

II/2

Highway Structures Economics

Introduction

The Through Truss

Early in the development of highways there were few crossings that required grade separation structures. Roads were generally dirt, and they followed the earth's contour without regard to the extent of either vertical or horizontal visibility. At stream-crossings, approach roads were curved to permit bridge-building perpendicular to the stream. Often the road had to be raised at the end of the bridge so that the bridge could clear the local high water mark. Loads were few and light; roads and bridges were narrow.

For the most part, construction depended on mulepower. Most mechanical equipment was light and could handle only light members. In general, the available heavy steam powered equipment lacked mobility; and any that had

mobility faced a problem of narrow roads. Except for rail, transportation equipment was also light; and construction sites, unless adjacent to railroads, were not readily accessible. Engineers therefore found it necessary to design bridges made up of relatively small, light members easy to transport and erect.

Nearly all early steel highway bridges were through trusses. These were economically satisfactory even for short spans (30 feet). Because roadways were narrow and loadings light, through and pony trusses presented virtually no problems related to heavy floor systems or to efficient use of materials.

A through truss gave maximum clearance over high water, with minimum increase in approach grades. Fabrication and material costs were low; and even in the short spans, dead loads were a relatively large part of the total load. Frequently, higher stresses were allowed for dead loads than for live loads.

Truss members were generally made up of



FIG. 1. EARLY HIGHWAY CONSTRUCTION, GEORGIA

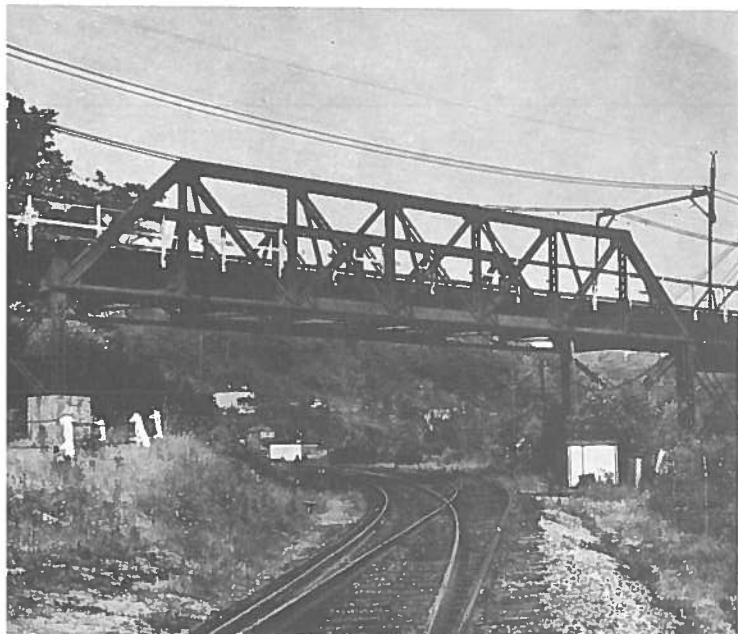


FIG. 2 THROUGH TRUSS, PENNSYLVANIA

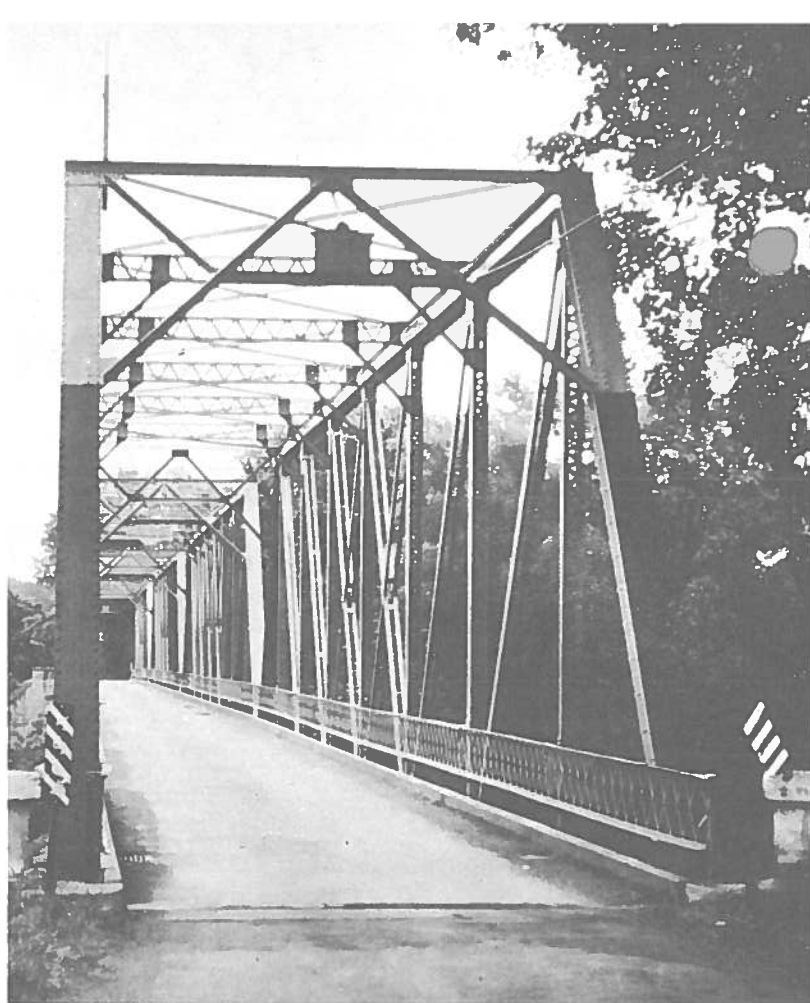


FIG. 3 THROUGH TRUSS, INDIANA



FIG. 4 RIVETED RAILROAD PLATE GIRDER, PENNSYLVANIA

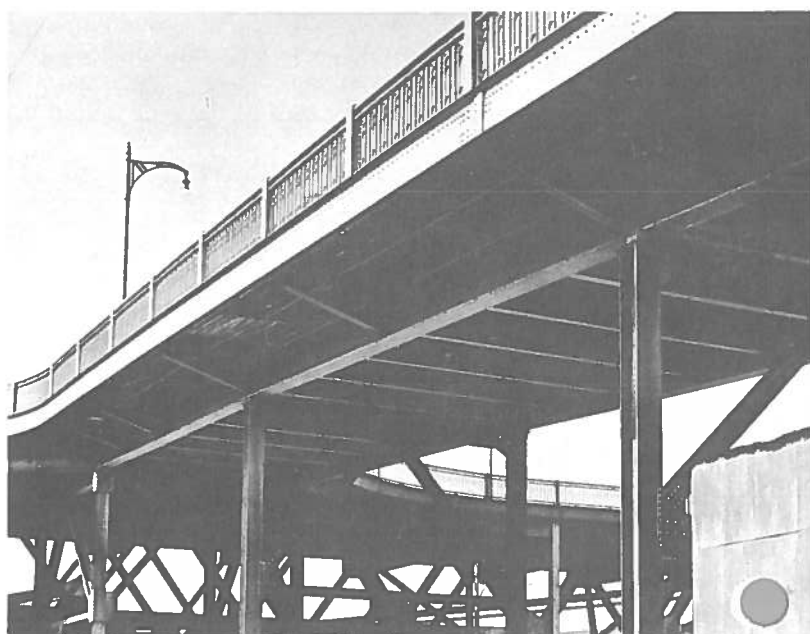


FIG. 5 ROLLED WIDE FLANGE BEAM BRIDGE, PENNSYLVANIA

angles and plates riveted together; or angles with lattice bracing were riveted to form open I-shaped or box-type members. Riveted angle and plate construction made up the floor beams—sometimes as little as 15 feet long. Over these beams, timber stringers were often used—floored with edge-laid timbers spanning the stringers transversely.

The steel in general use had a yield point of about 30,000 psi. High strength steels were available, but their use was restricted to longer spans—usually over 1000 feet. Stringers and floor beams on the longer spans were riveted. Stringers were spanned by steel buckle plate or timber. For the bridges utilizing buckle plate, some of which are still in use, brick or wood block provided the riding surface.

