge (1981); in Europe and Asia were selected to demonstrate the complete range of all types such as the double swing bridge; the floating movable bridge; the skewed Bascule Bridge, the double leaf bascule bridges. A Critical issue is fatigue resistant detailing. Fatigue problems (cracking); wearing surface types, and wearing surface problems unique to orthotropic steel decks are briefly summarized. A comprehensive reference list is provided to assist in obtaining more practicable information.



320 FT SPAN DANZIGER VERTICAL LIFT BRIDGE NEW ORLEANS Photo Courtesy of Ernst H. Petzold PE of Sverdrup Civil, Inc. [Photo # 3]

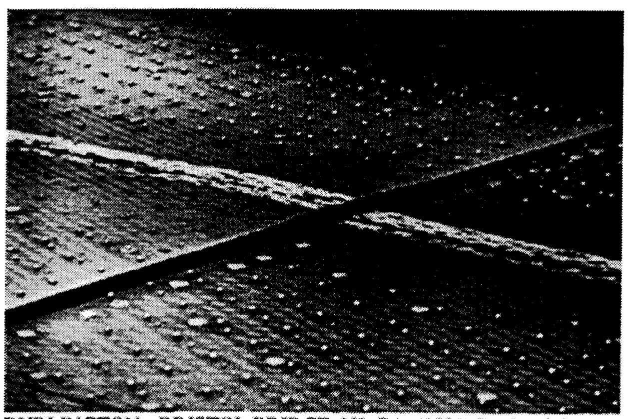


CORDOVA, ALASKA FERRY TERMINAL – STERN LOADING THE FERRY "E. L. BARTLETT" Photo Courtesy of Alaska Dept of Transportation / Public Facilities [photo by KJK PE] [Photo # 4]

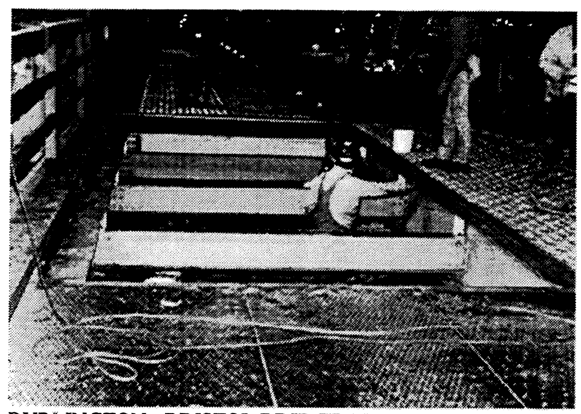
Lift Bridges

Burlington-Bristol Bridge

The origin of a system is like the roots of a tree; there are many sources. So is the origins and development of orthotropic steel deck bridges. A very brief summary is given in reference 1. In the 1920's American engineers began using steel plate riveted to steel beams for large movable bridges. The purpose was to minimize the dead load of the lift span. The predecessor of HNTB (Ash-Howard-Needles & Tammen) designed this lift span for the Burlington-Bristol Bridge Company, a private toll company– Figure 1 . The current owner Burlington County Bridge Commission is a private toll bridge company. Located across the Delaware River connecting Burlington New Jersey with Bristol Pennsylvania. The main span, which lifts, is 534-feet long. The design firm selected 0.625-inch (mm) deformed or "checkered" steel plate to save mass and money (see reference # 2). The deck did not have a wearing surface. Rivet heads were flatted to protrude about 0.25 inches. This system was selected instead of timber decking with asphalt-planks as a wearing surface. Approach spans used concrete decks. The total moving load including truss is 2,480 tons. The closely spaced steel floor stringers or rolled steel sections allowed the designers to assume some load sharing properties. The designer wrote ***"Since the plates are heavy enough to distribute some load to the adjoining beams, the individual stringers are designed to carry only 80% of the maximum wheel loading"***. Thus began the publishing of the idea of using the steel deck plate in harmony with rolled steel sections. The two lane bridge lift span details are shown in Figure 1. At the time of erection it was the longest lift span in existence (see reference #3). This bridge has a 20-foot wide roadway surface supported by steel truss superstructure comprised of laced members. In about 1995 the Burlington County Bridge Commission replaced the original flooring system with an open-grating system. This bridge was successful so the predecessor of HNTB (Ash-Howard-Needles & Tammen) designed another similar steel plate deck lift span bridge for the Port Authority of New York, New York in the 1930's.



BURLINGTON - BRISTOL BRIDGE, NJ -PA (1931) Deck prioir to removal 1995 Photo courtesy and by Sasha J Harding PE of Burlington County Bridge Commision [Photo # 5]



BURLINGTON - BRISTOL BRIDGE, NJ -PA (1931) Open Deck installation and removal of 1931 steel plate 1995Photo courtesy and by Sasha J Harding PE of Burlington County Bridge Commission [Photo # 6]

Harlem River

The MTA Bridges and Tunnels of New York, New York is the current owner operator of this bridge. Located across the Harlem River connecting Manhattan with Randalls Island– Figure 2. The main span, which lifts, is 310 feet long and completed in 1936 in reference #4. The owners' engineer describes different bridge deck systems in reference #5 & #6. The design firm selected 0.625-inch (16-mm) silicon steel plate to save mass and money (see reference # 5). The closely spaced steel floor beams allowed the designers to assume some load sharing properties. The sections of steel floor plate were butt welded together to form one continuous structural component. Welding was also used to connect the floor plate to the longitudinal sub-stringers (rolled steel shapes). The six lane bridge lift span details are shown in Figure 2. Each roadway is 30.5 feet wide on each side of the center median of the superstructure. The total superstructure width is 82.75-ft. The truss span weight is about 2025 tons with a deck weight of about 52.5 psf. The wearing surface over the steel deck was 1-inch thick mineral-surfaced planks, which were 24-inches long and 12-inches wide. The steel deck was painted with red lead and the planks were bonded to the deck with asphalt cement. "Performance of the planking on the Harlem River Bridge was quite good, and at the time of re-paving in 1962 about 70% was still covered by the original plank " reference # 7. The bridge was resurfaced with a 1-inch thick course of asphalt concrete. The deck corrosion under the first 27 years of use was considered minor. The owner is having engineering studies performed to replace the existing decking system with a welding Orthotropic decking with a new wearing surface. Re-decking with panelized orthotropic decks has been successful for various bridges (see reference # 1)

Guaiba Bridge at Porto Alegre, Brazil

In 1938 the AISC began publishing research findings of this system which called the "Battledeck floor" since they felt it had the strength of a battleship. Many of the ideas of stiffening steel plates had been in use by the ship building industry for decades. The James F Lincoln Arc Welding Foundation published the idea of an all welded together system steel bridge system (reference # 8). The Germans began the use of steel deck bridges as grade separation bridges for their freeway system the "Autobahn" in 1936. The evolution and variety of plate stiffening welded systems is described in reference #1. One of the leading German Autobahn engineers, later as a consultant design a large lift span in South America with a true Orthotropic deck (reference #9). The Guaiba Bridge at Porto Alegre, Brazil has plate girder with Orthotropic steel deck lift span of 183-feet (55.8 meters) was completed in 1960. In 1963 the AISC funded and published an authorized translation of the German design methods for use by North American engineers. This book is out of print and a code current version has not been published. The James F Lincoln Arc Welding Foundation published their edition of their textbook in 1967. In 1967 an American steel company published a design aid of tables based on the trapezoidal rib, which is the most material efficient of all stiffeners. The tables were developed using an IBM-360 computer and distributed free of charge. The typical engineer using a slide rule could use these complete plans for a bridge. German steel companies developed similar tables. Since then there has been an evolution of the code, excerpts of this aid that still comply with code minimum plate thickness requirements are reproduced in reference #1. Most North American bridges were designed using steel deck rib geometry described in the tables. The design engineer was able to use a slide rule or computer to complete the design. Many Japanese standard trapezoidal ribs are metric versions of American ribs and shown in tables in reference #1.

Danziger

The Industrial Canal Bridge or Danziger of New Orleans was built using one of the trapezoidal rib types proposed by the steel company– [see Figure 3]. The designers were Sverdrup and Parcel & Associates (main bridge) St. Louis, Missouri, and David Volkert and Associates (approaches) Mobile, Alabama . The General Contractors (joint venture) were the Williams Brothers Construction Co. and Ciambro Corporation New Orleans, Louisiana . Steel Fabricator was USS Fabrication Orange. Texas and the owner / operator is Louisiana DOTD. The total cost the bridge, including approaches, was about $38 million. This Orthotropic Bridge has a 320-ft span with a total deck width of 108.75 feet. It carries seven lanes of U. S. Route 90 traffic and replaced a 50-year-old double leaf Strauss Bascule Bridge. The Danziger Bridge needs to be raised only five times a day. Approximately 38,000 cars and trucks daily are expected to use the four westbound and three eastbound traffic lanes, each 12 feet wide, Vertical clearance for the movable span in the closed position is 50 ft above high water elevation, providing navigation for more than 90% of the waterway traffic without interrupting highway traffic. There are 150-ft high steel towers at each end. The length of the main span was chosen in order to clear the entire width of the canal at this location, thus eliminating the need for expensive protective fenders and dolphins and providing an unobstructed path to ocean ships and large, heavy barges using this waterway. The maximum vertical clearance at high water is 125 feet. After reviewing several alternatives, Sverdrup designed an Orthotropic deck, weighing about 2,200 tons, and can be raised 75 feet in less than two minutes to provide a 125-ft clearance over the canal. Sverdrup determined that the orthotropic deck system would not only be lighter and smaller, but also comparable in price to a two-truss design with greater lift - system requirements. A steel open-grid deck design disadvantages are rough and noisy riding surface, plus more surface area exposed to corrosion. A solid concrete deck is one way to avoid the rough surface, but on the Danziger would have weighed heavy 3, 700 tons. The main deck members are 14-ft deep rectangular girders 5.25-ft wide by 14-ft deep[see figure # 3]. Fatigue "cut-out" an oval slot at the base of the trapezoidal rib was used in all the crossbeam's webs . ASTM A572 and A588 were used for the main members while A-36 steel was used for other remaining components. The Orthotropic deck, which serves as the top flange of the main box girders, is ½-in. thick plate with welded trapezoidal ribs spaced at 26 in. o/c. framing into cross beams in 14.5-ft centers. The key dimensions of the American rib are a structural height of 10-inches, a base distance between the outside face of the trapezoidal legs of 12.75-in, bottom width of 6.5-inch and a plate thickness of 0.3125-in [323-mm x 254-mm x 8-mm or width x depth x thickness].The rib and its structural section properties are shown in a table of American ribs in reference #1. Rubberized asphalt was used as the wearing surface. The total weight of the lift span is about is about 2200 tons. The lift span was shop-fabricated in three sections both longitudinally and transversally. All field connections were made with high- strength ASTM A325 bolts. Those members are designated as fracture-blast-cleaned in accordance with SSPC-SP6-Commercial Blast Cleaning and inspected for surface flaws prior to fabrication, and shop-assembled, following progressive assembly method, in two sections in the transverse direction. Each section contains two main box girders, floor-beams (including end lift plate girders), diaphragms and cross frames. The total steel usage was 6.6 million pounds of steel. (see reference #10). The wind load acting on the lift span is transmitted to the towers through the span guides when it is raised. Two enclosed 140-ft towers support the operator house and the lift machinery, From the operator's cabin, the bridge operator has a clear view of the canal and the roadway. A house for the bridge tender is in the north leg of the west tower, 20 ft above the deck and about 90 ft above ground. The towers have also been designed so that, in theory, the lift span can be released from the cables to slide out horizontally for repair

Russian Vertical Lift Bridges

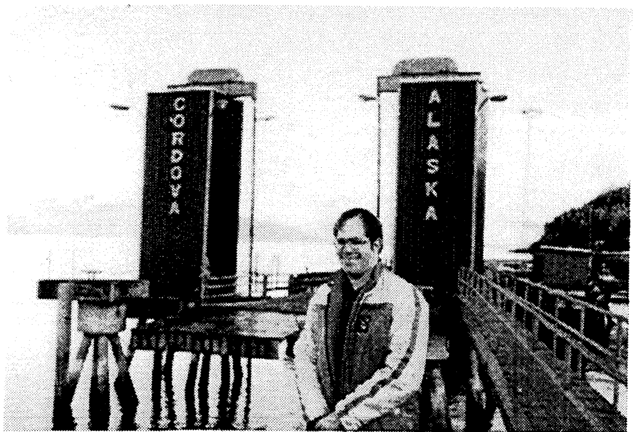
Russian engineers of Lengiprotranspmost have designed two long span vertical lift bridges. The crossing of the Severnaya Dvina River in the City of Arkhangelisk that was completed in 1990 and has a clear span of 84-meters. This bridge has its counterweights in four enclosed steel towers connected in two directions with steel box girders. They prefer to have elevators to transport personnel to the tower tops and equipment. Another bridge has a span of 120.45-meters (see reference #11). The open rib is very popular system and standard solutions have been developed for building in during the very cold winters. Russian engineers have design orthotropic bridges for other countries such as Turkey. Additional information on Russian bridges can be found in chapter 66 (see reference #1) and on the World Wide Web.

