# COMPARATIVE DESIGN OF THREE DIFFERENT TYPES OF INTERIOR STRINGERS FOR A SHORT SPAN SIMPLY SUPPORTED STEEL HIGHWAY BRIDGE

by

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TABLE OF CONTENTS

# I. INTRODUCTION

## I-1. Problem

The main function of a bridge is to carry vehicular or other traffic over a crossing, which may be a river, a canyon, or another line of traffic. Today more than 6,000 bridges of all types are built each year in the United States *(3). A relatively small proportion of all bridges built are of long span. Most have spans of 80 feet and below. For this reason, the design of a short span highway bridge becomes an increasingly important problem to structural engineers.

## I-2. Purpose

The purpose of this report is to demonstrate by examples the design of an interior stringer for three different types of short span steel highway bridges, namely, the beam bridge, the composite bridge, and the plate girder bridge and to compare the resulting members on the basis of their weight and overall depth.

## I-3. Scope

The basic concepts, AASHO specification requirements, and design procedures for these three different types of

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* Numbers in parentheses refer to corresponding items in the References.

bridges will be discussed in the following pages. An interior stringer of a bridge which has a span of 70 feet will be designed for the following conditions and the results compared using these three bridge types.

| | |
|---|---|
| Length.................................... | 71 ft 0 in. |
| Distance center to center of bearings.......... | 70 ft 0 in. |
| Clear width.................................... | 26 ft 0 in. |
| Live loading.................................... | H20-S16-44. |
| Wearing surface.................................. | 15 psf. |
| Concrete strength, $f'_c$........................... | 3,000psi. |
| M183 (A36) steel, $F_y$............................ | 36,000psi. |

6" slab.

6' - 6" center to center of stringers.

## I-4. Historical Review

The bridge was one of man's earliest requirements and, subsequently, one of his earliest inventions. It has been said that man knew how to build bridges before he knew how to build houses. The Chinese, who used both slab and arch construction, were either the earliest bridge builders or contemporary with the Babylonians. Records of ancient bridges in China have not been preserved, but masonry arches are known to have existed 2,000 B.C. (橋), Chinese character for a bridge, was first used about 1,000 B.C. (12 & 13). However, the 3,000-year-old, 40 foot, single span Caravan bridge over the river Meles at

Smyrna in Asia Minor (Turkey), is believed by archaeologists to be the oldest bridge still in use(13).

Bridges built before 1779 were generally made of timber or used masonry arches. In 1779, the first bridge was built of cast iron at Coalbrookdale, England, by Abraham Darby(13). The first bridge constructed with structural carbon steel was Ead's Bridge across the Mississippi River at St. Louis, begun in 1869 and completed in 1874(13).

## I-5. Types of Short Span Bridges

For individual spans of up to about 80 feet, highway bridges using structural steel can be categorized as follows:

(A) The Beam Bridge. (FIG.1.1)

The beam bridge, in which a concrete roadway slab is supported by wide-flange beams, is very popular because of its simple design and construction. This type of bridge is very economical for highway spans of up to roughly 80 feet. (Actually, the AASHO code minimum span-to-depth ratio limits its span to 76.5 feet.)

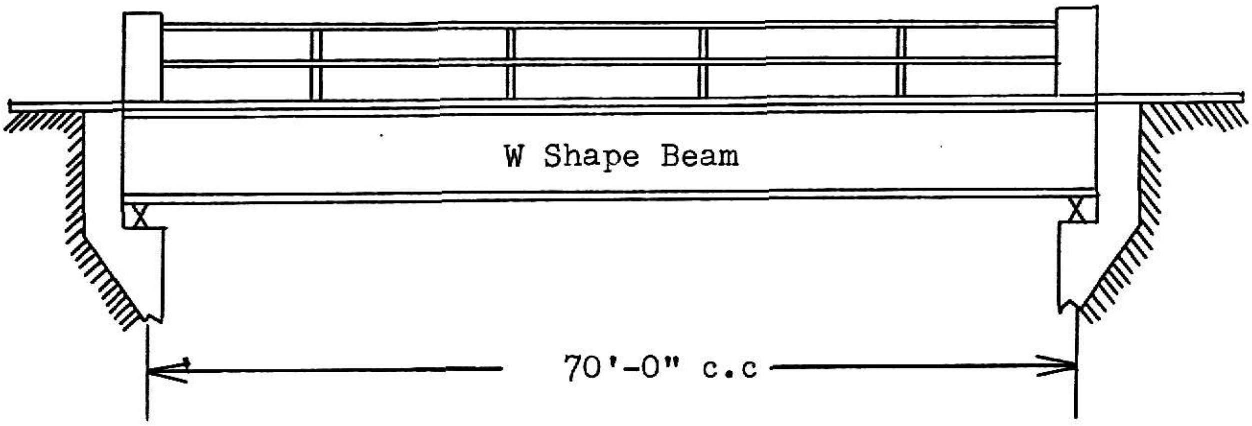
(B) The Composite Bridge. (FIG.1.2)

The composite bridge is one in which rolled steel beams and a composite concrete deck slab are constructed so that they act together as a composite section. The real impetus to composite construction in the United States came with the adoption of the 1944 AASHO specifications. Since about 1950 the use of composite bridge decks has rapidly increased until today they are commonplace all over the country. Simple span composite highway bridges have been economically used for spans from 50 feet to 120 feet.