Plate Girders

Railroads gave the first big push to the use of girder-type bridges in place of the early iron bridges and timber trestles. To minimize interruptions of train service, the railroads naturally sought a structural system allowing the shortest possible construction time. They also needed, for short spans, a structure with high shear capacity. Steel plate girders provided the solution for many of their short-span structures.

Girders could be and were completely fabricated at shops having railroad sidings. It was easy to load the girders on flat cars with shop equipment and ship by rail directly to the bridge site. The mobile steam powered railroad cranes (for righting derailed trains) that were already in widespread use had capacity for erecting bridge girders.

Because of railroad requirements for flat grades, elevations of approaches and bridges at many stream crossings were greater than high water clearance. Therefore, both deck girders and through girders were used. Because of stability requirements for nosing and locomotive and train sway, deck girders were sometimes spaced more than track width and needed floor beams and stringers as on through girders and trusses. The floor system when used was made up—as in highway bridges—of riveted floor beams and stringers.

Rolled Beam Bridges

The combined requirements of railroads and highways, augmented by building needs, were recognized by the steel industry. In the 1920's, to help meet these needs at the lowest practical

cost, wide-flange rolled beams large enough for use in bridges were produced. These not only provided ready-made, economic members but were less susceptible to fatigue. At this time, fatigue was usually considered to be taken care of in design by the use of large impact factors.

About the time large rolled beams became available, gasoline power had replaced mule-power. To handle the requirements of the automobile and truck, highways were improved throughout the country. Paving became a commonplace instead of a rarity in highway construction. Bridge sites became more accessible; and mobile construction equipment, transportable over the highways, was developed.

Highway bridge engineers, spurred by the need for wider bridges to handle the new traffic, recognized the advantages of deck-type bridge systems. The availability of rolled beams, decked either with reinforced concrete or timber, provided the economic feasibility for the deck system as a complete superstructure for highway bridges. For the shorter spans, weights and lengths of individual beams were little more than those of some truss members previously used. Further, the rolled beams were little deeper and sometimes less deep than the floor system required for the through truss. This allowed bridge elevations to remain the same.

As transportation, construction techniques, and construction equipment advanced, longer and heavier members were designed for the deck structures. As width requirements continued to increase, through truss construction became relatively more expensive. By the late thirties, rolled beam deck-type highway bridges were in general use for spans up to about 70 feet and were sometimes used for spans up to 90 feet. Use of riveted plate girders increased deck spans to 120 to 150 feet. Simple spans were used almost exclusively; continuous construction was rarely considered. Longer spans remained as through trusses. Over 400 feet or so cantilever construction and occasionally arches were used.

Continuous Construction

Bridge engineers were aware of the efficiencies of continuous construction. Through this method, maximum moments could be concentrated in short distances near the supports, resulting in a sizeable reduction in the relatively flat and long midspan portion of the moment curve; moreover, there could be a desirable reduction in the number of expensive expansion