Chelsea Bridge Study

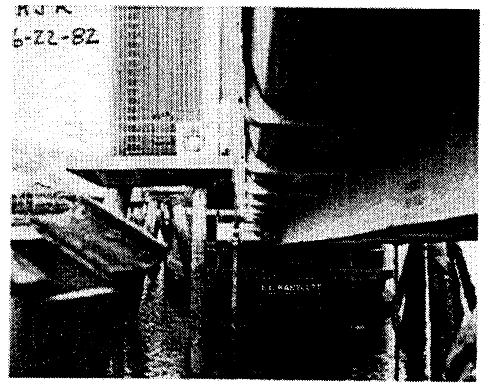
In 1998 HNTB presented a study for new lift span bridge (see reference #12). This Chelsea Bridge for the city of Boston was proposed to have a clear span of 453.67 feet and width of 58-feet. A tubular steel truss supports box girder floor beams at 18.88-feet on center. The Orthotropic deck would be .625-inch plate with 13.5-inch deep trapezoidal ribs. This paper present comparison of deck weights between four systems. The Orthotropic system had total weight of 1520 kips while the closest was the Exodermic Deck system at 2198 kips.

Cordova Alaska Ferry Terminal

In 1968 an articulating Ferry Terminal was designed for the Alaska Ferry system located in Cordova Alaska[see Photos # 4, 6 & 7, and Figure 4]. Campbell & Associates of Douglas Alaska designed the 16-foot wide by 119-foot long all steel ramp. The one end of the ramp hinges on the end of the City of Cordova's cargo dock. Two counter-weight towers are used to adjust the ramp to the deck level at the stern of the ferry. The elevation varies based on the tide and freeboard of that particular end loading ferry. This architecture is also common for ferries terminals located in British Columbia and the State of Washington. The counterweights are enclosed to protect them from freezing water and snow. There is about a twelve-foot tide. Their engineers selected a trapezoidal rib with spacing pattern from design aid booklets prepared by the American steel company [see figure #4]. The key dimensions of the American rib are a structural height of 9.5-inches, a base distance between the outside face of the trapezoidal legs of 13-in, bottom width of 6.5-inch and a plate thickness of 0.25-in [300-mm x 241-mm x 6-mm or width x depth thickness].The rib and its structural section properties are shown in a table of American ribs in reference #1. Also the spacing of the ribs was selected from this design aid. The steel deck is 0.375-inches thick, which also becomes the top flange of the two 46.375-in deep plate girders. The bottom flanges are 1-in x 18-inch plate. At every 10-ft is a transverse floor beam with 16 x 0.75-inch webs and 12 x 0.50-inch bottom flange. The top flange is the deck plate resulting in orthotropic action. Epoxy grit was used for a thin wearing surface of about 0.25 inches. The ramp is still in service after opening in 1970. Apparently there is no published paper on this project.



CORDOVA, ALASKA FERRY TERMINAL – Author during 1981 visit Photo Courtesy of Bill Gunderson PE of PND Engineering [Photo # 7]



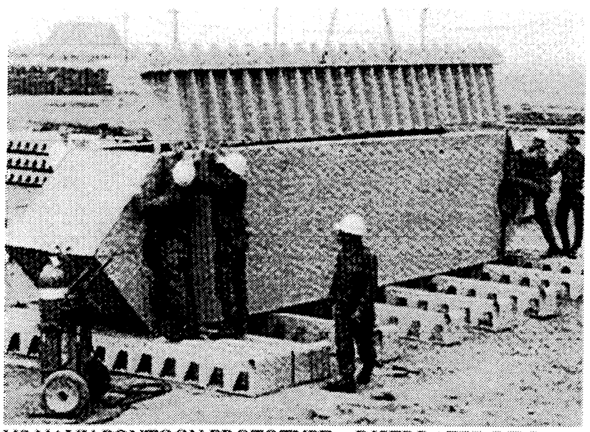
CORDOVA, ALASKA FERRY TERMINAL – STERN LOADING THE FERRY "BARTLETT" Photo Courtesy of Alaska Dept of Transportation / Public Facilities [photo by KJK PE] [Photo # 8]

Roll-on Roll-off Ramp Pontoon Dublin Ireland

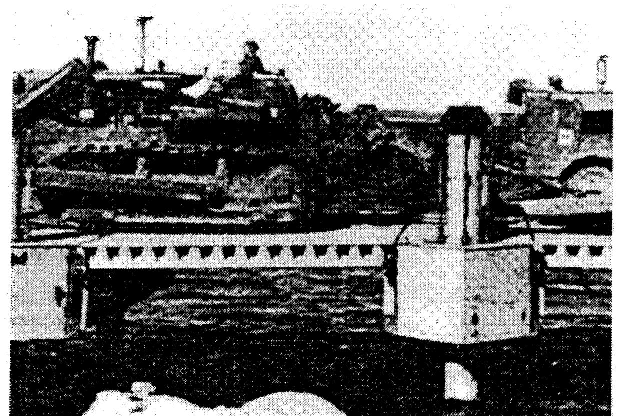
Another Orthotropic solution is to have boat side of the ramp float in water. The Roll-on Roll-off ramp at the end of the dock in Dublin Ireland has a welded steel pontoon (see reference #14). A guide pile for lateral stability is needed. The pontoon is 20-meters wide at ocean face and narrows to a 10-meter wide ramp at shore. The cross-section has nine trapezoidal ribs to stiffen the deck plate. Two plate girders are used for main superstructure. The entire bridge with steel pontoon was fabricated as a single 350 metric ton unit in the Netherlands and designed by Delta Marine Consultants of the Netherlands and Ove Arup of Great Britain . It was transported by an ocean going barge and erected as a single unit. An operator's booth is used to control ballast tanks to position the deck of the pontoon with the particular ship being unloaded. 180 metric tons of Roll-on Roll-off (trucks with container trailers) equipment is rapidly load on or off the vessel and be on the pontoon at one time. The owner - operator is the Merchant Ferries.

US Navy Pontoon Prototypes

The US Navy has been experimenting with biserrated trapezoidal ribs to reduce the total dead weight of the pontoon (see references #15 & 16). The idea of a welded- steel portable pontoon bridge was conceived and developed with a modular span of 20-ft. Three basic components are: the pontoon, the deck beams, and the bracing trusses. The pontoon consists of a wedge-ended box, measuring 6-ft wide, 4-ft deep and 30-ft in overall length. The watertight body is made of 0.25-inch thick plates, stiffened of a novel shape, which the Navy's engineer's designated as "biserrated-orthotropic." In the top and the bottom these ribs" spanning transversely 2-ft on centers, are shaped as trapezoid ribs about 6-inch deep. And have serrated upper edges. In the sides, they are of triangular shape and are placed vertical 1-ft on centers. The pontoon is also featured by 20-inch diameter cylindrical wells at each end for one mooring spud or support pile. The dead weight of the pontoon is about 5-tons, and when a float it has a draft of approximately l-ft. The key dimensions of the Navy's rib are a structural height of 12-inches, a base distance between the outside face of the trapezoidal legs of 11-in, bottom width of 3-inch and a plate thickness of with 0.1875-inch [279-mm x 305-mm x 5-mm or width x depth x thickness]. Weighing only 25 pounds per linear foot, the section possesses great flexural and torsional strength. It is made of two parts –a serrated- edge trapezoid rib and a top closure or flange plate. The channel, in turn, is obtained by bending a plate to that form after the strip is cut along two parallel serration lines. The "cut-out" or serration, which occurs in staggered pattern in the two faces serves two main purposes: first it makes possible the use of full-penetration welds for the flange-to-web interconnections; and secondly. it increases the depth of the section, and hence its strength by a height equal to the depth of the serration without an increase in the weight. There are 24 beams per module or span. In assembly, they are placed side by side, supported on the side shelves of the pontoons. forming a deck 24-ft wide. A pipe segment welded to the bottom at each end of each beam serves the dual role of a rocker support and connection sleeve for the anchorage-locking pin. Four lines of bracing trusses that fit between the pontoons and tie them together longitudinally. Each truss is composed of three tubular members arranged as a K, with a bottom chord and two diagonals pin- connected to the pontoons and to each other. The weight of the three-member assembly is about 650 pounds. The prototypes were built of A-36 steel.. The Navy engineers chose single-trough deck beams, in lieu of similarly stiffened panels. The pontoon is subdivided into six sub-assemblies, with the goal saving valuable space in overseas shipments. It can be mount it on top of an axle-and-wheel assembly to form a trailer. In more permanent installations, welded connections can be provided by means of seat and ring segments placed at bottom and top of the pontoon spud wells. The welded-steel pontoon bridge was conceived and developed to be mobile, lightweight, strong and durable. The first 80 prototypes modules came to about $6,500 each. The "cut-outs" or bi-serrations on the sides of ribs reduce the dead weight, however the interior of the rib is now exposed to corrosion. Actual units were built in 1966 and 1968 and it is known about the fatigue durability. It is not known if this pontoon is still manufactured. The army has a pontoon bridge with boat shaped pontoons rather than rectangular shaped by the Navy.



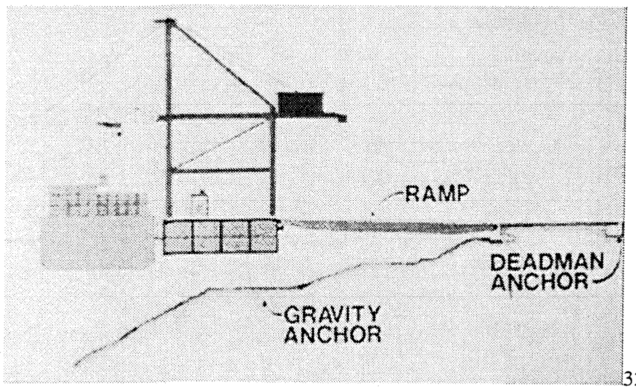
US NAVY PONTOON PROTOTYPE - BISERRATED RIBS (1966) US Navy Photo courtesy *MODERN WELDED STRUCTURES* Volume III - 1970– PUBLISHED BY THE JAMES F. LINCOLN ARC WELDING FOUNDATION [Photo # 11]



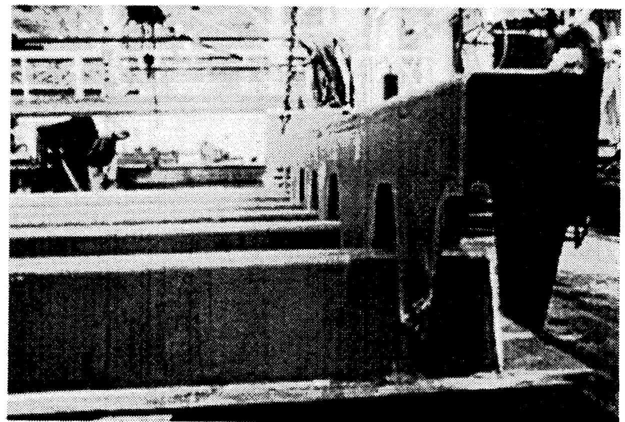
NAVY PONTOON LOAD TEST - BISERRATED RIBS - SPUD PILE THROUGH WELL(1966) US Navy Photo courtesy *MODERN WELDED STRUCTURES* Volume III - 1970– PUBLISHED BY THE JAMES F. LINCOLN ARC WELDING FOUNDATION [Photo # 12]

Valdez, Alaska Floating Dock Ramp Bridges

Valdez, Alaska is beside a deep fiord carved by a glacier. A floating concrete dock was built for the city[see photo # 9]. Connecting this floating dock at each end to the shore are two identical orthotropic steel twin ramps 29.5-ft wide and a simple span of 200-ft. Berger / ABAM Engineers of Federal Way Washington developed two identical steel ramps to resist the harsh corrosion forces from salt or sea water. There is one located at each end of the floating concrete dock owned and operated by the City of Valdez. The orthotropic superstructures were built using trapezoidal rips and a typical cross-section is shown in references #1 & 17. The key dimensions of the American rib are a structural height of 12-inches, a base distance between the outside face of the trapezoidal legs of 14-in, bottom width of 6.5-inch and a plate thickness of 0.375-in [356-mm x 305-mm x 10-mm or width x depth x thickness]. Each bridge was entirely shop fabricated and shipped to Alaska. The transverse floor or crossbeam was fabricated from a flat sheet of steel bent in a "U" shape [see photo # 10]. The rib was serrated for a tight fit around all the trapezoidal ribs. Next it was welded to all the ribs for a corrosion resistant detail. The remainder of the ramps were detailed be watertight. Open steel railing was used to ease of rain runoff and snow removal. A sidewalk area was provided for employee safety. The superstructure consists of twin rectangular boxes.