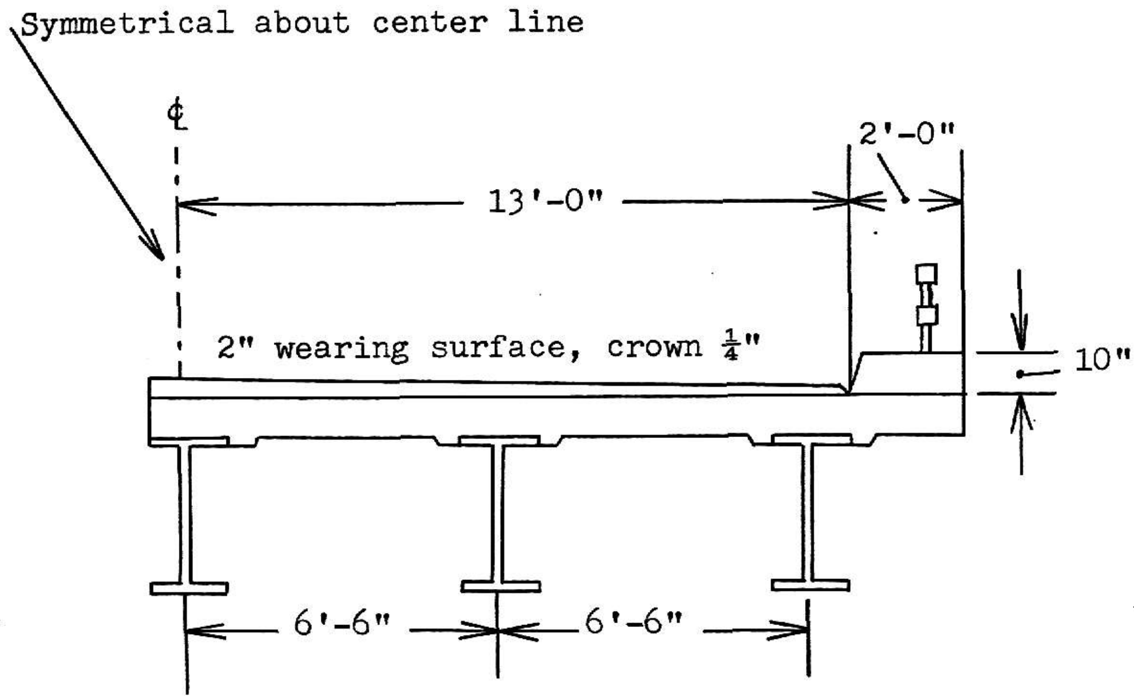
(C) Built-up Beam and Plate Girder Bridges.

In some bridges, beam depths may be limited by clearance requirements. Cover-plated beams (FIG.1.3.a) will often prove to be the best solution for situations of this type. Furthermore, built-up hybrid beams (FIG.1.3.b) are sometimes used, that is, using higher-strength steels for the more severely stressed sections of a beam, and lower-price steels elsewhere, permitting greatest over-all economy.

Plate girders are large I-shape sections built up from plates and rolled sections. (FIG.1.3.c.d) These deep flexural members carry loads which cannot be carried economically by rolled beams. The upper economical limits of plate girder spans depend on several factors, including whether the bridge is simple or continuous, whether the largest section can be shipped in one piece, etc. Generally speaking, plate girders are very economical for highway bridges for spans from 80 feet to 150 feet.




Bridge Profile




Transverse Section

Fig. 1.1. Beam Bridge.

