

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

by

ROBERT TEH HSIN LEE

Diploma, Taiwan Provincial Taipei Institute of Technology  
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Approved by:

  
Major Professor

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## 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.

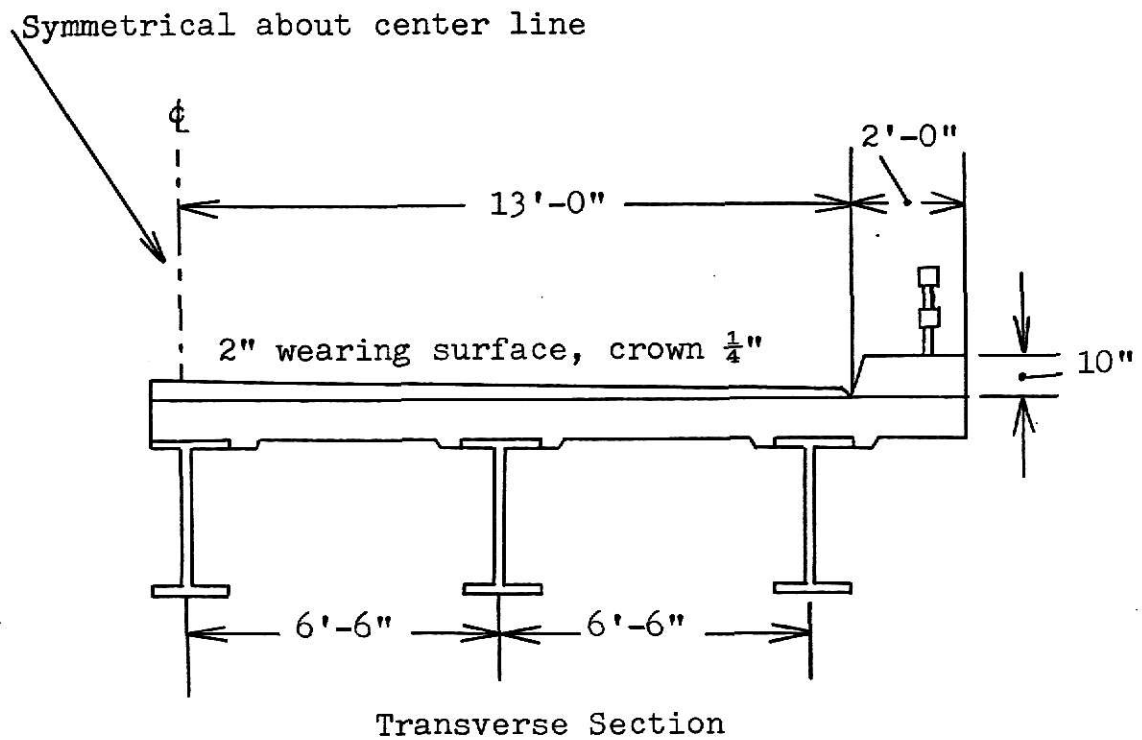
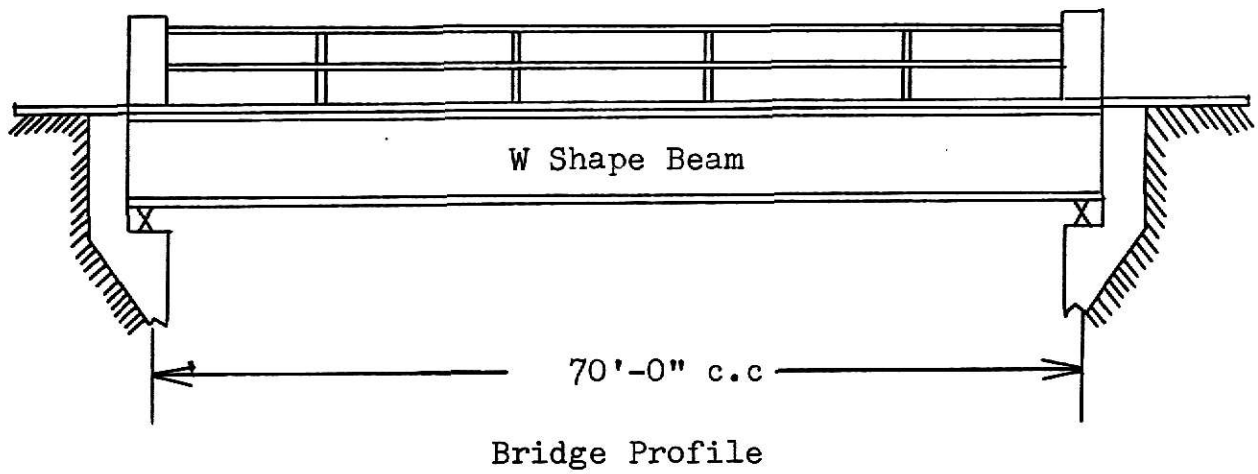


Fig. 1.1. Beam Bridge.