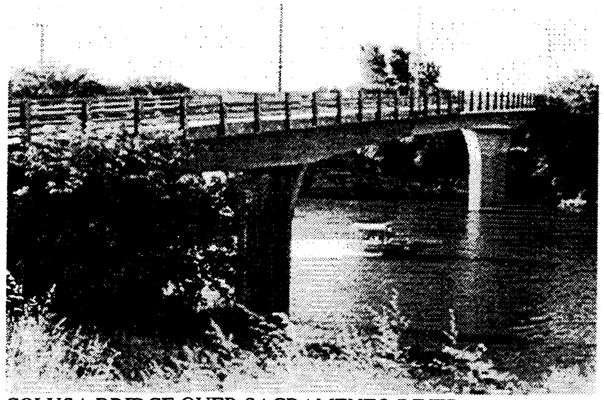
VALDEZ, ALASKA FLOATING DOCK RAMP BRIDGES – Cross-section drawing Courtesy of Warren Wilson SE of Berger / ABAM Engineers [Photo # 9]



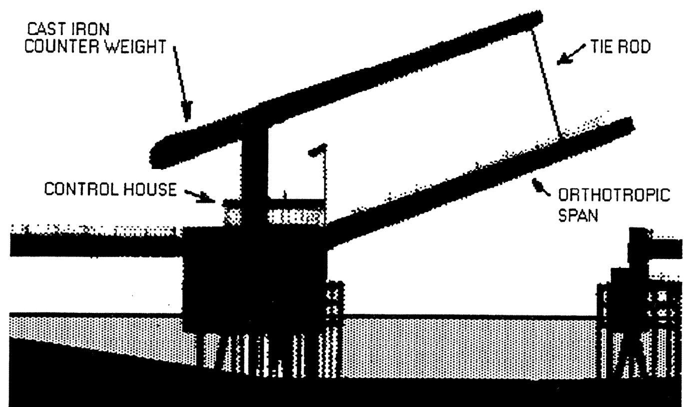
VALDEZ, ALASKA FLOATING DOCK RAMP BRIDGES – Cross-Beam fabrication Courtesy of Warren Wilson SE of Berger / ABAM Engineers [Photo # 10]

Colusa California Sacramento River

In Colusa California an existing swing was replaced in 1972 with a unique bridge– Figure 5 . The bridge is primarily prestressed concrete with a 105-ft removable orthotropic box section. Before highways riverboats were the primary mode of hauling heavy cargo through central California using the Sacramento River. Once the highway system evolved the swing bridge hardly opened as the used of shallow draft riverboats ended. The unique solution was half the price of a convention swing bridge. A solution that allowed cranes to pick-up this section was developed (see figure # 5) references #18 & 19. A trapezoidal welded steel box girder with an orthotropic deck was used to provide a lightweight removable section in a low-level concrete bridge. The bridge was designed by CH2M Hill Engineers of the Sacramento California office for Colusa County. The three requirements for a removable section can be summarized as follows: minimum weight; high torsional stiffness; and easy attachment of lifting cables In order to assure a minimum 100-ft clear channel, a 105-ft long removable section was needed. The weights of several types of removable bridge section were calculated as follows: normal weight concrete 550 tons concrete box; lightweight concrete 400 tons ; steel box with regular weight concrete slab 400 tons; steel box with light weight concrete slab 300 tons and steel box with orthotropic plate deck 200 tons. There are two practicable ways of lifting this span in order to create a passage through a low-level bridge. Simultaneous lift by two cranes operating from both cantilevers in the main span or lift by a single barge mounted crane. Truck-mounted traveling cranes in the range of 125 tons. Two cranes can provide a comfortable lift for a removable section of steel box girder with an orthotropic plate deck weighing about 200 tons. High strength, corrosion resistant, weathering steel ASTM A588 was specified throughout. The 0.375-inch thick deck plate is stiffened longitudinally by a means of closed trapezoidal ribs, spaced about 2-ft on centers. The ribs are cold bent into trapezoidal shape from a ¼ x 24 plate and are 8-1/2 deep. Transverse supports for the ribs are provided by the floor beams spaced about 14' on centers. At every other floor-beam there is a cross- frame consisting of two diagonal and one vertical members made of WT4 x 8.5. In longitudinal direction, the main carrying element is the trapezoidal steel box consisting of two inclined webs .0375-in x 72-inch, the .0375-in thick x 38.16-ft wide deck plate serving as the top flange and the .0375-in thick x 18.5-ft wide plate serving as the bottom flange. The bottom flange is stiffened by vertical .0375-in x 6 plates spaced 27-inch on centers. The overall depth of the removable section is 6-ft, which consist of 5.75-ft steel box and 3-inch epoxy asphalt overlay. The shear in the inclined web is transferred to a vertical web plate at each end by means of two 0.75-inch thick diaphragm plates. One issue for the fabricator was presented by the 1600-ft radius curve of the bridge. For bridge aesthetics, the webs and edges of the deck to be curved in plan, while the trapezoidal ribs were allowed to be made up a series of chords. The exterior of the steel box was painted with " light gray paint to achieve a uniform colored structure. In the deck overhangs the insulation completely envelops and conceals floor-beams and ribs, thus again giving the appearance of a single type structure. The removable unit has four 6" x 12" openings through the deck plate for access to lifting pins. Each pair of openings is located at the end of the unit arranged symmetrically about the centerline of the bridge. The cables are attached to the 4-inch diameter lifting pine which are held in place by the respective bearing stiffeners. All fillet welds were 0.25-inch thick and most were made in down-hand position (the easiest for a welder to complete). All plate splices were full penetration butt welds, and intersecting welds were avoided. For the connection of the orthotropic trapezoidal rib to the deck plate two 0.25-inch thick fillet welds were specified. Edge preparation was not used since a 0.25-inch thick weld on an inclined 0.25-inch thick plate very likely will penetrate over 90% of the plate thickness. Again most of these welds were made by submerged arc welding process. For the connection of the web to the flange, two 0.25-inch thick fillet welds were used top and bottom. Orthotropic steel deck plate bridges constructed in comparatively moderate climate zones have exhibited premature icing (compared to concrete decks), causing a slippery driving surface. In severe climates where roads and bridges are uniformly frozen, there is little difference between driving on the road or on the bridge. In moderate climates, particularly when there are fluctuations of temperature from freezing to thawing, the thin steel deck with any moisture on it will freeze first and thaw out last. This side by side existence of an icy bridge and unfrozen road may result in a traffic hazard to an unsuspecting motorist. A layer of 10-inch plus or minus urethane foam insulation was placed on the underside of the deck plate. The goal was not " expected to eliminate the differential freezing of the bridge altogether, but rather to bring closer the temperature at which the bridge and the road ice over. Currently there is a yellow "slippery bridge" sign posted at each end of the bridge.



COLUSA BRIDGE OVER SACRAMENTO RIVER , CALIFORNIASHERIFFS PATROL BOAT PASSING UNDERNEATH – Photo by Alfred R. Mangus P. E. July 2000 [Photo # 13]



BREYDON BRIDGE, GREAT BRITAIN – OVERHEAD COUNTER-WEIGHT AND MOVABLE SPAN PORTION OF THE BRIDGE. ALSO CONTROL HOUSE SHOWN. Adapted from *THE STRUCTURAL ENGINEER* VOLUME 66 NO 2-19 JANUARY 1988 [Photo # 14]

Bascule Bridges

Miller-Sweeney Bridge

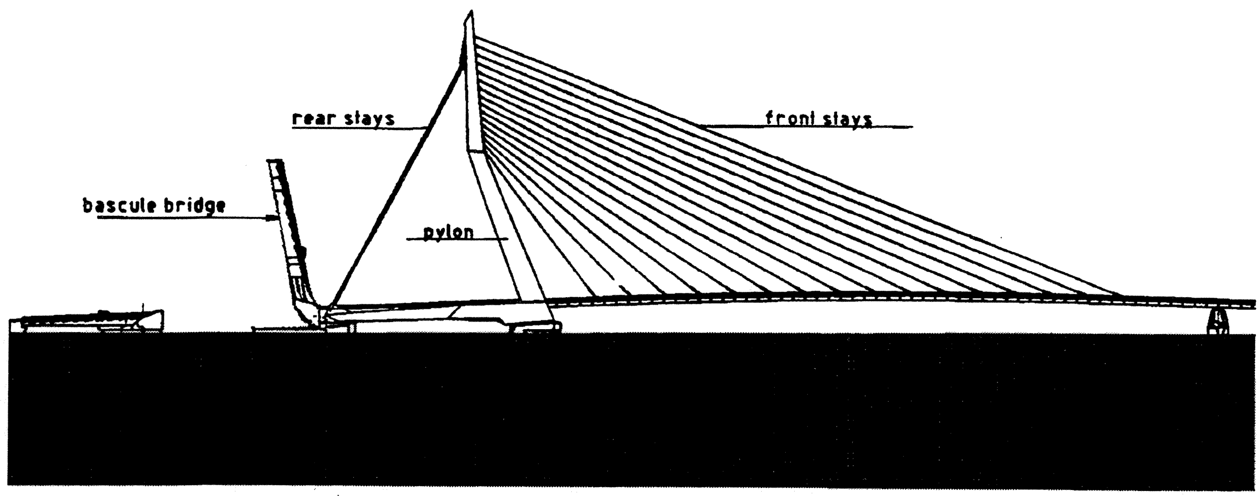
The Miller-Sweeney Bridge at Fruitvale Avenue is a four lane single leaf Bascule bridge, which movable span is 127.5-ft (with horizontal clearance between fender systems of 98 feet) – Figure 6. The Oakland Estuary is a navigable waterway between Alameda Island and Oakland California with access to the San Francisco Bay. The structure was designed by McCreary Koretsky International Inc. for the U.S. Army Corp of Engineers and opened to the public on December 12, 1973 and received Movable Span Award from the AISC for engineering and design excellence. The bridge was turned over to the County in 1975 and was named after local politicians Mr. Miller and Mr. Sweeney. Approximately 28,000 vehicles each working day use this orthotropic steel deck bridge. Bridge Statistics are as follows: Vertical Clearance MLLW (Low Tide) 21 Feet ;Vertical Clearance MHHW (High Tide) 15 Feet ;Vessel Channel Clearance 96 Feet ; Bridge Height Restriction for Vehicles None ; Bridge Length 215 Feet ;Width of Roadway 52 Feet ; Width of Traffic lanes 13 Feet and Pedestrian Sidewalks 6 Feet 1 Inch. Vessel and Vehicle Traffic statistics from the Alameda County WWW site. In 1989 the Loma Prieta Earthquake caused damage to the bridge inside the machinery pit and it was shut to all vessel traffic until repaired. In 1991 another mishap occurred when a fully loaded sand barge (weighing 4,000 tons) hit the movable span and caused extensive damage. The emergency drive is what kept the waterway open to vessel traffic as the main drive system would have damaged the bridge further if it were engaged. The bridge was closed to vehicles for months and cost almost a half million dollars to repair. The Bascule span uses trapezoidal rib with spacing pattern from design aid booklets prepared by the American steel company. The wearing surface failed by creep when the movable span was in the open position and was resurfaced in the summer of 1997 [see references #19, 20 & 21].