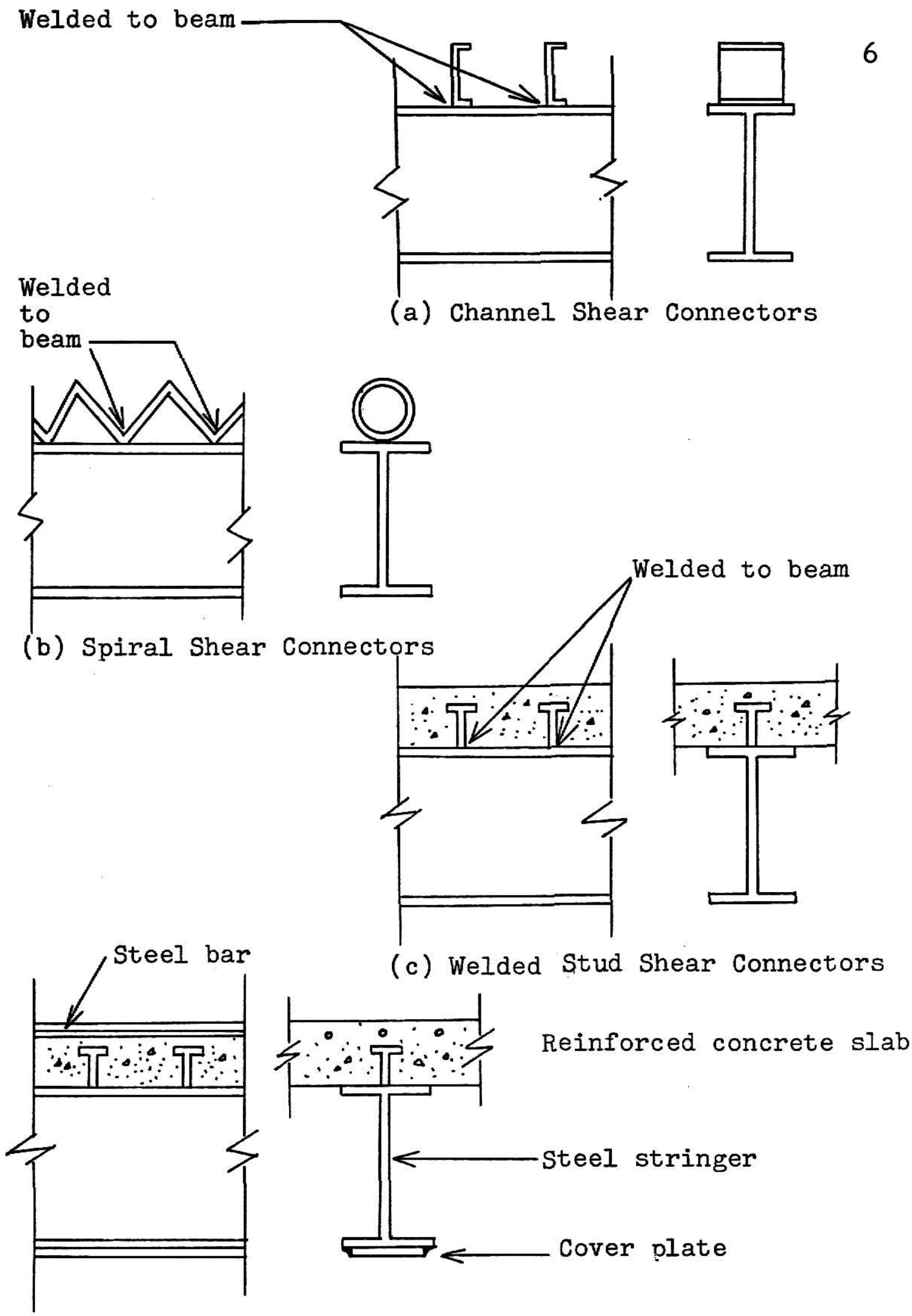



Fig.1.2. Composite Bridge Deck with Shear Connectors.

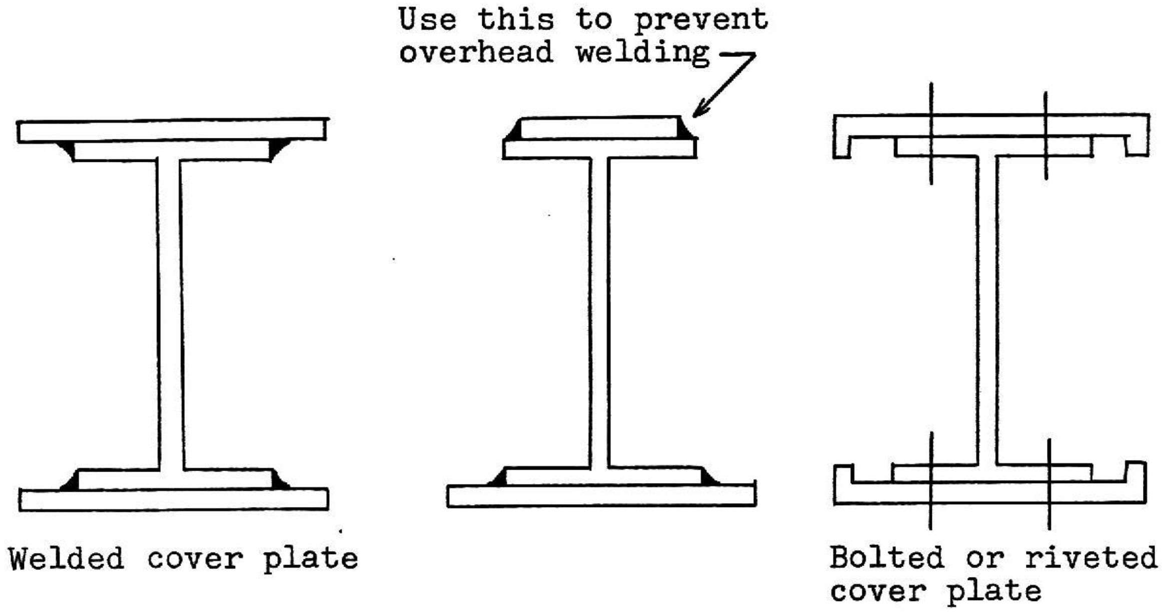
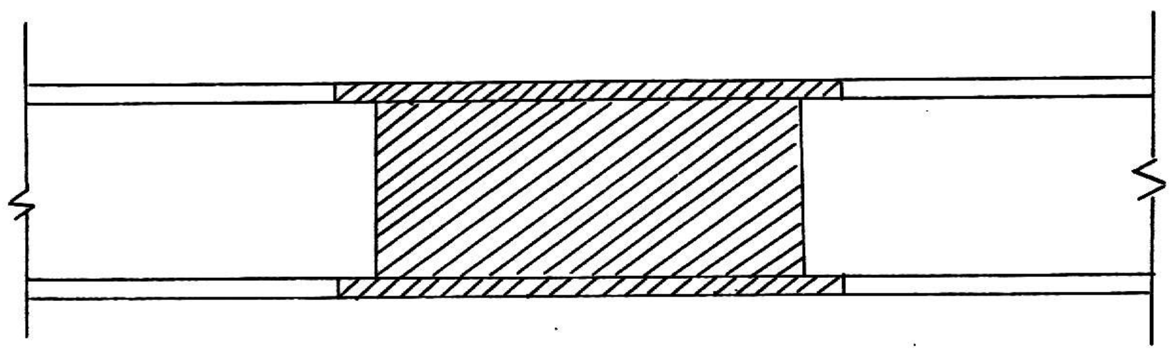



Fig.1.3.a. Cover-Plated Beam.



M188 (A441) Steel $F_y = 50{,}000$ psi.

M183 (A36) Steel $F_y = 36{,}000$ psi.

Fig.1.3.b. Hybrid Steel Beam.