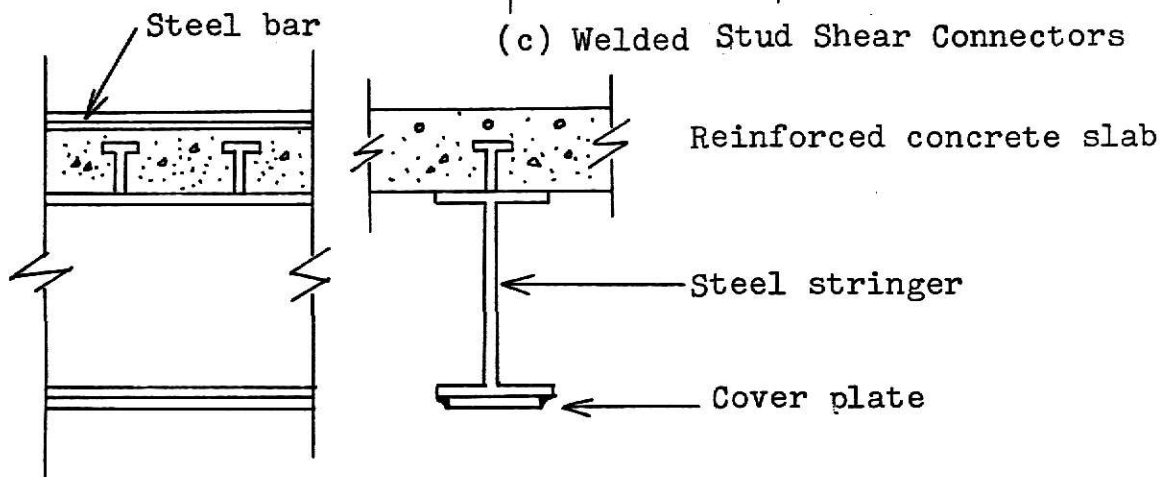
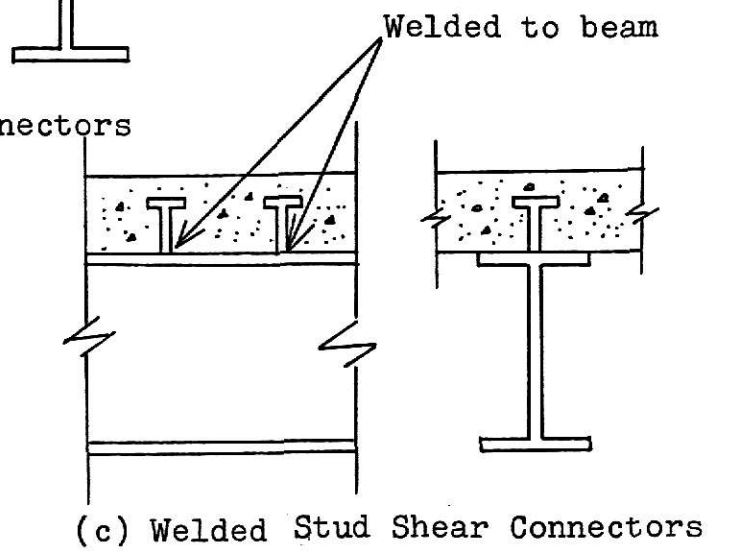
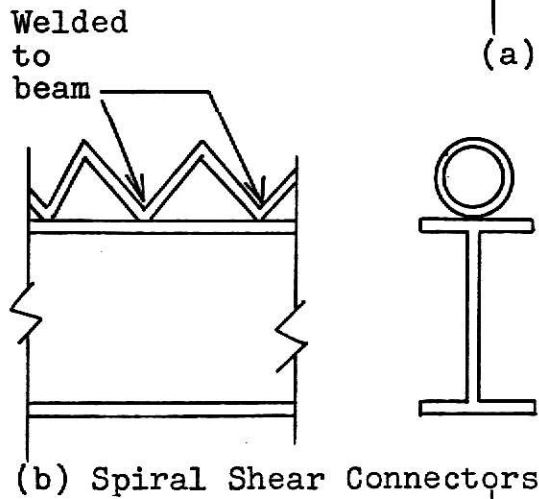
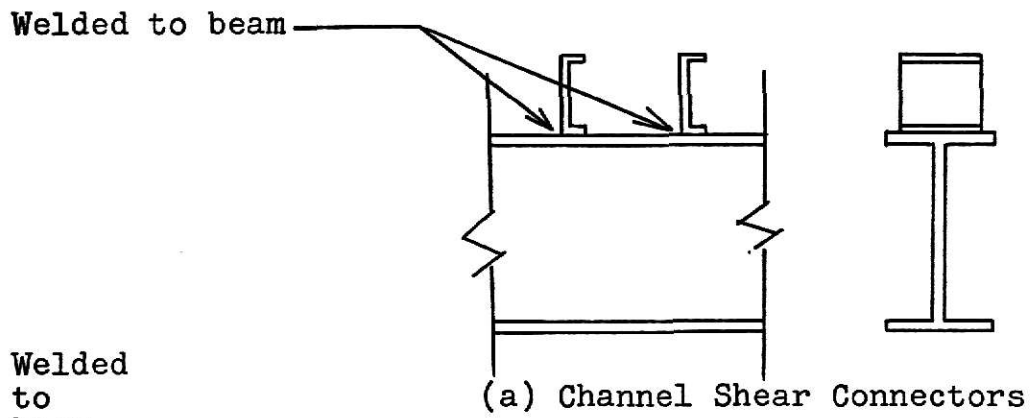


