Steel Box Girder Bridges

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THE STEEL BOX GIRDER bridge, because of its structural efficiency and aesthetics, has become increasingly popular as a bridge type. Because of four unfortunate erection accidents of box girder bridges in Austria, the United Kingdom, Australia, and Germany, this type of structure has received much recent attention. However, because of this attention, the behavior of steel box girder bridges is perhaps better understood than any other bridge type through extensive testing and improved mathematical models for use with computers. As a result of the United Kingdom investigations into the collapse of the Milford Haven and West Gate bridges, this will be the first long span bridge type with standard specifications covering its design and construction. The aerodynamically stable slender steel box girder bridge with cable stays will challenge the suspension bridge for the longest spans and may win. The steel box girder will be seen more commonly because of its aesthetics and economy in shorter spans as well, especially on horizontally curved alignment.

SPECIAL CONSIDERATIONS

In a discussion of box girders we encounter structural behavior and terms which may be of less significance for other structural types but which can be important in box girder design. One is *shear lag*. Conventional beam theory neglects deformations caused by shearing stresses in assuming that plane sections remain plane. This assumption is reasonable until the beam flange becomes unusually wide as is the case with box girder bridges.

If the tension flange of a simple span H-section could distort under the influence of the shearing stresses resulting from a mid-span load, it would warp, causing a maximum separation where the web transfers the shear to the flange (Fig. 1a).

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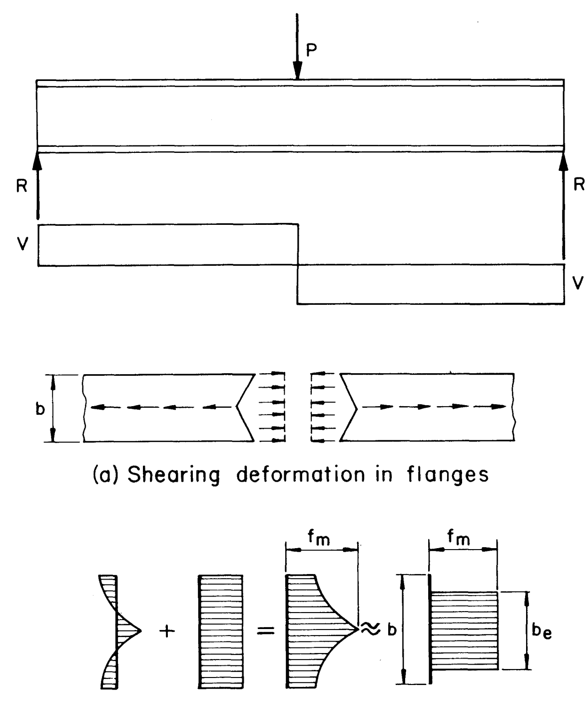
Because the flange is, in fact, not free to distort, the beam cannot warp at the symmetrical mid-span section (Fig. 1b). To get it back to its continuous shape again, a secondary stress must be superimposed upon the uniform stress distribution assumed. The sum of these tensile and compressive stresses results in equal and opposite forces.

The addition of this secondary stress and the uniform shear stress results in a shear diagram which is maximum at the mid-flange web connection and minimum at the flange tips. This secondary shear stress effect is, of course, maximum at the center of the span, where there is a sudden change in the shear diagram, and zero at the beam ends, where the flange cross section is free to distort.



(a) Shearing deformation in flanges

(b) Stress distribution

Figure 1

It is convenient to use the concept of *effective width*, which assumes that the maximum stress is distributed uniformly over a narrower width of the flange, which encompasses the same area as the area under the actual diagram. This reasoning applies to the compression flange as well as the tension flange, but the compression flange strength is influenced by buckling. The non-dimensional plot of effective width over width against span length over width shows the effect of shear lag to be much greater for a concentrated load than for a distributed load (Fig. 2).

Another consideration is the post-buckling strength of the stiffened compression plate (Fig. 3). A plate section under compressive forces while simply supported along its unloaded edges behaves differently from the behavior of a series of independent vertical column elements; at the Euler critical load the column elements would fail, whereas, with these elements acting together in the plate, orthogonal bands or elements can be considered acting horizontally between the edge supports, inhibiting collapse. In contrast to failure of the individual column elements by excessive deflection and collapse, the plate gradually develops buckling waves, but will not fail. The larger the slenderness ratio, the greater the post-buckling strength; the stouter the plate (smaller slenderness ratio), the lower the post-buckling strength. The horizontal bands restrain the elements from buckling to a greater degree near the supported edges. Those elements with the most buckling restraint resist the increased stress up until they yield. To complete the membrane action of our grid we should include diagonals between the horizontal and vertical elements to represent the shear stresses.

The area under the stress curve of the buckled plate being replaced by two rectangles with the same area was advanced by Von Karman in 1932. The rectangles are the maximum stress high and half the effective width wide. The ratio of effective width to width was estimated to be proportional to the square root of the ratio of the critical stress over the maximum stress. This was modified to conform to test performance at Cornell University by Winter. This modification tacitly considers plate geometrical and material imperfections because it is based on actual plate loading tests.

Of course the post-buckling strength of a compression plate is limited, especially for the low slenderness ratios common for box girder bridge flanges. Also the initial imperfections (out-of-flatness and residual stresses) have a marked influence upon the strength of the compression flange. This may be contrasted to the much higher post-buckling strength of a shear panel which resists shear by tension field action after buckling. It should be recognized also that the web's participation in bending resistance is thrown to the flange after it buckles.

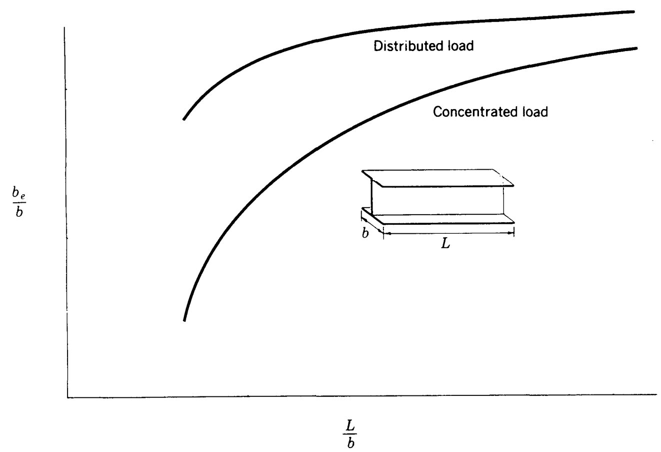


Fig. 2. Effective width due to shear lag

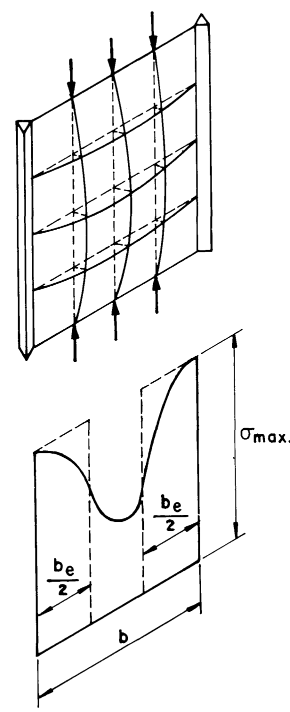


Fig. 3 Buckling of stiffened compression plate

Box girders are ideal for bridges because of their torsional rigidity. However, cognizance must be taken of the stresses resulting from the transfer of forces in the box. An eccentric load over one web of a box can be broken down into three basic actions (Fig. 4). These are: (a) symmetrical flexure, (b) pure torsion, and (c) distortion:

(a) The flexural action causes longitudinal flexural stresses and transverse shearing stresses.
(b) The pure torsional forces are resisted by transverse torsional stresses.