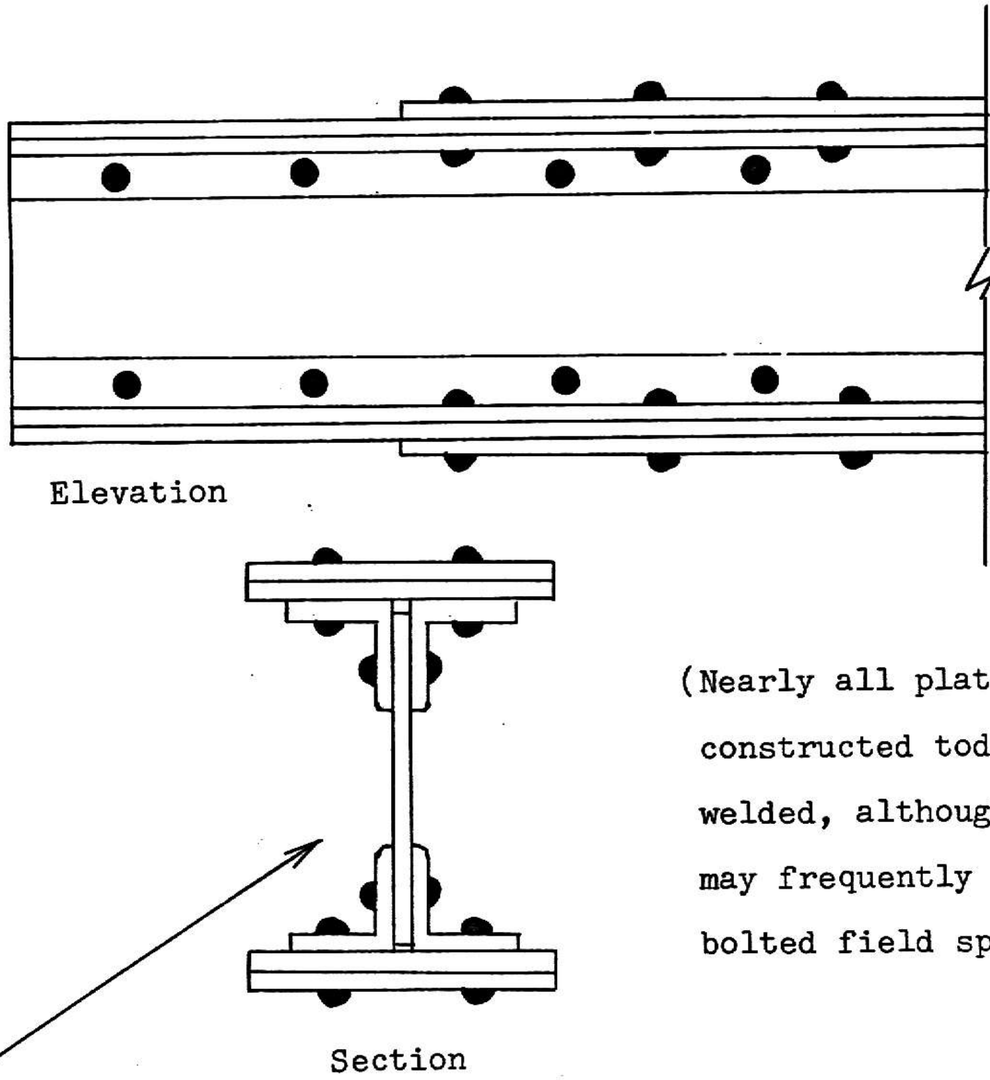



Fig.1.3.c Riveted or Bolted Plate Girder.

(Nearly all plate girders constructed today are welded, although they may frequently have bolted field splices)

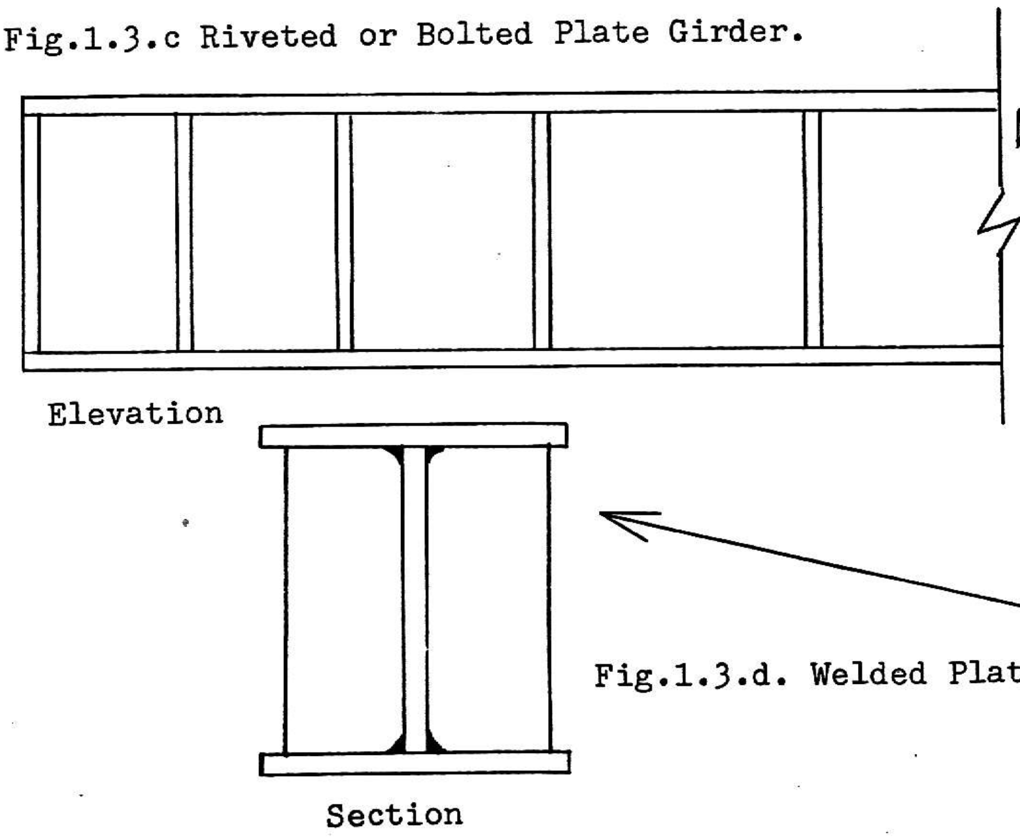



Fig.1.3.d. Welded Plate Girder

# II. THE BEAM BRIDGE

The general considerations in the design of a rolled beam bridge are:

(A) The strength requirement is that the cross section of a member be adequate to resist the applied bending moment and the accompanying shear force.