Breydon Bridge

The Breydon overhead counterweight Bascule Bridge was completed in May 1985 over the Breydon Waterway on the outskirts Great Yarmouth Great Britain. This double cantilever framing system originates from drawbridges used in castles. (Vincent Van Gogh's 1888 painting "The Langlois Bridge" a double overhead counterweight Bascule bridge is on the cover of HMS inventory of movable bridges in USA booklet). The bridge was built on the same alignment as a swing bridge, which was demolished in 1962. The Breydon Bridge is a nine- span viaduct, 247m-long, over Breydon Water on the outskirts of Great Yormouth, Norfolk. The viaduct forms part of the A 47 Trunk Road Introduction Breydon Water , an open stretch of mud flats covered by water at high tide. The Husband & Co. consulting engineers concept was reviewed by the Royal Fine Art Commission prior to final detailed design. The viaduct consists eight 26-m simple spans and one Bascule span carrying a 10m- wide roadway with two 1-m sidewalks between guardrails[Overall length between abutment bearings 247 m and width of 13-m]. The Bascule span allows passage of commercial vessels and large leisure craft through a 23m-wide navigational channel. The two side spans on either side of the main channel provide secondary navigation channels for smaller pleasure craft with restricted headroom. Three Options considered in the design of the movable span and the approach viaducts were a Bascule bridge below with counterweight below deck; a bobtail swing span and selected solution as discussed in this text. The advantages and disadvantages due the effects of constraints imposed by site conditions, navigation requirements, and the need to provide a structure with easy maintenance are described in reference #23 . Aerodynamic, hydraulic tests and investigations related to the unusual type of structure were performed due to the unusual aspects of construction. The deck of the Bascule span is simply supported for live load. For dead load the span has extra support from the hangers and counterweight system. The counterweight arms are simple cantilevers, and the towers are freestanding columns. The 30.875-m long by 13.0-m wide Bascule leaf provides a 10-m wide roadway with two sidewalks of 1-m. A 20-mm thick steel deck plate has been adopted which minimizes welding distortion and provides a flat surface for the epoxy bauxite wearing surface. The trapezoidal rib stiffened steel Orthotropic deck is supported by two longitudinal plate girders and transverse girders at 2.75 m centers. The deck plate forms the top flange of both the longitudinal and transverse girders creating the efficiency of orthotropic action. The cross- girder at the nose and tail of the leaf and at the hanger connection position is of box girder construction. Steel box girder construction is also adopted for the counterweight towers and frames. A 15 metric ton counterweight system at the extreme end of the Bascule leaf. Discussions were held with TRRL about the details selected for the orthotropic steel deck to give an acceptable fatigue life. [TRRL = Department of Transport and Road Research Laboratory (TRRL) Room C19/01 2 Marsham Street London SW1P 3EB United Kingdom]. Many components including the 250 metric tons counterweight frame, with its hanging rods fitted, was transported to site and erected using the a floating crane. Structural steel quantities were approach spans 645 metric tons ;bridge deck 260 metric tons; counterweight frame 250 metric tons; towers 91 metric tons; Cast-iron counterweight 350 metric tons.

Erasmus Bridge

Public Works Dept. of the City of Rotterdam, Netherlands is the owner and operator of the Erasmus Bridge. The construction of the Erasmus Bridge in the harbor area of Rotterdam, for which Architect Ben van Berkel made the original design in 1989, construction began in 1994 and was finished in the fall 1996. Van Berkel & Bos was the lead consultant for the Department of Works of Rotterdam. The main contractors were Heerema Vlissingen ; Grootint Dordrecht and Ravenstein Deest. The unique 139-m. tall pylon for the cable-stayed span, and has a movable Bascule section permits the passage of ships taller than Rhine River navigation height. The tower and bascule machine room are located downstream in the shallows adjacent to an island, so this unique shape does not impede river traffic. "The Swan" Bridge has two light rail tracks, 4 lanes of vehicular traffic ; 2 bicycle lanes ; and 2 sidewalks with the river walk.. The single leaf Orthotropic bascule is one of the largest in the world, with a deck measuring 172-ft by 117-ft (52.3 by 35.8-m), and an apron weighing 3.5 million lbs. (1,560-metric tons). In an open position, the fall stands 62-ft (19-m) "out of plumb." The bascule column has three functions; first the anchoring of the cable-stayed span of the bridge; second providing the foundation for the pivotal point of the bascule bridge; and third housing the bascule pit and mechanical equipment room. The 0.7-in (18-mm) thick orthotropic steel bridge deck is reinforced with trapezoidal stiffeners measuring 24-in (600-mm) center to center. The fully welded deck has a 0.3-in (8-mm) thick synthetic resin wearing surface, providing considerable savings on the structure's dead weight, compared to an asphalt mastic-wearing surface. The deck plate, box-shaped longitudinal girders, cross and main girders form a fully welded, Orthotropic steel bridge. The cross girders and consoles were fabricated in the form of girder plates. Girders are box profiles around the rotation point, where bending and torsion moments are greatest. The box girders absorb the large torque and bending load with a minimum of distortion. At the 2/3 front end of the bascule deck cantilever the forces and required rigidity are less, and the cross section transitions a girder plate of the same depth as the box section. To limit the diagonal eccentricity of the deck, the sideward twist was placed in front of the main rotation point wherever possible. The weight of the Bascule Bridge was almost completely balanced by the counterweight, except for the front bearing pressure. The ballast was located eccentrically in a diagonal direction to compensate for the obliqueness of the bridge. This equally distributes both the weight responses in the main rotation points, and the bending moments caused by the bridge's own weight in both main girders. The moving time is limited to 120 seconds for the opening and 135 seconds for the closing of the bridge [see references # 24 & 25].



ERASMUS BRIDGE , ROTTERDAM, NETHERLANDS , – OPEN BASCULE SPAN - Drawing adapted from *WELDING INNOVATION QUARTERLY* Volume XV No 2 - 1998– PUBLISHED BY THE JAMES F. LINCOLN ARC WELDING FOUNDATION [Photo # 15]

Swing Bridges

Walpole Island Bridge

The Walpole Island Bridge was completed in 1970 on Indian Reserve No. 46 connecting Walpole Island to the mainland of Ontario Canada. – see Photo # 1 and Figure 7 [see references # 1 & 26]. It crosses the Snye River or Chenal Ecarte near Wallaceburg Ontario. The Canadian firm of Wyllie & Ufnal Limited designed this bridge in 1967. Only the center 218-ft outer diameter swing span is orthotropic, while the remaining spans are 7.5-inch thick reinforced concrete on steel girders. The key dimensions of the American rib are a structural height of 8-inches, a base distance between the outside face of the trapezoidal legs of 11.50-in, bottom width of 6.5-inch and a plate thickness of 0.3125-in [356-mm x 305-mm x 8-mm or width x depth x thickness]. A trapezoidal "cut-out" with a snug fit suitable for welding was made for the continuos ribs. The transverse floor beams are at 11.33-ft on center and the .50-inch thick deck plate is used for the top flange. Two variable depth plate girders a 2.25-inch thick wearing surface was used. The superstructure was fabricated into four units and field spliced with A-325 bolts. A control tower controls the opening of the two-lane bridge of 28-ft clear between the 5.5-ft wide concrete sidewalks with aluminum exterior railing. This bridge remains operational.

Double-Leaf Swing Bridge near Naestved, Denmark

A double-leaf swing bridge started in November 1995 and was opened in September 1997 to span a 44-m wide dredged navigation channel that connects the city of Naestved, Denmark with the sea. The Danish Road Directorate the owner / operator considered a total of 14 proposals from five different consulting firms: five bascule bridges, four swing bridges and five lift bridges. The double-leaf swing bridge design by ISC Consulting Engineers, Copenhagen, Denmark was selected based on its building and maintenance cost advantages, aesthetics, functionality, reliability and environmental compatibility. The successful design has: low total costs compared with other bridge concepts; attractive design ; efficient superstructure, two identical swing sections with cylindrical concrete piers ; simplified bearing scheme; shear locks at the center and the abutments; durability; low maintenance and finally corrosion protection for the interior surfaces of the center spine box sections. Vessel traffic through the swing bridge is about 1000 openings per year. Therefore bridge is designed to be operated from a panel located on one side, affording the operator a good overall view of road and vessel traffic. The bridge carries a two-lane road and a bicycle path, with a total width of 49.93-ft [14 m]. The bridge alignment is nearly straight, crossing the centerline of the channel at an 80 degree angle[see reference #1 or # 27].A vertical clearance of 26.2-ft [8-m] above mean water level, 15-ft [4.6-m] above a road on the south side of the channel, and 8.5-ft [2.6-m] for the bike and pedestrian path on the north side of the channel. The two piers are located on the banks to avoid any reduction of the navigation width of 138.8-ft [42-m], equal to the distance between the sides of the superstructure in the full open position. The steel superstructure consists of two identical 160.5-ft [48.95-m] long swing sections with a deck shaped like a parallelogram, its ends parallel with the channel. Both sections rotate in a counter clockwise direction. The rotating sections are supported on roller-bearing slewing rings 16.4-ft [5-m] in diameter. These rings are located on the two circular concrete piers 183.7-ft [56-m] apart, measured perpendicular to the channel. The two swing orthotropic sections of the bridge are all welded and comprised of single 19.7-ft [6-m] wide the center spine box section. The full deck is width on 11.5-ft [3.5-m] long cantilevered tapered plate girders on each side of the center spine box girder. The depth of the box girder varies from 11.5-ft [3.5-m] at the bearing supports, to 5-ft [1.5-m] deep at the cantilever tips at the center of the channel and at the abutments. Full-size transverse plate diaphragms are at 13.1-ft [4.0-m] intervals are perpendicular to the bridge axis and in the same plane as the cantilevered cross girders. Because of the skew, the box section is shaped with a horizontal bottom flange at the center of the bridge and at the abutments. The bridge orthotropic steel deck and the bottom flange are stiffened by means of cold-rolled rectangular rib sections, with bulb rib sections applied as web stiffeners. A finite element computer analysis of the cantilevered crossbeam determined the peak stresses in the web, which occur at the edge of the cutout for the through sections. A 0.787-inch [20-mm] thick web plate is required to achieve sufficient fatigue strength. Local bending in the web brackets due to shear forces transferred from the deck plate to the web seem to have been neglected in existing orthotropic bridges. Fatigue cracks have occurred at the lower corners of the rib "cut-outs" in crossbeam webs of a few existing orthotropic bridges. Danish steel grade Fe 510 C-E has been applied for all primary parts of the box girders. There is a 2.36-inch [60-mm] thick layer of cast asphalt, instead of an epoxy-wearing surface. The exterior of the steel superstructure has been sandblasted and further protected by painting according to Danish code. The interior of the center spine box sections is protected by means of dehumidifying systems, thus the cost and weight of paint are saved. The dehumidifying equipment is located inside the box girders above the two concrete piers. The dry air is circulated forward through the hollow stiffener sections and returns through the box sections. Access to the box sections for inspection is through man- holes provided in the bottom flange of the box girders inside the pier. The two bridge sections were fabricated, welded and treated for corrosion protection at the fabricator, and some of the mechanical and electrical installations were pre-mounted as well. Casting of the counterweights was also executed before transportation of the bridge sections. The two bridge sections, each 300 metric tons., were transported by barge along the navigation channel and lifted on top of the concrete piers at the site. The paving of the asphalt wearing surface and the mounting of the hydraulic pins was completed after erection. The hydraulic cylinders are located in the box section of the bridge, with access for maintenance provided through the piers. The total time of operation including activating the traffic signals, the opening and closing of the bridge, but excluding the time needed for the ships to pass through, is about 3.5 minutes.

