USS

STEEL / CONCRETE
COMPOSITE

BOX-GIRDER BRIDGES

A Construction Manual

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A CONSTRUCTION MANUAL

United States Steel wishes to express its thanks to the Mississippi Valley Structural Steel Co. (St. Louis, Mo.) and The Midwest Steel & Iron Works Co. (Denver, Colo.) for granting permission to photograph their respective facilities and operations.

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CONTENTS

FOREWORD

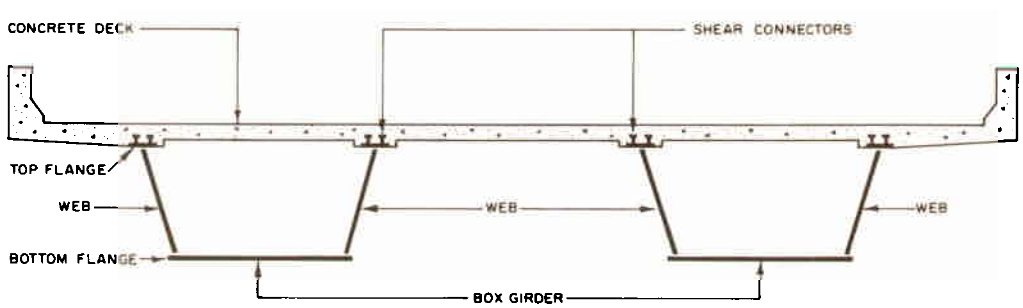
Steel/concrete composite box-girder bridges are no longer rarities; the number, successfully designed and built, has grown significantly. At this time, many more are being designed and will, quite soon, be bid for construction. In the United States, this increasing popularity is due, in large measure, to a growing emphasis on aesthetics, as well as to the box girder's potential for structural and economic advantages.

The torsional stiffness of box girders is many times greater than that of I-section beams and girders. Consequently, box girders have superior transverse load distribution characteristics, and this in turn leads to more efficient designs. It is well known that torsion has particular significance in bridges that are curved in plan; hence, box girders have a special advantage for curved bridges because of their ability to resist torsion without extensive use of diaphragms between the girders. The result is an uncluttered, exposed surface that helps make box girders more corrosion resistant, and easier to maintain since there are fewer nooks and crannies in which debris and water can accumulate.

It is safe to assume that the next decade will see a substantial growth in the use of steel/concrete composite box girders for the superstructures of urban and rural highway structures.

United States Steel wishes to express its gratitude to the various fabricator/erector organizations who generously provided much of the material on which this book is based.

For their kindness in making available the extensive data needed for the three case histories, particular thanks are due: Mr. Thomas Alberdi, Jr., Deputy Design Engineer-Structures, Florida Department of Transportation; Mr. William A. Kline, Chief Bridge Engineer, Wisconsin Department of Transportation; and Mr. Carl E. Thunman, Jr., Engineer of Bridge and Traffic Structures, Illinois Department of Transportation.

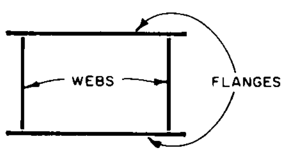


CROSS SECTION OF TYPICAL BOX-GIRDER BRIDGE

INTRODUCTION

With the emergence of the steel/concrete, composite box girder as a relatively new type of structural member for intermediate-span highway bridges, many designers and contractors have become aware of their lack of sufficient experience in this kind of construction. This shortcoming has, at times, led contractors to submit unit bid prices or construction costs higher than necessary. With this in mind, United States Steel offers this manual to provide engineers, fabricators and erectors with a fuller understanding of box girder construction, and to present guidelines that can assist them in the development of designs that will help realize the full economies of box girders.

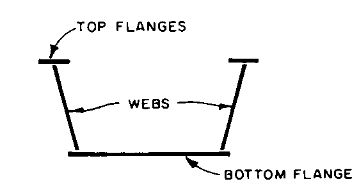
In simplest terms, the steel box girder may be defined as a longitudinal bending member with four steel plates (two webs and two flanges) arranged to form a closed box-like cross section (Fig. 1). Many early box girders were built exactly this way.



STEEL BOX GIRDER

Figure 1

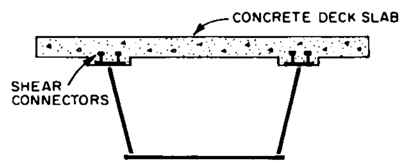
However, in modern highway structures, a more common arrangement is the "trough"- or "tub"-type girder (Fig. 2). In this case, two steel webs with narrow top flanges similar to I-girder flanges are joined at their bottoms by a full-width bottom flange. At fabrication



TROUGH-TYPE STEEL BOX GIRDER

Figure 2

and erection stages, the section may be completely open at the top or it may be "quasi-closed" by a top lateral bracing system. To close the top opening and complete the box, a reinforced concrete deck slab is added which acts compositely with the steel section by means of shear connectors attached to the top flanges. In its final form, this girder is referred to as a steel/concrete, composite box girder (Fig. 3).



STEEL/CONCRETE COMPOSITE BOX GIRDER

Figure 3

This manual deals primarily with steel/concrete composite box girders of moderate length (up to 350'-0") whose design is governed by the Standard Specifications for Highway Bridges[1] of the American Association of State Highway and Transportation Officials, hereinafter referred to as the Specifications. For general information, however, the list of references given on page 27 includes a wider range of box girder types.

The main body of the text consists of five parts or sections.

Part I pertains to fabrication and erection, and explains why the preparation of shop detail drawings for curved box girders may be beyond the scope of normal drafting room practice and thus may require special procedures. Part I also discusses the importance of shop assembly, shop handling, and shipping practices. Consideration is given to the lifting of large box girder sections, taking into account over-all stability of horizontally curved pieces and local stability of the thin, wide plate elements that make up girder sections. Typical construction loads are discussed, with emphasis on how these loads may affect the profile geometry of the steelwork. The concrete placing sequence for casting the deck slab is also discussed.

Part II shows how elementary principles of solid mechanics apply to the design and construction of steel box girders. Cross-sectional, torsional and flexural stiffnesses are considered, as well as local and lateral torsional stability.

Part III compares temporary with permanent bracing material taking into account factors such as cost, aesthetics and structural behavior.

Part IV examines the responsibilities of the various parties involved in the construction of a steel box girder bridge. American and foreign practices are outlined. Also considered are the Specifications and the way they have influenced American practice.

Part V presents guidelines for good construction practices for steel box-girder bridges.

The balance of the text includes a reference listing, and an appendix containing 1) several analytical examples, and 2) case histories of box-girder bridges where certain difficulties were encountered and overcome. Study of these experiences should help the reader avoid similar situations.

As was noted earlier, USS presents this publication with the hope that it will not only lead to increased understanding of box girder performance, but that it will also help impart a higher degree of confidence to engineers and contractors involved in this type of construction.

It should, however, be noted that this manual is not intended to serve as a handbook, in the sense of offering hard and fast solutions, nor is it an attempt to give fabricators and erectors explicit directions as to equipment and technique. The intent is, rather, to give bridge building professionals—contractors, fabricators, and engineers—generalized concepts and a broader understanding of box girder construction. It might also be added that only by reading the text completely, can maximum benefits be derived; extracting information in piecemeal fashion may lead to erroneous conclusions.

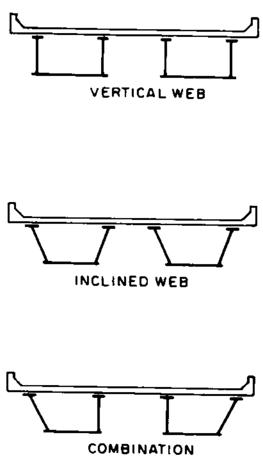
PART I

CONSTRUCTION CONSIDERATIONS

Fabrication

Detailing: Developing Pattern for Cutting Web Plates

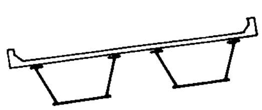
Composite box girders are built with two webs both of which are either vertical or inclined or sometimes in a combination of vertical and inclined (Fig. 4). Inclined webs not only offer pleasing aesthetics, but also allow the bottom flange plate to be narrower and thicker, thereby creating more efficient bottom flanges.



CROSS-SECTIONAL GEOMETRICS

Figure 4

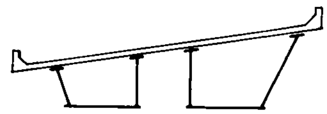
When the bridge cross section is superelevated, customary geometry for the steelwork is obtained by rotating the entire flat cross section into its superelevated position (Fig. 5).



SUPERELEVATED CROSS SECTION

Figure 5

This procedure simplifies design and fabrication by maintaining the symmetry of the girder sections and keeping the webs at a constant depth. Occasionally this practice is violated in order to achieve certain architectural effects. For example, the soffit of the boxes shown in Figure 6, has been kept level

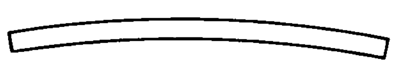


SUPERELEVATED CROSS SECTION, LEVEL SOFFIT

Figure 6

in the superelevated cross section, resulting in a different depth for each of the four webs. Furthermore, if this arrangement is used in areas of superelevation transition, the depth of each web will vary continuously. Such geometric complication obviously adds to the effort expended for design and fabrication.

If a girder were on tangent alignment both horizontally and vertically, the detailing and cutting of an inclined, constant-depth web plate would pose no special fabrication problem. The plate pattern would be rectangular and its dimensions easily computed. But this case is unrealistic since gravity is ignored. In actual construction, all girders will have some vertical deflection due to dead load which, in effect, introduces a vertical curve. When a vertical curve must be built-in because of camber or because of roadway profile and camber, it becomes more difficult to establish the developed shape of the plate, that is, the shape of the flat plate pattern from which the web will be cut. (Figure 7 illustrates a plate pattern with a vertical curve.)



CURVED PLATE PATTERN

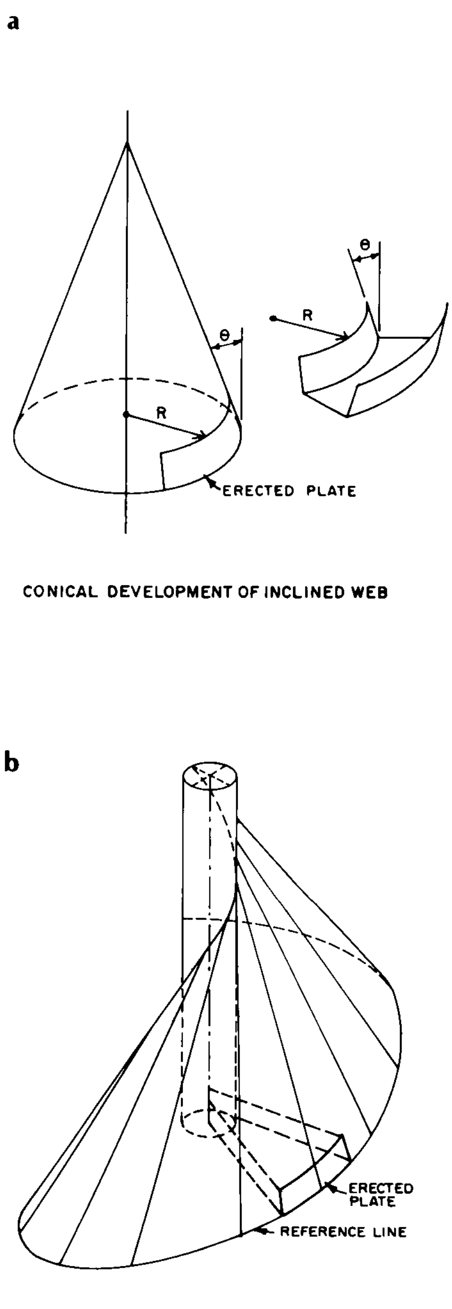
Figure 7

Another, apparently simple case is that of an inclined web on level grade, but on curved horizontal alignment. In this case, the web is a sector of a right circular cone with constant inclination, but the radius of curvature varies over the height of the web (Fig. 8). A cone is an easily calculated developable surface and the web plate pattern for this case again would seem to present no particular detailing or fabrication problem. But, here again, the concept of the example is false since dead load deflections would require that bridge girders be cambered to a non-level profile even if the final grade of the bridge were level.

This leads to the prevailing general case of an inclined web plate that is curved both horizontally and vertically. Mathematically, such a web is a warped or non-developable surface which, theoretically, cannot be built by conventional bridge fabricating procedures. It is possible, however, to model the surface approximately as a large number of small developable surfaces (Fig. 8a). The degree of accuracy of the approximation depends on the number of increments or small developable surfaces into which the plate is broken. With a reasonable number of increments this approach gives results that are acceptably accurate and within normal fabrication tolerances. Since the required computations are beyond the scope of ordinary bridge detailing practice, they are usually performed with a digital computer.

Some agencies, design consultants, and fabricators have developed their own computer programs for calculating or checking this complicated web geometry. A sample printout from such a program appears in Appendix A. The program that produced this printout may be obtained by fabricators from USS Engineers and Consultants, Inc., a subsidiary of United States Steel Corporation. The use of this or similar programs can help simplify the complex problem of providing a developed plate pattern for cutting the webs.

In all previous reference to horizontal curvature, circular curvature was implied; solutions are unobtainable for webs on spiral alignment. Bridges on spirals can be accommodated, however, by using a series of compound curves for the *structural* geometry to approximate the spiral geometry of the *roadway*. Any small discrepancies that may occur between the roadway alignment and the structure can be taken up in the cantilever overhang in the deck slab.



CONICAL DEVELOPMENT OF INCLINED WEB

CONVOLUTE DEVELOPMENT OF INCLINED WEB

Figure 8

Shopwork: Fit-up, Work Sequence, Shop Assembly, Handling

In many respects, actual fabrication of steel box girders will involve the same operations that are required for I-girders. Cutting and welding of main elements are performed principally by automatic processes. Some shops have installed numerically-controlled cutting machines capable of cutting flange and web plates to shapes defined by coordinates.

Since good fabrication procedure demands careful layout, fitting-up and welding, a number of shops have, for reasons of efficiency, constructed jigs (Fig. 9). Such jigs can be built with adjustments to accommodate changes in depth, width, web plate slope, and radius of curvature as they not only vary on a particular project but also from project to project.

Proper welding sequence helps minimize welding distortion. Many box girders are left with some residual twist or other geometric discrepancy because of a lack of symmetry or compensation in placing welds. With high residual stresses, handling of girders in the shop and field can cause additional distortions. These effects will be of more concern in some box girders than in others.