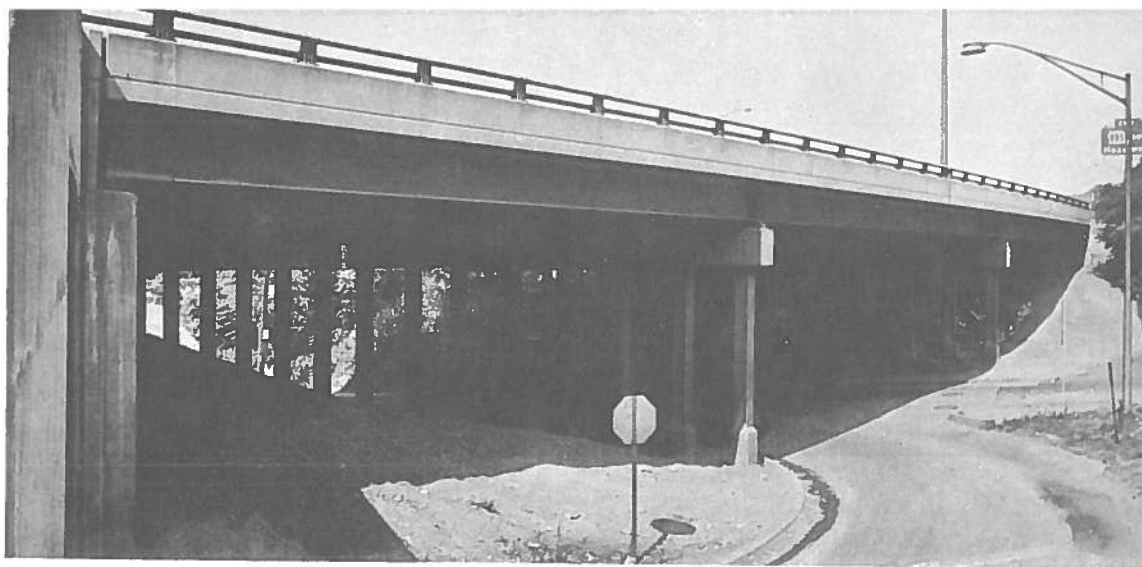


FIG. 6 CONTINUOUS WIDE FLANGE BEAM BRIDGE, PENNSYLVANIA

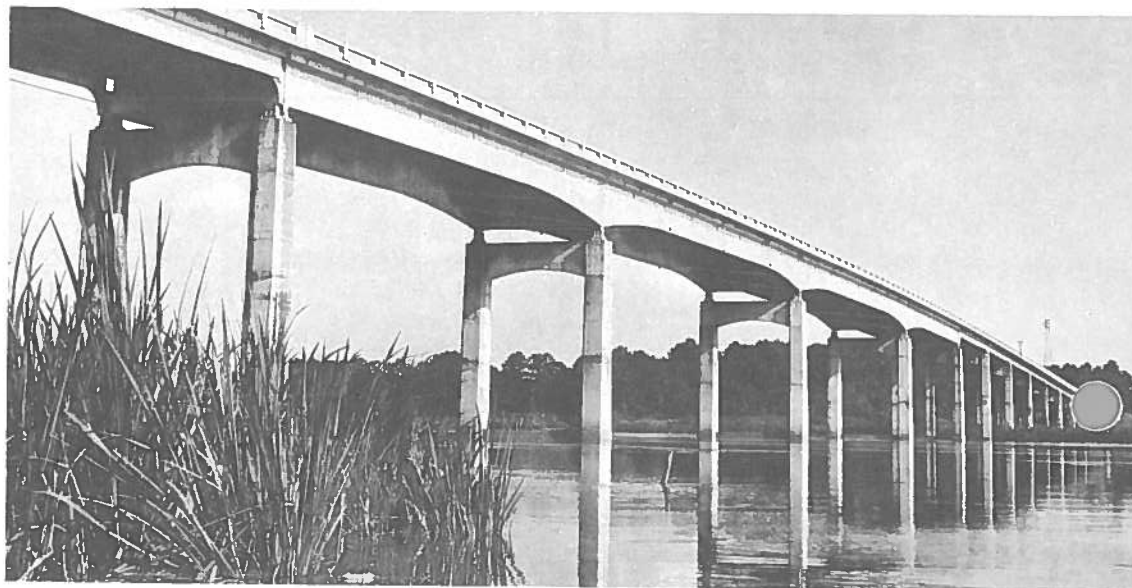


FIG. 7 CONTINUOUS RIVETED PLATE GIRDER BRIDGE WITH FLOOR BEAMS, GEORGIA

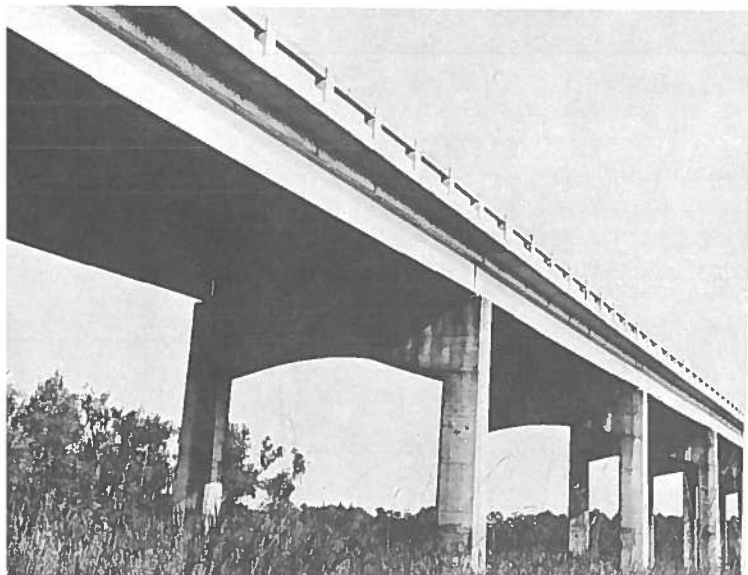


FIG. 8 COMPOSITE WIDE FLANGE BEAM BRIDGE
WITH COVER PLATES, GEORGIA

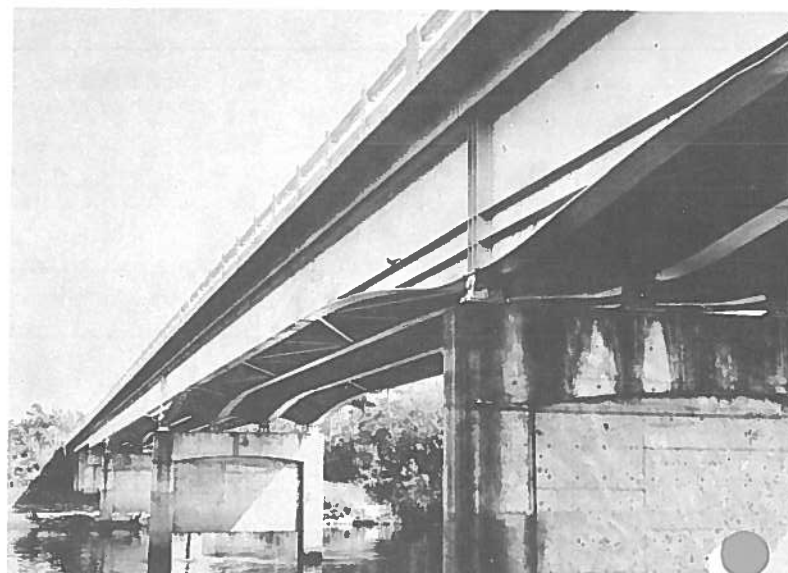


FIG. 9 CONTINUOUS COMPOSITE
WELDED PLATE GIRDER, GEORGIA

joints and bearing assemblies.

At this time classical methods of analysis had not been extensively used on continuous members of variable cross section. Not only was analysis a problem; there was also much concern about possible differential pier settlement and its effect on continuous construction. There was a general lack of highly trained foundation exploration teams; in some areas, soil mechanics was not considered a fully developed science.

To obtain part of the advantage of continuity, cantilevers with suspended spans came into use on plate girder spans. The upper span length of plate girders was extended well above 100 feet, and economies were improved in spans under 100 feet. However, while cantilevers reduced the number of bearing assemblies, they still required the same number of expensive expansion assemblies as did simple spans. Further, hanger and hanger assembly costs were added.

In the late 1930's, moment distribution began to be used as a tool in the analysis of indeterminate structures. This procedure simplified the problem considerably and designers began to investigate more thoroughly the potential of continuous construction. Initial studies were made primarily on bridges founded on rock or on virtually unyielding piles. Then, highway construction was brought almost to a standstill by the Second World War.

After the war, highway construction surged. Under 70 feet, simple spans made of rolled beams were still dominant. From 70 to 90 feet, cantilever spans of rolled beams or riveted plate girders were common, although some continuous construction was being used. Spans over 90 feet and up to 150 feet were riveted plate girders—some simple spans, some cantilever, and some continuous.

In the late 1940's, college graduates entering the field of engineering were versed in the easier design solutions for indeterminate structures, and were eager to put these approaches to work. With this knowledge available increasing numbers of investigations were made on the use of continuous girder bridges.

Nearly all girder bridges were still using riveted construction. Possible extension of deck plate girder span length was being investigated, and some continuous spans exceeding 300 feet were built. With the addition of riveted cover plates, primarily in negative moment regions, continuous rolled beam construction for deck structures was found economical in spans up to 100 feet.

Welding

Post-war highway construction changed radically from prewar construction. Greater emphasis was put on speed of travel, both by individuals and by trucking firms eager to compete with the railroads. It was no longer feasible or desirable to make a road fit a bridge location by putting curves at each end of the bridge. Instead, bridges were designed and constructed to fit the requirement of a total system. Skews and horizontal curves in bridges became usual rather than rare, and bridges over highways became a necessity for speed and safety.

During the war, considerable experience was gained in the use of welding. To conserve materials and speed construction, welded ships were produced at a fast rate. The technology of welding was greatly increased and its application to bridges was realized. In Europe, where materials were still in short supply, it was important to make as efficient use of steel as possible. Welded construction, with savings of about 15 per cent in material, was common for most steel bridges—long or short span. This resulted in more functional structures with less material and, therefore, less dead load. Girder spans could be, and were, extended beyond the previous economic lengths.

Although most of the fabricating shops in this country were geared to riveted construction, designers investigated the possibilities of welded bridges. Extensive use was made of welded cover plates on rolled beam structures and of weldments for bearing assemblies. Since less equipment and less costly setups were required for welding, smaller fabricators found the means to handle larger bridge projects and rapidly gained experience in such work.

Composite Construction

During this period, composite construction—with the concrete deck slab an integral part of the superstructure beams—was finding favor with some designers. New types of connectors (to resist the horizontal shear between concrete deck and steel beam) were developed and marketed by the manufacturers. Through research and sales promotion, they added impetus to composite construction and furthered economies in bridge construction.

Studs, channels and spirals were the primary shear connectors used for composite construction. Besides tying the deck slab to the steel beams and girders, forming an integral unit with

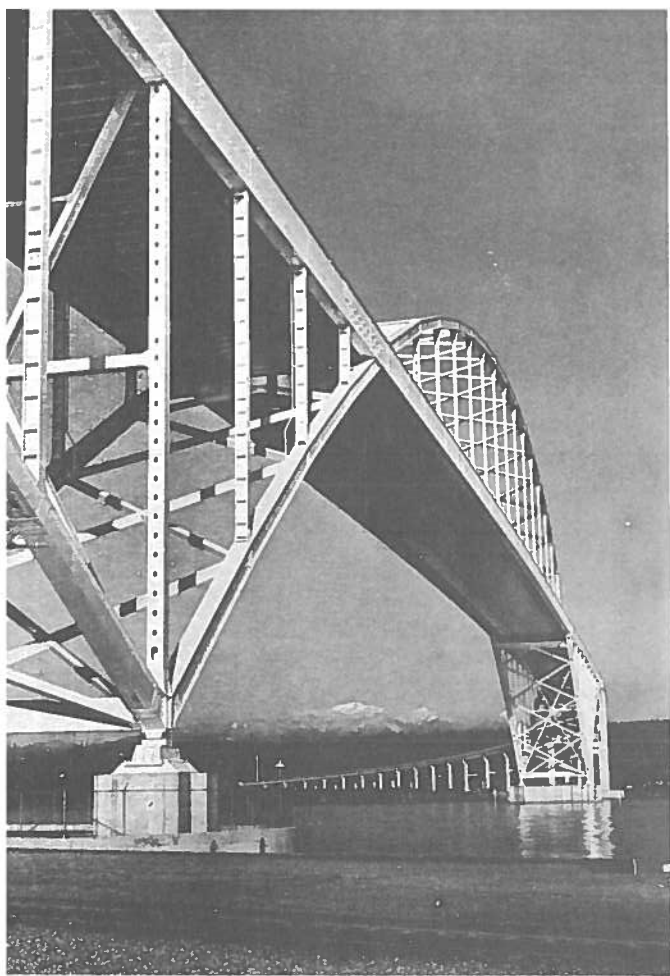


FIG. 10 ORTHOTROPIC PLATE BRIDGE, CANADA (B.C.)