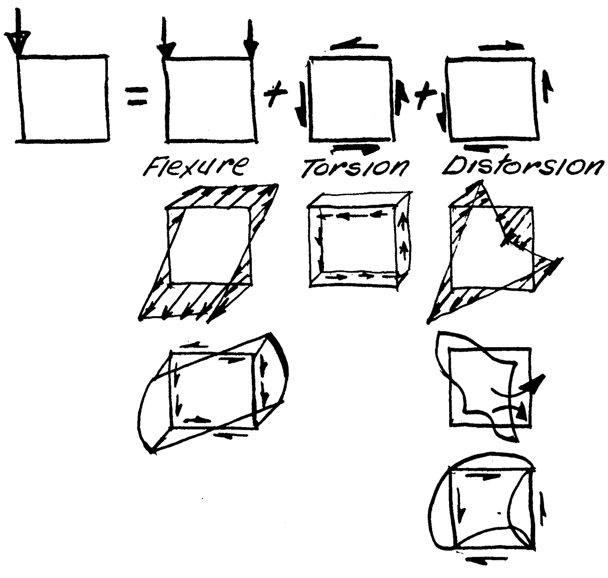


Fig. 4. Box girder under eccentric load

(c) The distortional forces cause longitudinal stresses which are maximum at the corners, bending stresses transverse to the plane of the cross section, and distortional warping stresses.

However, the design approach should be to select the box section based upon the classical elastic theory neglecting the effects of shear lag and distortional stresses, then to superimpose these secondary effects and modify the section where and if necessary.

HISTORICAL DEVELOPMENT

On March 18, 1850 the first track of the Brittania Bridge over the Menai Straits in Wales was opened to traffic and the adjacent tube was opened to traffic that October (Fig. 5). This was the first metal box girder bridge and was fabricated of wrought iron plate.

In designing this bridge Robert Stephenson consulted with an iron ship builder and a mathematician and decided upon the rectangular design. He then tested a one-sixth scale model and decided that the structure would be stiff and strong enough acting alone to eliminate the suspension chains which were part of the original design.

Each of the 472-ft 1500-ton central box girders were floated into position and lifted up to their supports with hydraulic jacks positioned above the bearings, This was a 108 ft lift. This was certainly a great feat in its day and the bridge still stands, 123 years later, carrying heavier loads. It is a monument to one of the great engineers who introduced in this bridge the box girder concept, riveting, and continuity in design. The girders are continuous through the towers.

The box girder suffered the criticism of being very expensive compared to other types, and wasteful of material compared to an open trusswork structure. There was also the problem of train passengers being subjected to engine fumes as the car traveled through the boxes.

The system was not revived until a hundred years later. This, however, was a gradual development that was preceded by orthotropic plate deck bridge construction. Orthotropic plate deck construction was an outgrowth of steel battle deck bridge construction.

Battle deck construction was advocated in the 1930's in an attempt to reduce the dead load of highway structures. Typical of this construction would be a 2-in. asphalt wearing surface carried by a $\frac{3}{8}$ to $\frac{3}{4}$-in. steel deck plate welded to rolled beams spaced from 10 to 33 in. apart (Fig. 6). These stringers were carried on floorbeams 16 to 25 ft apart, which were framed into the main longitudinal trusses or girders. Design information was distributed by AISC for these battle decks.

The battle deck participated with the stringers as part of their top flanges in carrying the load. It did not participate with the floorbeams or contribute to the stiffness of the main carrying members. For these reasons the battle deck bridge did not provide the economy sought, but it did provide valuable information about the structural behavior of bridge decks.

The Germans, concurrently with US engineers, experimented with cellular construction. After World War II, when steel was in short supply and many structures had to be replaced, the orthotropic plate deck bridge was rapidly developed and used for German bridges.

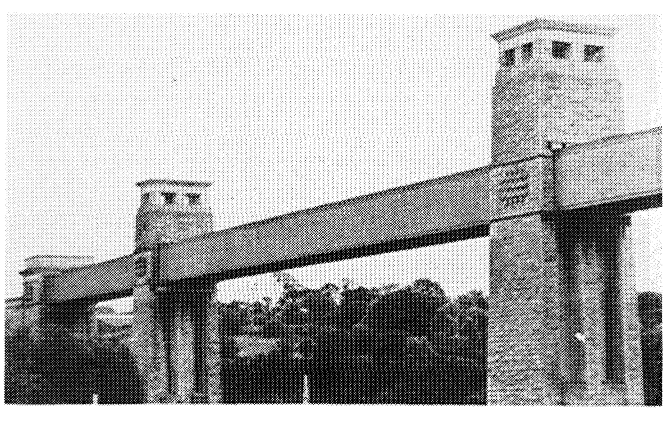


Fig. 5. Brittania Bridge

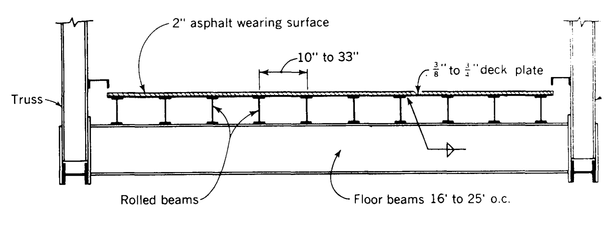


Fig. 6. Steel battle deck

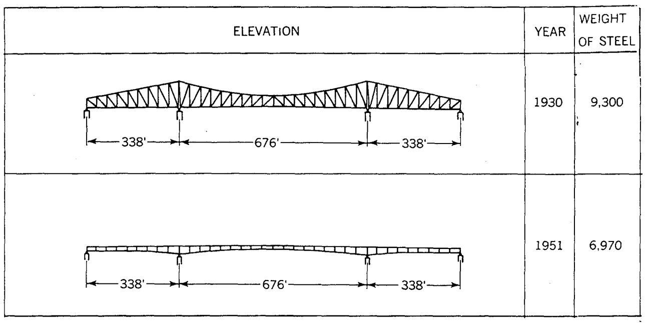


Fig. 7. Duesseldorf Neuss Bridge

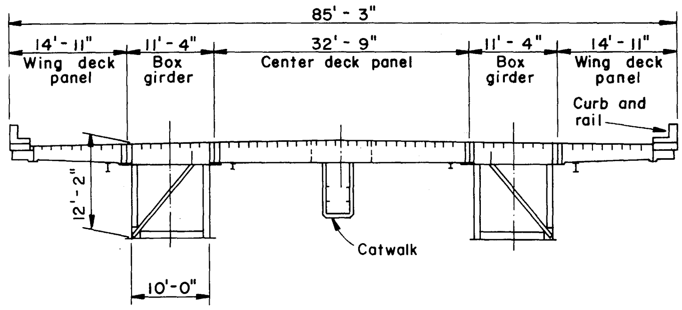


Fig. 9. San Mateo–Hayward Bridge—cross section

The orthotropic plate deck was first used with plate girders, but soon evolved into its more common use with the torsionally stiff box girder for long span bridges. The considerable savings achieved in the 1951 vs. the 1930 Duesseldorf Neuss Bridge with use of this system was increased for subsequent structures (Fig. 7). The steel box girder is becoming more popular now as used in conjunction with cable supports in the cable stayed bridge.

AISC's *Design Manual for Orthotropic Steel Plate Deck Bridges* was based upon the German developments and is the design reference used for these bridges in this country and elsewhere.

The first major orthotropic steel plate deck bridge opened to traffic in the U. S. was the San Mateo-Hayward box girder bridge in California, opened to traffic on October 31, 1967. The San Mateo-Hayward Bridge main channel center span is 750 ft, the flanking channel spans are 375 ft, and the approach spans are 292 ft. The channel and 292 ft approach spans are all orthotropic plate deck box girders. The shoreward 208-ft spans are steel boxes with a composite steel and concrete deck (Fig. 8).

The steel orthotropic plate deck spans consist of steel box section anchor spans cantilevering beyond the piers in alternate spans to engage shorted suspended simple spans in the other bays. The hinges are concealed in the interior of the boxes, giving the bridge the appearance of an uninterrupted continuous structure. The suspended system can accommodate differential support settlement without inducing large stresses in the girders.

The typical 292-ft approach span cross-section depth is 12 ft. The girder depth transists from a 12-ft depth to a depth of 30 ft over the channel piers. The two 10-ft wide box sections, including their 11 ft-4 in. top orthotropic plate deck flanges, were erected first (Fig. 9). A feature of this project was that all the girder sections were assembled in an assembly yard with all the loading conditions which they would experience in the erected state. They were fitted, match-marked, disconnected, and shipped to the site. Erection fit-up was a foregone conclusion when the sections left the assembly yard.