With an I-girder or an open-section box girder, the torsional stiffness* of the section is so low that the erected girder will easily accommodate itself to the geometry of the bearings and diaphragm connections in the field. A box girder that is even "quasi-closed" with lateral bracing at the top is high in torsional stiffness. Such a girder may not accommodate itself to the bearings and connections easily. For this reason, it may be advisable to shop assemble a box girder structure, reaming the connection holes at this stage and determining the bearing fit. In shop assembly, the girder is blocked up in the exact, or relative, position it will occupy in the field. Long, continuous girders are often shop assembled in three contiguous shipping pieces. Although shop assembly does add to fabrication cost, in the long run, over-all costs may be reduced significantly.

* Torsional stiffness is covered in detail in Part II "BOX GIRDER BEHAVIOR CHARACTERISTICS AND INFLUENCE OF BRACING."

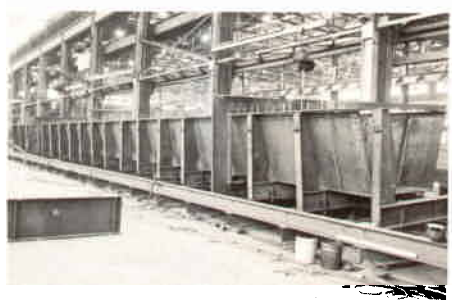


Figure 9

Figure 10 shows a typical box girder shop assembly.

For downhand welding positions and general ease of fabrication, shop handling becomes an important factor in the efficient production of box girders. Special handling jigs have proven worthwhile in many shops. Figure 11 shows a curved box girder section in a rotated position for a particular shop operation. The collar supporting the girder can be rotated about the pin located midway between the two vertical posts, allowing the girder to be shifted off the vertical in either direction. The collar surrounding the girder also serves as a lifting harness (Figs. 12a and 12b). Set screws clamp the girder into the harness which is adjustable in the width direction of the girder and has lifting holes at all four corners, permitting lifting in any direction. Spreader beams are used for lifting with a single crane. Figure 13 illustrates this type of lift and also shows that slings, with cushioning material at the flange edges, may be used in lieu of more elaborate collars or harnesses. Light sections have even been handled with slings without flange edge cushioning (Fig. 14). (Additional information regarding the lifting of box girder sections will be found under the heading "STEEL ERECTION AND DECK PLACEMENT.")

Figure 10

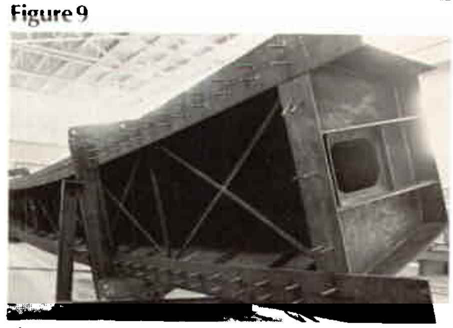


Figure 11

Figure 12a

Figure 12 b

Figure 13

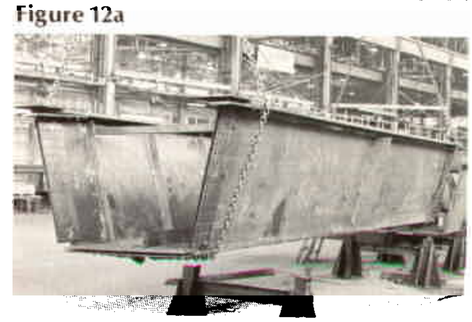


Figure 14

Shipping: By Barge, Rail, Truck

A good design will also take into account shipping feasibility. At the design stage, consideration should be given to length and weight of shipping pieces, location of field splices, and over-all dimensions of the cross section. A length of approximately 120′-0″ and a weight of approximately 90 tons are two maximums for pieces that can usually be handled efficiently either in the shop or field. Lower limits may be imposed by other circumstances.

Box girders may be shipped by barge, rail or truck, or in any combination of these means. Obviously, barge transit is very efficient if both fabricating plant and construction site are located on the same waterway. Under such circumstances, there is virtually no limit either to the size or weight of shipping pieces that can be handled on a water route.

In rail transit, the standard railroad flat car is 53′-6″ long, 10′-8″ wide and 3′-6″ above rail, with an average capacity of about 140,000 lb. The maximum permissible height of load above rail for unrestricted movement varies with the width. While allowable heights must be checked for each project, following is an approximate guideline for allowable heights at different widths:

Width	Height
7′-0″	15′-6″
10′-0″	14′-8″
10′-8″	14′-2″

Narrower loads may be shipped in standard gondola cars which are 65′-0″ long and 8′-6″ to 9′-0″ wide.

For restricted rail movements, widths of up to approximately 13 ft can be handled, depending on the route, the configuration of the load, and the mid-ordinate and end-of-car overhang on curved alignment. Long pieces may be shipped supported on bolsters on two flat cars at opposite ends of the load, connected by idler cars. Such bolsters run as much as 1′-6″ in height above the car floor, reducing the net height available for the load by that amount. In recent years, truck-train "piggyback" cars have been used for shipping long loads. These cars are up to 85′-0″ long and can therefore handle loads over 100′-0″ long if overhang beyond the end of the car is accommodated by means of end idler cars. High blocking is required to provide clearance above the idler car, but the blocking need not accommodate movement as in the case of a bolster. Once again, it must be emphasized that each project's particular requirements for rail movement must be individually investigated.

The sequence in Figure 15 shows (a) a truck-train flat car preparatory to loading; (b) a shorter-than-car-length box section being lowered onto the car; (c) the box section at rest on the car; and (d) the details of the end brackets which restrain the load longitudinally.

Figure 16 shows a long box section that overhangs the end of the car, and which therefore requires high blocking for idler car clearance. This illustration also shows a typical hold-down device.

Figure 17 shows a girder loaded for shipment in the upside-down position.

Shipment by truck provides access to sites that may not be served by water or rail transportation. Few specific guidelines can be stated, however, due to the variety of regulations and conditions that exist in states and municipalities. An approximate measure of the width that can be shipped unrestrictedly is 8 ft, and loads up to about 12-ft wide can generally be hauled with permit and special escort. Truck economics will depend on topography, distance and route.

The photo sequence in Figure 18 shows (a) long-distance truck transit on an interstate highway; (b) a truck on a secondary highway; (c) a truck on a haul road; and (d) a truck at a jobsite.

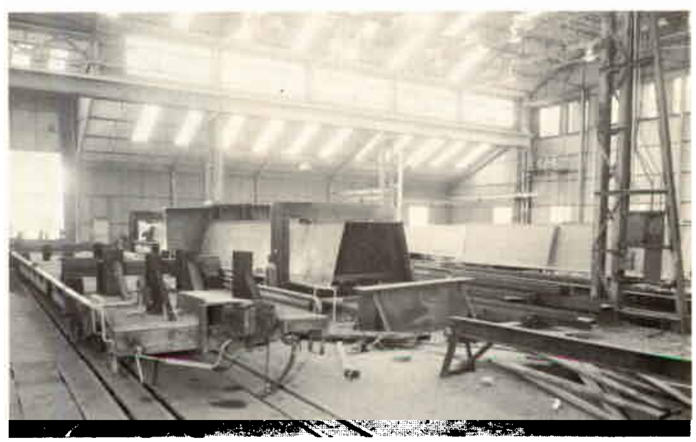


Figure 15a

Figure 15b

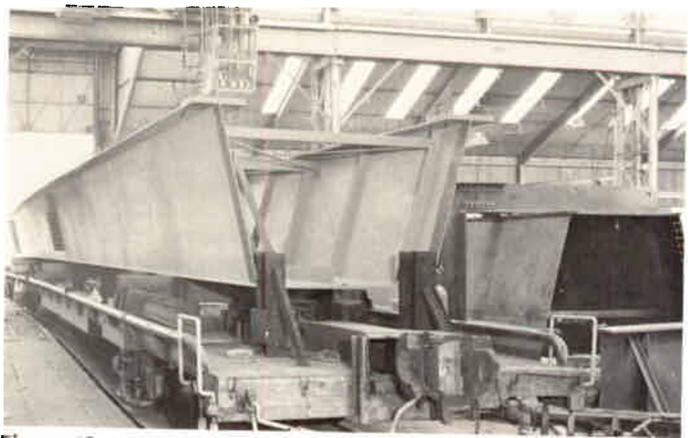


Figure 15c


Figure 18a

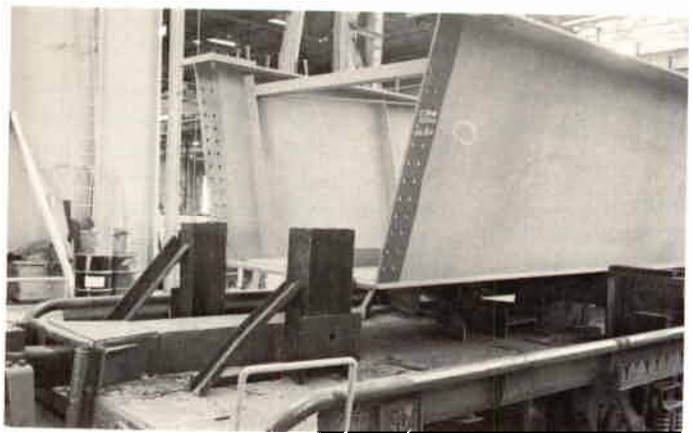


Figure 15d


Figure 18b


Figure 16

Figure 18c

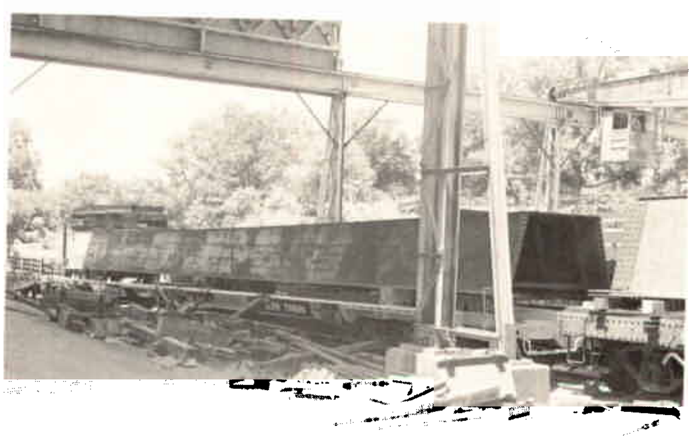


Figure 17


Figure 18d

Steel Erection and Deck Placement

Shortly after delivery to the site, the steel box girder shipping pieces must be lifted into position, spliced, and in some cases, braced or shored in order to insure stability and strength in the erected state—even before placement of the concrete deck completes the girder. Form work is then constructed, imposing additional, although temporary, loading on the partially completed girders. Further loads result from deck concreting equipment, and finally a large increment in loading is produced by the weight of the deck concrete itself. The ultimate success of the construction depends on the maintenance of adequate strength, stability and stiffness all through these stages.

Lifting: Local Plate Stability and Over-all Stability

Lifting, the first important step, has already been discussed in connection with shop operations in which common types of equipment and procedures were mentioned. Obviously, the relatively low lifting in the shop with overhead cranes is not as critical as field lifting with boom-type cranes, where conditions are much harder to control. In any case, local plate stability and over-all stability are important considerations in every lifting operation.

As used here, the term local plate stability means the ability of the plates—that make up the girder—to resist buckling as individual elements when subjected to compression stress. Over-all stability concerns equilibrium of entire girder sections during lifting, a particularly important consideration with sections that are horizontally curved.

Local plate stability will, most likely, be worth investigating for two box girder components—the wide, thin bottom flanges and the narrow top flanges.

Local Stability of Bottom Flanges

The stress conditions for which the bottom flanges are designed should first be examined for the finished structure. In its final condition, a simple span box girder's bottom flange will be in tension, with buckling, therefore, not a problem. Similarly, in continuous box girders, although the bottom flange in the negative bending regions over interior supports is designed for compression and will usually have been furnished with stiffeners to prevent buckling, the bottom flange in the positive bending regions is designed for tension just as in the case of the bottom flange of a simple span. Since stiffening is not required for tension flanges in the finished bridge, it is probable in both such cases that the original design will not call for stiffening in these regions. This makes such flanges—normally thinner than compression flanges—vulnerable to buckling if they are put into compression as a result of erection loads. For example, if a box section is lifted at the quarter points, the weight of the girder will cause the bottom flange to be in compression in the vicinity of the lifting locations. While this flange may easily sustain a stress of 20 ksi, or more, when working in tension under service loads, its critical buckling stress as a wide unstiffened compression plate may be much lower. Such a case is illustrated in Appendix B.

For this reason, designers will sometimes provide nominal stiffeners on tension bottom flanges for handling and erection, particularly in the case of large girders. When tension flange stiffening is not provided in the design, the contractor must be aware of the buckling potential and be prepared to exercise additional care when handling and lifting. Obviously, lifting girder sections near the ends will eliminate most of the compressive stress in bottom flanges. But this cannot be done with curved sections since these must be lifted along their longitudinal gravity axis or they may overturn. This is discussed at greater length on page 29.

Local Stability of Top Flanges

A different kind of instability condition exists with top flanges: lateral torsional buckling, i.e., the tendency of narrow compression flanges to twist and buckle sideways. This same condition is a well-known design consideration in the compression flanges of I-girders. For composite girders, this type of buckling is only a serious concern until the slab has been placed and hardened, after this the flange is braced by the slab.

Lateral torsional flange buckling is prevented by using crossframes or diaphragms to limit the unbraced length of the compression flange, and/or by limiting the magnitude of the compressive stress. For straight girders, the Specifications give allowable flange compressive stresses as a function of flange width (b) and unbraced length (L) as follows for steels having yield strengths of 36,000 and 50,000 psi:

F_y	F_b
36,000	$F_b = 20{,}000 - 7.5\left(\frac{L}{b}\right)^2$
50,000	$F_b = 27{,}000 - 14.4\left(\frac{L}{b}\right)^2$

If there are lateral forces producing lateral bending—in addition to the vertical loads—the total flange stress is obtained by adding the maximum compressive stresses that can occur simultaneously at a point. In other words, what is being dealt with is the flange tip stress. For lateral torsional buckling computations, it is conservative to require that this total stress should not exceed the values given by the formulas above. Note, also, that in composite girders, lateral bending stress is caused only by loads that exist prior to and during the wet concrete stage of construction.

If the girder flange is curved, lateral bending moment[2] will be produced which can be taken as

$$M_{Lat.\,Curv.} = \frac{Md^2}{20Rh}$$

where

M = Vertical bending moment in the girder (ft-lb)
d = diaphragm spacing along flange (ft)
R = radius of centerline of top flange (ft)
h = depth of girder, center to center of flanges (ft)
$M_{Lat.\,Curv.}$ = lateral bending moment (ft-lb)

The lateral bending stress for curved girders having vertical webs is calculated by applying this moment transversely to the girder flange and computing the stress by conventional methods.