FIG. 11 CONTINUOUS ROLLED BEAM BRIDGE, IOWA



FIG. 12 CONTINUOUS ROLLED BEAM BRIDGE, OHIO

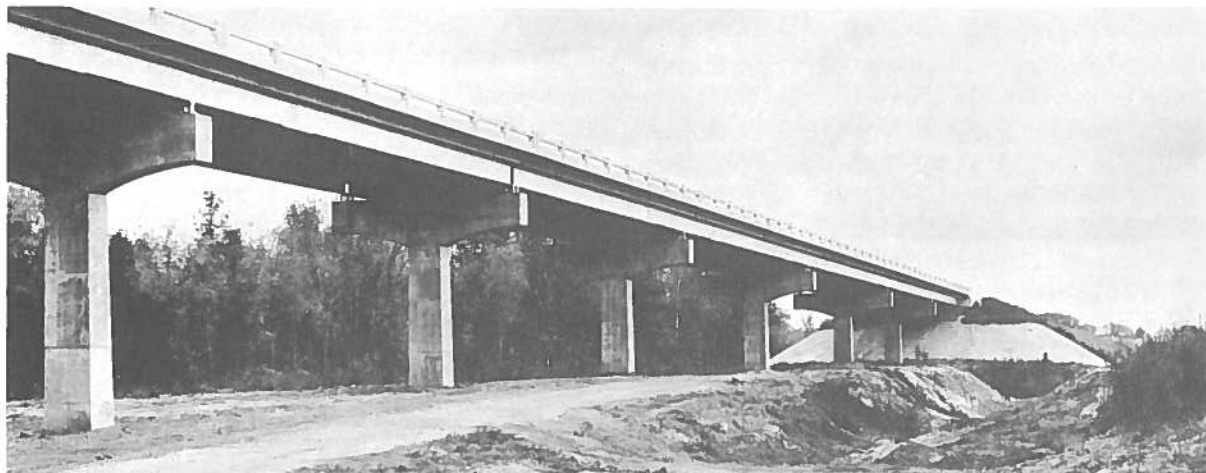


FIG. 13 CONTINUOUS COMPOSITE ROLLED BEAM SPANS, GEORGIA

a large capacity for overloads, composite construction allowed a reduction in the structural steel required for the primary beams. Most of this was reflected in smaller areas for top flanges of beams and girders, although bottom flanges were also somewhat reduced.

The greatest materials saving was apparent in plate girder spans, where top flanges could be reduced to a minimum consistent with erection requirements. Rolled beams were made asymmetrical by the addition of welded cover plates to the bottom flanges only. This not only increased economy in the spans where rolled beams were previously applicable, but also increased their usable length of span.

Composite construction led to a further change in the application of plate girders. For spans 150 feet and over, girders with floor beams and stringers had been considered the most economical construction. However, with the concrete deck supplying a large part of the top flange, economies were available in longer span multi-girder bridges without floor beams and stringers.

With the experience gained on welded cover plates, designers pushed toward all-welded plate girder construction in spans greater than 100 feet. Fabricating shops acquired automatic and semi-automatic welding equipment; and portable radiographic equipment became available for inspecting welds. In a further development of connections, high strength bolts (which make joints less influenced by fatigue) were introduced and began to supplant field riveting on some structures. Except for truss spans, riveted construction declined rapidly, as did the art of riveting itself.

As welding came to the fore, new steels were produced with the toughness needed for welded construction. These steels, with yield points ranging from 33,000 psi up to 100,000 psi, were made available and were quick to catch on. With the reduction in weight made possible by welding and by the higher strength steels, continuous girders began to be used in spans greater than 250 feet.

Labor rates increased rapidly in all industries during the 1950's. Fabricating shops were no exception, and fabricating costs for steel bridges started on a steady climb. Materials costs also increased, but not at a comparable rate. It therefore became more and more important to simplify fabrication wherever possible. There was a further increase in the economic span length

of plate girders requiring more material than trusses. "In line" fabrication methods were exploited wherever practical in bridge design and construction.

By late 1950, except for the larger fabricating shops, few shops were capable of handling riveted construction efficiently. Where riveting had previously been specified, welding or high strength bolts were used. Towers for suspension bridges had been designed and constructed using shop welding, with high strength bolts for field connections; and the new 100,000 psi yield point steels were being used for welded construction of truss members.

Orthotropic Plate

As esthetics in bridge construction received greater consideration, the demand increased for girder type structures in the longer span ranges. In all long spans, dead load plays a dominant role; and methods were sought to reduce this loading wherever possible. Higher strength steels aided in this quest; an additional means of reducing dead loads was to utilize stiffened steel decks having a two-purpose function—both as the top flange of the girder and as the deck.

Appropriate design procedures were devised for the "orthotropic" plate, and girders with stiffened steel deck flanges are currently in place. Some of these structures have spans of more than 700 feet. Probably for all bridges in excess of 400 to 500 feet, trusses still offer maximum economies although they may not be considered as pleasing esthetically.

Advances

Bridge design and construction has now become very sophisticated. Methods of analysis have advanced so that multi-redundant indeterminate structures can be handled with comparative ease. The electronic computer has provided tremendous aid in this direction.

Nearly all state highway departments and bridge consultants have (or have access to) excellent foundation investigating teams. Soil mechanics have improved steadily, and it is now possible to predict future settlement of piers founded on elastic materials. With this knowledge, continuous structures can be designed taking into full account the stresses that may be induced by future pier settlement.

Research facilities have improved, particularly at the university level, and are in considerable demand for work in the highway field. Many re-