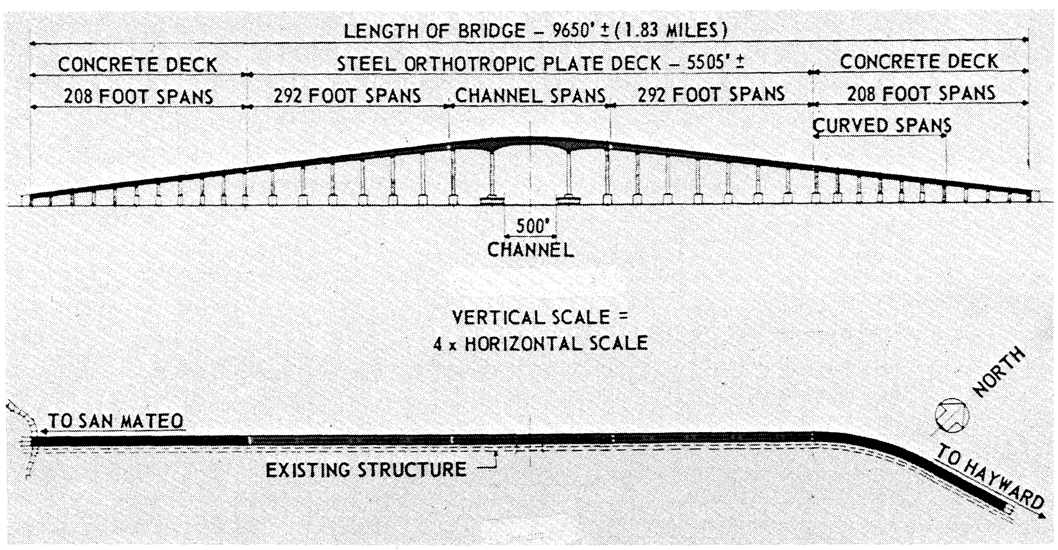


Fig. 8. San Mateo-Hayward Bridge—plan and elevation

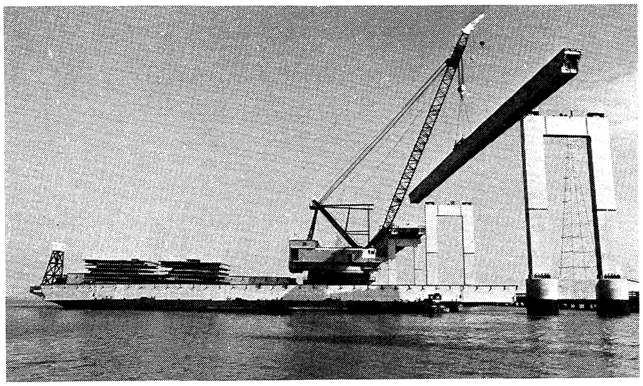


Fig. 10. "Marine Boss" derrick

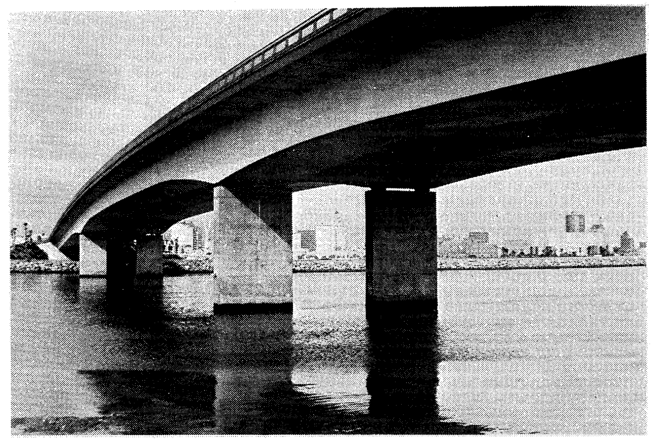


Fig. 12. Queen's Way Bridge

ERECTION BY LARGE LIFTS

San Mateo-Hayward Bridge—Murphy Pacific Corporation's philosophy of bridge erection is that the closer a bridge can be preassembled and erected to a single unit, the more economical will be the erection. Murphy Pacific applied this concept to the erection of the San Mateo-Hayward Bridge by use of a huge capacity marine derrick especially designed and built for this project. This derrick, named "Marine Boss," had a 550-ton lifting capacity (Fig. 10).

The center orthotropic deck plate section was placed between the box sections directly after the boxes were erected. The 375-ft, 520-ton suspended center channel box girders deflected 11 in. under their own weight. In order to force the orthotropic plate deck center and wing sections, which were added later, to participate with the box sections in resisting all the loads, towers and tie back cables were used to deflect the girders back upward while these panels were placed (Fig. 11).

Queen's Way Bridge—The Queen's Way orthotropic plate deck box girder bridge (Fig. 12) carries traffic to the present berth of the Queen Mary in Long Beach, Calif. It was opened to traffic in 1970. It is two separate adjacent 3-lane box girder structures resting independently on two lines of piers. Murphy Pacific increased the capacity of the Marine Boss from 550 tons to 650 tons to lift the 617-ton center box sections which closed the main span of this bridge. The spans are 350 ft—500 ft—350 ft.

Bryte Bend Bridge—The attractive composite steel box girder Bryte Bend Bridge over the Sacramento River in California was opened to traffic in 1971 (Fig. 13). The main symmetrical spans between the edge river piers and the mid-river pier are 370 ft; the flanking spans are 281 ft. The shorter approach spans have the same configuration but transist to a shallower depth. The boxes are continuous over the center pier for a length of 300 ft on each side of it. An interior hinge support connects it to another length of box which is continuous over another pier.

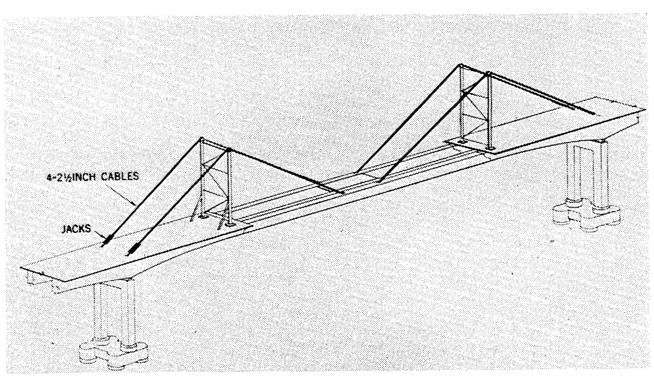


Figure 11

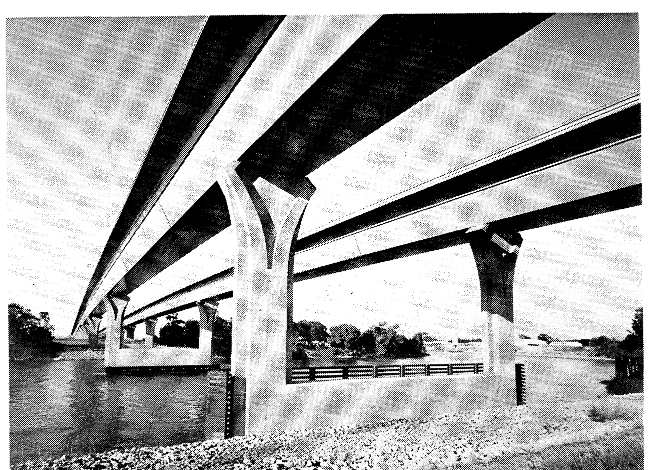


Fig. 13. Bryte Bend Bridge

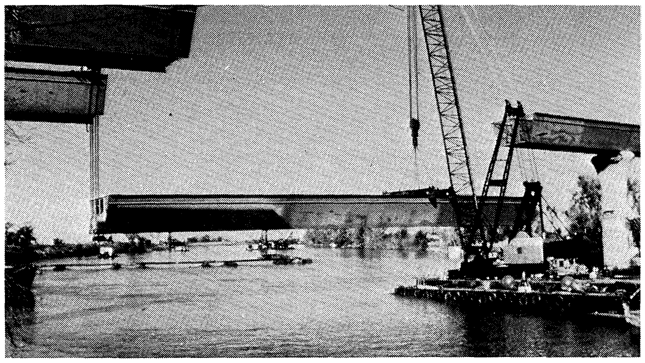


Fig. 14. Bryte Bend Bridge

The shallow river depth precluded Murphy Pacific's Marine Boss from traveling up the river for the erection of this structure, so a new derrick barge 87 ft x 200 ft with an 11-ft draft and a 250-ton lifting capacity was constructed to make long box section lifts. A 220-ft long section weighing 307 tons was lifted by jacks at one end and the derrick at the other end (Fig. 14). The eight 25-ton hydraulic jacks which were used in conjunction with the derrick were positioned atop the cantilevered end of the erected box section which extended over the mid-channel pier.