For curved girders having inclined webs, additional lateral flange bending stresses exist due to the horizontal component of the web shear. This additional lateral bending moment is expressed as[3]

$$M_{Lat.\,Inclined\,Web} = \frac{Hd^2}{24DL}(V_{Left} - V_{Right})$$

where

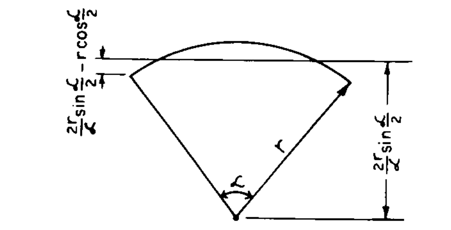
H = horizontal projection of web (ft)
d = diaphragm spacing along flange (ft)
D = vertical projection of web (ft)
L = span length, between vertical supports (ft)
V_{Left} = vertical web shear at left end of span (lb)
V_{Right} = vertical web shear at right end of span (lb)
$M_{Lat.\,Inclined\,Web}$ = additional lateral bending moment (ft-lb)

Again the lateral bending stress is calculated by applying this moment transversely to the girder flange.

I-girders are sometimes fitted with a temporary lateral truss at the top flange to prevent lateral torsional buckling prior to placement of the permanent bracing in the field. Similarly, box girders with internal bracing installed in the shop acquire the advantage of having the top flanges automatically braced when delivered.

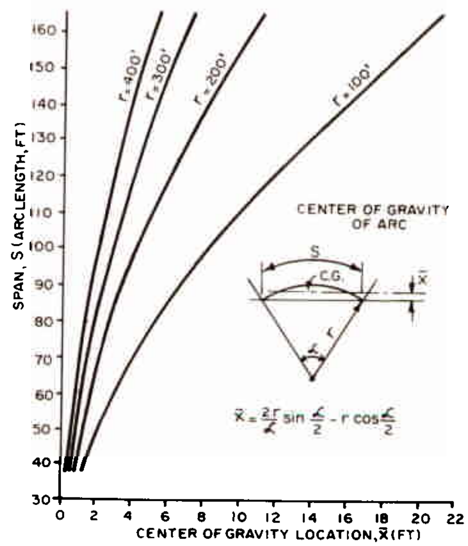
Over-all Stability of Shipping Pieces

As stated previously, over-all lifting stability is also important. For curved box girders, it is essential that lifting locations be determined for each curved section so that it will remain level and stable. This amounts to locating the longitudinal gravity axis of the girder section. The simplest approach is to consider the curved box girder section an arc. The formula for determining the offset distance between the center of gravity and a chord joining the ends of an arc is easily derived or can readily be found in handbooks (Fig. 19). For convenience, the formula may be put in graphical form, with the center of gravity location plotted against arc length for varying radii. Figure 20 is for radii from 100 to 400



CENTER OF GRAVITY OF CIRCULAR ARC

Figure 19



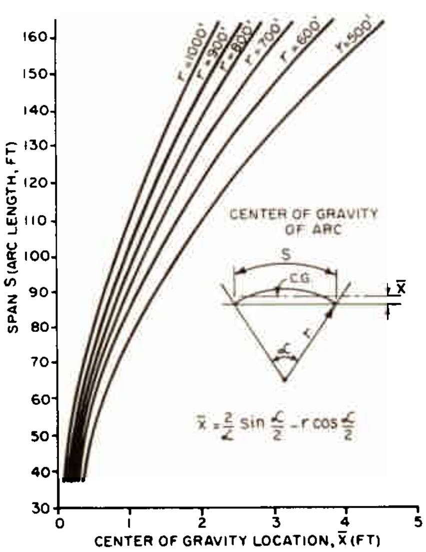
ARC CENTER OF GRAVITY, RADIUS 100' TO 400'

Figure 20

feet; Figure 21 for radii from 500 feet to 1,000 feet. A more accurate treatment of the problem is to consider the box girder section a sector of an annulus. The center of gravity for such a sector is given in Fig. 22. Again the relationship may be put in graphic form. Since the formula involves three independent variables (a, r and α), it is most convenient to construct the graph in dimensionless form, as in Figure 23, with the ratio a/r plotted against the ratio $\bar{x}$/r for varying values of α. Obtaining the value of $\bar{x}$/r from the graph, it is then a simple matter to multiply by r to get $\bar{x}$. It should be noted that when the value of "a" approaches zero, $\bar{x}$ converges to that for an arc.

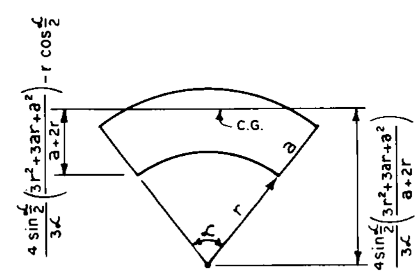
Applying the sector of annulus method to the girder section shown in Appendix B, a value of 9.11 ft is obtained as the distance from a line through the inside corners of the sector to the longitudinal gravity axis. The sector is taken as that area bounded by the outside edges of the girder top flanges. A more precise calculation, taking moments of the weights of all components of the girder section about the axis through the inside corners of the sector and dividing by the weight, yields a corresponding value of 9.30 ft.

A third calculation, considering this same girder section as simply an arc, produces a value of 9.11 ft, identical to that obtained by treating the girder section as a sector of an annulus. For girder sections that are fairly long and narrow, use of the arc analysis is probably of sufficient accuracy; for girder sections that have larger width-to-length ratios the sector analysis would produce results that are different from the arc analysis and more nearly correct.



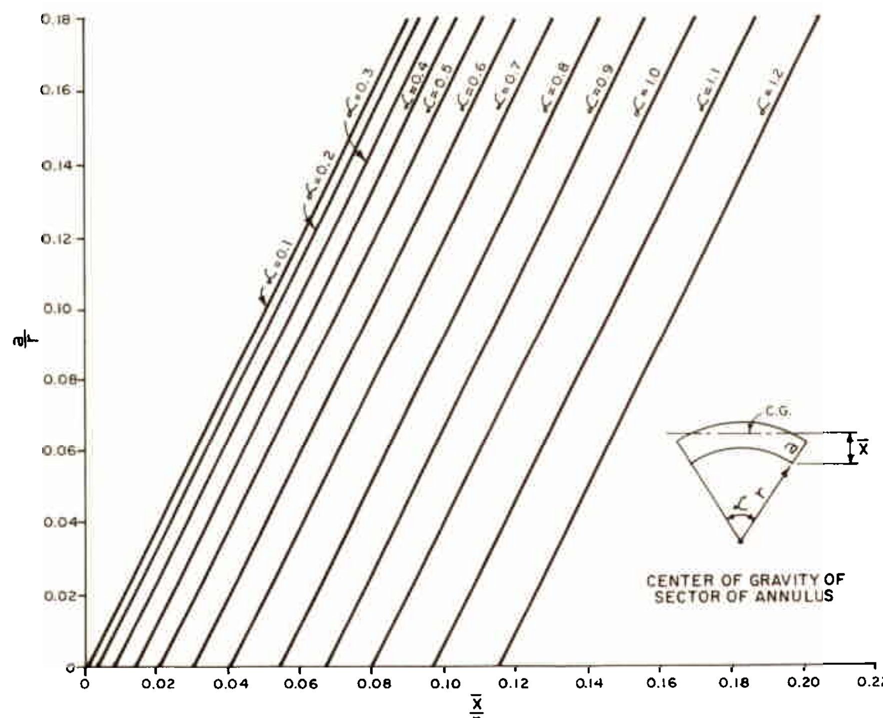
ARC CENTER OF GRAVITY, RADIUS 500' TO 1000'

Figure 21



CENTER OF GRAVITY OF SECTOR OF ANNULUS

Figure 22



SECTOR CENTER OF GRAVITY

Figure 23

Construction Loads

After steelwork erection, the next stage is that at which considerable additional load is applied to the steelwork in the form of construction loads and the weight of the deck itself.

Types of Loads

Construction loads[5] include the weight of form work and equipment, as well as wind effects. Of particular importance is the effect of eccentricities in construction loading, which produce torsion on the steelwork at a time when the steelwork may have very little torsional resistance. Common sources of such loads include 1) concrete finishing machines that are often mounted on rails supported outside of the steelwork, with resulting eccentricity, 2) the form work for the slab cantilever overhang is frequently eccentrically mounted, and 3) wind loads acting on any equipment of significant height above the structure will have eccentric effects. An unsymmetrical deck placement sequence may also produce torsional loading.

Trouble-free construction is, most often, the result of the contractor's awareness of the significance of construction loading, and his efforts to reduce or counter the effects of eccentricity.

Over-all Stability of Erected Steelwork on Substructure

At each increment of construction loading, the steel structure should be checked for uplift and over-all stability as it rests upon the substructure. A design with adequate strength and stability for the finished bridge will normally be framed and detailed in such a way that over-all stability will not be a problem during construction. It is simply a matter of the contractor recognizing a situation where uplift becomes a possibility and then taking whatever preventive measures may be necessary.

Meeting Profile Geometry Requirements

In box-girder bridge design, the most common difficulty arises from construction loads and their eccentricities; this results in a failure to meet geometric requirements. Earlier, it was stated that open-section girders offer little resistance to torsion at the construction stage. (A more detailed definition of torsional resistances is given in Part II "BOX GIRDER BEHAVIOR CHARACTERISTICS AND INFLUENCE OF BRACING.") When a steel structure is excessively flexible, it may deflect and twist under construction loads to such an extent that the required profile geometry of the bridge cannot be met. Since girders are cambered for vertical load but not normally for twist, the twisting deformation from eccentric loads is perhaps the most frequent cause of geometric discrepancies. Thus a box girder that lacks proper shoring or bracing may have its flanges at the wrong elevations, making it difficult or impossible to set forms and screed points correctly. Part V discusses these geometric problems and ways to avoid them.

Deck Placement Sequence

Common practice, in continuous I-girder construction, is to cast the deck slab in positive bending regions first, and then in negative bending regions to minimize cracks at the top of the slab (Fig. 24). The same rationale applies to box girders. A second consideration is to keep the concrete placement as symmetrical as possible both longitudinally and laterally, but particularly laterally. This minimizes unbalanced or eccentric loading, and avoids differential deflection of various parts of the structure. Furthermore, if there is any possibility of uplift at the end bearings, it may be an added advantage to cast continuous units first in the positive bending regions in the end spans.

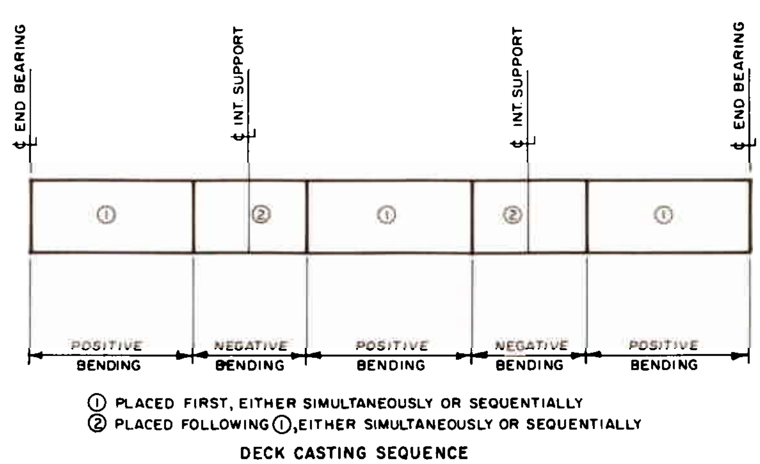


Figure 24

PART II

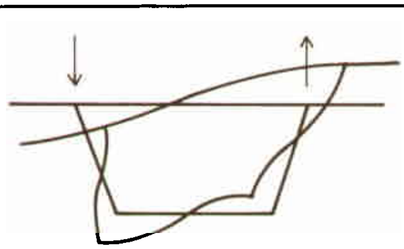
BOX GIRDER BEHAVIOR CHARACTERISTICS AND INFLUENCE OF BRACING

The structural mechanics of box girders is dealt with in this section. Understanding these characteristics should help make certain that appropriate design and construction measures are taken.

Box girders may undergo several different types of deformation or distortion during construction. One may think of the resistance of the girder to these deformations as forms of "stiffness."

Cross-Sectional Stiffness—Distortion of Box Section

One form of stiffness is cross-sectional stiffness or the ability of the girder to maintain its cross-sectional shape. A distorted cross section (Fig. 25) arises primarily from torsion



DISTORTED CROSS SECTION

produced by eccentric loading, equivalent to a couple acting on the girder about its longitudinal axis. The distortion is a local phenomenon most pronounced at the point of application of the eccentric load and diminishing with distance away from the load application point. Similar distortion may also be present in horizontally-curved girders as a result of the eccentric center of gravity of the girder.

Preventing Distortion with Internal Crossframes

During construction, cross-sectional distortion may cause discrepancies in the profile geometry of the girder flanges. Such distortion can be controlled by the use of internal diaphragms or crossframes (Fig. 26).

The Specifications require that all steel box girder designs must provide internal diaphragms, at least at the support locations. For ordinary straight box girder bridges designed in accordance with the Specifications, *intermediate* crossframes or diaphragms, that may be needed for construction, will not normally be required in the finished structure, and may be installed as temporary members, or if desired, internal crossframes may be left as permanent members with no adverse structural effect on the girders other than a slight weight addition. In any event, the details connecting the crossframes to the girder can have a significant effect on fatigue strength and should therefore be reviewed by the engineer.

In practical design, the location and spacing of crossframes for straight girders is a matter of judgment. In addition to the internal diaphragms over the supports, designs for straight bridges often call for crossframes at the middle of simple spans, or at the maximum positive moment sections and adjacent to field splices of continuous spans. Proportioning the bracing members is usually done on the basis of L/r or minimum material requirements. The maximum permissible L/r is 140 for secondary compression members and 240 for secondary tension members. If angles are used, the Specifications require a size of at least 3 inches x $2\frac{1}{2}$ inches, and the width of outstanding legs must not be more than 16 times the thickness of the angle. At least two fasteners or corresponding welds must be used for the end connections.

Figure 26

Curved bridges differ from straight bridges in that internal crossframes are required to provide necessary support for the curved top flanges. This is in addition to the cross frames functioning as distortion control. For this reason crossframes are generally spaced at regular intervals in curved bridges, and are retained as permanent members.