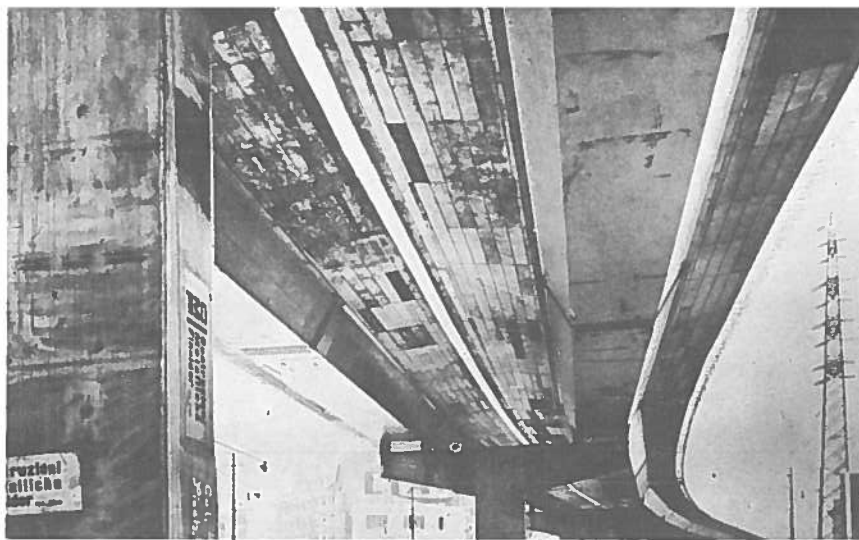


FIG. 14 STEEL BOX GIRDERS, ITALY



FIG. 15 CONTINUOUS PLATE GIRDER SPANS, VIRGINIA

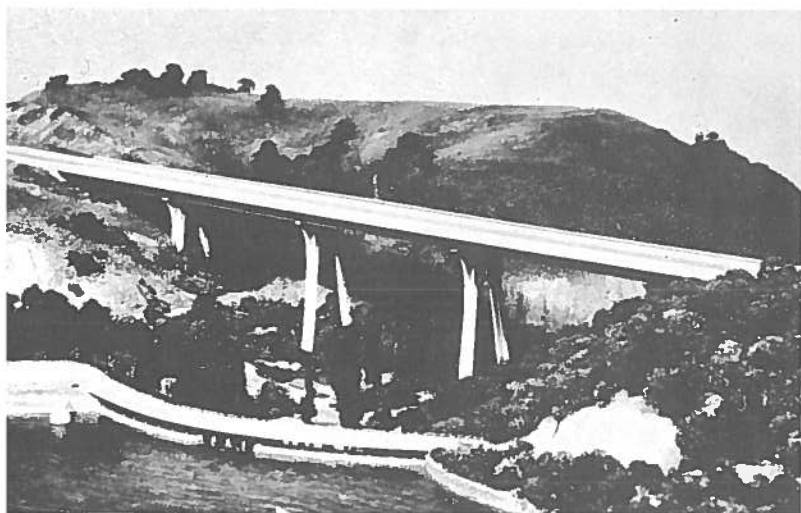
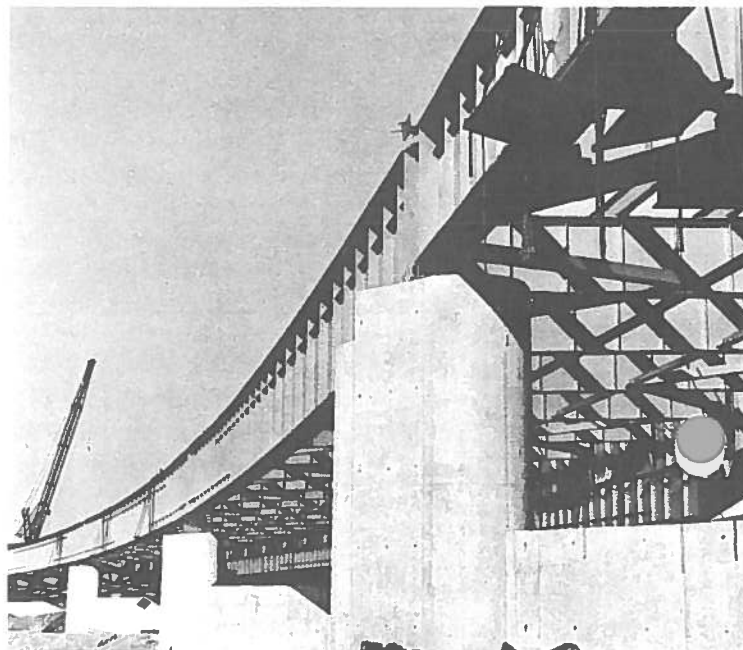


FIG. 16 COMPOSITE PLATE GIRDER BRIDGE,
MAXIMUM SPAN 350 FEET, CALIFORNIA

FIG. 17 CURVED PLATE GIRDER BRIDGE, ILLINOIS



search projects directly related to bridges have led to improved design solutions and, to a large degree, many past unknowns have been clarified. This, in turn, has led to a better knowledge of the true action and capacity of bridge structures.

Construction and fabrication techniques and equipment have kept pace with design. Methods have been devised for continuous structures whereby all splices can be made at the bridge site prior to the erection of beams. Full length beams covering several spans are erected without the need for false bents. In fabrication, semi-automatic and automatic equipment and procedures are in use for welding, punching and drilling. All these have improved the economics of fabrication.

Materials have also improved. Higher strength steels are available for specific as well as for general use. Weldable steels are available in almost all strength ranges applicable to bridges.

All of these factors have been considered and are currently being used to economic advantage. In design, structures are being viewed more integrally, rather than as a combination of related but independent pieces. These structures have greater total capacity and at the same time make more efficient and economic use of materials.

Traffic and its related problems have increased at an almost alarming rate. By means of traffic counts and loadometer readings, much information has been gained on the actual loads and load frequencies that bridges are subjected to. However, while more knowledge has been gained about actual loads, control of these loads has not necessarily kept pace.

In highway design the accent is on speedy movement of traffic. This, plus the related needs of other methods of travel, has brought bridge engineers into closer contact with one another. Mutual assistance in the solution of common problems has made better use of available engineering talent. Various engineering groups outside of highway groups, such as ASCE, have also been active in the interchange of ideas and knowledge. And industry has helped by spreading the best available knowledge of particular materials or products.

However, in many respects, bridges have remained unchanged. The basic objective is the same—to cross a relatively wide expanse usually with a relatively narrow elevated path. The primary design problem is therefore longitudinal. Although trusses and other forms have been supplanted in many spans by beams and girders,

the structures still perform the same function in a similar manner. Hence, the great strides in bridge engineering have been in better representations of the same systems.

The resulting structures are much more functional and at the same time, have gained in aesthetic values. Spans once considered economically and technically impractical are now designed as a matter of everyday engineering practice.

Short Span Bridges

General

In this discussion of economy, short span bridges will be considered those for which rolled beams (in areas where they are available) are generally applicable. For simple spans, this definition provides for a maximum of about 80 feet. For continuous construction, the maximum intermediate span is about 110 feet, and a maximum end span is about 90 feet.

Deck structures utilizing rolled beams are now in vogue because they are the most economical type of construction for these spans. However, in some areas where fabricating costs are relatively low, there is use of welded plate girders rather than rolled beams in both simple and continuous spans over 80 feet.

Composite construction, which received serious consideration in the 1950's, is now in general use and is applicable for spans over 40 feet. The minimum span for continuous composite construction is about 60 feet. For shorter spans (less than 40 to 60 feet), where composite action is not advantageous, fabrication should be kept at a minimum; therefore beams not requiring cover plates are generally selected.

Beam Spacing

In all short span deck bridges, it will be economical to use designs that do not require floor beams or excessive deck thicknesses but do require a minimum practical number of longitudinal beams. The validity of the latter requirement can be shown by considering the properties of available wide flange rolled beams. The section modulus of WF beams can be expressed as a constant times the product of the area and depth of the beam. Published data indicate that the

constant is equal to about $\frac{1}{3}$ for nearly all WF beams used in bridges. Hence, four beams having the same depth and total area as five beams will have the same total capacity. Since the total area is constant each of the four beams must have a greater area than each of the five beams, the greater individual beam area will nearly always allow selection of beams with greater depth and consequently greater total capacity.

To achieve maximum economy, the typical 28' roadway bridge with a total width of 33' to 34' should have four beams at a spacing of about 8'-6". This provides a distance of 25'-6" out to out of beams, and requires deck cantilevers of 3'-9" to 4'-3" for the remaining roadway plus safety curbs and handrails. With this spacing and a design concrete strength of 4,000 psi (the minimum suggested for durability), a 7" deck will be necessary. Since this is the minimum deck depth allowed in many states, the deck will be used at its maximum efficiency. The 4' \pm deck cantilever balances the deck moments in the 8'-6" distance between beams, and also produces nearly equal longitudinal moments in the interior and exterior beams.

As a general guide for all spans and bridge widths, a spacing of 7'-0" to 8'-6" for rolled beams should be used wherever practical. However, at the upper limit of the rolled beam spans, it may be desirable to decrease the beam spacing and increase the number of beams. For the same typical 28' roadway bridge, better results will be obtained with five beams at a spacing of about 6'-6".

There are, of course, exceptions to these general guides to transverse spacing of wide flange beams. Occasionally, when it is not possible to obtain an efficient section for the beams when the suggested minimum number is used, economy may be found by adding an extra beam. Some economies may also be available in certain short span designs—such as the T-Beam, or those which use widely spaced WF beams and a two way deck slab—that deviate from the above general guides.

Spans to 40'

At the lower end of short span bridges non-composite beams of A36 steel are used almost exclusively. Some gain in efficiency of materials can be achieved even in these spans by the use of composite construction. However, the actual material savings in pounds is small, and the corresponding cost savings is more than offset

by fabricating costs.

Beam depths required will be between 21" and 30", so that there is ample latitude for selection of efficient sections. Since the total weight of the steel will be small, each fabrication item will be reflected as a relatively large percentage of the materials cost; simplicity in details is therefore especially desirable.

By AASHTO specifications, diaphragms or cross frames are required at intervals not to exceed 25'. Since, in these short spans, the stiffness of the concrete deck is large compared to the stiffness of the diaphragms, the latter serve primarily as beam spacers and should be as light as practical. Lightweight channels (depth $\frac{1}{3}$ that of the beams) will be ample, and should be field connected to the beams by bolting or welding to plates that have been either shop or field welded to the beam web. For spans over 25' and up to 50', one such diaphragm will be required at the center line of the span.

Over intermediate supports of continuous bridges, channel diaphragms similar to those at the span center line are satisfactory. At expansion joints, which occur at the ends of simple spans and at the ends of continuous units, the end of the concrete deck should be protected by increasing the deck thickness—this provides a concrete edge beam. Slotted holes should be called for in the ends of the steel beams, to allow placement of reinforcing steel required for the edge beam.