Fremont Bridge—The center arch span of the world's longest tied arch bridge was jacked up into place by Murphy Pacific in mid-March 1973. This was the 902-ft center span (weighing 6000 tons) of the Fremont Bridge in Portland, Oregon. It was erected upon river piling downstream from the bridge site.

When the end spans were completed, the center arch was barged up to the site. Then it was lifted with the use of 32 hydraulic jacks (Fig. 15).

Fig. 15. Fremont Bridge

ERECTION BY BALANCED CANTILEVERING

San Diego-Coronado Bay Bridge—Construction of the steel orthotropic deck box girder San Diego-Coronado Bay Bridge was completed in June 1969. The method of erection was balanced cantilevering.

A special design feature of this bridge is that the main span girders are cantilevered from the adjacent anchor spans for dead load and act continuously with the adjacent spans for live load only. This design was ideal for erection by the balanced cantilever method. The main channel span lengths are 660 ft—660 ft—560 ft (Fig. 16).

The erection sequence used five temporary erection bents. Three were used for the longer flanking span on the Coronado side of the bridge (Fig. 17). Two were used for the shorter span on the San Diego side.

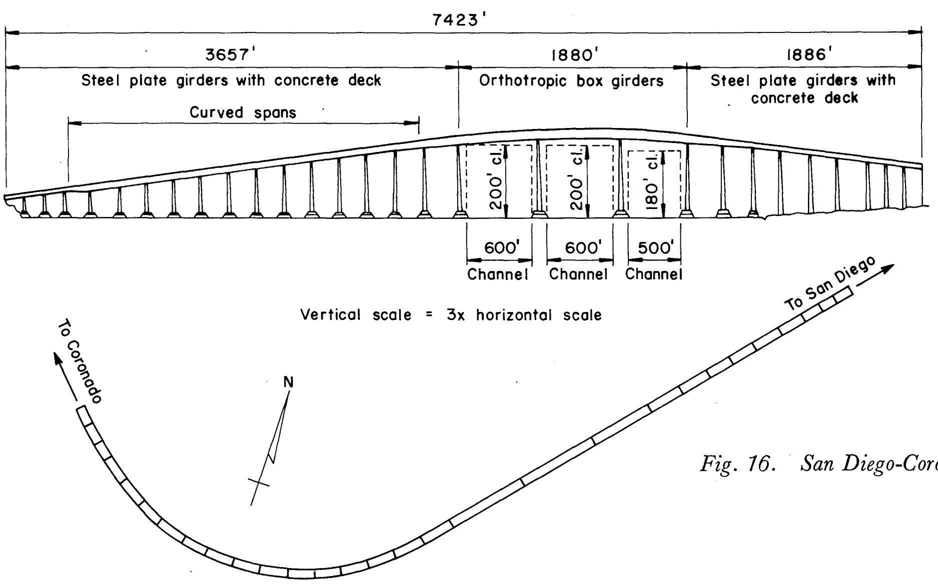


Fig. 16. San Diego-Coronado Bridge—plan and elevation

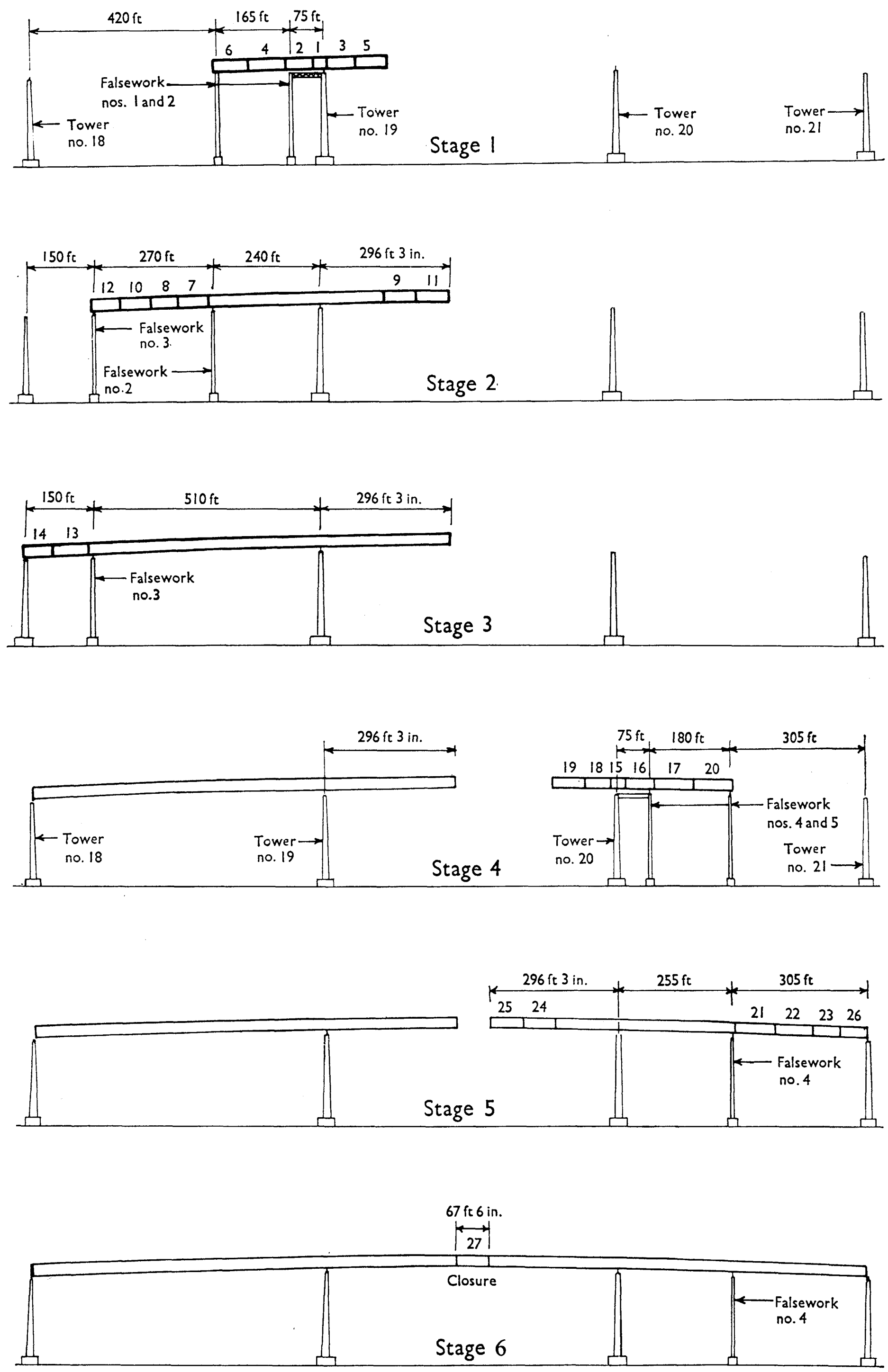


Fig. 17. Erection scheme, San Diego-Coronado Bridge

The 34-ft wide by 25-ft deep cross section of the box was maintained over the three main channel spans. The 15-ft wing sections were added after each of the 27 box sections was spliced to the previously erected section (Fig. 18).

The boxes were held while the required initial minimum splicing was done. When the splicing was completed, the wing sections were connected. The final section was placed, of course, with no attempt to induce continuous action for the dead loads across the center span. The additional live loads only are carried with box continuity across this mid-channel span.