For curved girders, then, crossframe spacing may be influenced by the lateral support requirements of the curved top flange.[2] Similarly, the force delivered from the curved flange to the crossframe may also govern the crossframe member sizes. Otherwise these sizes may be dictated by the minimum L/r of 120 for main compression members and 200 for main tension members, and the limiting angle leg width of 12 times the thickness.

Several analytical methods[6,7,8] are available to calculate the out-of-plane stress due to cross-sectional distortion as a function of diaphragm spacing and diaphragm member size. These methods have been used in inverted form to calculate diaphragm spacing and member size necessary to hold transverse distortion stresses to a specified value. Recent research[26] has developed particularly simple formulas for crossframe spacing and required area of crossframe diagonal as follows:

$$\text{Crossframe spacing} = S \leq L\left(\frac{R}{200L - 7500}\right)^{1/2}$$

where S = required diaphragm spacing (ft)
L = span length between supports (ft)
R = center line girder radius (ft)

Area of crossframe diagonal =

$$A_b \geq \frac{0.02\ Lb\cos\theta}{d^2}$$

where A = area of crossframe diagonal (inch^2)
L = span length (inches)
d = depth of box (inches)
b = width of box bottom flange (inches)
θ = angle of diagonal with respect to horizontal (degrees)

Some typical crossframe arrangements are shown in Figure 27.

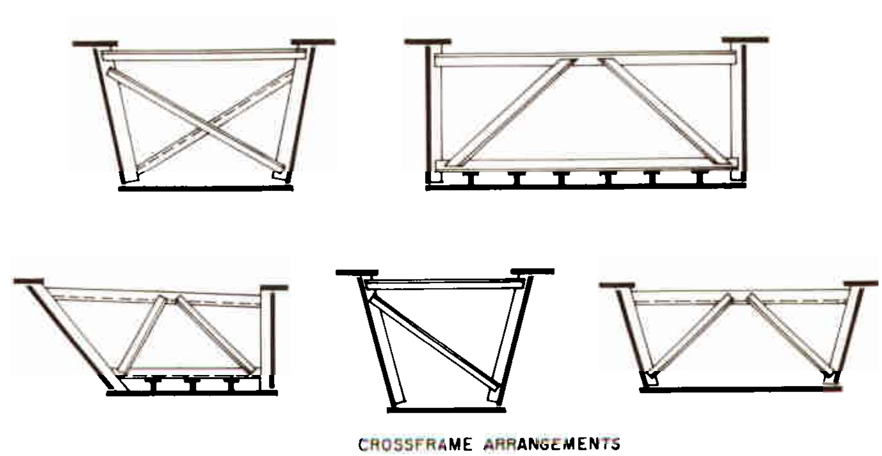


Figure 27

Torsional Stiffness— Rotation of Box Section

Another very important type of stiffness in box girder construction is torsional stiffness. To repeat what has been stated earlier: torsion may often be present in straight girders during construction because of eccentric loads, and is always present in horizontally curved girder spans. The effect of torsional load is twisting deformation (Fig. 28), i.e., angular movement about the longitudinal axis.

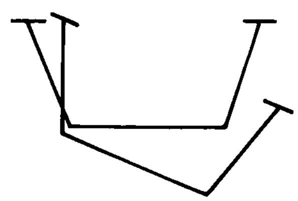


Figure 28

The magnitude of twist is directly proportional to the torsional load and inversely proportional to the torsional stiffness.

It is helpful when discussing torsional stiffness, to consider elementary (St. Venant) torsional theory with regard to open and closed sections. The approximate torsional stiffness of an open section made up of plate elements whose widths are great compared to their thickness is given by

$$K = \frac{1}{3}\sum bt^3$$

where b and t are the width and thickness, respectively, of the individual plate elements. The approximate torsional stiffness of a closed section made up of plate elements is given by

$$K = \frac{4A^2}{\Sigma b/t}$$

where A is the area enclosed within the section and b and t are again the width and thickness, respectively, of the individual plate elements.

As an example, these formulas are applied to the sections in Figure 29a and Figure 29b. The section in Figure 29a is open by virtue of a slot; Figure 29b is exactly the same as Figure 29a, except there is no slot so the section is closed.

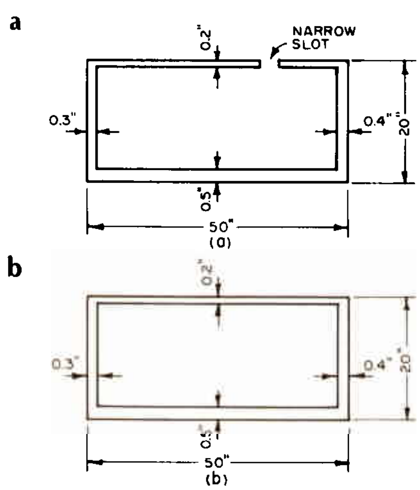


Figure 29

Open Section (a)

$$K = \frac{1}{3}\Sigma bt^3 = \frac{1}{3}[(20)(0.4)^3 + (50)(0.5)^3 + (20)(0.3)^3 + (50)(0.2)^3] = 2.82 \text{ in}^4$$

Closed Section (b)

$$K = \frac{4A^2}{\Sigma b/t} = \frac{4(20 \times 50)^2}{\frac{20}{0.4} + \frac{50}{0.5} + \frac{20}{0.3} + \frac{50}{0.2}} = 8571 \text{ in}^4$$

Ratio

$$\frac{\text{Closed Section Stiffness}}{\text{Open Section Stiffness}} = \frac{8571}{2.82} = 3039$$

Taking the ratio of the two St. Venant stiffnesses, we see that the closed section is over 3,000 times as torsionally stiff as the open section.

Preventing Rotation with Top Lateral Bracing

In its final condition, the cross section of a composite steel-concrete box girder is closed by the deck slab. But, prior to placement of the slab, the girder is an open box section and therefore very flexible torsionally. An effective way to increase torsional stiffness before the deck forms and concrete are placed is by means of a horizontal lateral bracing system, either full-length or partial-length, placed near the plane of the top flanges. As mentioned earlier, this type of section is sometimes referred to as a "quasi-closed" section. Tests have shown that a properly designed bracing system, although not as effective as a solid plate of the same order of thickness as the webs and bottom flange, provides a torsional stiffness exceptionally greater than that of the open section.

Torsional characteristics of girders with truss-like lateral bracing may be calculated by transforming the bracing into an equivalent plate. Formulas [9] are available for computing equivalent plate thicknesses for various sizes and types of lateral bracing. Having an equivalent plate thickness permits the evaluation of torsional stiffness by the St. Venant formula for closed sections as previously described. It also enables the determination of torsional shear flow across the top of the box at any section by the following formula:

$$s = \frac{T}{2A}$$

where T = applied girder torque (inch-lb)
A = enclosed area ($inch^2$)
s = shear flow (lb/inch)

Multiplying this shear flow by the width of the box, b in inches, gives a transverse shear force which may be used to evaluate the force in the bracing. Some designers take T as the maximum torque in any lateral bracing panel, and calculate the maximum transverse shear in the panel by

$$s = \frac{Tb}{2A}$$

The lateral bracing member is then proportioned to carry this shear much as a diagonal in a truss bridge is designed to carry the vertical shear, satisfying stress and L/r limitations. Other solutions[10] have been developed which compute a panel shear in the same manner as above, but treat the lateral bracing panel as a rigid frame made up of the lateral bracing diagonal, the girder flanges, and the transverse struts; the forces in this frame are calculated by any of the commonly used methods of frame analysis.

Recent research[26] suggests the following formula for the required cross-sectional area of a lateral bracing diagonal member:

$$A_b \geq 0.03b$$

where

A_b = required area ($inch^2$)
b = bottom flange width (inches)

Properly designed *curved* box girders should include the lateral bracing and internal diaphragms as part of the design. Experience has shown that, for construction, it is also advisable to use a full-length top lateral bracing system, either temporary or permanent, in *straight* box girders having a span length greater than 150 ft. As previously discussed, closing the box at the top with a horizontal truss is an effective means of attaining a torsionally stiff section prior to placing the slab. An alternate concept, utilizing corrugated stay-in-place forms to function as lateral bracing, is currently being studied by researchers. The principle is the same, and more is said about it on pages 20 and 22.

Truss-type lateral bracing is most often connected to the webs of the box girder at a small distance below the top flanges to avoid interference with deck forming. The connection often consists of gusset plates welded to the web on each side of the transverse web stiffener (Fig. 30a).

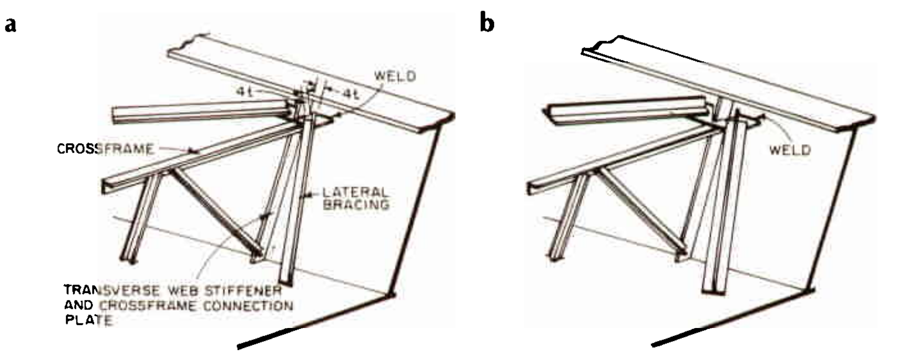


Figure 30

For larger lateral bracing members a single gusset plate is sometimes used, slotted to accommodate the transverse web stiffener (Fig. 30b).

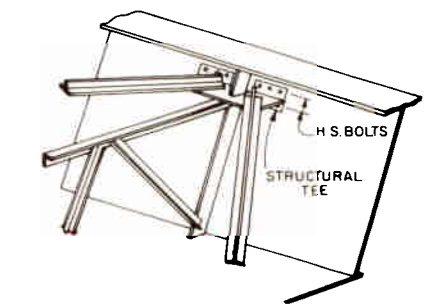
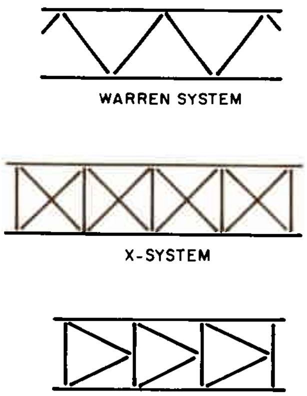


Figure 31

For regions where stresses in the box girder will vary in magnitude or even reverse, fatigue becomes an important consideration. The connections shown in Figure 30 qualify as AASHTO fatigue "Category E" details. This fatigue category is subject to a very restrictive stress-range limitation and such connections should only be used where the stress-range limitation will not affect the design of the main elements of the girder.

A more favorable bolted connection, shown in Figure 31, is often used in lieu of the welded connection, especially in regions where the top flange is in tension. This bolted connection would qualify as an AASHTO fatigue "Category B" detail.

The lateral system itself may be a tension-compression Warren or K system, or an X-type pure tension system with light rods or straps (Fig. 32).



PERMANENT BRACING SYSTEMS

Figure 32

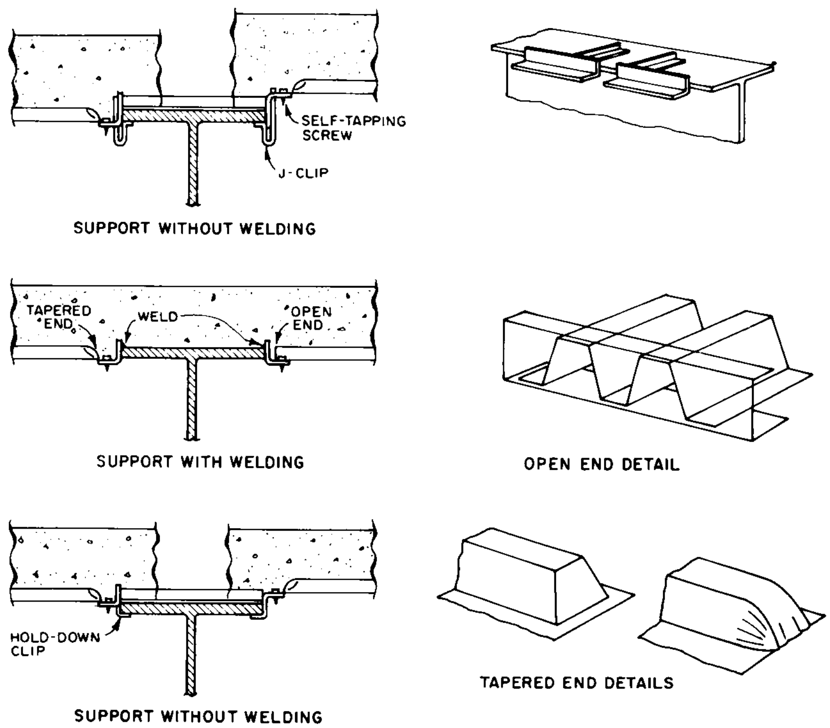
Preventing Rotation with Stay-in-Place Forms

In discussing the concept of stay-in-place forms for closing the box section, it should be noted that stay-in-place *metal* forms have gained widespread use as an alternate to conventional wood forming for the deck area within box girders (Fig. 33).

Figure 33

Used simply as forming, these corrugated panels act as beams spanning between the box girder flanges to support the wet, deck concrete. The panels are normally supported by seat angles (Fig. 34). Self-tapping screws connect the panels to the seat angles or, for more positive attachment, puddle welds may be used.

Stay-in-place forms offer the advantage of speedy installation and freedom from stripping. Since this concept has proven economical, and since the corrugated panels do physically close the box girder at the top, such panels may serve as a substitute for top lateral bracing, subject to proper investigation and approval by the Engineer. In addition, the panels can be installed in the shop and thus be available to stiffen the girder during handling and shipping, as well as during erection.



STAY-IN-PLACE FORM DETAILS

Figure 34

When this forming functions as a girder closure element, it is subjected to shear under torsional loading; therefore, two factors must be considered in determining the feasibility of such an application: 1) the strength of the corrugated panel in shear, and 2) the strength of the connection of the corrugated panels to the girder flange and to one another.

A 1973 research project[11] at the University of Maryland developed formulas for calculating an equivalent flat plate thickness for corrugated panels. These were based on the elastic deformation characteristics of the sheet plus the effect of fastener slippage at the seam and at girder connection locations. Preliminary results of tests verify the validity of the formulas and indicate that such stay-in-place forms do have adequate shear strength for use as lateral bracing under the proper conditions. A design procedure is presented by which the panels are selected from manufacturers' catalogs solely on the basis of flexural strength for a particular load and span, then transformed to an equivalent plate, permitting torsional characteristics to be calculated.