Yumeshima-Maishima Floating Swing Bridge, Osaka Japan

Japan Assoc. of Steel Bridge Construction with Japan Engineering Consultants Ltd. performed Structural engineering designed of the world's first floating swing bridge. It is under construction in the Port of Osaka owned by the Osaka Municipal Government [population of 16 million.]. Main Contractors [see – Figure 8 – Figure 9 and references # 28,29,30] expected to complete construction in the fall of 2000 and will connect two reclaimed islands separated by a waterway is about 400m.. The ordinary fixed bridges or movable with foundations fixed to seabed, such as swing bridge, Bascule Bridge. rolling bridge or transporter bridge were investigated concluded from a comparative study that those ordinary bridges would be difficult or economically infeasible; instead, a unique bridge was proposed and is the first of this type in the world. Many important engineering issues such as the dynamic response to the wave, wind, earthquake and vehicle loads. The bridge rests on the two hollow steel pontoons and is supported horizontally by rubber fenders and dolphins. selected as an arched skew floating highway bridge with total length of 940-m and width of 38.8-m for six traffic lanes with doubled rib tied arch. The entire bridge can swing around a pivot axis near the Maishima end and is moved with tugboats. The structure is designed to be strong and stable enough to withstand typhoon-level wind and waves. The swinging operation begins when a pivot axis is inserted and the transitional side bridges are jacked up. Then the reaction walls are released from their mooring function and rotated. and the bridge is swung about the pivot axis by tugboats. The superstructure, with pontoons, were assembled at a ship fabricator's dry-dock. The complete bridge section will be towed to the site and joined to the mooring systems. Japanese engineers use *Specification for Highway Bridges* but this code does not cover the design of floating bridges. Comprehensive experiments was achieved through cooperation among various Japanese universities, Osaka Municipal Government, the Ministry of Transport, heavy industrial companies, fabricators, design consultant companies and a fender manufacturing company. Peer review was performed by the Technical Committee on Movable Floating Bridge (Chairman: Prof. E. Watanabe) from 1990 to the present time. Total orthotropic deck area is 129,171 square feet [12 000 square meters] Substructure 7100 metric tons and Superstructure 24,500 metric tons of steel Total cost is estimated at $400 million US dollars. The roadway superstructure is 2.3-m deep and built intregal with the bottom chord of the tied arch. The superstructure was fabricated seven longitudinal sections with six bolted splices. A 12-mm steel deck was used with three types of rib stiffeners. The key dimensions of the Japanese rib are a structural height of 9.45-inches, a base distance between the outside face of the trapezoidal legs of 12.59-in, and a plate thickness of 0.236-in [320-mm x 240-mm x 6-mm or width x depth x thickness] see reference # 1 for section properties. This trapezoidal rib shape is used in the traffic areas. Ribs are spaced 330-mm apart. Open ribs 160-mm deep by 15-mm thick are used in the sidewalk areas and inside the tied arch bottom chord. Open ribs 160-mm deep by 13-mm thick are used in the curb areas. Figure 9 shows additional steel detailing. The roadway has an 80-mm wearing surface while the sidewalks have a 40-mm wearing surface. There are other papers discussing this complex bridge.

Brief Summary Fatigue of Orthotropic Bridges

Since 1997 in several heavily loaded highway bridges in the Netherlands, fatigue cracks were observed in the welded connection between the longitudinal trapezoidal stiffener web and the deck plate of the Orthotropic Bridge deck. Depending on the crack initiation point they could be found and repaired relatively easy. In some cases cracks are relatively large and their repair is difficult. The Caland Bridge , a highway plus railroad lift-span, was completed in 1969 in the harbour area of Rotterdam, Netherlands has been extensively monitored with strain gages. The meter lift span has four vehicular lanes, two railroad tracks, a bicycle path and sidewalk for pedestrians. The superstructure cross -section consists of a 10-mm steel deck with trapezoidal stiffeners. The original surfacing system was based on bitumen binders (mastic asphalt). Part of the wearing has been replaced on polyurethane resins and both the original and replacement had a thickness of 50-mm. [see reference # 31].

The Van Brienenoord twin Bridges were completed in 1958 and 1990 the harbor area of Rotterdam, Netherlands [see references # 32,33]. The owner operator of the bridges is the Rijkswaterstaat, the Department of public works of the Netherlands. The A-16 freeway crosses the Nieuwe Mass River using main span of a tied arch. The 1958 is an open rib Orthotropic steel deck, while the 1990 version uses closed ribs (trapezoidal) suspended by inclined cables from the arch. At the side of each tied arch is a single leaf bascule bridge. Plate girders are used for the main beams and transverse floor beams. The length of the 1990 bascule span is 60.24-m and a width of 27-m [six traffic lanes] and has a weight (including ballast) of 1670 metric tons. A single control tower operates both parallel bascule bridges. The cracks propagate through the deck plate and the wearing course and grow in longitudinal direction parallel to the deck plate weld. This type of crack has been found in this bascule bridge deck and another thirty year old bridge deck. One longitudinal crack of 800 mm was found. Both steel decks are 12-mm thick on the movable bridges, surfaced with a relative thin wearing course of about 8-mm thickness. The trapezoidal stiffeners can not be observed during regular inspection from underneath of the bridge deck. The long adjacent parallel cracks could cause a deep deflection of the deck plate above the longitudinal trapezoidal ribs and repairs by grinding and filling the groove by a butt were performed. The dimensions of the rib is the German Steel Company "Krupp FHK 2/325/6" with a structural height of 325 mm, a base distance between the outside face of the trapezoidal legs of 300 mm, bottom width of 105 mm and a plate thickness of 6 mm [300 x 325 x 6] . The plate thickness of the crossbeam web support of the continuous longitudinal stiffener is 10 mm. The surfacing on fixed bridges thick wearing surfacing of 40 to 80 mm. The number of trucks in the heaviest loaded lane on this bridge amounts to be about 7000 trucks a day in 1997.

Japanese research has been extensive since they have the world's longest span Suspension Bridge; Cable-Stayed Bridge and floating bridge. All three bridges use orthotropic steel decks with wearing surfaces. A two part magazine article in Japanese in "Bridges and Roads" Oct 98 and Nov 1999 discusses the most common locations of fatigue cracks in their bridges. M. Shigeyuki; Ohta. K and N. Kazuhiro of PWRI Public Works Research Institute also have a study of rapid cool down of steel deck bridges. A nice graph compares, the around the clock, rapid "cool-down" of a bridge deck above a river. The slippery surface on two California's smaller Orthotropic bridges has required the posting of yellow warning signs. The most common ribs used in Japan are 320 x 240 x 6-mm (96 bridges); 320 x 260 x 6-mm (37 bridges) and 300 x 220 x 6-mm (32 bridges) from a table of 257 bridges utilizing 44 trapezoidal rib types. The Japanese also use the other rib types.

Dr Fisher's studies on Orthotropic steel decks have created a next generation system [see references # 1, 34, 35]. An internal baffle plate positioned inside the trapezoidal rib makes the deck have a longer fatigue cycle life. This detail has been used on decks for suspension bridges in New York and California. Practicing design engineers have combined research findings and testing into new code design formulas and repair techniques [see references # 1, 36]. The durability of their designs is also very important to them.

Brief Summary of Wearing Surfaces for Orthotropic Bridges

Researchers in the Netherlands and Japan have been testing new material systems. Caltrans research testing and wearing surfacing studies started in the 1960's [see references # 1,37, & 38]. A very large bridge San Mateo Hayward justified the widening of Ulatis Creek Bridge on I-80 in the City of Vacaville California. The twin existing two lane concrete bridges were widen to have 3 lanes of traffic in 1966. The high-speed lane 1 (no truck traffic) on the eastbound bridge was widened with the open-rid Orthotropic deck system of the proposed San Mateo Hayward Bridge. Four wearing surface materials were applied in 4 adjacent deck areas. The winner of the contest was epoxy asphalt. The economics of the test was justified because the original wearing surface remains part on the 1968 San Mateo Hayward Bridge. Also the Ulatis Creek Bridge's Orthotropic lane on I-80 remains in service with epoxy asphalt. Caltrans will be testing again for the two new orthotropic suspension bridges on I-80. The wearing surface interacts with the steel deck and may be thinner over bolted splice plates. Suppliers, researchers and design engineers are monitoring all the various products and performance. Some wearing surfaces have failed too quickly. Caltrans has had to resurface one orthotropic bridge with trapezoidal ribs after 22 years of service. [See references # 39].

Brief Summary of Design of Orthotropic Bridges

Birds and other creatures have nested in the handholes for bolt splice for trapezoidal ribs used bridges built in the 1960's. Expanding Inert Foam has been placed inside the cells or trapezoidal ribs in California. This material is believed to prevent internal corrosion; nesting of creatures; and possibly help in delaying "cool – down" of slippery decks. Fishers' orthotropic steel deck fatigue studies now part of the code. [See references # 1,37,38 & 39]. Many engineers believe this is best available solution at the moment. The coordination of orthotropic deck design research has not occurred since the 1960's. Every designer is left to state of the art literature search. Some major projects have had funds to perform specific research. A world conference was held when four box girder bridges within a two-year period collapsed killing people [see references # 1]. Traditional hand calculator methods are used to look at the design. Most practicing design engineers today have very powerful personal computers. A variety of "Finite Element " programs are available. In addition non-linear finite element analysis is also performed. Many engineers and firms have created "in-house" spreadsheets and other proprietary software. Comparing designs between countries is more complicated than just translating the languages (also more difficult because engineering slang or jargon varies with each country). Complicating the issue is that every country has it's own vehicle live loading. An interesting graphic comparison between code minimum design vehicle loads of Germany, Belgium, Sweden, Norway, Finland, Netherlands, Italy, Spain, USA (HS 20), Switzerland, United Kingdom, France and Japan is shown on pages 62 & 63[see references # 41]. The author goes into more detail comparing the United Kingdom BS = British Standard code vs. USA (HS 20). Japanese engineers M. Shigeyuki; Ohta K and N. Kazuhiro of PWRI Public Works Research Institute in their "Bridges and Roads" Oct 98 and Nov 1999 discusses the design vehicle loads for their bridges. Some Japanese codebooks are available in English versions. Quite a few American Orthotropic bridges have been fabricated in Japan. These complexities make it more difficult to compare design and maintenance issues.

Everything finally constructed is really a test structure. Engineers biannually monitor the actually real world performance of bridges. The FHWA is proposing the design of a bridge now be 100 years. Earlier orthotropic bridges have not been durable because the primary goal was to reduce steel weight. A moderate sized orthotropic bridge in the USA had a lot of fatigue cracks. The DOT owner decided to pour a concrete deck replacing the asphalt-wearing surface. This converted into a hybrid bridge using composite structure similar to a box girder with concrete deck. Since this fatigue retrofit was done a few years ago, the long-term results are not known. The DOT did not want to have endless studies on the fatigue cracks. Many small or moderate sized orthotropic bridges are not documented in the literature. If the project was documented it can be very hard to find, since most computer data base search engines limited to about the last 20 years. The majority of orthotropic bridges built in the 1960's are still in service and performing in an acceptable manner. The Author is interested in identify Orthotropic bridges, especially in North America, not included in this article. His employer, Caltrans will be maintaining the two major Orthotropic suspension bridges as they become part of Interstate I-80, plus has Caltrans the largest inventory of nine other Orthotropic bridges under one political jurisdiction in North America [see reference # 1].