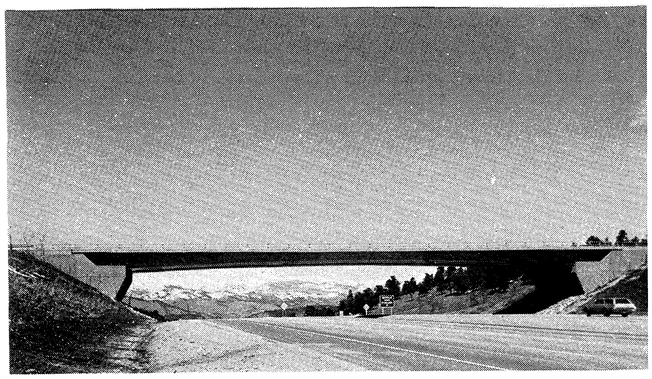


Fig. 19. Genessee Interchange

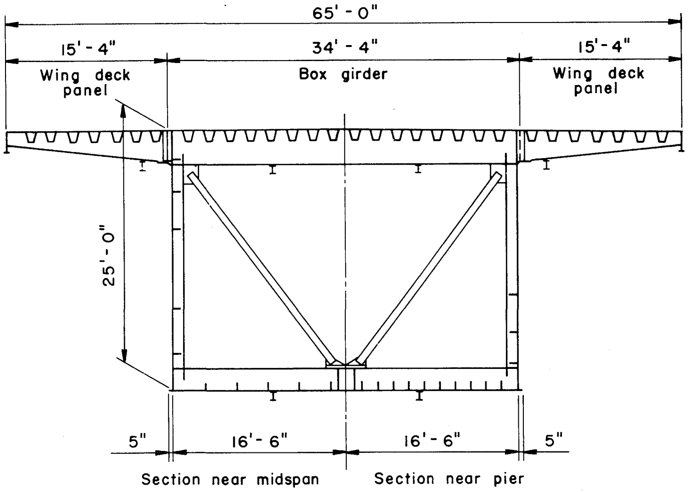


Fig. 18. San Diego-Coronado Bridge—cross section

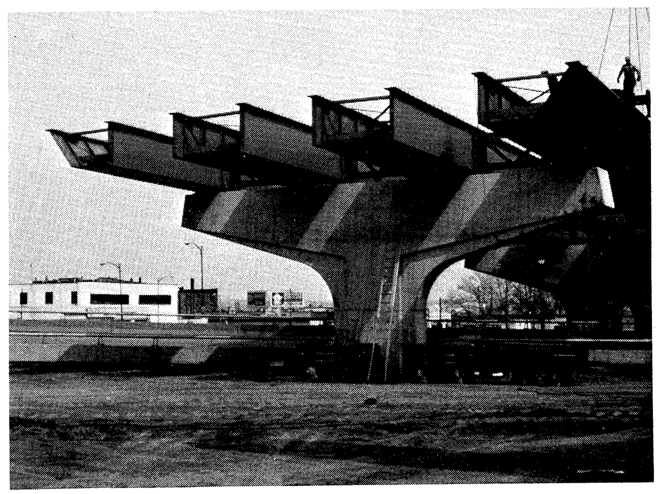


Figure 20

MODERATE SPAN HIGHWAY BRIDGES (MOBILE CRANE ERECTION)

The shorter span highway grade crossings using steel box girders compositely with a concrete deck account for the greatest number of the steel box girders built in this country. The aesthetics of the Genesee Interchange box girder bridge in Jefferson County, Colorado (Fig. 19), which was a 1971 American Institute of Steel Construction prize bridge winner, attests to the fact that there will be more of these steel box girder bridges built in the future.

These shorter span highway grade separation steel box girder structures are subject to the AASHO specifications. AASHO adopted criteria for the design of single cell steel box girder bridges with composite concrete decks in 1969. These provisions are intentionally simplified and conservative and are intended for highway bridges of only moderate length (Fig. 20).

These criteria are based upon folded plate theory and diaphragms are placed only at support locations without the use of interior diaphragms. Shear lag, which results in non-uniform stress distribution, is limited by restricting the tension flange plate widths to one-fifth of the span length for simple spans, and one-fifth of the distance between points of contraflexure for continuous spans. The post-buckling extra strength of slender compression flange plates is ignored.

These bridges are erected with mobile cranes, usually without temporary bents. The minimum amount of steel required in a design of this type occurs with a box girder span-to-depth ratio of approximately 25 using the AASHO rules.

The 1971 annual supplement to the 1969 AASHO specification has included load factor design for steel highway bridges. These provisions also apply to the design of composite steel box girders.

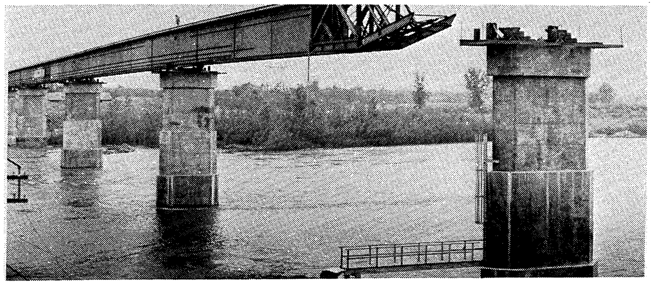


Fig. 21. Erection of Kansas City Southern RR Bridge

RR BRIDGE ERECTED BY LAUNCHING

Kansas City Southern RR Bridge—Current AREA specifications do not include provisions for box girder design. The Kansas City Southern RR Bridge, a 1972 AISC prize bridge award winner, is a steel box girder bridge designed within the applicable AREA provisions, but based upon box girder behavior.

The method used in erecting this bridge was launching. The lengths of full box girder sections were assembled upon dollies at each shore and then this train of spliced boxes was shoved out over the river piers by the force delivered to wire rope falls by a 200 HP hoisting engine (Fig. 21).

Three-in. wide launching rails, permanently welded to the bottom of the box flanges at the web line, slid over skidway units mounted on the abutment and the piers as they were shoved out from the shore. A nose with sloping rails for mounting the skidway supports on the piers was attached to the front of the boxes. This nose was extended with a 30-ft length of light trusswork behind it for the purpose of reducing the maximum negative moment at the pier it cantilevered beyond, which would be larger with the heavier box section. The bridge was completed and opened to traffic in 1971.

FOREIGN CONSTRUCTION

The steel box girder bridge developed earlier and more ambitiously in Europe and the United Kingdom. Use was made of wide shallow boxes for longer and longer span lengths.

Designs employed cable stays in conjunction with the box, reducing the bending requirements of the cross section. The penalty of increased axial stress in the box was easily accommodated with additional longitudinal stiffening.

The Germans were the forerunners in these developments and the appearance of these long graceful cable stayed box girder bridges, in addition to their economy, increased their popularity in other countries. The most common erection procedure for box girders in Germany is by cantilevering, but the various elements of the boxes are erected separately instead of lifting prefabricated lengths of the box.

CONSTRUCTION FAILURES

Unfortunately, there have been erection failures of four box girder bridges. These were:

(a) The *Fourth Danube Bridge* in Vienna on November 6, 1969 (Fig. 22). Failure was attributed to: distribution of the erection loading being different from uniform distribution assumed, greater temperature differential than assumed, and fabrication imperfections. But more importantly, it introduced doubts in Europe and the United Kingdom about the linear plate buckling theory.

The Fourth Danube Bridge in Vienna was repaired and opened to traffic at an additional cost of only 3.5% of the total cost of the bridge. There was no delay in the overall motorway scheme because the approach roads were still not ready when it was repaired.