Preventing Rotation with External Diaphragms

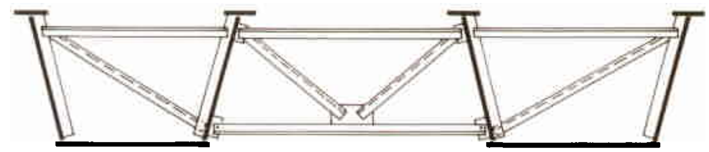
When closure of the box at the top is undesirable, torsional resistance in multiple girder systems can be furnished by temporary diaphragms or crossframes between the girders. (Obviously such external diaphragms may have to be backed up by internal diaphragms at the same locations.) If the diaphragm material is sufficient, the girders will be effectively prevented from twisting at these locations (Figs. 35 and 36).

Preventing Rotation with Shoring

When site conditions are suitable, shoring may offer a third alternative for controlling excessive torsional flexibility of the girders during construction. Figure 37 shows a very simple jackpost shoring used during construction of the Omaha Viaduct in Omaha, Nebraska. It should be noted that this bridge was built prior to adoption of the box girder specifications by AASHTO. Moreover, since this structure had diaphragms between girders, as well as lateral bracing within the girders, the shoring in this case actually served only to support the steel girder section until the diaphrgams were connected.

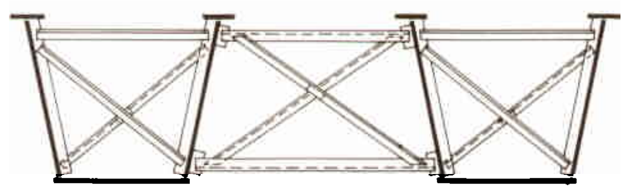
Care should be exercised in locating the bearing points for shoring. Concentrated reactions will exist at these points and there must be adequate back-up to distribute these reactions into the material of the girder. Hence, shoring will, ordinarily, be placed directly under webs, web stiffeners, or solid diaphragms.

Because long shoring pieces are uneconomical, shoring is a viable alternative to temporary lateral bracing and diaphragms mainly in low clearance structures.



(A)

Figure 35



(B)

OPEN WEB EXTERNAL DIAPHRAGMS

Figure 36

Figure 37

Flexural Stiffness—Differential Deflection Between Boxes

A third type of stiffness is flexural stiffness. This may be important during deck casting operations in wide structures that have multiple girders. If multiple casts must be made with longitudinal construction joints, the girders supporting the earlier pour may deflect below the level of the adjacent unloaded girders (Fig. 38). This makes it difficult to form and key-in the construction joint for the succeeding cast.

Strangely enough guarding against this differential deflection may be more important for the construction of box girders than in I-girder construction, since for stringer-type I-girders, the Specifications require diaphragms at 25 foot intervals and the diaphragms prevent differential deflection. The Specifications do not require diaphragms between box girders.

In the absence of diaphragms, the magnitude of differential deflection depends on the non-composite stiffness of the individual girders. Since there is no way that this stiffness can be increased—short of redesigning the girders—the remedy for excessive differential deflection finally becomes a question of whether to introduce temporary external diaphragms between box girders, or to use temporary shoring.

Preventing Differential Deflection with External Diaphragms

Connecting diaphragms between the girders creates a grid system and thus tends to equalize girder deflections. It should be recognized, however, that under transversely eccentric construction loading such a diaphragm system, if left in place, could change the deflection pattern of the over-all structure to an extent that would alter the elevations of the top of steel, change the concrete stresses, and even cause uplift at some bearing. This is another reason for avoiding eccentric construction loads and keeping the placement sequence as symmetrical as possible. Since the external diaphragms are not needed after the entire slab is cast and functioning, external diaphragms used to control differential deflection should be removed at this time. More will be said about temporary versus permanent diaphragms a bit further on.

Preventing Differential Deflection with Shoring

Temporary shoring can be very effective in controlling deflection. Since deflections due to uniform load are proportional to the fourth power of the span length, even one line of shoring at midspan can reduce the differential deflection to 1/16 of what it would be without the shoring. The decision whether to use diaphragms between girders or to use shoring, depends to a great extent on the height of the girders above the ground, as well as on whether ground conditions will permit the landing of shoring.

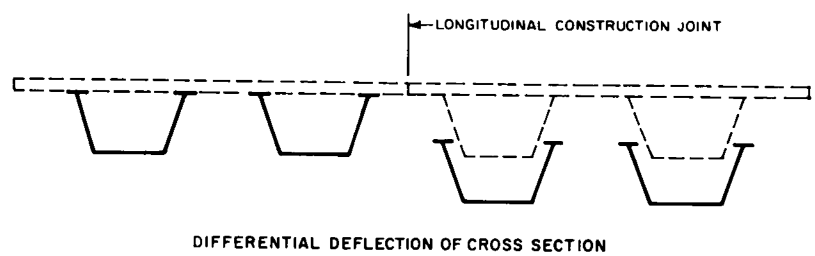


Figure 38

PART III

TEMPORARY VS. PERMANENT STIFFENING MATERIAL

The relative values of temporary versus permanent bracing material must be evaluated not only with regard to diaphragms between girders, but also to internal crossframes and lateral bracing.

For straight bridges the Specifications do not require diaphragms between box girders. It is, of course, true that introducing permanent, external diaphragms between girders represents an unnecessarily high degree of bridge design conservatism. Moreover, cross-frames or diaphragms between girders may mar the bridge's appearance. On the other hand, some of the problems encountered in removing such material must be anticipated. For example, by the time the deck has been constructed and the diaphragms can be removed, the steel erection contractor may have left the job site. In this case, either the steelwork contractor must return to remove the diaphragms, or the task falls to the general contractor. Because diaphragms are usually under stress, disconnecting them may present difficulties. Care must be taken to avoid damage—to any part of the bridge—from shock loads produced by cutting stressed members. Particular attention should be given to the deck itself which may not be up to full strength at the time of these operations.

There is yet another—obviously important—consideration when evaluating permanent versus temporary bracing, and that is the salvage value of the material relative to the cost of removal. From an economic standpoint, only when the material can be re-used in subsequent box girders on the same project will the removal of diaphragms and bracing be worth the effort. Nonetheless, external diaphragms that are not part of the original design, should always be removed because the structural behavior of the box girders and the load distribution may be significantly changed if they are left in place.

PART IV

RESPONSIBILITIES OF OWNER, DESIGNER, CONTRACTOR

As has been shown, designing a box girder for its function as a member of the finished bridge does not, of itself, guarantee that it will have adequate stiffness and stability under construction loads; some extra bracing and stiffening is often required during the construction phase. But this leads to the inevitable question: how is responsibility distributed—among owner, designer, and contractor—for determining a) whether or not such extra material is needed, b) its designing and c) taking into account the construction conditions?

The Common Law of Contracts requires only that the owner and designer working together must produce a design that can be built; the contractor is required to erect the structure upon whose construction he has successfully bid, regardless of any lack of construction information on the plans. However, a distinction should be made between legal responsibility and the more arbitrary form of responsibility: the ethical position dictated simply by good professional and business practice. This latter responsibility is the issue at hand.

To gain some perspective on the whole question of responsibility, it may be helpful to compare European and American bridge engineering practice. The European bridge designer is often the builder as well, and design-construct firms are usually staffed with highly qualified engineers. In many cases, owners solicit competitive designs—a practice that frequently leads to innovative bridge designs and construction methods. Under these circumstances, the engineer-builder is directly involved throughout all of the project phases, thus he is in a good position to give every construction step as much attention as may be required.

The situation is markedly different in the United States. Bridges are designed either by the owner or by a consultant to the owner. The end product in American design procedure is a set of contract plans upon which a construction contract is competitively bid by companies that only perform construction services. Certain parts of the work, such as detailing, fabrication and erection, may in turn be subcontracted by the general contractor to firms specializing in those areas.

The contract documents usually establish procedures whereby detail drawings and construction procedures must be submitted to the owner or his consultant for approval. But, after the design phase, the Engineer is involved in the project only in the somewhat indirect way described. He is usually in the position of *reacting* to proposals and developments that take place during construction. Rarely, does he take the initiative. In America, this involvement of the engineer, primarily in design operations is made evident in the manner that design specifications have evolved.

American design specifications deal with the ultimate *performance* of the *completed* structure; they do not contain provisions for bracing *during* construction. For straight box girders, the only requirement given in the Specifications regarding secondary material concerns the use of internal diaphragms at the support locations. To quote the provisions: *"Diaphragms, crossframes, or other means shall be provided within the box girders at each support to resist transverse rotation, displacement and distortion."* The Specifications continue: *"Intermediate diaphragms or crossframes are not required for steel box girder bridges designed in accordance with this Specification."*

Because of these provisos, permanent, intermediate crossframes or bracing make the design conservative to an unnecessarily high degree, as far as final service load capabilities are concerned.

For curved box girders, a Tentative Specification[13] has been formulated which *does* require a rational analysis of intermediate cross frame requirements within girders, but does not require closure of the girder to create torsional stiffness. Again, there is no specific mention of construction conditions.

Thus, since the emphasis in design is on the service behavior of the *finished* structure, it is not surprising that some designs have been made without adequate consideration of *construction* conditions. The official commentary[14] on the Specifications for straight girders does, however, acknowledge the special problems that exist during construction and offers the following guidance:

"In order to maintain the geometry of the box girder section during fabrication, hauling, erection and placement of the deck, it may be necessary to provide removable or construction bracing until the deck is completed. If details of this bracing are not shown on the contract drawings, then the need for such bracing should be brought to the attention of the contractor, either on the contract drawings or in a special provision." The Commentary then gives an example of such a special provision: *"The individual box girder section geometry shall be maintained throughout all phases of construction, including placement of the concrete deck. It is the contractor's responsibility to provide the bracing required to maintain the geometry of the box girder section. This bracing may be permanent or temporary at the contractor's option, but must be done without damage to the elements of the box girder. The bracing shall provide support to the top flanges when the box girder webs slope, or when the deck falsework is constructed in a manner which causes transverse horizontal forces in the girder flanges before and/or during placement of the concrete deck. Permanent or removable bracing may be required between individual box girders to prevent rotation of an exterior box girder caused by loading the deck cantilever."*

Apparently, the need for special bracing during construction is recognized, despite the fact that the Specifications deal only with the finished girder. Thus, it is now common practice for the designer to make some provision for the wet concrete stage of construction; at this stage, the design often includes bracing for stiffening or stabilizing the steelwork. However, in Appendix C, discussions of some early box-girder bridge case histories reveal that designers underestimated the amount of bracing needed because of a lack of box-girder construction experience. Furthermore, the contractor does not have the right to assume that the designer has considered construction conditions. Hence, the contractor himself must be aware, at least in a general sense, of the likely behavior of a box-girder bridge during the deck placement stage as well as the necessary bracing requirements.

In all probability, it was proper for U.S. practice not to extend the designer's initial considerations much beyond that of providing bracing for the wet concrete stage, because the contractor—as an independent entity—needed as much freedom as possible in erection procedures and material usage.

It must be assumed that a qualified contractor is best equipped to know how to utilize his equipment and material on a given job. Thus, it is his responsibility to make provisions for all construction loads and stresses that result from fabrication, handling, transportation and erection.

PART V

GUIDELINES FOR GOOD PRACTICE

Actual experience coupled with information derived from earlier sections of this publication (Parts I through IV) can provide some guidelines for good practice in steel box-girder construction as follows:

1) Vertical crossframes should be located at the lifting points of each shipping piece and at each side of a field splice. Construction of straight box girders of spans under 150 ft. usually requires only one panel of horizontal, lateral bracing on each side of a lifting point. Straight box girders with spans greater than 150 ft. may need a full-length lateral bracing system; this is to prevent distortions that may be brought about by temperature changes occurring prior to concrete slab placement.

2) It is preferable that eccentric construction loads be avoided, but if they cannot, a full-length, properly proportioned internal lateral bracing system will reduce box-girder twisting to acceptable values. If the design does not provide such a lateral system, a temporary external crossframe system between boxes can provide the required resistance to twisting.

3)When lifting straight box-girder pieces with thin, unstiffened bottom flanges, lifting points located near the ends of the piece should be used.

4) Curved box-girder pieces may require lifting points away from the ends of the piece. When these curved box pieces have thin unstiffened bottom flanges, the flange stresses should be investigated to determine the need for stiffening material as described in Appendix B. If stiffeners are welded to the flange, no attempt should be made to remove them after erection. In addition, the ends of these added stiffeners should always be finished with a coping whose radius is greater than 6 inches, and the toe termination of the connection welds should be ground smooth.

5) Normally, curved box-girder designs call for a full-length lateral bracing system with internal crossframes whose number and spacing are a function of the degree of curvature. Usually, where a system such as this is provided, no additional bracing will be required for construction. If the original design does not specify bracing or diaphragms, some should be added during fabrication and made permanent, subject to the approval of the engineer.

6) Where a steel box girder has an aspect ratio (depth/bottom flange width) greater than about 2.0 and the design does not specify a permanent lateral bracing system, a temporary exterior crossframe system between boxes should be added. This will give additional stability to the boxes and enable them to withstand wind gusts more effectively during construction.

7) When temporary bracing or crossframes are added for construction purposes the preferable practice is to fasten them with bolted connections that have a minimum of two bolts per connection.

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COMPUTER SOLUTION: PLATE PATTERN COORDINATES OF SLOPED WEBS ON CURVED BOX GIRDERS

This Appendix section shows output from the USS computer program PATC for web plate geometry. The program is available through USS Engineers and Consultants, Inc., a subsidiary of United States Steel Corporation.

In Figure 39, the first part of the printout is a verification of the input data and indicates that the program requires horizontal geometric data, profile grade data, stationing and camber ordinates. The computed results are given in the form of a table entitled "Plate Pattern Coordinates." For the particular section of the plate represented here, X and Y coordinates, illustrated in Figure 40, are printed for the upper and lower edges of the developed plate pattern at intervals of about 4 ft. These results include the effect of horizontal and vertical curvature, super elevation, and camber, and would enable a fabricating shop to lay out the web plate for cutting.