(B) The stability requirement is that the member be adequate against lateral torisonal buckling.

(C) The stiffness requirement is that the member be adequate to resist excessive deflections under service condition.

All of these requirements will be discussed in the paragraphs to follow.

## II-1. Navier's Flexure Formula

Beams ordinarily will first be selected based on their ability to carry the maximum bending moment, M, without exceeding the allowable unit fiber stress, $F_b$. The fiber stress of a particular section can be computed using Navier's century-old flexure formula.

$$f_b = MC/I < F_b$$

where, $f_b$ = computed flexural stress.

M = applied bending moment about the neutral axis at the section under consideration.

C = distance from the centroidal neutral axis to the extreme fiber.

$I$ = moment of inertia of the section about the same axis.

The value of $I/C$ is constant for a particular section and is known as the section modulus

$$S = I/C = M/F_b$$

The basic allowable bending stress, $F_b$, tension or compression, is taken as a fraction of the yield strength, $F_y$. For highway bridges (AASHO specifications) $F_b = 0.55\ F_y$. (see AASHO Table 1.7.1 for details)

## II-2. Selection of Rolled Beams

Rolled beams generally prove to be the most suitable and economical bridge stringers. There are two types of beams currently rolled: American Standard or S shapes and Wide Flange or W shapes.

(A) The S shapes, the first beam sections rolled in the United States, are rolled in sizes varying from 3 to 24 inches, in depth. For each given depth, there are 2 to 5 sections of varying weight, depending on web and flange thicknesses. In order to vary the area and weight within a given nominal size, the web thickness and the flange width are changed by an equal amount as in FIG.2.1.a.

(B) The W shapes, which vary in depth from 4 to 36 inches, have from 1 to 47 weights for each depth. In order to vary the area and weight within a given nominal size, the flange thickness, and the web thickness are changed as shown in FIG.2.1.b. The W shapes have more steel concentrated in their flanges than do the S shapes and thus have larger section moduli values for the same weight. Also, as the name implies, the flange of the wide-flange shapes are wider, thus resulting in greater lateral stability and easier connection of the flanges to other members. For these reasons the W shapes have almost completely replaced the S shapes.

A table is given in the "Manual of Steel Construction" entitled "Allowable Stress Design Selection Table" (for shapes used as beams)(15). From this table steel shapes having sufficient section moduli can be quickly selected. The table has the sections arranged in various groups having certain ranges of section moduli. The boldfaced typed section at the top of each group is the lightest section in that group and others are arranged successively in the order of their section moduli. Normally the deeper sections will have the lightest weights giving the required section moduli, and they will generally be selected unless their depth causes a problem in obtaining the desired clearance, in which case a shallower but heavier section will be selected.

## II-3. Holes in Beams

It is often necessary to have holes in steel beams. They are obviously required for installation of bolts and rivets. The presence of holes of any type in a beam certainly does not make it stronger and in all probability weakens it somewhat. When the holes are symmetrical with respect to the centroidal neutral axis of the gross section, no shift of this axis is caused by the holes, (FIG.2.2.b) but if the holes are not symmetrical as described above, the neutral axis is shifted away from the holes. (FIG. 2.2.c) Then, hypothetically, the line of zero flexural stress jumps abruptly up and down the length of the beam while the stresses in the flange change accordingly. This is not likely to happen in reality; tests seem to show that flange holes for rivets or bolts do not appreciably change the location of the neutral axis. As a matter of fact, smooth transitions take place as shown in FIG.2.3. AASHO requires that rolled beams be proportioned by the moment of inertia method(2). Two values of the moment of inertia will be calculated when holes are present. The neutral axis is assumed to remain at its normal position for both calculations. For compressive stress the gross moment of inertia is to be used regardless of the presence of rivet or bolt holes. (This provision assumes that holes on the compressive side of the beam have less effect on flexure since those holes are filled by rivets or bolts.) For tensile stress the net moment of inertia is to be used. Should a hole be present in only one side of the tension

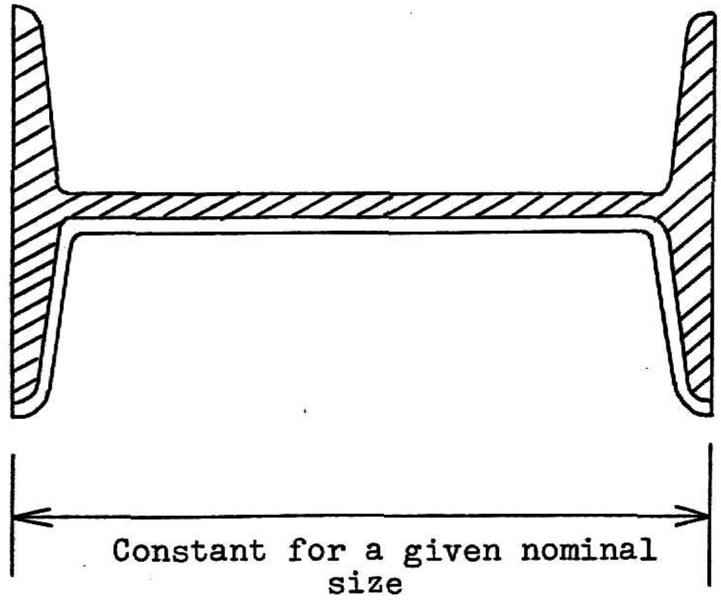



Fig.2.1.a. S Shape Beam.