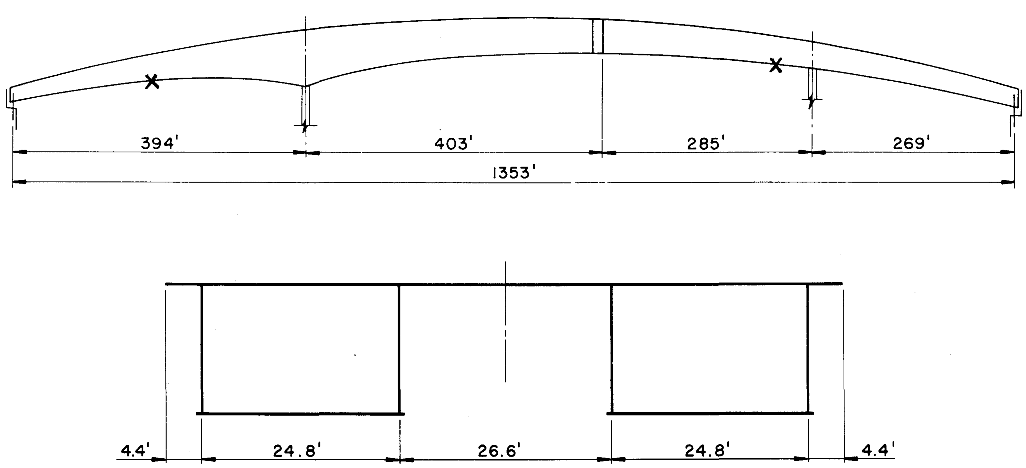


Fig. 22. Fourth Danube Bridge (failure location indicated by X)

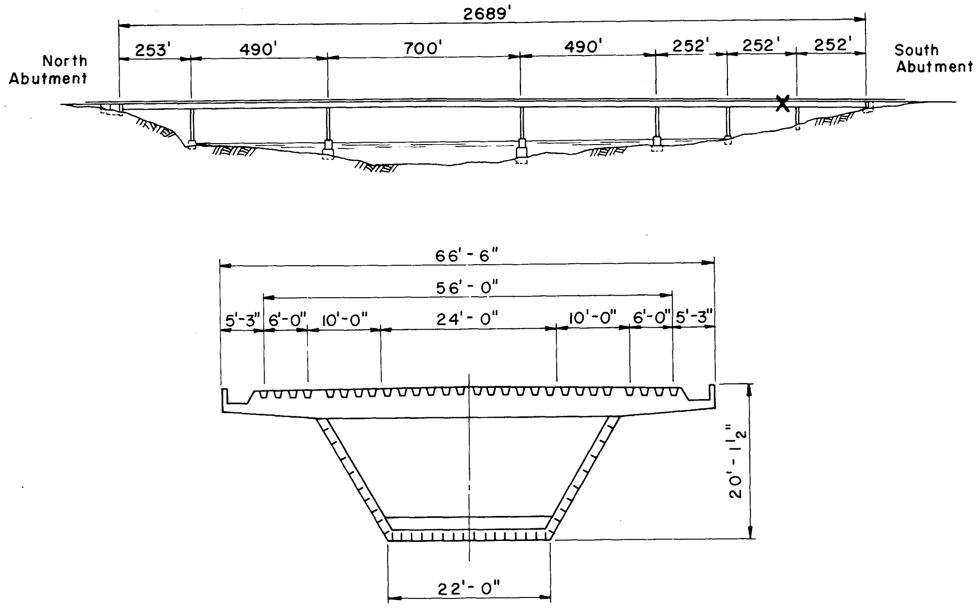


Fig. 23. Milford Haven Bridge (failure location indicated by X)

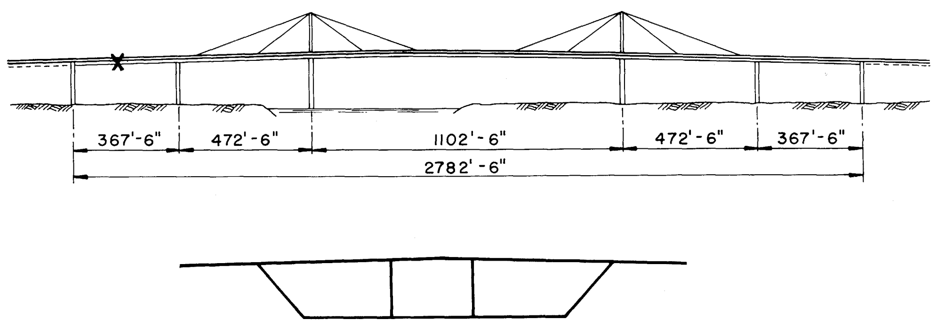


Fig. 24. West Gate Bridge (failure location indicated by X)

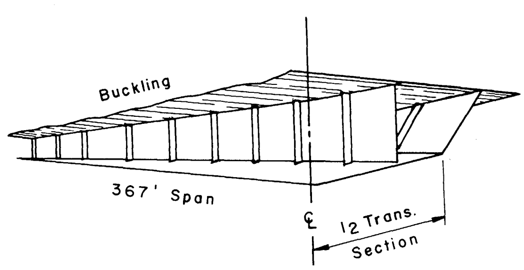


Fig. 25. Buckling of flange plate, West Gate Bridge

(b) The *Milford Haven Bridge* in Wales on June 2, 1970 (Fig. 23). Failure was attributed to the inadequate design of a bearing diaphragm.

The Milford Haven Bridge with design revision is in the stage of reviewing erection schemes. Recently it was decided to erect by a jacking system with lateral restraint against wind oscillations.

(c) The *West Gate Bridge* in Melbourne, Australia on October 15, 1970 (Fig. 24). Failure was attributed to a combination of erection errors and perhaps some design inadequacies. The 367 ft-6 in. cable stayed flanking approach spans were being erected by jacking at the piers of full span lengths of the boxes, one half of the cross section wide. This left about 10 ft of flange plate projecting toward the center line. This plate buckled during jacking, because of inadequate bracing (Fig. 25). Attempts were made to align the buckled plate for the longitudinal splice by loading the boxes with concrete blocks. Finally it was decided to allow more flexibility in the plate by removing bolts from a transverse splice in the top flange. This allowed the buckle to spread and precipitated collapse.

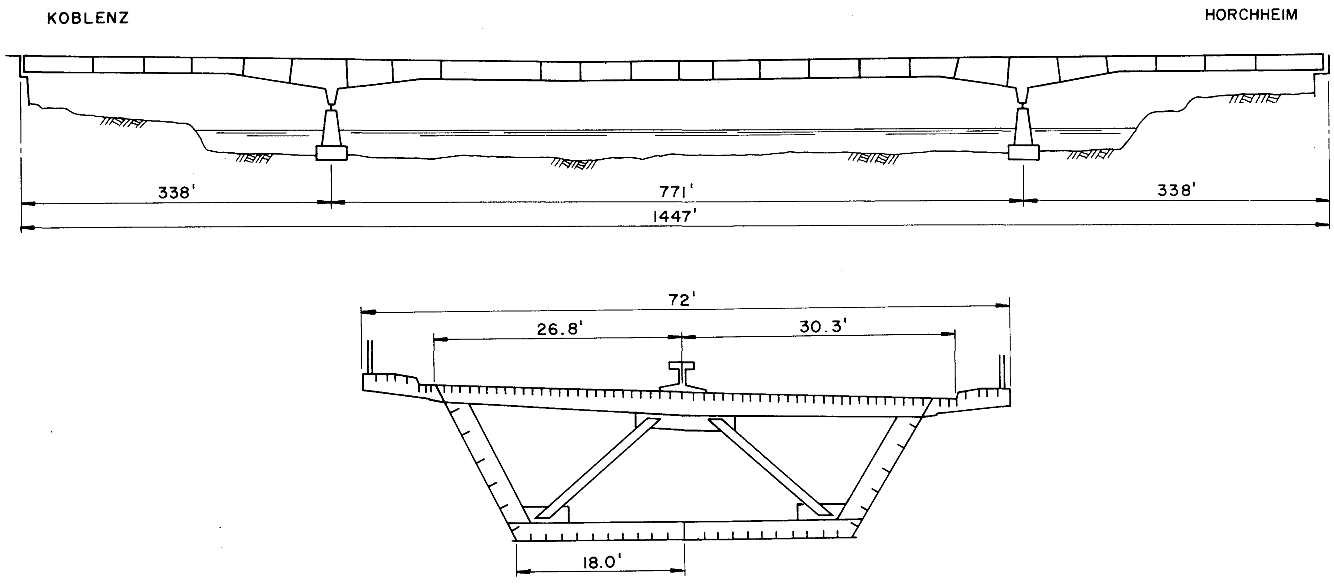


Fig. 26. Koblenz Bridge (failure location indicated by X)

The West Gate Bridge construction, with changes from a steel deck plate composite with concrete to an orthotropic steel plate deck system, recommenced early in 1972. The eastern 367 ft span which had been erected prior to the accident on the western side is being shored up while the changes are being made in the deck plate, converting it to an orthotropic steel deck plate. The remainder of the bridge will be erected by the cantilever method.