Continuity should always be considered for these spans. Savings will be obtained primarily through reduction in number of bearing assemblies and expansion joints. For example, a four span continuous bridge will require five bearing assemblies per beam, whereas four simple spans will require eight bearing assemblies. Cover plates should not be used, since the cost of fabrication will more than offset the material savings.

An end to intermediate span ratio of 1 to $1\frac{1}{3}$ will provide near-balanced continuous design—i.e. negative moments at all intermediate supports will be nearly the same and positive moments in each span will also be nearly the same. Since the negative moments will be greater than the positive moments, the design will be for the negative moments. Necessary splices should be shown as optionally welded or bolted.

A design for short spans that deviates from the general guides is one making use of widely spaced longitudinal beams, transverse beams

and a two-way deck. Beam spacing on constructed spans has varied from 12' to 14'. Transverse beams, forming the additional supports required for the 2-way slab, have been spaced from 15' to 20'. The advantages of this design include reduced requirements for the deck, more efficient use of longitudinal beams, and reduced number of bearing assemblies. The chief disadvantage is the additional fabrication required for transverse beam connections. Bearings should be either flat steel plates or elastomeric pads. The flat steel plates will be inexpensive since cutting of curved surfaces is not required. Both steel plates and elastomeric pads should bear on thin sole plates welded to the beam flange. On continuous spans, sole plates at intermediate supports should be wider than the beam flange. Round or slotted holes for anchor bolts can then be placed in the extension beyond the beam flange, so that the full beam section can be utilized for moment resistance.

Spans 40' to 110'

Rolled wide flange beams and welded plate girders have been used economically in spans from 40' to 110'. When the general beam spacing guides can be followed, wide flange beams will usually provide the greater economy. Spans in the upper limit of this range may require an excessive number of wide flange beams. When six or more beams (or beams heavier than 170 pounds) are necessary, welded plate girders will frequently be more economical.

Wide flange rolled beams can be used for simple spans up to about 90'—with 80' as about the economic limit. There are indications that ASTM A441 steel will provide some economies where deflections are not a controlling design criteria. In all cases, simple spans should be investigated as composite beams.

Partial length welded cover plates added to the bottom flange will improve the economies of composite construction. It is preferable to use a single unspliced plate of constant width and thickness. The width should be less than that of the beam flange and the ends should be cut square, so that the attaching fillet welds can be continuous around the plate perimeter. This arrangement provides the best resistance to fatigue (by eliminating the danger of undercuts at corners), and also simplifies welding procedures.

Composite welded plate girders should be investigated for simple spans over 80'. Girders of ASTM A 36 and A 441 steels will cost about

the same. Spacing should be much the same as for rolled beams except on wider structures, where it is better to increase the spacing of the welded girders if a change from the 8'-6" figure is necessary. The reason for this is the increase in efficiency that can be obtained with an increase in depth for the same total beam area.

A minimum top flange, consistent with erection requirements, will usually satisfy design stresses. Top flanges will rarely exceed 12" in width and $\frac{3}{4}$ " in thickness and will often be only $\frac{1}{2}$ " thick. The 12" width provides ample space for placing shear connectors, such as studs, in groups of sufficient numbers to allow a longitudinal spacing that will not interfere with the deck reinforcing steel. As an aid to deck construction, the connectors can be spaced longitudinally in multiples of one half the deck steel spacing. This prevents interference of the connector with the deck steel and also provides a guide for deck construction.

The bottom flange generally should be made from three plates of two sizes: a center plate covering approximately the mid 60% of the span, and two end plates butt welded to the center plate. Plate sizes correspond to the requirements of the moment curve. Splice costs will usually offset any material savings available through increasing the number of changes in flange plate size beyond the two that are recommended.

Frequently, $\frac{7}{16}$ " unstiffened or $\frac{5}{16}$ " stiffened web plates will satisfy the shear and buckling requirements for welded plate girders in spans under 100'. While the $\frac{5}{16}$ " stiffened plate has less material, the final cost after fabrication will usually be less with the $\frac{7}{16}$ " web. If unstiffened web plates thicker than $\frac{7}{16}$ " are required, it will be economical to use thinner webs with transverse stiffeners. For maximum savings, the stiffeners should be welded.

Non-composite construction is economical for continuous spans having end spans less than 50' and intermediate spans less than 65'. In these spans, it will be advantageous to use beams requiring cover plates in the regions of negative moment. Only one cover plate should be used on each flange; plates should be of constant size, with a width less than the beam flanges. Beams requiring positive moment cover plates usually will not offer economies. Other fabrication should preferably be restricted to connections for angle or channel diaphragms and to beam end preparation for field splices.

It is desirable to locate field splices at or near points of dead load contraflexure. Designing and detailing the splices for both welding and bolting will permit the contractor to select the method suitable for his equipment. In these spans, splices at each point of contraflexure may not be necessary. Here, too, a design option will allow the contractor and fabricator freedom to select the cheapest construction method.

Composite construction should be considered for continuous spans greater than 65'. Usually WF beams can be selected requiring partial length cover plates in both positive and negative moment regions. In the positive moment region, a cover plate on the bottom flange alone will be effectively balanced by the concrete deck serving as top flange cover plate.

Studies to date indicate that costs are about the same for composite construction used only in the positive moment regions as for composite construction over the total continuous span. Where composite construction is used in the negative moment region, top flange cover plates will be smaller than those on the bottom flange. Longitudinal reinforcing steel in the concrete deck provides the additional resistance needed for live load moments. Although the cost of reinforcing steel per pound is less than that of structural steel, this usually will be offset by the cost of the additional connectors.

In areas of relatively low cost fabrication, welded plate girders may be less expensive than wide flange beams for continuous spans in excess of about 80'. The girder in the positive moment regions, should have the same characteristics as those of a simple span composite welded girder. Symmetrical girders are necessary in non-composite negative moment regions. Where composite construction is utilized in negative moment regions, the top flange plate will be smaller than the bottom flange plate. In either case, flange plates required at the point of maximum moment (over the support) should be continued for a short distance of about $\frac{1}{4}$ the span into end spans, and $\frac{1}{10}$ the span into intermediate spans. A reduction in flange plate size should be made at these points. No other reduction should be made in the negative moment region.

Maintaining composite construction over the total span is advantageous in satisfying fatigue requirements at the dead load points of contraflexure. At these points in the partially composite girder, fatigue may govern plate sizes; this will

not ordinarily be true in the totally composite structure.

An empirical factor of $S/5.5$ is authorized in the AASHO specifications to determine the number of wheel loads for which each beam is designed. This factor is based on studies confirming the ability of the concrete deck to distribute live loads transversely. No credit is given in this factor for additional distribution effects resulting from the use of heavy diaphragms. Although the empirical factor is authorized, the use of a more sophisticated procedure such as grid analysis is not prohibited.

When designs are made on the basis of the empirical factor, diaphragms should be of minimum size. Three angles, one placed horizontally near the bottom flange and two placed diagonally, will usually be adequate and will result in a minimum amount of material. The angles can be connected by welding or bolting to vertical plates welded to the beam webs. At the ends of simple spans and at the ends of continuous units, diaphragms must also serve to support the end of the concrete deck—the recommended support is a channel of minimum weight consistent with the edge beam requirements. Connection to the beam web should be similar to that for the cross frames.

In some areas elastomeric pads have recently replaced the more expensive fabricated bearing assemblies. While these pads have been used primarily in concrete construction, they can also be economical in steel construction. For spans under 80', a single-thickness pad will allow the necessary movement for expansion and contraction; laminated pads will be required for movement in excess of that anticipated for an 80' span.

It is common in simple span construction for each span to have one fixed end and one expansion end. On many of these structures, armored expansion joints are used over each pier. In addition to being expensive, they are rarely leak-proof. Extruded or preformed plastics give promise of providing a better and less expensive joint requiring less maintenance. These are applicable not only for simple span joints but also for joints at the ends of continuous units.

Precambering of rolled beams is expensive and unnecessary for the typical bridge, since some inaccuracies will always exist for which provision must be made. This is usually done by means of a variable coping depth over the beams. The same method can be used in obtain-

ing the correct deck thickness and final deck elevation for the uncambered beam. The minor additional coping concrete required will cost less than the precambering.