PATTERN COORDINATES FOR CURVED GIRDERS WITH SLOPING PLATES

BOX GIRDER AG9

0. CONTINUATION PIECES

HORIZONTAL CURVE DIRECTION	RADIUS	PLATE EDGE RADII UPPER	LOWER	VERTICAL HEIGHT OF PLATE
LEFT	643.530	654.655	653.592	4.250

PROFILE GRADE DATA

PVC STATION	V.C. LENGTH	INITIAL GRADE (PCT)	FINAL GRADE (PCT)
1952.694	460.000	5.00000	-5.00000

WEB A8 --- PORTION 2

PLATE START STATION	2017.9370	PLATE END STATION	2064.5300
NO. PLATE DIVISIONS	12	S.E. TRANS. START STA.	0.0000
SUPERELEV. RATE-START	0.0680	S.E. TRANS. END STA.	0.0000
SUPERELEV. RATE-END	0.0680		

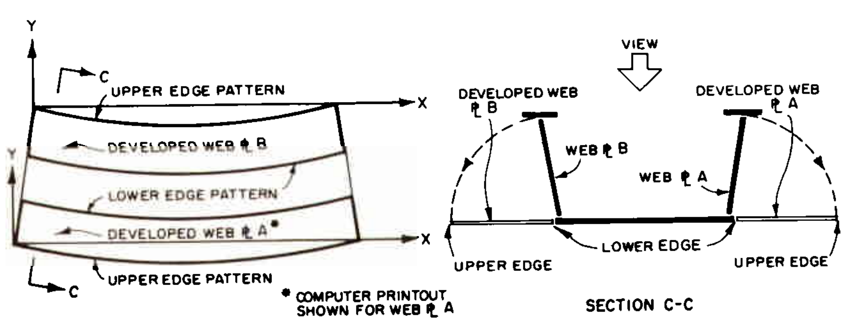
DEAD LOAD CAMBER DIAGRAM (INCHES)
STA.D0 1993.1840 STA.D5 2042.6900 STA.D10 2112.6900

D0	D1	D2	D3	D4	D5	D6	D7	D8	D9	D10
0.96	0.74	0.46	0.20	0.04	0.00	0.25	0.74	1.31	1.70	1.82

POINT	STATION	PLATE PATTERN COORDINATES(FEET) UPPER EDGE X	Y	LOWER EDGE X	Y	CAMBER (IN.)	SUPER-ELEV. RATE	Y-NO CAMBER UPPER EDGE (INCHES)
0	2017.937	0.0000	-0.0000	0.0480	4.3783	0.3299	0.0680	-0.00
1	2021.819	3.9521	-0.0311	3.9918	4.3473	0.2280	0.0680	-0.49
2	2025.702	7.9042	-0.0572	7.9357	4.3213	0.1545	0.0680	-0.89
3	2029.585	11.8562	-0.0767	11.8795	4.3020	0.0917	0.0680	-1.20
4	2033.468	15.8082	-0.0895	15.8233	4.2892	0.0372	0.0680	-1.43
5	2037.350	19.7602	-0.0980	19.7670	4.2808	0.0215	0.0680	-1.56
6	2041.233	23.7121	-0.0991	23.7107	4.2797	0.0058	0.0680	-1.60
7	2045.116	27.6641	-0.0971	27.6544	4.2817	0.0433	0.0680	-1.56
8	2048.999	31.6160	-0.0903	31.5981	4.2885	0.1126	0.0680	-1.43
9	2052.881	35.5678	-0.0761	35.5417	4.3028	0.1819	0.0680	-1.20
10	2056.764	39.5195	-0.0546	39.4851	4.3242	0.2526	0.0680	-0.89
11	2060.647	43.4712	-0.0310	43.4286	4.3479	0.3885	0.0680	-0.49
12	2064.530	47.4228	-0.0000	47.3720	4.3787	0.5243	0.0680	-0.00

UPPER EDGE CHORD - NO CAMBER 47.4219

Figure 39



DEVELOPED WEB PLATE PATTERN COORDINATES

Figure 40

Illustrative Example: Investigation Of Bottom Flange For Local Buckling During Lifting Operations

Here, the example illustrates stability considerations for unstiffened bottom flanges during lifting operations.

The girder section shown in Figure 41, is taken from an actual design, and represents one of the shipping pieces of a three-span continuous unit. In the finished structure, this peice will resist positive bending moment, with the bottom flange in tension. It has an approximate over-all length of 142 ft, weighs 99 tons, and is assumed to be lifted at the quarter points. Although the section happens to have a slight horizontal curvature, and a variation in width and depth, these factors do not affect the example.

Stresses will be checked at the right quarter point where the girder is lifted, and at the flange transition 21 ft from the right end of the girder. It is therefore necessary to calculate the moments at these locations.

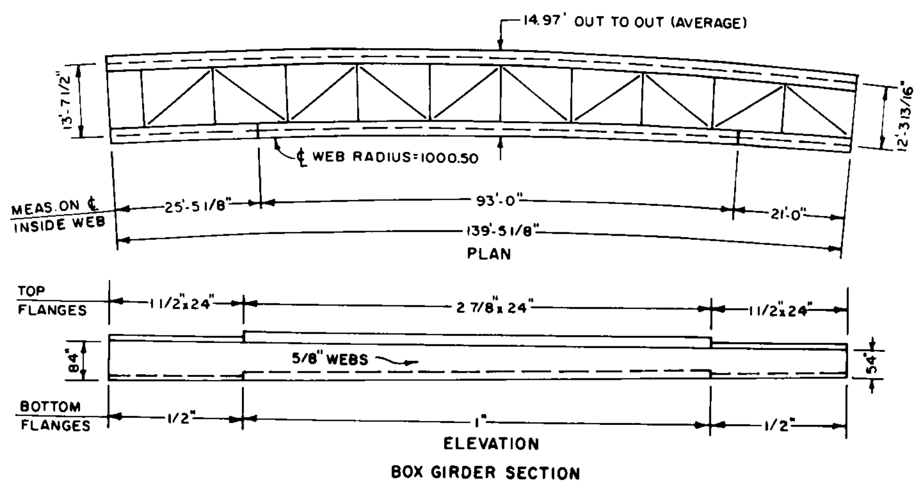


Figure 41

Total weight of girder = 198000# = 198.0 kips
Avg. out-to-out width of box = 14.97′
Avg. length of box

$$= \left[1000.50 + \frac{(14.97 - 2.00)}{2}\right] \times \left[\frac{139.43}{100.50}\right] = 140.33$$

$$\text{Wt. per foot} = \frac{198.00}{140.33} = 1.411 \text{ kips/ft.}$$

$$\text{Mom. @ 1/4 Pt.} = \frac{1.411\left(\frac{140.33}{4}\right)^2}{2} = 868^{k'}$$

Mom. @ Sect. Transition

$$= \frac{1}{2}(1.411)\left[21.0\left(\frac{140.33}{139.43}\right)\right]^2 = 315^{k'}$$

The moment diagram is shown in Figure 42.

Since the member varies in depth and width, the depth and width at the sections in question are calculated.

Width of Box (c-c webs) @ 1/4 Pt.

$$= 147.81 + 1/4(163.50 - 147.81) = 151.73''$$

Girder Depth @ 1/4 Pt.

$$= 54 + 1/4(84 - 54) = 61.50''$$

Width of Box @ Transition

$$= 147.81 + \frac{21.00}{139.43}(163.50 - 147.81) = 150.17''$$

Girder Depth @ Transition

$$= 54 + \frac{21.00}{139.43}(84 - 54) = 58.52''$$

Section properties and stresses can then be calculated. The girder sections at the quarter point and section transition point are shown in Figure 43.

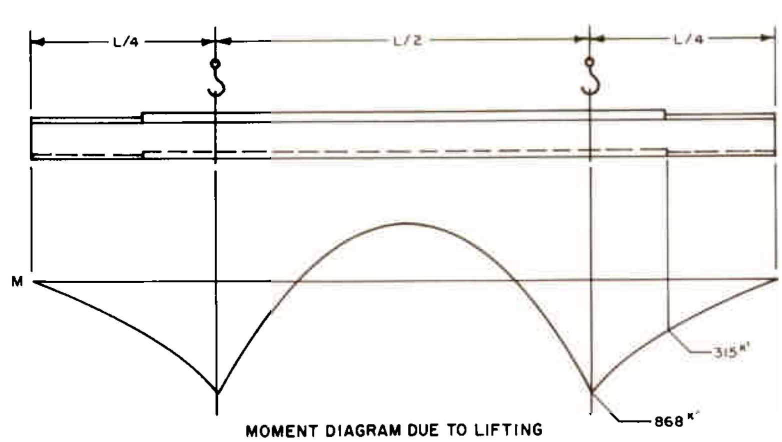


Figure 42

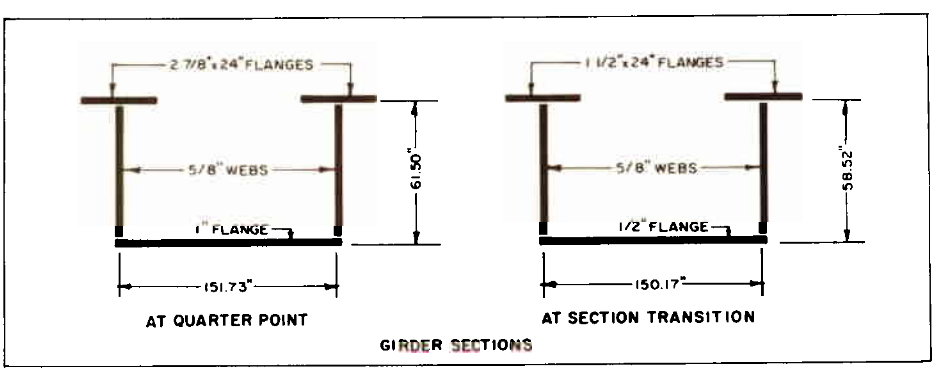


Figure 43

Section Properties and Stresses at Quarter Point

Material	A	d	Ad	Ad²	I_o	I
2-flanges $2\frac{7}{8}'' \times 24''$	138.00	31.69	4,373	138,587	—	138,587
2-webs $\frac{5}{8}'' \times 60.5''$	75.63	—	—	—	23,067	23,067
1-flange $1'' \times 152.35''$	152.35	−30.75	−4,684	144,056	—	144,056
	365.98 inches²		−311 inches³			305,710

$$d_s = \frac{-311}{365.98} - 0.85, \quad 311 \times 0.85 = \quad \frac{-264}{305{,}446 \text{ inches}^4}$$

$$d_{Bot.\ of\ steel} = 31.25 - 0.85 = 30.40 \text{ inches}$$

$$S_{Bot.\ of\ steel} = \frac{305{,}446}{30.40} = 10{,}048 \text{ inches}^3$$

$$f_b = \frac{868 \times 12}{10{,}048} = 1.04 \text{ ksi (calculated stress)}$$

$$\frac{b}{t} = \frac{151.11}{1} = 151.11 > \frac{13{,}000}{\sqrt{F_y}} = \frac{13{,}000}{\sqrt{36{,}000}} = 70.1 \therefore$$

Allowable stress must be calculated from the following formula given in AASHTO, Article 1.7.49(D)(2):

$$F_b = 57.6\left(\frac{t}{b}\right)^2 \times 10^3 = 57.6\left(\frac{1}{151.11}\right)^2 \times 10^3 = 2.52 \text{ ksi (allowable)}$$

Since the allowable stress (2.52 ksi) is greater than the calculated stress (1.04 ksi), no stiffeners are required.

Section Properties and Stresses at Section Transition

Material	A	d	Ad	Ad²	I_o	I
2-flanges $1\frac{1}{2}'' \times 24''$	72.00	29.76	2,143	63,767	—	63,767
2-webs $\frac{5}{8}'' \times 58.02''$	72.53	—	—	—	20,345	20,345
1-flange $\frac{1}{2}'' \times 150.79''$	75.39	−29.26	−2,206	64,545	—	64,545
	219.92 inches²		−63 inches³			148,657 inches⁴
					63 × 0.29 =	−18
						148,639 inches⁴

$$d_s = \frac{-63}{219.92} = 0.29 \text{ inches}$$

$$d_{Bot.\ of\ steel} = 29.51 - 0.29 = 29.22 \text{ inches}$$

$$S_{Bot.\ of\ steel} = \frac{148.639}{29.22} = 5{,}087 \text{ inches}^3$$

$$f_b = \frac{315 \times 12}{5{,}087} = 0.74 \text{ ksi (calculated stress)}$$

$$\frac{b}{t} = \frac{149.54}{0.5} = 299.08 > \frac{13{,}300}{\sqrt{36{,}000}} = 70.1 \therefore$$

Allowable stress is calculated from the following formula

$$F_b = 57.6\left(\frac{t}{b}\right)^2 \times 10^3 = 57.6\left(\frac{0.5}{149.54}\right)^2 \times 10^3 = 0.65 \text{ ksi (allowable)}$$

At the quarter point, the calculated stress of 1.04 ksi is less than the 2.52 ksi basic allowable stress for design. At the flange transition, however, the calculated stress of 0.74 ksi exceeds the 0.65 ksi allowable design stress. Although this is a 14-percent overstress, there is little possibility that buckling would actually occur because of the safety factor which the allowable stress equation includes; often, as much as a 33-percent overstress is allowed for temporary construction stresses. The designers of this bridge also investigated the stresses at the left transition point and added nominal longitudinal stiffeners on the bottom flange; no buckling occurred during construction. The main thing to be learned from this example is that a primary unstiffened-tension flange can be put into a critical, compression-buckling condition during construction.

Case Histories

Many steel/concrete composite box girder projects have been successfully and economically constructed in recent years. Bridges, wherein the designer provided ample bracing (either temporary or permanent) for the erection and wet-concrete stages, were constructed with few, if any, unusual construction problems. Figures 44 through 74 are photographs of the fabrication, transportation, and erection of some of these noteworthy projects at Vail Pass, Colorado; Omaha, Nebraska; Baton Rouge, Louisiana and Pittsburgh, Pennsylvania.

Figure 44 Vail Pass Colorado Bridges: crossframes and lateral bracing arrangement.

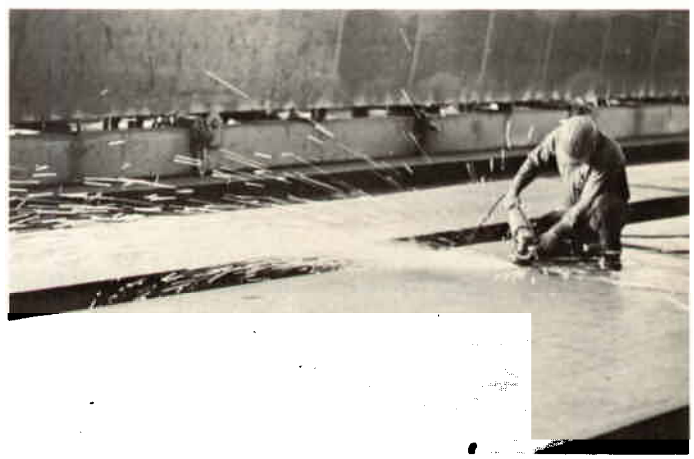


Figure 45 Vail Pass Colorado Bridges: bottom flange preparation.