(d) The *Koblenz Bridge* over the Rhine River in Germany on November 10, 1971 (Fig. 26). Failure seems to have been caused by an inadequate stiffener detail which was separated from connection to the compressed flange for about 18 in. to accommodate a transverse butt splice (Fig. 27). However, *Engineering News-Record* (11/23/72) reports that investigators now attribute the failure to the in adequacy of the linear plate buckling theory where the plate is extremely wide in comparison to its longitudinal dimensions and is stressed across its width.

Until this failure occurred, German designs of box girder bridges had seemed to be immune to the problems experienced elsewhere, and in other countries consideration was given to the special methods and design requirements developed by German engineers. Their preference for erecting individual box section elements instead of complete sections seemed more susceptible to erection problems, but they did not have erection failures until the Koblenz Bridge.

Design and construction have been reviewed and revisions are under consideration.

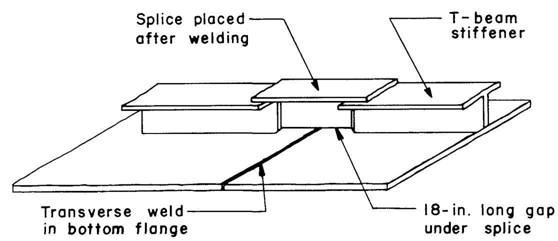


Fig. 27. Stiffener splice, Koblenz Bridge

THE MERRISON REPORT

As a result of the two failures of steel box girder bridges in the United Kingdom and Australia, a committee was established to inquire into the basis of design and method of erection of steel box girder bridges by the United Kingdom Department of the Environment. The chairman of this committee was Dr. A. W. Merrison, Vice Chancellor, University of Bristol. The committee and report have been referred to as the Merrison Committee and the Merrison Report. Dr. Merrison is not an engineer, but a physicist. The other committee men are engineering educators or practicing engineers. This committee issued an interim report which contained two appendices, A and B. Appendix B is addended to the interim report pamphlet and is concerned with contractual procedures.

Appendix A is a large document with subsequent amendments to bring it up to date. It is dated September 1971 and is titled "Interim Design Appraisal Rules." Although these criteria were developed, within a short period of time, for the assessment of the Milford Haven

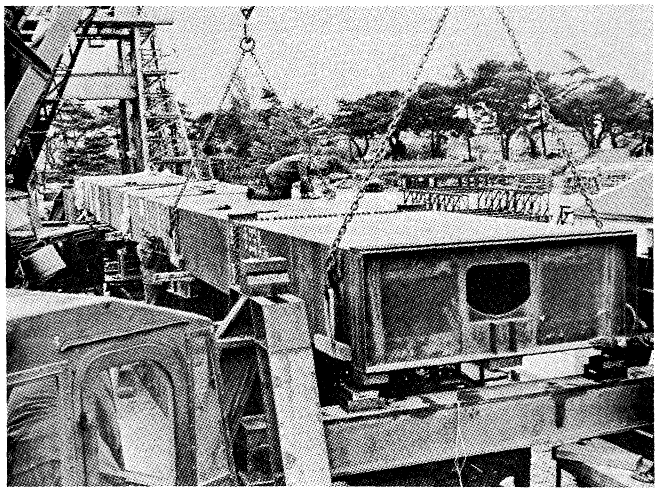


Fig. 28. Box girder test

Bridge, they were applied, with revisions, to all box girders under construction as Interim Design Rules. These rules are being condensed and summarized into general rules for the design of box girder bridges, and it is estimated that they will be available in the Fall of 1973.

The Merrison Appraisal Rules were modified in the drafting of the Interim Design Rules. The Interim Design Rules use nominal imperfections but measured residual stresses for the calculation of flange stress. These rules do not allow for membrane stresses in stiffened panels.

As part of the testing recommended by the Merrison Committee, diaphragms were tested to failure at university laboratories. Full box girders were also tested to collapse. A 140-ft girder, 8 ft wide x 3 ft deep was supported in a test rig to act as a two span continuous structure. It was designed within the interim rules of the Merrison Committee and loaded to study different failure modes of the box (Fig. 28).

The Merrison Committee was concerned with two questions. The first was: Has the fundamental knowledge of structural mechanics available to the engineer reached a stage at which it is reasonable to undertake with confidence the design of large thin-plate box girders? The second was: Has guidance in the use and detailed application of this fundamental knowledge been accessible to the engineer?

The answer to the first question was that "current knowledge of structural mechanics justifies engineers proceeding to the design of large box girder bridges, including the use of designs in which the effects of possible buckling are taken into account."

In answer to the second question, the Merrison Committee continued, "but the simple fact that a structure incapable of sustaining specified loads was not exposed by presently accepted checking procedures leads us to the conclusion that the guidance in the use and application of the available knowledge was and is inadequate."

The Merrison Committee *Interim Rules for the Assessment of Steel Box Girder Bridges* are conservative. However, this committee did an excellent job of assembling this information in a limited time and are to be congratulated for their work. The results of this work in its final condensation as design rules, which should be distributed in the fall of 1973, will probably be the best reference available on this subject. It has undergone extensive criticism by the design profession, researchers, fabricators, and contractors, with an attempt to correct excessively conservative design and fabrication provisions and to simplify the design application.

Concurrent with this work, the Highway Research Board Steel Superstructures Committee has assigned a task group to compare the Merrison provisions with domestic practices.

BRIDGE REVIEW BY MERRISON RULES (ERSKINE BRIDGE)

At the time the Merrison Report was issued in 1971 there were 49 steel box girder bridges in various stages of construction in the U. K. and 30 more in the design stages. The revised provisions of Appendix A were applied to these structures, some of which were stiffened during construction. The Erskine Bridge over the Clyde Estuary in Scotland is a continuous high strength steel box girder cable stayed bridge which was reviewed under the Merrison provisions and, as a result, critical compression elements were stiffened (Fig. 29).

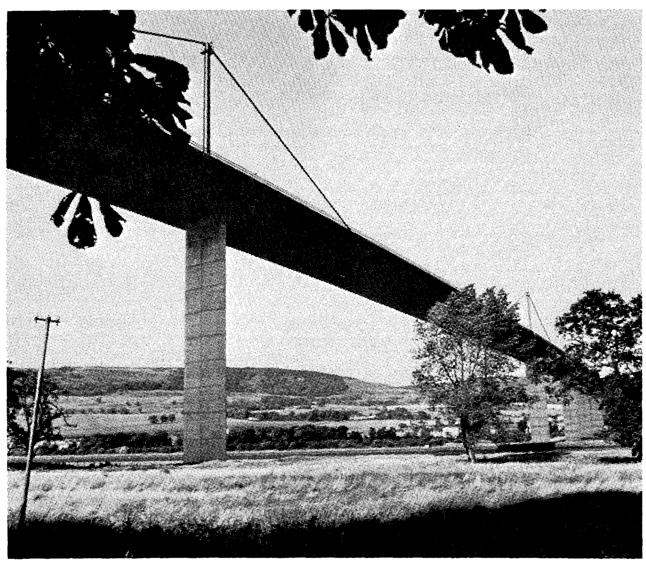


Fig. 29. Erskine Bridge

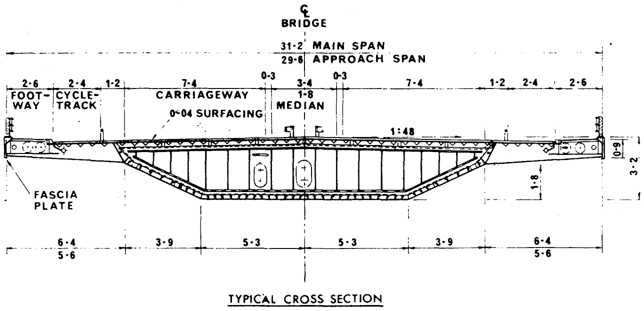


Fig. 30. Erskine Bridge—cross section

The trapezoidal shallow depth cross section was designed for a wind velocity of 130 mph (Fig. 30).

A unique method of cantilever erection was used for the erection of the box sections (Fig. 31). Two launching girders 84 ft long weighing 30 tons each were supported at each side of the erected end spans of the box. The arms of these girders extended beyond the end of the erected box providing a platform for the special launching bogie carrying the next box section to be spliced. The launching girders lowered the section into position by rotating down through the required angle. The carrying bogie adjusted to maintain level support for the new box section by means of hydraulic jacks. A C-shaped truss frame mounted on the outside of each launching girder is used to move it forward. This frame travels on bogies upon the deck.