The subject of deflections is of considerable importance, especially where wide flange beams of higher strength steel are used. Wide flange beams of A441 steel are economical in the 80' span range, provided the design is governed by stresses and not by deflections. The AASHTO specification requires that live load deflection be limited to $1/800$ of the spans; this will often be difficult to meet with wide flange beams of steels other than A36, unless composite construction is utilized. When both A441 steel and composite construction are used in the same structure, the economies are additive, producing an attractive cost picture.

The composite wide flange rolled beam example in Chapter 4 illustrates some of the economic considerations previously discussed. For this two span continuous structure, the requirements for negative moments are satisfied almost precisely by a 36 WF 135 pounds, with 10" x 1½" top and bottom cover plates in the negative moment region. The beam is the minimum weight in its section depth, and requires cover plates of maximum thickness allowed by AASHTO. This results in a negative moment section having maximum economies.

Unfortunately, the positive moment section is not as economical as the negative moment section, because—if constant depth is maintained—the minimum weight section satisfying the requirements of negative moment must necessarily be used for the full length of the structure. Had the negative moment section required the next larger section (36 WF 150) the 135 pound beam could still have been used in the positive moment region. A larger cover plate would have been necessary than the 10" x ¾" one shown in the example and the economies of the positive moment section would have increased.

The box girder design example shown in Chapter 7 illustrates another type of construction and its related economies. The design of this example, as noted in the discussion and calculations, was made for transverse distribution of the live load by the composite concrete deck only. The addition of suitable transverse cross frames would have improved this distribution.

The main advantage of the box girder over the plate girder lies in its torsional rigidity, which improves the transverse live load distribution

and reduces the total load for which the structure must be designed. This, in turn, reduces the amount of steel necessary to resist the forces and loads to which the structure is subjected.

Quantities and costs for simple span and continuous span box girders have been compared with quantities and costs for the latest wide flange beam design suggested by the Bureau of Public Roads for the same span. The assigned unit prices reflected those to be anticipated in two different areas of the country. Judging from these comparisons, the simple span box girder appears to be about 10% cheaper than the wide flange beam design, and the continuous box girder about 10% cheaper than the simple span box girder. The continuity savings will increase for units of more than two spans.

Medium Span Bridges

General

This discussion will consider medium span bridges to be those with spans from 110' to 350'. Except where absolute minimum clearances are required, welded deck plate girders are generally applicable. For spans above 150', absolute minimum clearances can usually be provided by utilizing through trusses. Since these are special applications, they will not be discussed here.

All medium span bridges containing two or more spans should be designed and constructed as either continuous or cantilever bridges. Continuous construction is preferred, since it makes more efficient use of materials and provides a greater reduction in number of expansion joints. Continuous construction also obviates the expensive fabrication required at cantilever joints.

Composite construction has been used to economic advantage for all spans in medium span bridges. This is true both for construction similar to rolled beam design (multiple girders) and for construction with girders, floor beams and stringers.

Where floor beams and stringers are not used on two-lane structures, girder spacing should be about the same as for rolled beam bridges. For structures with more than two lanes, girder spacing should increase with increasing span length. Girders for floor beam and stringer construction should be spaced inside the curb faces, so that

cantilever deck moments will closely equal deck moments between and over the stringers.

In general, medium span girders should use webs with transverse stiffeners. Longitudinal web stiffeners should also be used for girders in the upper part of the medium span range.

Spans 110' to 175'

Composite construction is very suitable for these spans. In two-lane bridges, of either composite or non-composite construction, maximum economies will be obtained with four girders. For bridges with more than two lanes, the best solution will be provided by five or more girders at 7' to 9' spacing.

Studies made for girders in this span range indicate that ASTM A36 and A441 steels can be used at about the same cost. Where A441 steel is utilized, webs should be made as thin as practical using both transverse and longitudinal stiffeners. Girders utilizing A36 steel may show some economies with longitudinal stiffeners; transverse stiffeners are necessary in any case.

It is generally more economical to build plate girders in the medium span range with straight soffits. In such girders, A441 steel should be utilized in the negative moment region while A36 steel should be used for positive moments. Haunched girders, while providing greater efficiency in use of materials, are more expensive in fabrication. The increased cost of haunched fabrication will usually not offset the material savings.

In the regions of negative moment, no more than three different flange plate sizes should be shown, requiring two butt welded splices. In the regions of positive moment, three plate sizes requiring four butt splices may also be desirable. Because of cost variations, showing flange plate splices as optional will give the fabricator the opportunity to select the cheapest method for his shop.

Bracing should consist of cross frames and a lower lateral system. Cross frames generally require less material and avoid fabrication and erection problems if they are composed of two horizontal angles (one at each flange level) and two diagonal angles forming a cross between girders. Cross frames spaced as near 25' as practical should go between all girders.

The lower lateral system should also be made up of angles, resembling the cross frames in geometric form. The lower horizontal member of the cross frame also serves as a part of the lateral

system. Lateral bracing should connect pairs of girders and should be used in alternate bays only.

Bearing assemblies for these spans are usually rockers or rollers. Prior to 1935, these were almost always castings. However, welding has made it feasible to fabricate these assemblies from plates at much lower cost. Some laminated elastomeric pads have also been used as bearings for these spans. They appear to be even less expensive than weldments.

Twin box girders (Chapter 7) are expected to show maximum economy in spans from 110' to 175'. The economies indicated in short spans should improve with increased span lengths. Transverse live load distribution characteristics, which are considerably better than those of the usual plate girder, will remain about the same as for the shorter spans. More efficient use of the wide bottom flange will be an added advantage of the longer span box girder. Although diaphragms or cross frames are not shown in the example, their use is recommended. When used, they should be considered in the design and account taken of their additional aid in transverse live load distribution.

In addition to relatively low cost, box girders provide a structure with low maintenance. Depth to span ratios are minimal; this, together with the clean straight lines, makes for a functional structure with high esthetic appeal.

Spans 175' to 350'

As in spans under 175', continuous construction should be used wherever possible. Plate girders utilizing floor beam and stringers may show economies for non-composite construction. For composite construction, the multi-girder system (similar to that for rolled beams) has been applied to spans up to 350'. In these spans, the girders should be spaced about 14' on centers. This tends to limit the multi-girder system to roadway widths in excess of about 40'. For two lane structures, two composite girders should be used with floor beams and stringers. Spacing of girders should be as given in previous discussion.

With span lengths above 175', use of the higher strength steels becomes more economical. They have a particular advantage in the regions of negative moment where A36 steel flange plates may have excessive thickness. By present specifications, webs must be of the same material as flanges. Therefore, where A441 or A514 steels are used in the flanges they must also be used in the webs.

Girders utilizing the higher strength steels should have stiffeners, bracing and connection details of A36 steel. Higher strength steels for these items will reduce neither thicknesses nor widths.

Estimates made for each structure will be necessary to determine the relative economies of haunched girders vs. girders with straight soffits. An advantage of straight soffits for these spans lies in obtaining web depths that may not require horizontal splices. The additional cost of these splices plus the additional cost of haunching may offset the material savings to be gained through use of a haunch.

Lateral bracing, cross frames and bearing assemblies should be the same as those suggested for spans between 110' and 175'. Obviously, larger bearing assemblies are necessary since reactions are greater. However, welded bearing assemblies will still provide the least costly solution. Elastomeric pads, laminated or otherwise, are generally not considered applicable for these spans.

In selecting flange plates, it will usually be advantageous to use the maximum practical width. This is particularly true for the higher strength steels, where the lower yield point of thicker steel requires a corresponding reduction in allowable design stresses. Maximum width of plate will of course result in minimum thickness, maximum yield point and maximum allowable stress.

Box girder bridges, utilizing a single box subdivided into cells about 12' wide, have indicated economies for spans greater than 175'. A variation of this, using two relatively narrow box girders, has also shown promise. This type of structure is very similar to regular plate girder construction, in that floor beams and stringers are used to span the space between the boxes. Usually the two box girders are spaced so that they are near the roadway edges but require deck slab cantilevers, thus making efficient use of the deck thickness required to span the stringers.