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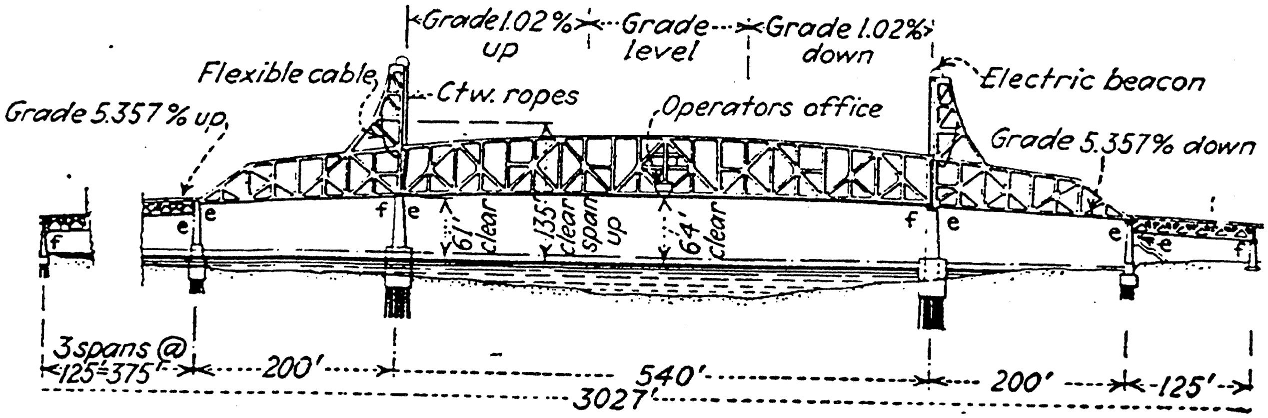
Light Steel Floor Governs Long Lift-Span Design

By E. E. PAUL
Assistant Engineer, Ash-Howard-Needles & Tammen. Consulting Engineers, New York, N. Y.

May 14, 1931 — Engineering News-Record

Burlington-Bristol bridge

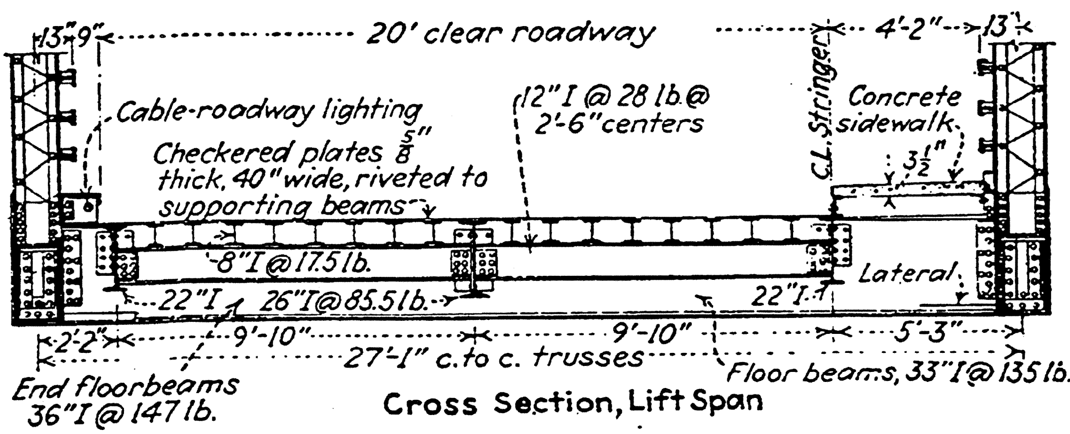
steel-plate floor on lift span effected $40,000 saving in structure.



Elevation

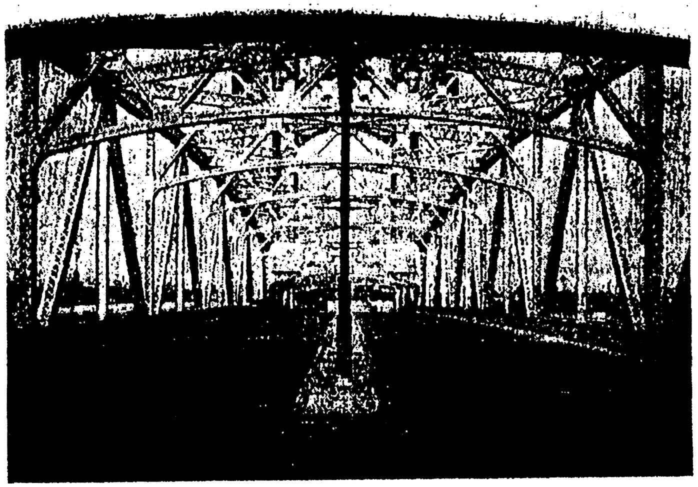
The adopted floor consists of ⅝-in. deformed steel traffic plates slightly over 3 ft. wide carried on longitudinal stringers spaced about 13 in. centers; these stringers are supported by cross-beams, in turn carried on the main stringers. The floor plates are riveted to the stringers, and the rivet heads are flattened to a height of about ⅛ in. No attempt was made to have the plates act as flanges for the stringers. With the arrangement used, the maximum stress on the plate occurs with moderate-weight trucks having narrow tires. Heavy trucks with wide tires will deliver a large part of their load directly to the stringers, thus producing smaller stresses in the plate. Since the plates are heavy enough to distribute some load to adjoining beams, the individual stringers are designed to carry only 80 per cent of the maximum wheel load.

Although the steel-plate floor was estimated to cost approximately $12,000 more than the next cheapest fireproof type, the light weight of the floor effected an estimated saving of $42,000 in the cost of the structure as a whole, after taking account of the effect of reduced weight upon the floor system, the trusses, the ropes, the machinery, counterweights, towers and piers. Roughly, the study showed that each pound added in the floor meant an additional cost of about 12c. in the trusses, towers, piers, etc.



Cross Section, Lift Span

The Burlington-Bristol Bridge – Figure 1



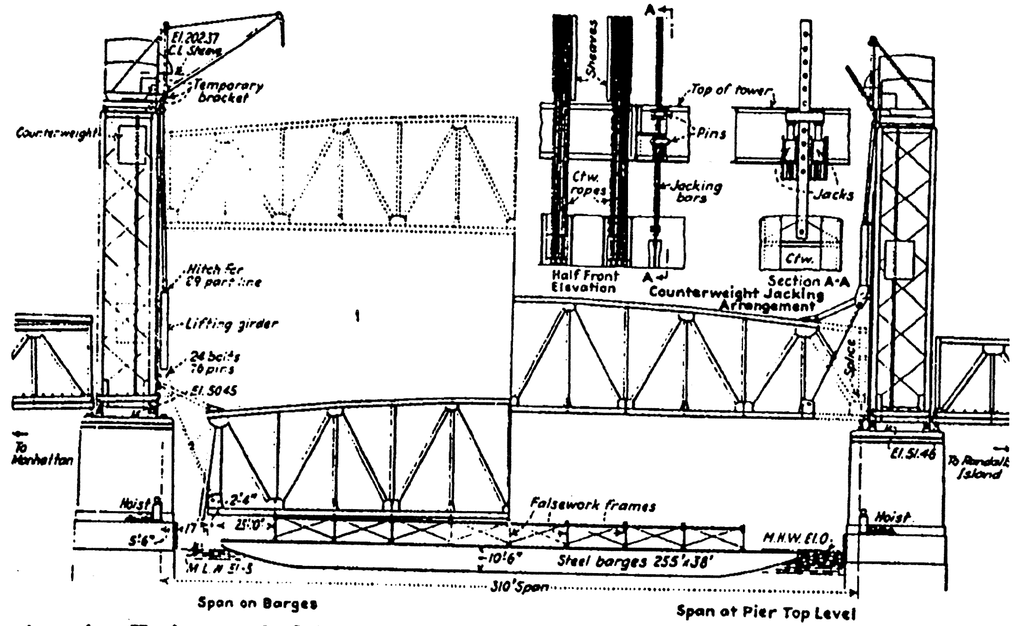
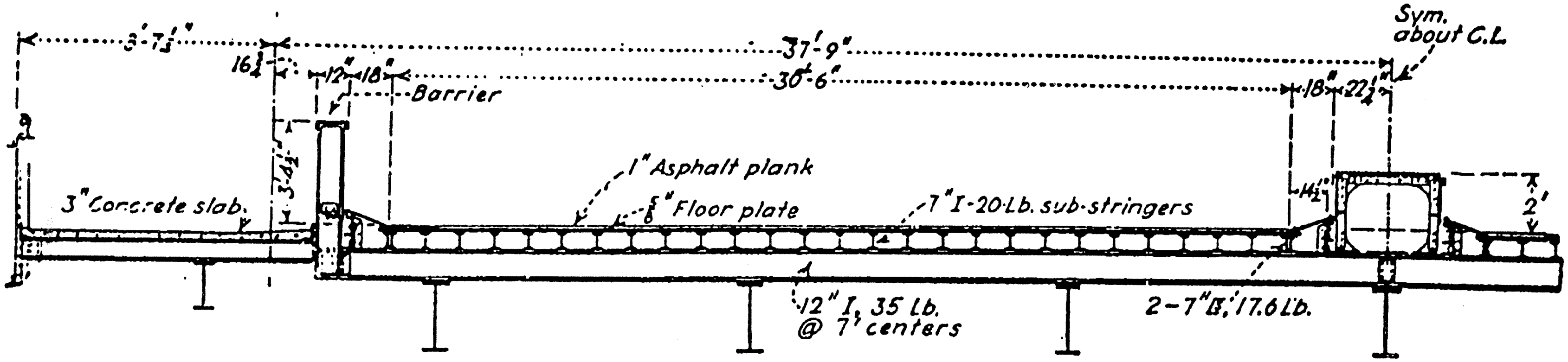
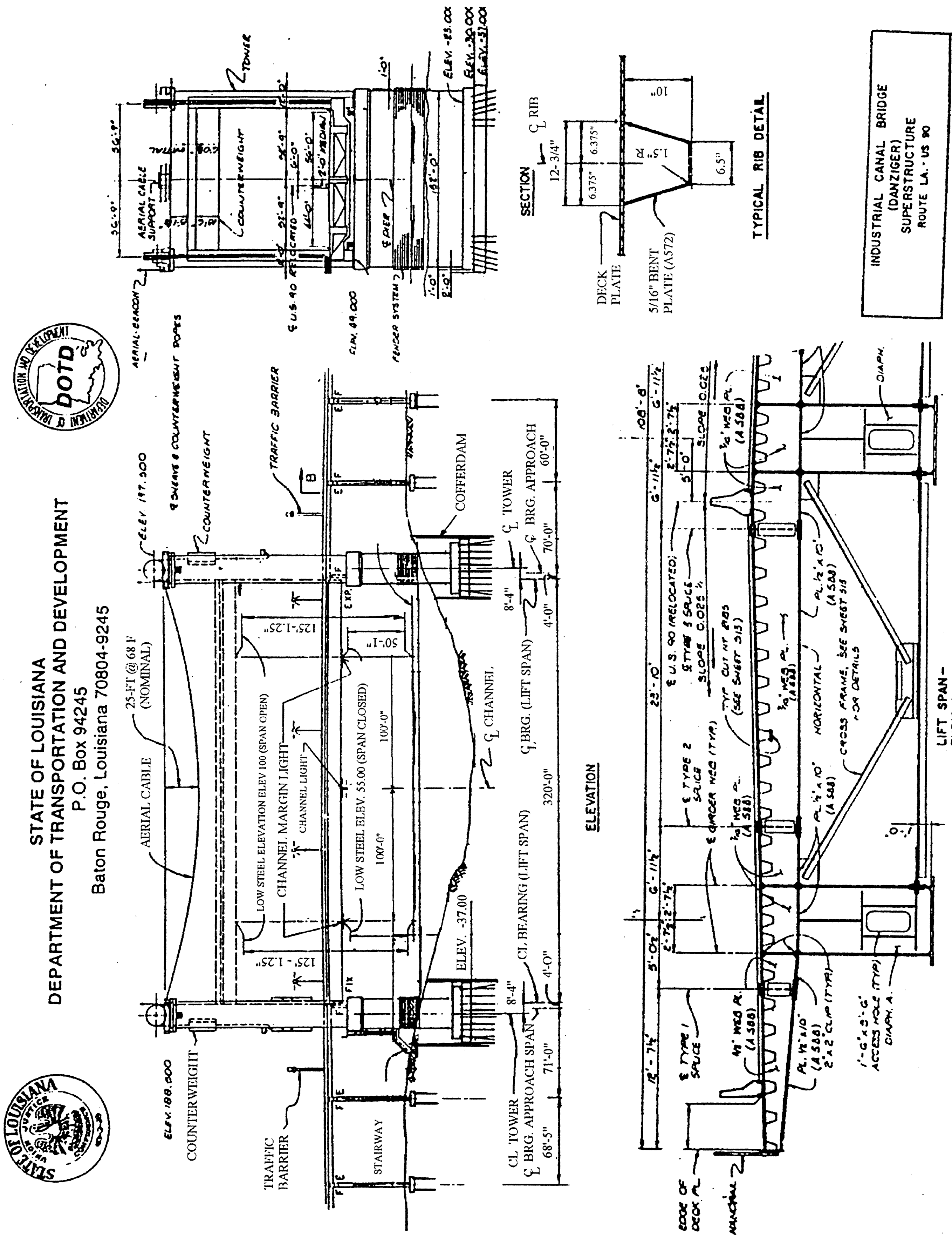