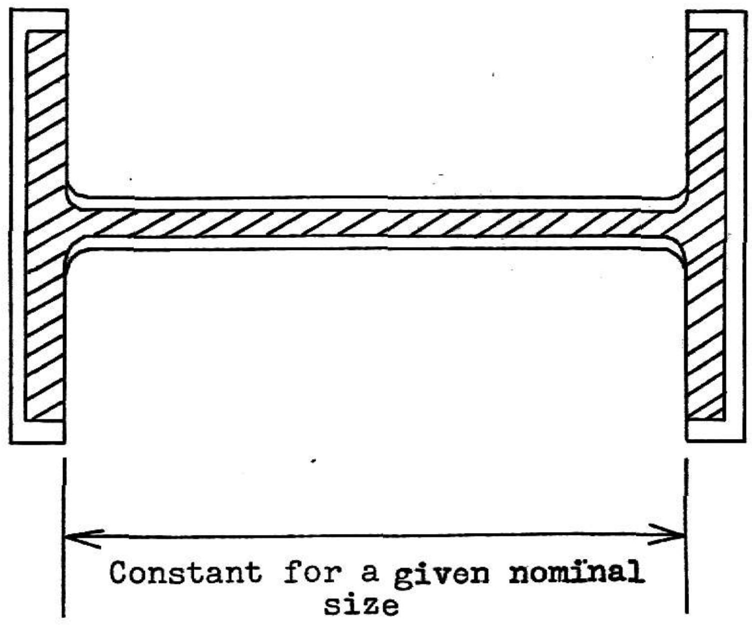



Fig.2.1.b. W Shape Beam.

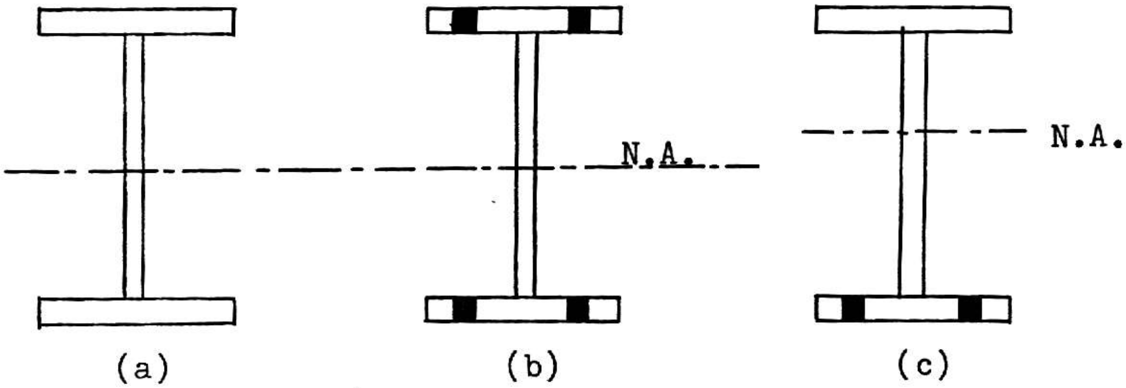



Fig.2.2. Cross-Sectional Area and Neutral Axis.

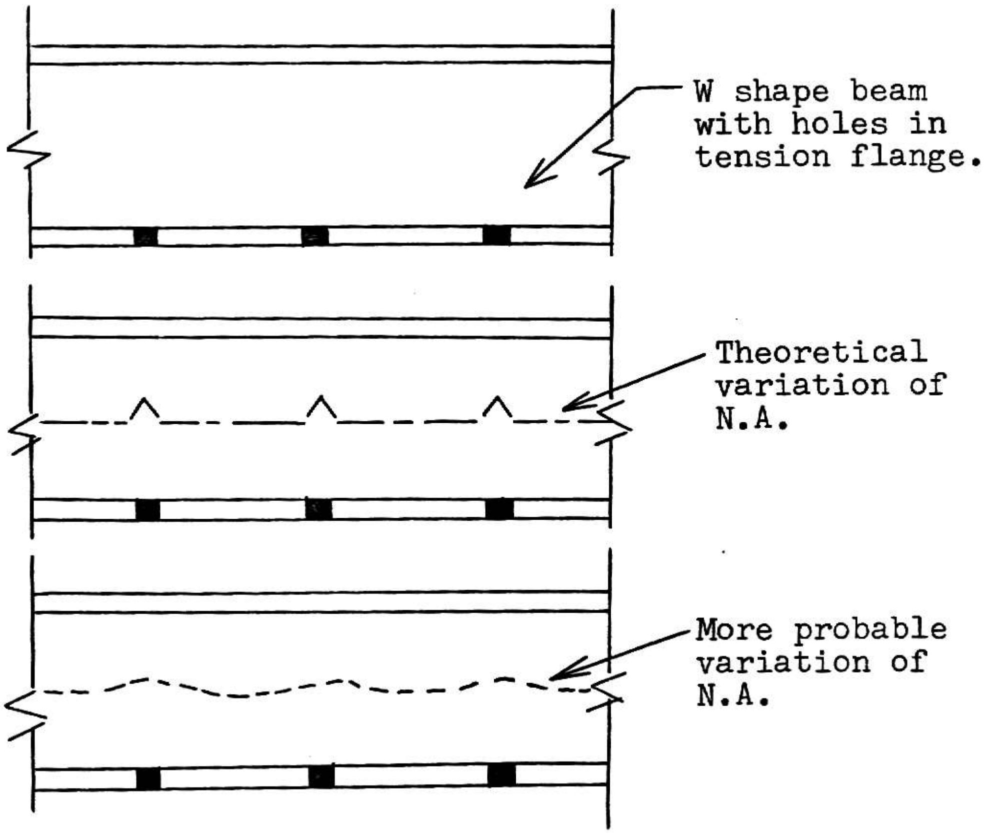



Fig.2.3.

flange of a W section, there will be no axis of symmetry for the net section of the shape. The usual practice is to subtract the same area of holes from both sides whether they are present or not.