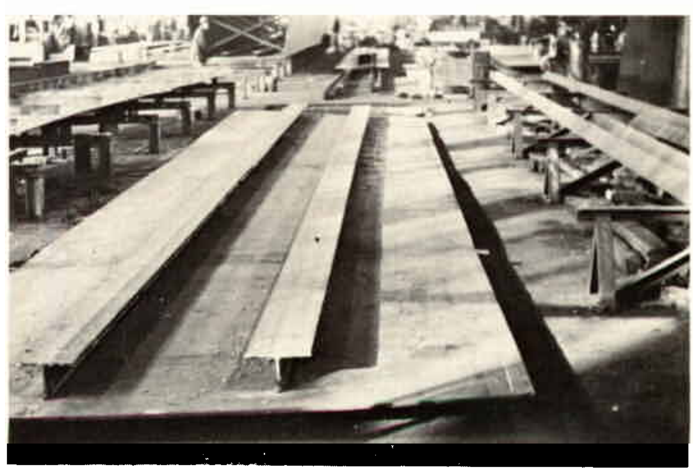


Figure 46 Vail Pass Colorado Bridges: bottom flange stiffeners.

Figure 47 Vail Pass Colorado Bridges: web to top flange fabrication.

Figure 48 Vail Pass Colorado Bridges: fit-up jig.

Figure 49 Vail Pass Colorado Bridges: end diaphragm fabrication.

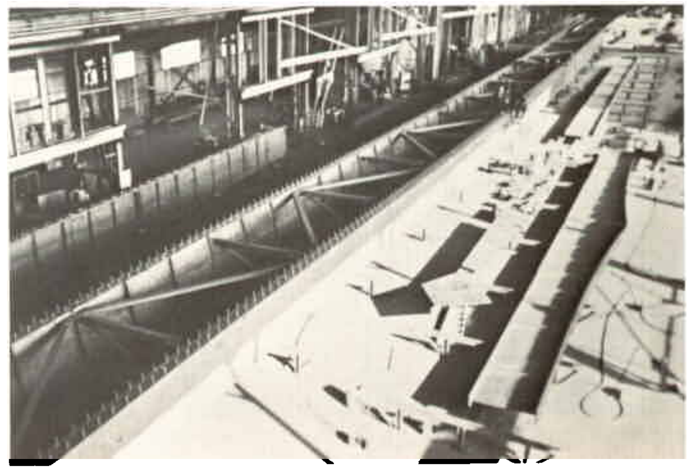


Figure 50 Vail Pass Colorado Bridges: shop assembly.

Figure 51 Vail Pass Colorado Bridges: erection of shipping piece.

Figure 52 Vail Pass Colorado Bridges: erection of shipping piece.


Figure 53 Vail Pass Colorado Bridges: erection of shipping piece.

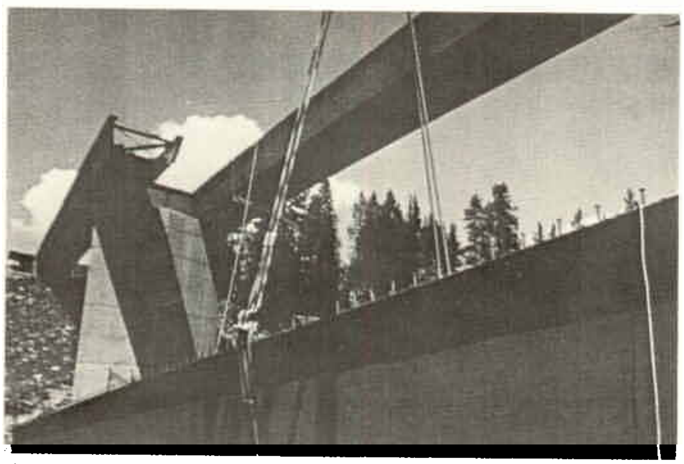


Figure 54 Vail Pass Colorado Bridges: erection of shipping piece.

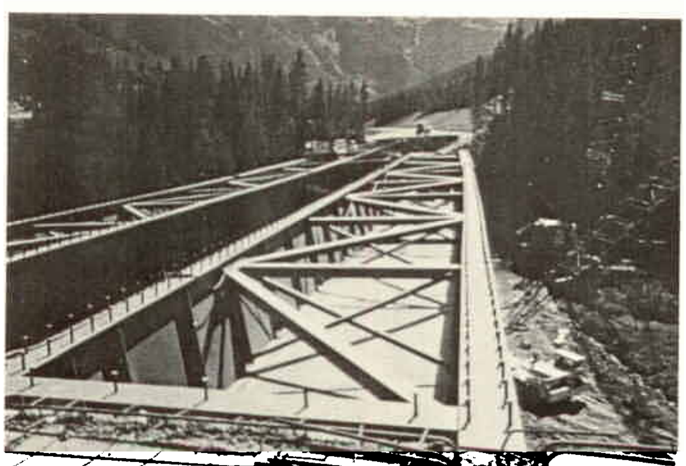


Figure 55 Vail Pass Colorado Bridges: erected steelwork.

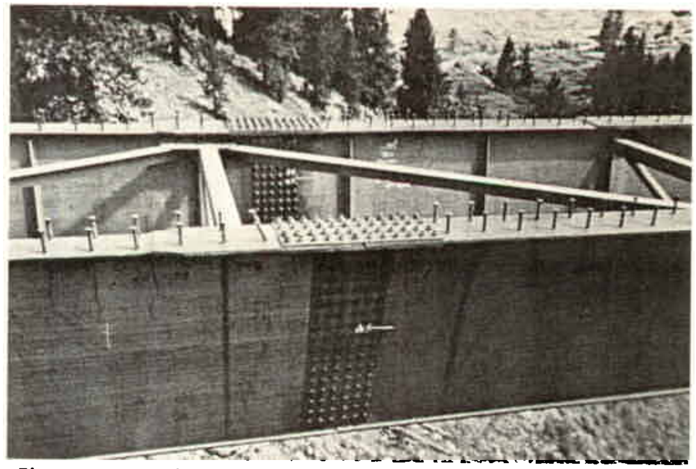


Figure 56 Vail Pass Colorado Bridges: field splice and flange transition.


Figure 57 Vail Pass Colorado Bridges: erection.

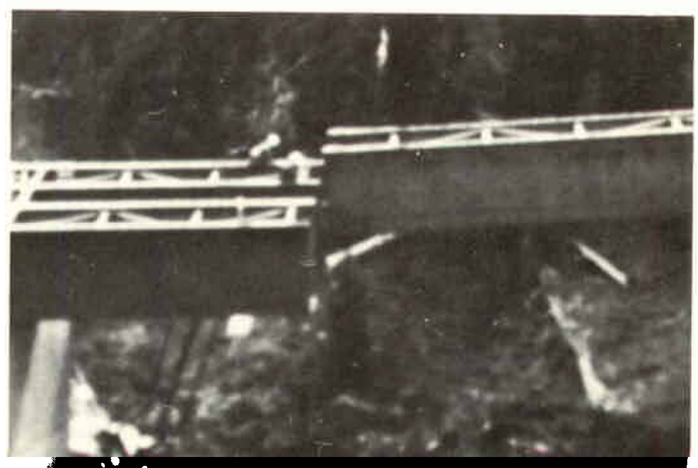


Figure 58 Vail Pass Colorado Bridges: erection.

Figure 59 Vail Pass Colorado Bridges: formwork for concrete deck slab.

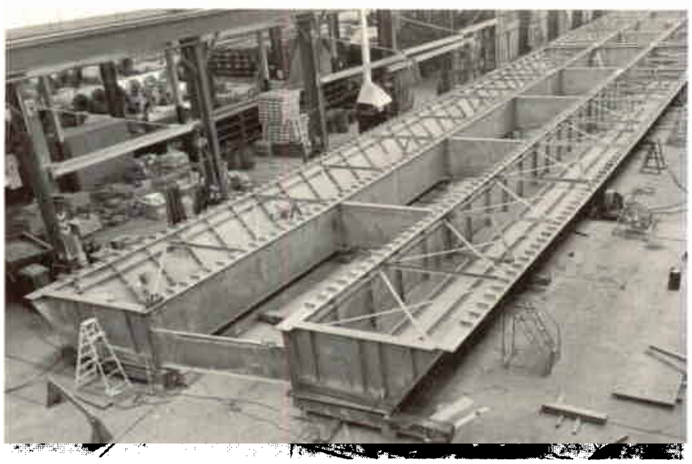


Figure 60 Omaha Nebraska Viaduct: shop assembly showing crossframe lateral bracing arrangement.

Figure 61 Omaha Nebraska Viaduct: erection of curved-box section.

Figure 62 Omaha Nebraska Viaduct: erection of curved-box sections.

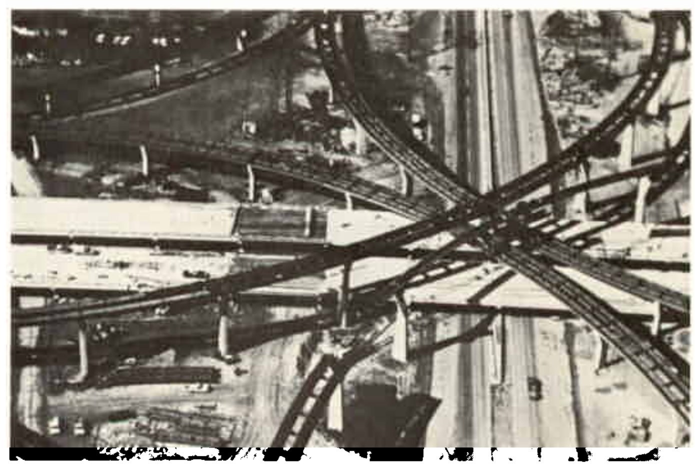


Figure 63 Louisiana Airline Highway Interchange: aerial view of erected steelwork.

Figure 64 Louisiana Airline Highway Interchange: erected steelwork.

Figure 65 Louisiana Airline Highway Interchange: erected curved-box girders.

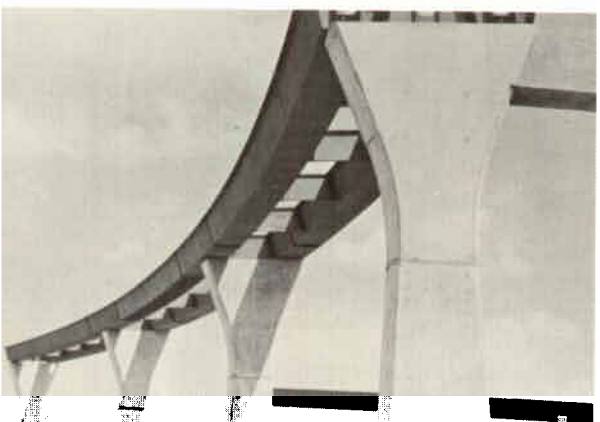


Figure 66 Louisiana Airline Highway Interchange: erected sections of curved-box girders.

Figure 67 Louisiana Airline Highway Interchange: erected sections of straight-box girders.

Figure 68 Louisiana Airline Highway Interchange: finished structures.

Figure 69 Louisiana Airline Highway Interchange: finished structures.

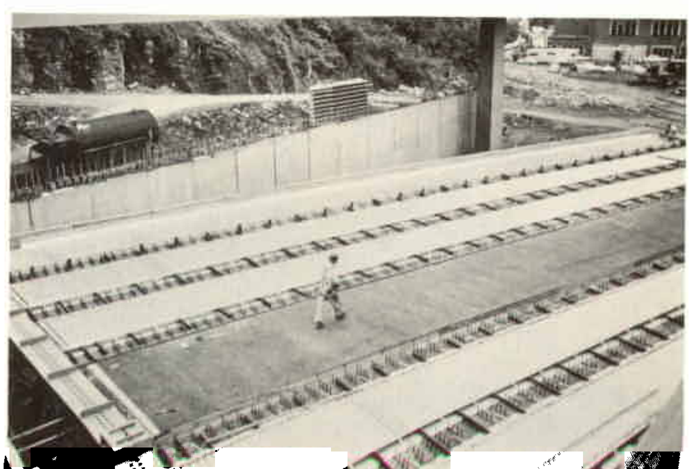


Figure 70 Pittsburgh's Fort Duquesne Bridges: stay-in-place forms, two-cell boxes.

Figure 71 Pittsburgh's Fort Duquesne Bridges: finished structures.

Figure 74 Pittsburgh's Fort Duquesne Bridges: finished structures.

Figure 72 Pittsburgh's Fort Duquesne Bridges: erection of box-girder span to bents.

Figure 73 Pittsburgh's Fort Duquesne Bridges: erection of curved-box sections.

However, a great deal more can be learned from studying cases where difficulties *were* encountered, and the following illustrate some unusual experiences. Discussion centers on 1) what caused these difficulties to occur, 2) what was done to correct them, and 3) how they might have been avoided.

Clearwater Bridge

The Clearwater Bridge was the first steel/concrete composite box girder bridge in Florida. It is a 206'-3" simple-span bridge carrying U.S. Route 19 over State Route 60 in Pinellas County (Fig. 75). The bridge is oriented in a nearly due north-south direction. Each girder was erected in three pieces, with two welded field splices (Figs. 76, 77 and 78). The girder sections were supported on falsework bents located at each field splice.

The design included internal crossframes on 23'-3" to 26'-0" centers; there were no external diaphragms between girders. No lateral bracing, temporary or permanent, was used near the plane of the top flanges.

After the field splices were completed, the falsework was removed, and forming preparations were begun for the first placing of the concrete deck—a cast of 41'-8" in length centered at mid-span. At the time of the removal of the falsework, all girders were observed to be straight and both top flanges of each box were at approximately the same elevation (Fig. 79).

Very soon thereafter, it was noted that some of the girders were curving horizontally outward from the bridge center line, and twisting, thereby causing differential elevations between the top flanges of each box. This movement was so great that formwork for the concrete deck could not be properly constructed (Figs. 80 and 81).