Fig. 1. Asphalt plank on welded steel plates give the Triborough lift span a lightweight deck



Triborough Lift Span



The Industrial Canal Bridge or Danziger Bridge – Figure 3
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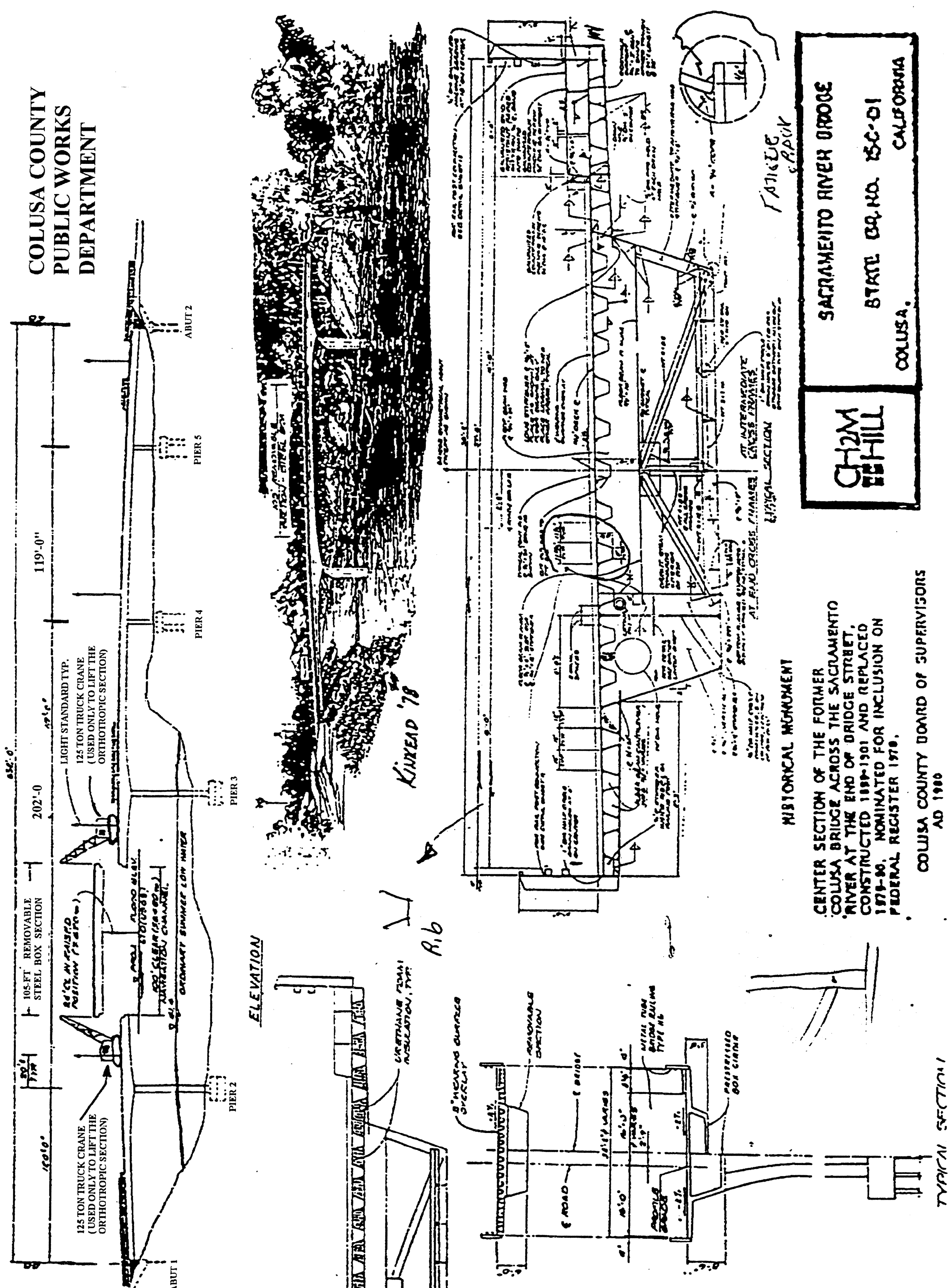
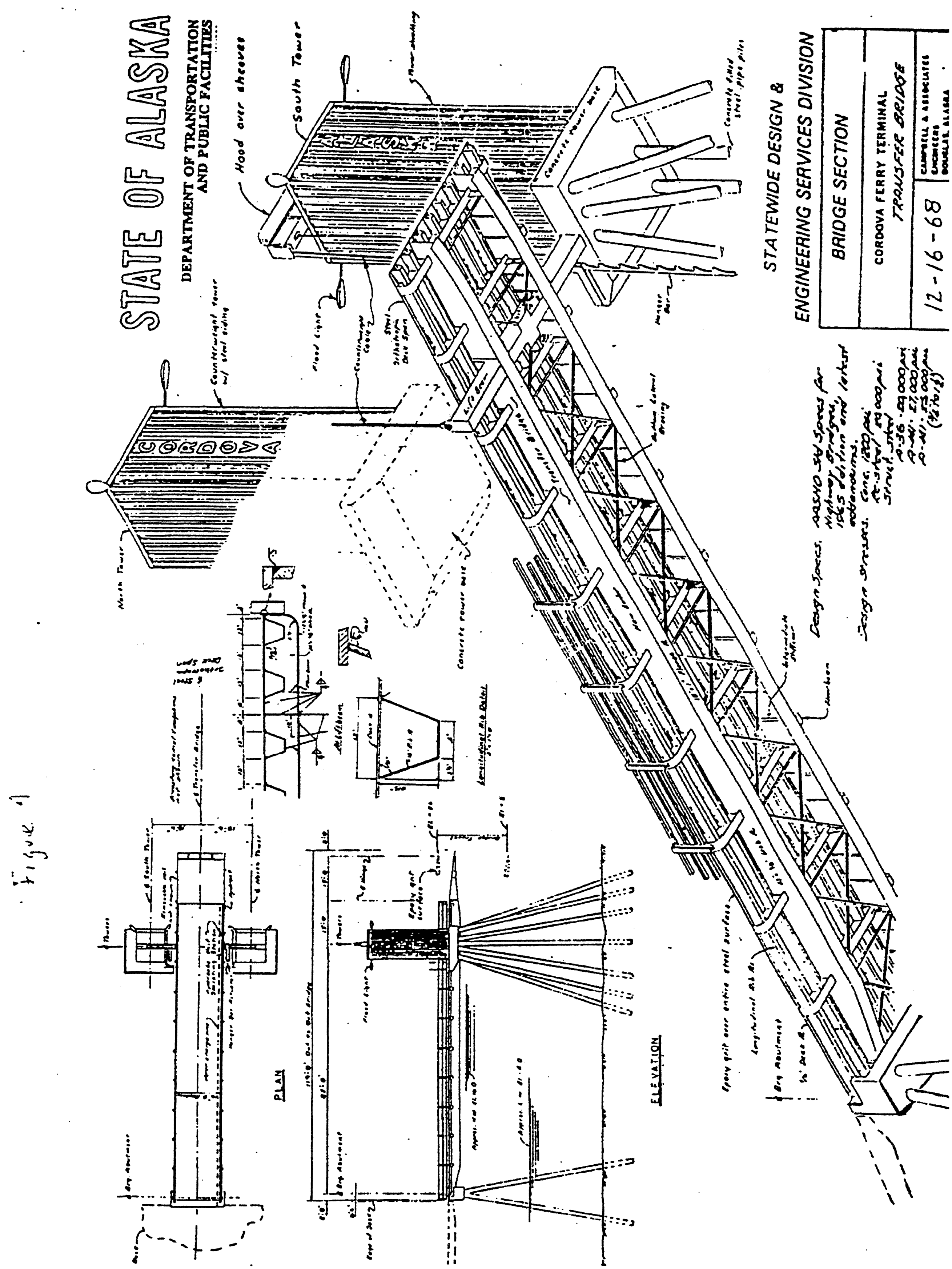
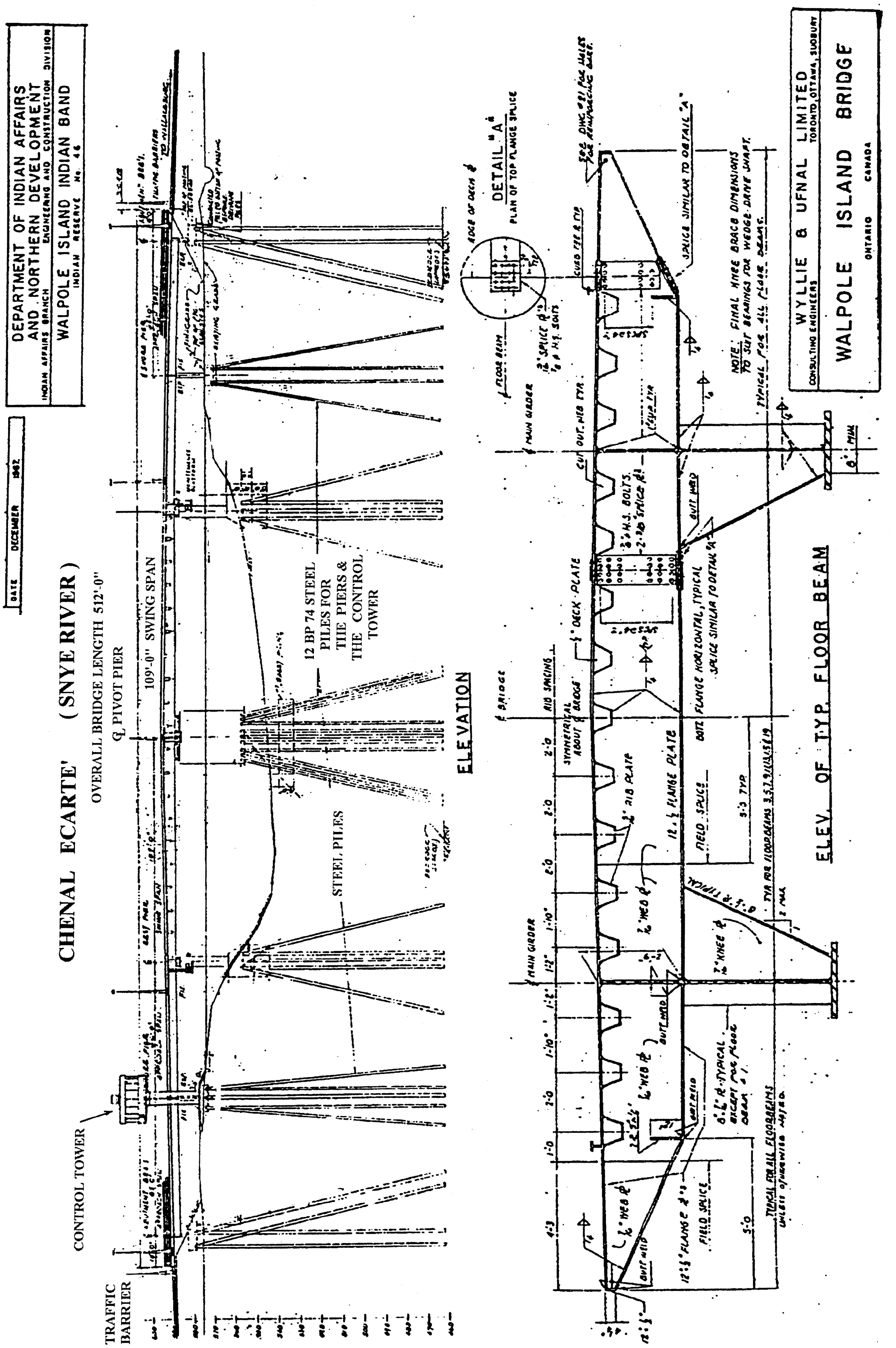