## II-4. Lateral Support of Beams

The basic predicted strength of beams sometimes can not be attained because failure occurs by instability at lower loads. That is, when the compression flange of a beam is long enough, it may quite possibly buckle unless lateral support is provided.

A beam which is wholly encased in concrete or which has its compression flange incorporated in a concrete slab is certainly well supported laterally. In this case the allowable bending stress in the compression flange is $0.55\ F_y$.

If full lateral support is not provided, the allowable compressive stress is to be reduced as indicated in the following formula:

$$F_b = A - B(L/b)^2 \quad \text{(AASHO Table 1.7.1)}$$

where A and B are coefficients depending on $F_y$ as in Table 2.1, L is the distance in inches between points of lateral support and b is the flange width in inches.

Table 2.1. Coefficients A and B, and the limits for maximum values of L/b.

| Yield point (ksi) | | 36 | 42 | 46 | 50 |
|---|---|---|---|---|---|
| A | (psi) | 20,000 | 23,000 | 25,000 | 27,000 |
| B | (psi) | 7.5 | 10.2 | 12.2 | 14.4 |
| Maximum | (L/b) | 36 | 34 | 32 | 30 |

## II-5. Shear

Unless the beam is very short and is subjected to high concentrated loads, the shear stress rarely governs the design of beams. It is however still necessary to check the shear stress.

The maximum shearing stress occurs at the neutral axis and is given by

$$f_v = VQ/It_w$$

where, V = external shear at the section in question.

Q = statical moment of that portion of the section lying outside (either above or below) the line on which $f_v$ is desired, taken about the neutral axis.

I = moment of inertia of the entire section about the neutral axis.

$t_w$ = width of the section.

For sections having an I shape, such as W shape rolled beams or built-up girders, the maximum value is only slightly greater than the average shearing stress $V/A_w$. For the purpose of design the average stress is often used; it is assumed that the allowable value is specified on the basis of the average rather than the maximum shearing stress value. In calculating the average shearing stress, the effective web area is considered $A_w = h_e t_w$, where $h_e$ is the effective depth of beam taken as the total depth for rolled beams. For a plate girder only the web depth is used.

The allowable shearing stress, $F_v$, is 0.33 $F_y$. (AASHO Table 1.7.1)

## II-6. Bearing Stiffeners

Suitable stiffeners shall be provided to stiffen the webs of rolled beams at bearings when the unit shear in the web adjacent to the bearing exceeds 75% of the allowable shear for girder webs(2).

## II-7. Maximum Deflections

The deflection shall be computed in accordance with the assumptions made for the loading when computing the stress in the member.

The maximum deflection due to live load plus impact shall not exceed 1/800 of the span, except on bridges in urban areas used in part by pedestrians whereon the ratio preferably shall be 1/1000. (2) The AASHO code also handles the deflection problem by limiting its depth-to-span ratio to a minimum value of 1/25 in order to prevent large deflections.

# III. THE COMPOSITE BRIDGE

The term composite bridge defines a system in which interaction of a concrete slab with a steel beam is accomplished by means of a mechanical device called a shear connecter. The concrete slab becomes the compression flange and the steel section resists the tensile stresses. The tension portion of the beam on a bridge is usually not encased since fireproofing is generally not necessary on a bridge. The shear connectors may be in the form of channels, spirals or studs serving to tranfer the longitudinal shear from the concrete to the steel beam and also serving to hold the concrete from uplifting. (FIG.1.2.a,b,c)

## III-1. Advantage of Composite Construction

The obvious advantages of composite construction are as follows:

(A) Saving in steel of 20 or even 30 percent compared to non-composite construction.

(B) Reduction in depth of members.

(C) Composite sections have greater stiffness than non-composite sections and therefore have smaller deflections, —perhaps only 20 to 30 percent as large as non-composite sections.

(D) Economical use of rolled sections for longer spans.

## III-2. Methods of Constructing Composite Bridges

A composite bridge may be built with or without temporary supports (shoring). When shores are not used, the steel beams support their own weight, the forms, and the weight of the slab during casting and curing of the slab. Only the loads applied after the slab has hardened are resisted by the composite section.

When the steel beams rest on temporary supports, it may be assumed that all of the loads are carried by the composite section.

Shoring usually will not be used in bridge construction for three reasons:

(A) Shoring is a delicate operation, especially if settlement of the temporary supports is difficult to prevent, which is usually the case in bridge construction.

(B) Tests have shown that the ultimate strengths of composite sections of the same size are the same whether shoring was used or not. If lighter steel sections are selected for a particular span because shoring is used, the result is therefore a smaller ultimate strength.

(C) After the concrete hardens and the shoring is removed the slab will participate in composite action in supporting the dead loads. The slab will be placed in compression by these long-term loads and will have substantial creep and shrinkage parallel to the beams. The result will be a large decrease in the stress in the slab with a corresponding increase in the steel stresses. The problem consequently is that most of the dead load will be supported by the steel beams anyway and composite action will really apply only to the live loads as though shoring had not been used. A common practice when shoring is used is to reduce the calculated effective area of the concrete flange by a factor of 3.