Spans Greater Than 350'

Each bridge in this span range should be considered a special study and thoroughly examined before deciding which of the various possible solutions is most applicable to the specific structure. Orthotropic plate girder bridges, box girders, trusses, and arches have all been used to advantage. The final choice depends on spe-

cific bridge site i.e., vertical clearance, foundations, number of traffic lanes, etc. Continuous or cantilever trusses probably offer the most economical solution for spans in excess of 500' but less than suspension bridge range.

Orthotropic plate girder bridges are considered more aesthetically pleasing than trusses, and have been used for spans up to about 750'. However, the economy of this construction for spans above 600' is doubtful. The girders supporting the steel deck may be either of the conventional type or they may be box sections. In any event, continuous or cantilever construction is a must.

For all of these structures welding has proven to be advantageous. On trusses it has been primarily in shop fabrication for the truss members. Field connections for the truss members are generally made with high strength bolts. On the orthotropic plate girder bridge, welding is almost a necessity—fabrication of the stiffened deck and attachment of this deck to the girder webs could hardly be accomplished by other means. Field splices of the girder sections are usually made with high strength bolts.

Other Considerations Continuity

The primary reason for using continuous construction is to obtain the economies it offers over simple and cantilever spans. Although not considered a design feature, the continuous structure also has greater overload capacity. This is true because maximum positive and maximum negative moments occur with the loads at different locations. Thus, yielding at a given location under an overload redistribute the moments and does not mean that failure is imminent.

The maximum dead load moment in continuous spans occurs at the supports. For interior spans, this moment is about $\frac{2}{3}$ that of the maximum dead load moment of a simple span that has the same length. The moment decreases rapidly and equals zero at about $\frac{1}{4}$ the span length away from the support. At the center of an interior span the dead load moment is about $\frac{1}{3}$ that of a corresponding simple span.

The envelope of maximum live load moments for continuous spans will show near equal moments at the span ends and at the center. For an interior span, these moments are about 60% of the maximum live load moments for the equal length simple span. The combined live and dead load continuous beam moments are greater

at the supports than at the center of the span. Since both the live and dead load support moments reduce rapidly, the maximum beam or girder section required is not only less than that of a simple span, but also occurs over a shorter distance.

Two span continuous structures are not as efficient as those containing more than two spans. This is true because each span in a two span structure has one pin rod end and therefore does not realize the full advantage of continuity. Conversely, the intermediate spans of a three or more span unit has a degree of fixity at each end.

The most advantageous ratio of interior to end spans in continuous construction varies with the ratio of dead to live loads. As the dead to live load ratio increases, the most efficient ratio of interior to end span approaches 1.22. For small dead to live load ratios, as will occur in spans less than 60', interior spans should have a length about $\frac{1}{3}$ greater than end spans. For spans from 60' to about 110', the ratio should decrease to about 1.3. For spans greater than 100', a ratio of 1.3 to 1.25 will provide balanced designs. However, these ratios are more important for wide flange beam spans. Since in plate girder construction the resistance to moment at any section can be changed readily by increasing or decreasing flange plates, minor deviations from the suggested ratios will have negligible effect on overall economy.

Curved Girders

In the past, bridges constructed on horizontal curves made use of straight beams or girders placed on chords between the supporting piers. Curved steel girders were rarely considered since there was little readily available design information. Recent publications, including Chapter 12, Volume I of this Handbook, have begun to fill this void. Curved girders have been designed and are now in service.

The curved girder has greatly improved the aesthetics of bridges on horizontal curves. In addition it allows continuous construction in much the same manner as for straight girders on tangent bridges. Form work and the placing of deck reinforcing steel are also simplified.

The primary difference between the curved girder bridge and the straight girder bridge of the same span length arises in the outermost and innermost girders. Because of the torsional

stresses resulting from curvature, additional loads are induced in both these girders. These loads increase moments in the outer girder and decrease moments in the inner girder. Depending on the radius of curvature, the change in moments will be from 10 to 30%. Although the increased load on the outside girder is about equal to the decreased load on the inside girder, additional steel will be required in the curved* girder bridge. The amount of this increase will usually be on the order of about 5% to 10%.

Girder spacing for these bridges should be the same as that for tangent bridges having the same span. Composite construction and continuity should be used wherever practical. Bracing details will vary somewhat. Recommendations have been made that cross frames or diaphragms be placed closer together than the spacing normally used on tangent bridges. This is intended to offset the effect of radial loads horizontal to the flanges.

Cross frames and diaphragms act as supports in resisting the radial loads. Therefore, the closer the transverse bracing is spaced the less the effect of the radial components. However, even when cross frames are spaced at more than 20', welded girder bottom flange plates of the greatest practical width consistent with the thickness will usually prevent these radial stresses from exceeding 5% of the allowable design stress. In the top flange the radial components need be considered only for dead load stresses of the girder, provided the concrete deck is so detailed and constructed that it can serve as the top lateral system.

Although fabrication and erection costs for curved girders will be greater than those for straight girders, the advantages previously mentioned may offset these cost increases.

Steels

Most steels used in bridges have ASTM Specifications that establish minimum properties regardless of the supplier. The steels primarily used in bridge construction are ASTM A36, A572 Grade 50, A588 and A514. These steels are all weldable steels and have minimum yield points of 36,000 psi (A36), 50,000 psi (A572 G50), 50,000 psi (A588) and 90,000 psi to 100,000 psi (A514). A588 and A514 grades can be used in unpainted bridges where the enhanced atmospheric corrosion resistance of these steels is desired to help minimize maintenance costs.

Each of these steels has definite applications in bridge construction; some of these applications have been mentioned earlier in this chapter. However, it is difficult to know which steel will furnish greatest economies without an evaluation for the specific structure. Since total fabrication costs for most plate girder spans will be about the same regardless of the steels used, mill prices of the specific steels in the sizes desired will be a helpful guide in determining their economy for a specific structure. In general, as span lengths increase steels with greater yield strengths should be used.

All Steel Superstructures

Chapter 9 illustrates two examples of designs for all steel construction. They are for the most part designed in accordance with the AASHTO specifications. Each of these examples makes use of construction similar to that used for decks on orthotropic plate girders. In Design 1, the deck forms the total superstructure; in Design 2, welded longitudinal beams and transverse wide flange beams support the stiffened steel deck.

The weight of these structures, for a 50' span, varies from about 35 to 45 pounds per square foot. This steel weight is considerably more than that of a wide flange beam span of the same length. However, these weights include the steel deck; with fabrication set up for mass production, costs for this type of structure could be satisfactory.

The two biggest advantages of all steel superstructures are:

1. The rapidity with which they can be placed and opened to traffic.
2. They require neither form work nor false work, allowing complete freedom during construction for traffic to move underneath.

In addition, these superstructures can easily and readily be widened when future traffic volumes make this desirable. The use of corrosion resistant steels in such structures should obviate maintenance.

Future Bridges

One of the most promising economies for the immediate future is in the use of steels of different strengths in the same beam cross section. In present designs, when high strength steel is used in the webs. The thickness of these webs is governed by buckling and therefore the strength properties can not be fully utilized. Since the unit price of steels generally increases with strength, economies will obviously result from the use of high strength steel flanges in combination with lower strength steel webs.

Girders of this type are commonly referred to as "hybrid" girders. Present data indicate that steels with yield points of 36,000 psi can be used as webs with girder flanges of 50,000 psi yield point steel. Girders with flanges of 100,000 psi yield point steel will probably require webs with steels having a 50,000 psi yield point.

Considerable work is being done to prepare suggested design guides for the hybrid girder. These recommendations should be forthcoming in the very near future. Substantiation will be provided so that designers can interpret the information to suit their particular needs.

It is well known that composite girders have toughness and safety in excess of that of non-composite girders. This has not yet been recognized in bridge design specifications, partly because there has been insufficient published information on composite construction as it relates to bridges.

Design procedures allowing different factors of safety for dead and live loads are also to be anticipated. In many respects this is a return to the past, as noted in the introduction to this chapter. However, the application of these factors of safety will differ from the past. The newer procedures will give designers more information on the actual capacity of beams for overloads, yield loads and failure loads.