The steel erection of the Erskine Bridge was completed in April, 1971.

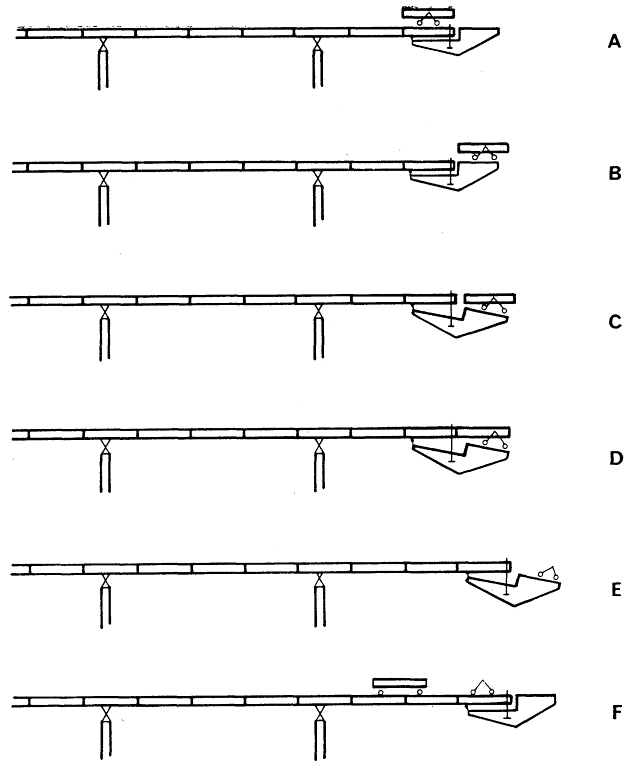


Fig. 31. Erection scheme, Erskine Bridge

RECENT CONFERENCES

An International Association of Bridge and Structural Engineers colloquium on the design of plate and box girders was held in London on March 25 and 26, 1971. At this conference Professor Massonnet, in a discussion of the report by Professor Dubas, concluded that for stiffened compression flanges of box girders, the required longitudinal stiffness based on the linear theory of plate buckling is to a significant degree insufficient in the post-critical range. Under the conditions of the performed test, the relative stiffness of the longitudinal stiffeners should be enlarged by a factor of 4 to 5; otherwise the security against collapse would be dangerously overstated.

Concurrent with the London conference, the box girder subcommittee of the ASCE-AASHO Task Committee on flexural members, under the chairmanship of Professor T. V. Galambos, published a second progress report on steel box girder bridges in the April 1971 Journal of the Structural Division.

From March 29 to April 2, 1971, the International Conference on Developments in Bridge Design and Construction was held at the University College, Cardiff in Wales. This conference was concerned with box girder bridges and delegates interested in this subject represented many countries. The proceedings of this conference were published in a hard covered book which was edited by K. C. Rockey, J. L. Bannister, and H. R. Evans, all of the Department of Civil and Structural Engineering, University College, Cardiff, Wales. The publisher is Crosby Lockwood & Son, Ltd., 26 Old Brompton Road, London SW 7, U. K. There are many valuable papers in these proceedings and it would be a useful reference for anyone studying the subject of box girder design.

LATEST INTERNATIONAL CONFERENCE

The Institution of Civil Engineers, Institution of Structural Engineers, and Institution of Highway Engineers jointly sponsored an international conference on steel box girder bridges which was held in London on February 13 and 14, 1973.

There were 19 papers presented at this conference. These papers covered structural concept, fabrication, research, analysis, and erection. Papers presented at a conference in the United Kingdom are subjected to a very lively discussion. The conference time is not consumed by the laborious reading of the papers by their authors. Instead a Reporter-General, who has studied the papers for his session in detail, summarizes the important parts of the papers. All of the conference participants are mailed copies of the papers prior to the conference and are familiar with the contents of the papers by the time of the conference. The conference time is utilized for an open discussion of the papers after they are briefly introduced. This provides for a most interesting and informative interchange. The conference proceedings are issued after the conference and contain both the papers and the discussions.

Dr. O. A. Kerensky stated in his steel box girder bridge paper entitled "Conception": "In conclusion, may I reiterate my faith in the future of box girder bridges. Properly conceived, carefully designed and fabricated, and erected to normal standards of good workmanship, they offer a safe, economic and elegant solution in many situations—often second to none!"

Dr. F. Leonhardt and D. Hommel, in their papers "The Necessity of Quantifying Imperfections of All Structural Members for Stability of Box Girders," concluded: "In several codes, for slender columns under compression, imperfections are considered by assuming unavoidable initial eccentricities quantified by a certain fraction of the buckling length ranging between 1/1000 and 1/500. This eccentricity will cover not only the geometrical imperfection, but also an unavoidable eccentricity of the load incidence at the end of the columns. For safety against buckling of orthotropic plates similar values must be assumed. However, it seems reasonable that the factors should not be constant, but decrease with the actual size of the panel or of the span of the cross-girder and depend also on the type of stiffener, especially on its sensitivity against torsional buckling. The thickness of the plate does not seem to be relevant. For I-shaped stiffeners, the initial displacements f_x or f_y might be between $l/300$ for a panel length of 2 m and $l/600$ for panel or girder lengths of 10 m or more. For triangular or trapezoidal box stiffeners these values could probably be 30% smaller. The initial distortion of steel plates should be limited to about 1/300 of the stiffeners spacing but locally not exceeding 4 mm over 3 m length. However, such quantities should not be fixed and specified before a much larger number of measurements of actual imperfections in bridges can be evaluated and before more test results of the influence of such imperfections on the real strength of stiffened plates can be obtained . . .."

Professor C. Massonnet and R. Maquoi presented "New Theory and Tests on the Ultimate Strength of Stiffened Box Girders" and concluded that the linear buckling theory is completely inadequate for the design of compressed stiffened plates. They also pointed out that the linear theory makes the designer believe that the stiffeners whose rigidity γ is greater than γ^* (the relative rigidity based on linear buckling theory) remain rigid up to collapse. They have developed a non-linear theory of the post buckling resistance of large stiffened box girders. The non-linearity that they are concerned with is the geometrical non-linearity (change in geometry) and the material non-linearity (yielding of some parts of the plate).

This is just a sampling of the papers, of course, and there was much discussion related to all the papers, especially regarding "Conclusions of Research Programme and Summary of Parametric Studies" by A. R. Flint and M. R. Horne.

The consensus of this conference was resounding confirmation of the steel box girder bridge concept and a unanimous expression of confidence in the future of the steel box girder bridge. The conference proceedings will be issued in the near future and will provide valuable information to the designer of steel box girder bridges.

The most important result of this conference was that the structural advantages of the steel box girder and the soundness of this bridge construction concept were definitely confirmed. It is also important to note that the conference conclusions were consonant with the initial conclusions of the Merrison Committee.

FUTURE OF THE STEEL BOX GIRDER BRIDGE

Not only has confidence in the steel box girder bridge been confirmed, but perhaps no other bridge type has been studied more extensively and a better understanding of its behavior known to the design profession. We are certain to see many more of these aesthetic, economical bridges built in the United States, as well as in Canada, where the cable stayed box girder has the lead in North America in the number constructed to date.

The Eleventh Edition of the AASHO *Standard Specifications for Highway Bridges* (*1973*) contains provisions for orthotropic steel plate deck bridges. These specifications were prepared by the ASCE Task Committee on Orthotropic Plate Bridges under the chairmanship of C. G. Schilling. This committee also has prepared a commentary on these specifications which will be published in the January 1974 ASCE *Journal of the Structural Division*. Although these specifications are concerned with the bridge deck, they include provisions which pertain to steel box girders as well. These rules consider the most recent research and experience with steel box girder bridges, including some recommendations of the Merrison Committee. They will provide an excellent guide for the design of orthotropic plate decks for steel box girder bridges.

The renewed confidence in the economically aesthetic steel box girder bridge insures it to be the bridge type of the future. It will challenge the suspension bridge in its use with cable stays, and without cable assistance will challenge other conventional types for the moderate span highway and railway bridges.