## III-3. Effective Flange Width

According to the AASHO code, in composite beam construction the assumed effective width of the slab as a T-beam flange shall not exceed the following:

(A) One-fourth of the span length.

(B) The distance center to center of beams.

(C) Twelve times the least thickness of the slab.

For beams having a flange on one side only, the effective flange width shall not exceed one-twelfth of the span length of the girder, nor six times the thickness of the slab, nor one-half the distance center to center of the next beam.

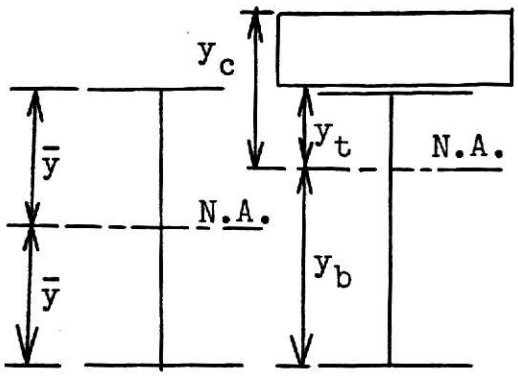
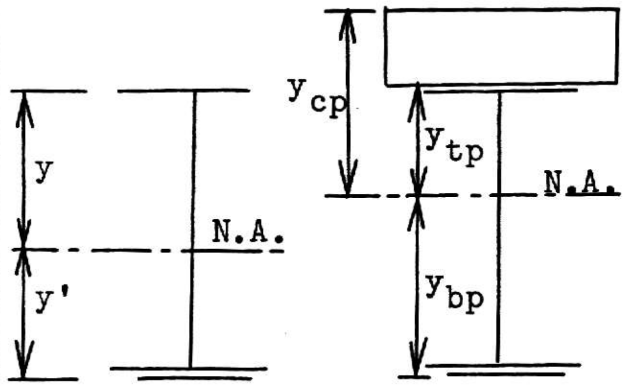
## III-4. Design of a Composite Beam

For composite design it is customary to replace the concrete with an equivalent area of steel, whereas the reverse procedure is used in reinforced concrete design.

In transforming the concrete slab into equivalent steel, the depth of the transformed area is held constant, but the width is reduced to 1/n times its actual value, where $n = E_s/E_c$. The value of n to be used will depend on whether the loads are long- or short-term. If loads are long-term (such as the composite portion of the dead load), the effect of creep and shrinkage are approximated by reducing $E_c$ to 1/3 its normal value. Thus, when stresses due to long-term loads are dealt with, the substituted steel area is only 1/3 what it would other wise have been.

The calculation of actual stresses for unshored construction requires the definition of several sets of section properties for the calculation of bending stresses. The terms are given and defined in Table 3.1.

Table.3.1. Notation for Composite Design

| Steel Section - No Cover Plate | Steel Section - With Cover Plate |
|---|---|
|   |   |
| Steel section moment of inertia = $I$ | Steel section moment of inertia = $I_p$ |
| Composite section moment of inertia = $I_c$ | Composite section moment of inertia = $I_{cp}$ |

$n = E_s/E_c$ = Elastic modular ratio

$n' = E_s/E_c'$ = Plastic modular ratio

When the plastic modular ratio is used all properties above are referred to as $y_c'$, $I_c'$, $y_{cp}'$, etc.

Section Modulus for Calculation of Stresses

(loading to be considered)

| Member | Unshored Dead Load | Live Load | Dead Load on Composite Section | Location of Stress |
|---|---|---|---|---|
| No Cover Plate | -------- | $S_c=I_c/y_c$ | $S_c'=I_c'/y_c'$ | Top of Concrete |
| | $S=I/\bar{y}$ | $S_t=I_c/y_t$ | $S_t'=I_c'/y_t'$ | Top of Steel |
| | $S=I/\bar{y}$ | $S_b=I_c/y_b$ | $S_b'=I_c'/y_b'$ | Bottom of Steel |
| With Cover Plate | -------- | $S_{cp}=I_{cp}/y_{cp}$ | $S_{cp}'=I_{cp}'/y_{cp}'$ | Top of Concrete |
| | $S_{st}=I_p/y$ | $S_{tp}=I_{cp}/y_{tp}$ | $S_{tp}'=I_{cp}'/y_{tp}'$ | Top of Steel |
| | $S_{sb}=I_p/y'$ | $S_{bp}=I_{cp}/y_{bp}$ | $S_{bp}'=I_{cp}'/y_{bp}'$ | Bottom of Steel |

Members constructed without shoring may have part of the dead load carried by the steel members and part by the composite section. The moments due to different loads are defined as follows:

$M_D$ = moment due to dead load on the steel member. (weight of the slab)

$M_d$ = moment due to dead load on the composite member. (weight of curbs, railing and wearing surface which are cast after the slab has hardened)

$M_L$ = moment due to live load on the composite member. (weight of the vehicles)

Two equations are given for stress calculation. The first equation of each pair pertains to the portion of the member without a cover plate and the second pertains to the portion of the member with a cover plate.

$$f_c = M_d/(n'S'_c) + M_L/(nS_c)$$
$$f_t = M_D/S + M_L/S_t + M_d/S'_t$$
$$f_b = M_D/S + M_L/S_b + M_d/S'_b$$

or,

$$f_c = M_d/(n'S'_{cp}) + M_L/(nS_{cp})$$
$$f_t = M_D/S_{st} + M_L/S_{tp} + M_d/S'_{tp}$$
$$f_b = M_D/S_{sb} + M_L/S_{bp} + M_d/S'_{bp}$$