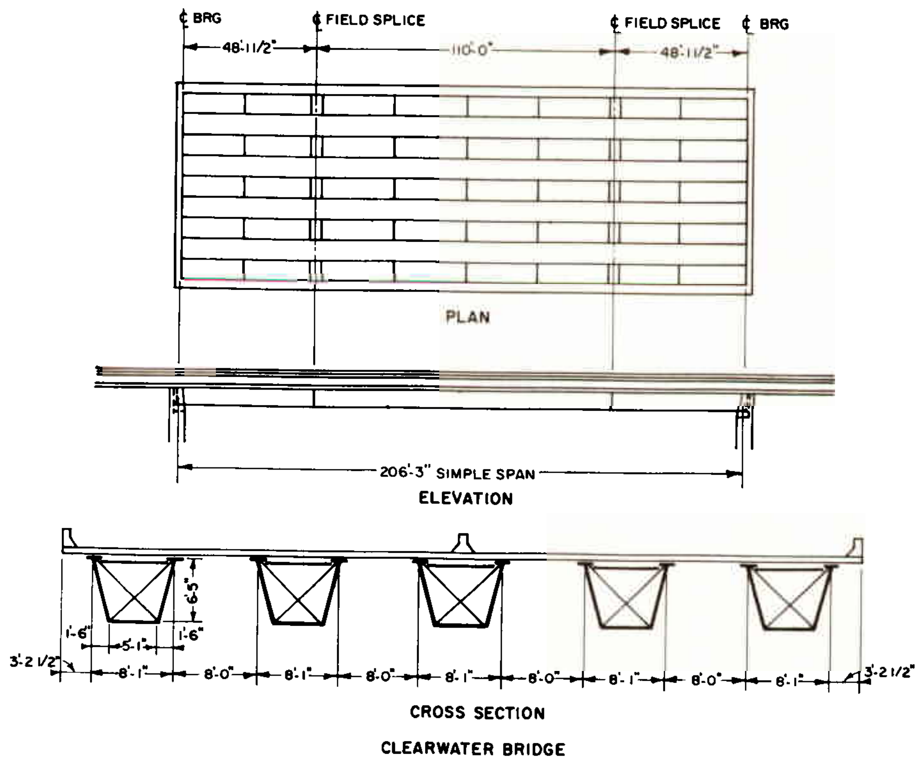


Figure 75

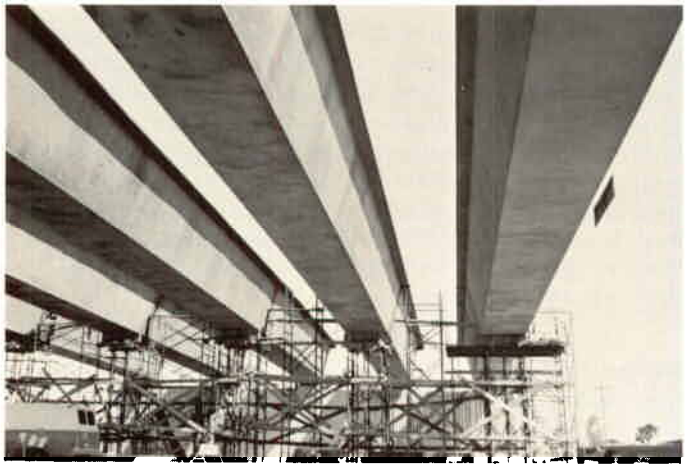


Figure 76 Early Erection Stage.

Figure 77 Further Erection.

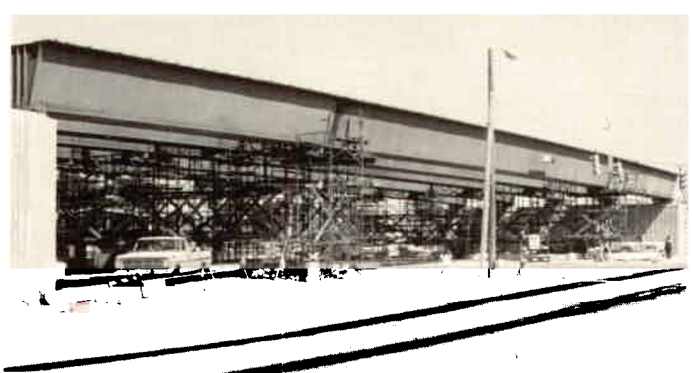


Figure 78 Erection Completed.

Figure 79 Erected Girders Prior to Distortion.

Figure 80 Bowing of Girder.

Figure 81 Twist of Girder.

Work was suspended for the next few days, to allow time to determine the cause and devise necessary corrective measures. During this period, the bowing and twisting of some of the boxes increased each day until the falsework was once again installed to stop the movement. Flange elevation differentials as much as 11 inches were reached.

The cause of the initial box movement was determined to have been initiated by differential thermal expansion of the girder webs, with further propagation occurring from the resulting eccentricity of the girder weight. Due to the north-south alignment of the bridge, the outside webs of the exterior boxes were exposed to long periods of full sun, the most easterly box receiving its exposure during the morning hours and the most westerly box during the afternoon. This exposure, coupled with the fact that the insides of the girders had been painted white for inspection purposes, while the outsides were covered with a typically darker shade of shop paint, resulted in maximum temperature differentials between the webs of the exterior box girders.

Similarly, but to a lesser degree, the outside webs of the first interior boxes also had more sun exposure than their inside webs due to cross-slope elevation differential and shielding from the sun by adjacent girders. The box located on the bridge center crown-line had the same amount of sun exposure on both webs and had very little movement.

The exterior boxes twisted outward with the exterior top flange downward and the interior top flange upward; the first interior boxes moved in a manner similar to that of the exterior boxes but with the magnitude of the movement only about half of the exterior box movement. Inasmuch as no system of lateral bracing was provided near the plane of the top flanges, the girders behaved as open sections with low torsional rigidity and with the center of torsional rotation below the bottom flange.

The remedial measures taken involved moving the top flanges back to the correct position and holding the boxes in this position until the center 41′-8″ slab placement was completed. Taking advantage of symmetry of the movement, boxes on the opposite side of the center line were used to "pull" each other back into position. This was accomplished by connecting cables to the top flanges of the boxes at two crossframe points located approximately 26 ft from each side of the bridge center line. Hydraulic jacks at the falsework bents were used to move the box girders. Then the cables were tightened. By alternating these operations, removing the cable slack after each jacking operation, the boxes were correctly aligned with only a small cable force applied. Also, during this time, the exterior webs were shaded from the sun and a water sprinkler system lowered the steel temperature and minimized temperature differential. When the box girders reached the correct position, two continuous 24 WF beam diaphragms were attached to the top flange of each box girder adjacent to the cable pull points. Then the falsework jacks were removed and the boxes were held in the correct position by the cables and WF diaphragms. The first section of the deck slab was then placed and, after a period of curing time for the concrete, the cables and WF diaphragms were removed. No further problems with girder twisting occurred.

In conclusion, this example serves to illustrate the extreme torsional flexibility of a long span girder of open section. It shows that internal crossframes alone are not effective in preventing box girder twist, and that a lateral bracing system is probably a desirable feature in box girder bridges of this length. It also shows the importance of considering thermal effects in a structure of this size. That these thermal effects were not reversible but, to the contrary, cumulative, was apparently due to one or more forms of inelastic behavior such as slippage at the bearings, buckling or yielding.

Because thermal distortion is always a potential problem in the construction of any major structure, care should be taken to eliminate situations that could create differential temperature conditions.

Stoneschool Road Bridge

The Stoneschool Road Bridge was one of the first steel/concrete composite box-girder bridges constructed in Wisconsin. Located in Walworth County, it carries Stoneschool Road over State Highway 15. It is a two-span continuous bridge, skewed approximately 44°. The design incorporates temporary, removable internal crossframes on 19'-0" to 24'-0" centers, without external diaphragms between girders. The girders are unusually deep and narrow (Figs. 82, 83 and 84).

One of the first problems encountered involved the internal crossframes. These had been added by contract change order, and were attached to the girder merely by a single bolt at each contact point; this allowed excessive rotation (later corrected by field welding the connections). Furthermore, despite the fact that the cross frames had single diagonals, these members were designed only for tension. They bent badly when they were put in compression by handling and wind loads during the early construction stages.

The first girder was erected February 25 and 26, 1971. Field splices were made, but the bearing assemblies were not bolted down. On February 27, a period of high winds, the girder and rocker assembly slid 1 $\frac{1}{2}$ inches out of alignment, despite the fact that two cranes had been left attached to the girders for stabilization. This girder had to be reset and the remaining girders were then placed in position.

The deck forming was nearing completion when high winds once again blew the girders out of alignment, leaving them in the wrong positions, both vertically and horizontally, producing serious discrepancies in the deck forming geometry. At this point, it was decided to attempt to realign the girders and install bracing between them.

Efforts to align the girders by lifting and pulling horizontally with cranes were initially unsuccessful because of the restraining effect of the deck forming. Eventually, however, the girders of the north span were pulled into position and the bracing between girders was welded into place in that span. But the south span remained out of alignment and it was necessary to remove the formwork from that span to correct the alignment. The forming for the overhang was left intact while this was being accomplished, and the resulting eccentric load caused an overturning tendency on the outside girder; this had to be kept in check by attaching a crane to the girder. When correct vertical and horizontal alignment were obtained in the south span, the bracing was installed and the deck reformed. The entire bridge deck was then placed. Bracing between girders was removed after the deck had cured.

Difficulties continued to arise even after the deck was placed. Longitudinal and transverse cracks were noticed in the deck after one month. Two months after the deck placing and subsequent to the removal of the bracing between girders, high winds again occurred, and additional longitudinal cracks were observed in the deck along the edges of the girder flanges.

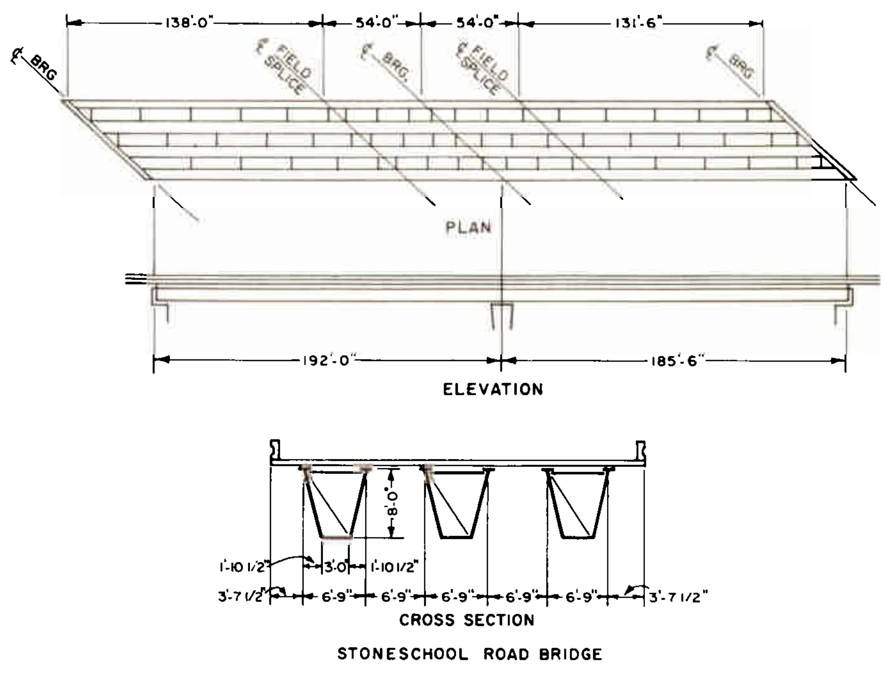


Figure 82

Stoneschool Road Bridge (continued)

Later, investigation revealed a single root cause for the construction problems occurring on this structure: the unusual proportions of the box girders. While aesthetically pleasing, deep girders with relatively narrow bottom flanges tend to be somewhat unstable if bracing is not provided between girders. Furthermore, the deep girder profile exposes a large area to the wind producing an overturning tendency about the bottom of the box. Because the boxes were narrow and lacked interconnecting bracing, the transverse stiffness was insufficient—prior to placement of the slab—to prevent the girders from deflecting laterally and "walking" out of alignment in high winds. It was even observed that the absence of lateral bracing resulted in a girder with so little torsional stiffness over the long spans that the weight of a man was sufficient to deflect a girder flange.

The longitudinal cracking of the decks during high winds was evidently due to the continuing tendency of the girders to overturn about their bottoms, in spite of the presence of the slab. Most of the difficulties on this structure could have been eliminated by the use of permanent lateral bracing within the box girders, and permanent diaphragms between girders—at least, at the support locations.

Box girders with skewed ends as on this structure are more difficult to fabricate and erect than square-ended girders. On large skews, deflection caused by the dead load of the concrete, changes the geometry at the end crossframe or diaphragm enough to raise one corner free of its bearing when field bolted. For this reason, some fabricators make it a practice to detail the end crossframes of girders with skewed ends, so that they will fit properly when the girder has been placed in position and the dead load applied.

Figure 83

Figure 84

ILLINOIS BRIDGES

Illinois' first steel/concrete composite box-girder bridges had a number of erection problems. Here, the basic difficulty was that contractors were supporting forming for the slab overhang, plus rails for concrete finishing machines, on cantilevers off the outside webs of the exterior boxes. Since these girders were not laterally braced they twisted far out of geometric tolerance.

Figure 85 shows the solution that was developed by the Illinois Department of Transportation; it has now become standard practice. Depending upon the contractor's forming system, additional ties between top flanges may be required. Ties are not required at strut locations.

In this solution, crossframes are temporarily bolted between girders, and then removed after placement of the deck. The crossframe members are attached to connection angles on the web backed up by stiffening angles on the opposite side of the web (Fig. 85, Section A-A). After the deck is cured, all this material is removed except for the bolts through the web. These are left to seal the holes. In this way, the uncluttered appearance of the finished structure—when viewed from below—is not measurably affected.

This case is a graphic example of an effective means to solve the problem of excessive girder twisting by using diaphragms between girders rather than internal lateral bracing.

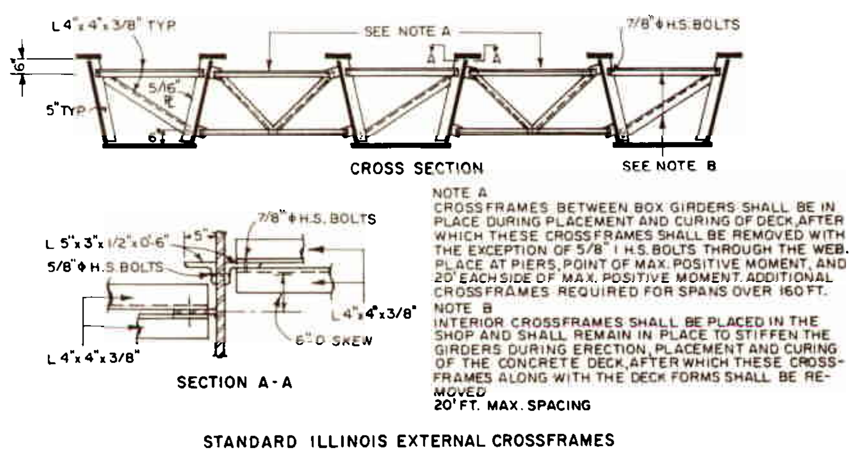


Figure 85

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