The Colusa California Bridge – Figure 5

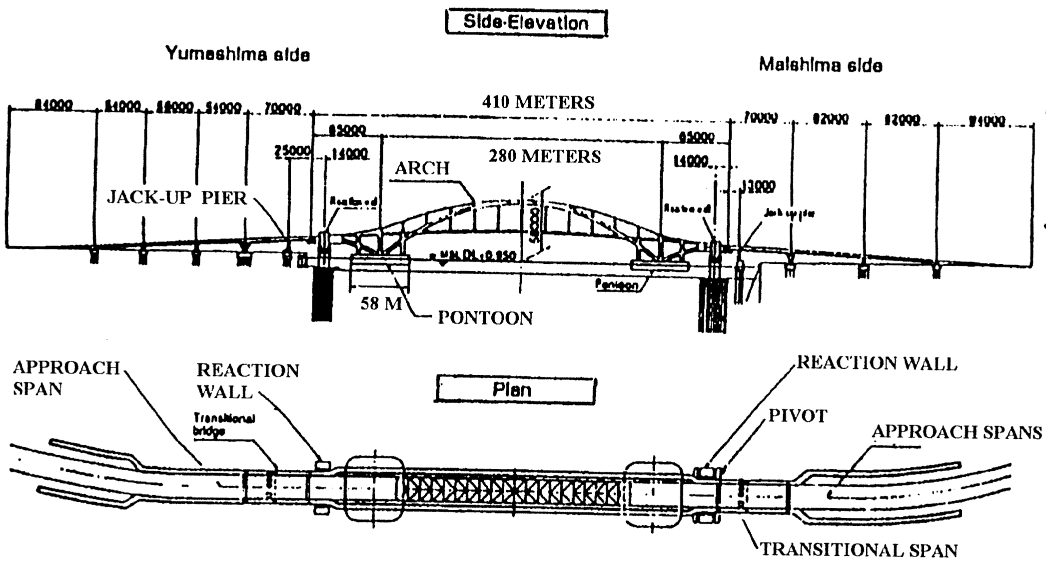


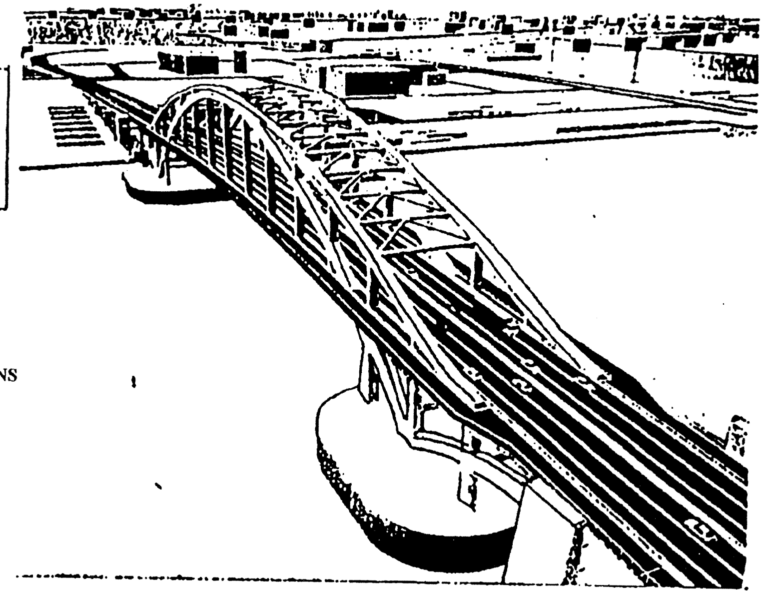
The Cordova Alaska Ferry Terminal Bridge – Figure 4



The Walpole Island Bridge – Figure 7

OSAKA MUNICIPAL GOVERNMENT





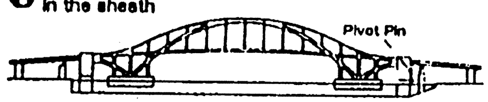
The Yumeshima-Maishima Bridge

Movable Floating Bridge

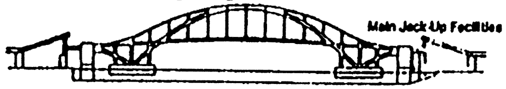
Swinging Procedure

Start of Swinging ▸ Insert of Pivot Pin ▸ Trandition Bridge of Jack-up ▸ Open Position of Reaction Wall ▸ Bridge Swinging ▸ Open of Waterway ◂ Reverse Swinging ◂ Standing Position of Reaction Wall ◂ Trandition Bridge of Jack-Down ◂ Original Up-Position of Pivot Pin ◂ End of Swinging

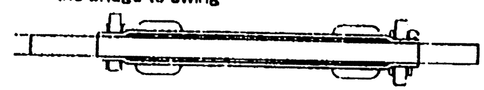
1. Swing center is made by inserting a pivot pin axis in the sheath



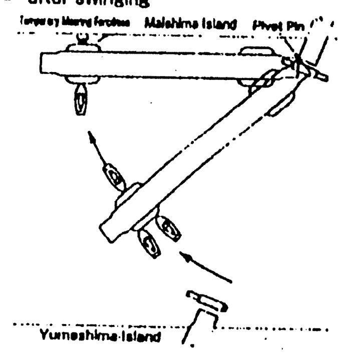
2. Transitional bridges are jacked up and separated from the floating bridge



3. Reaction walles take opening position for the bridge to swing



4. The bridge is temporarily moored after swinging



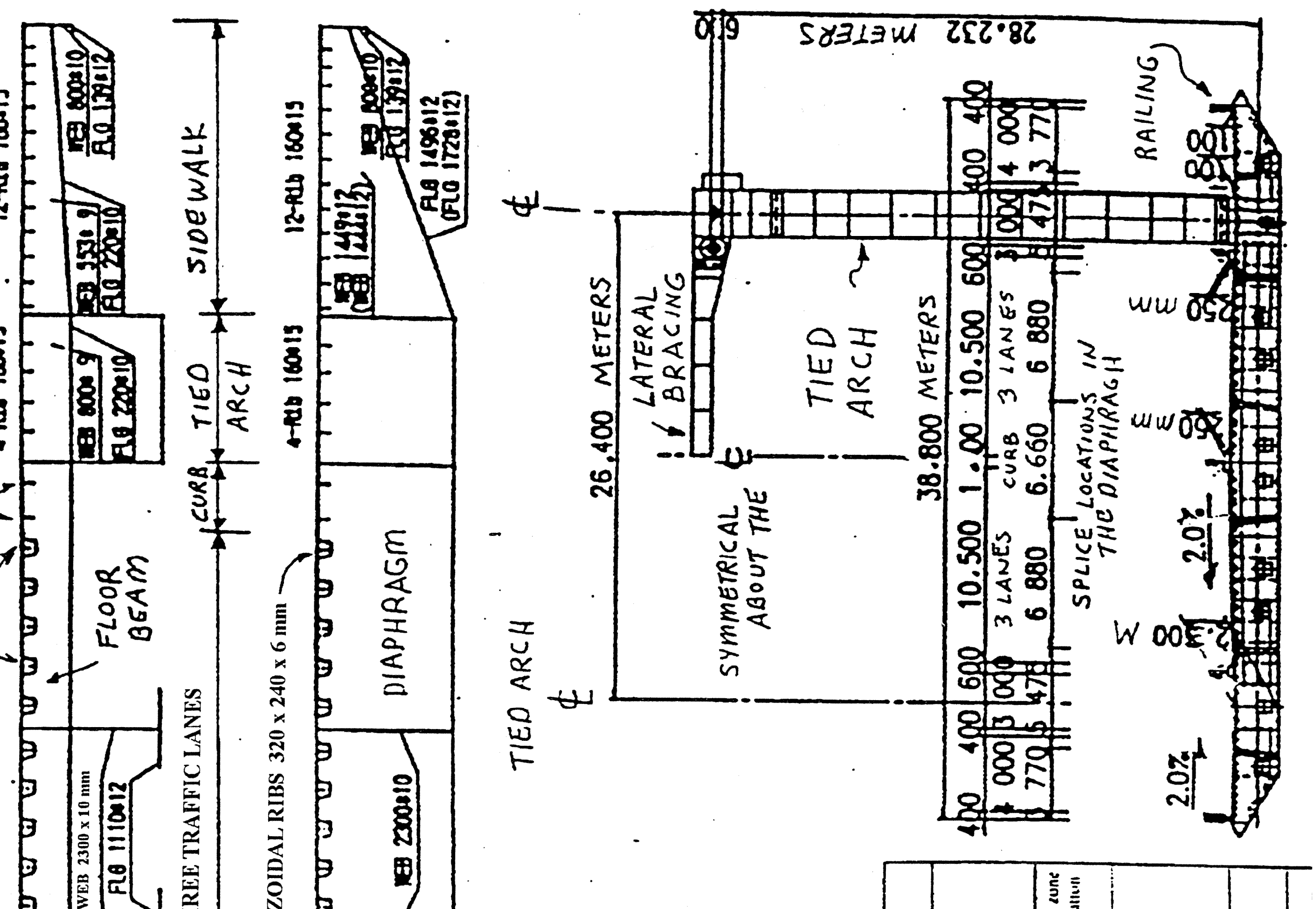
SUPERSTRUCTURE CONTRACTORS

HITACHI ZOSEN CORP.
MITSUI ENGINEERING & SHIPBUIDOING CO.,LTD.
YOKOGAWA BRIDGE CORP.
HARUMOTO CORP.
MITSUBISHI HEAVY INDUSTRIES, LTD.
KAWASAKI HEAVY INDUSTRIES, LTD.
MATSUO BRIDGE CO., LTD.
KATAYAMA STRATECH CORP.

Design Specifications and Approximate Steel Weight

Bridge type	Movable floating bridge(Swing type)		
Road standard	Grade 4, Type 1		
Live load	B live load	Wind load	V_{10}=42m/s
		Waves	$H_{1/3}$=1.4m, $T_{1/3}$=5.7~7.7s
Bridge length	410m		
Effective width	31.2m(6-lanes roadway with 2-side walks)		
Spans	65.7m+280.0m+65.7m		
Waterway width	Usual use	135.0m	Emergency 200.0m
Under clearance		DL+26.0m	——
Super elevation	2% straight line		
Longitudinal gradient	5% both grades (V.C.L140m)		
Pavement thickness	Roadway/80mm Walkway/40mm		
Expected Earthquake	Level I	Specification for Highway Bridge	
	Level II	Plate boundary and fault type	
Approximate steel weight (Unit in tons)	Pontoon	6,600	
	Floating bridge	18,200	
	Mooring structure	8,200	
	Total	33,000	

The Yumeshima-Maishima Bridge – Figure 8



Standard Japanese Trapezoidal Ribs used for Orthotropic Steel Decks

Top Deck Plate; Wearing Surface; 12 mm; 300 mm; Type; Main Girder Web

A × H × t	$\bar{s}$ (mm)	Moment of Inertia I (cm⁴)	MAX Span L_0 (m)
320 × 240 × 6	150	7.97 × 10[illegible]	2.20
	200	8.12 × 10[illegible]	2.25
	250	8.21 × 10[illegible]	2.35
320 × 260 × 6	150	9.41 × 10[illegible]	2.30
	200	9.57 × 10[illegible]	2.40
	250	9.71 × 10[illegible]	2.50
320 × 240 × 8	150	9.84 × 10[illegible]	2.35
	200	10.02 × 10[illegible]	2.45
	250	10.18 × 10[illegible]	2.55
320 × 260 × 8	150	11.62 × 10[illegible]	2.45
	200	11.83 × 10[illegible]	2.55
	250	12.02 × 10[illegible]	2.65

Summary of Ribs used for Orthotropic Steel Decks in Japan

		Cross Section	Shape	Range of Depth (mm)	Range of Span (m)	Comments
1	Open Ribs	Flat Plate		130~160	—	Normally used as tension zone. Requires special investigation for buckling limits
2		Bulb Angle		180~200	1.3~2	
3		Angle		200~250	2~3	
4		Split "T"				
		Closed Ribs		[illegible]	[illegible]	Wall thickness

The Yumeshima-Maishima Bridge – Figure 9