Fig.1.2. Composite Bridge Deck with Shear Connectors.

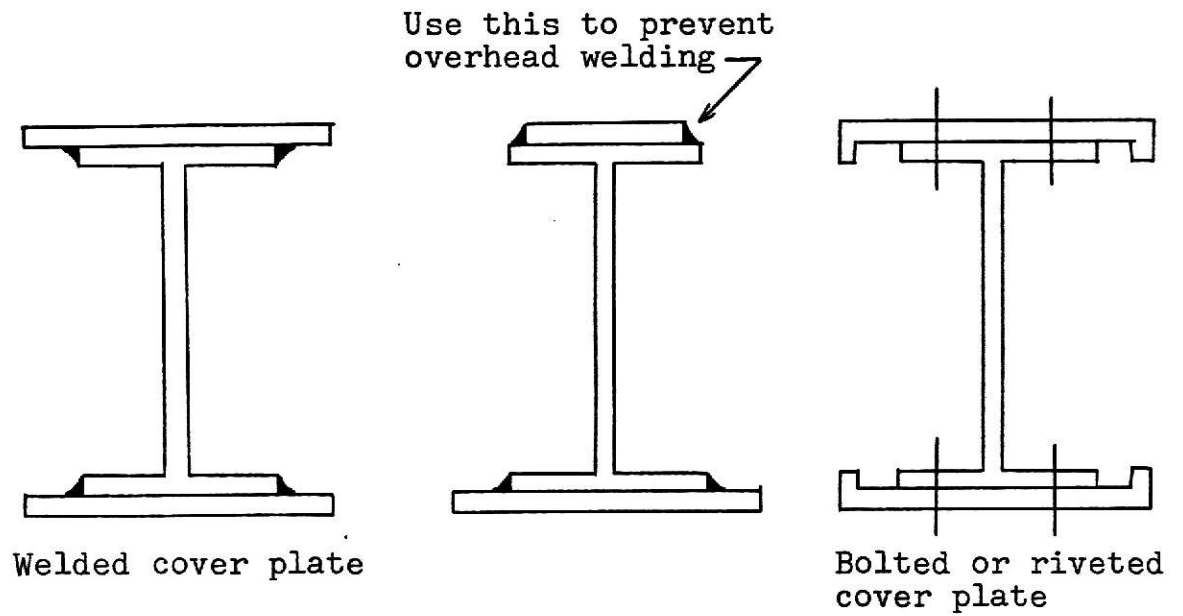
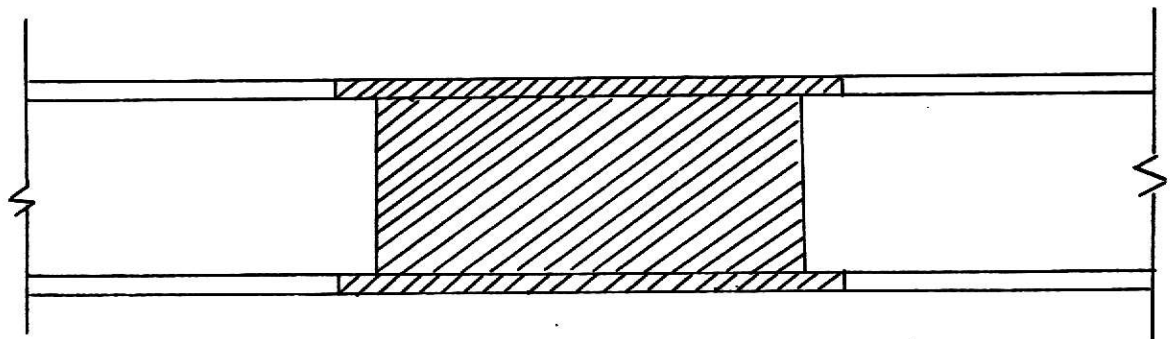


Fig.1.3.a. Cover-Plated Beam.



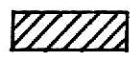
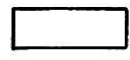
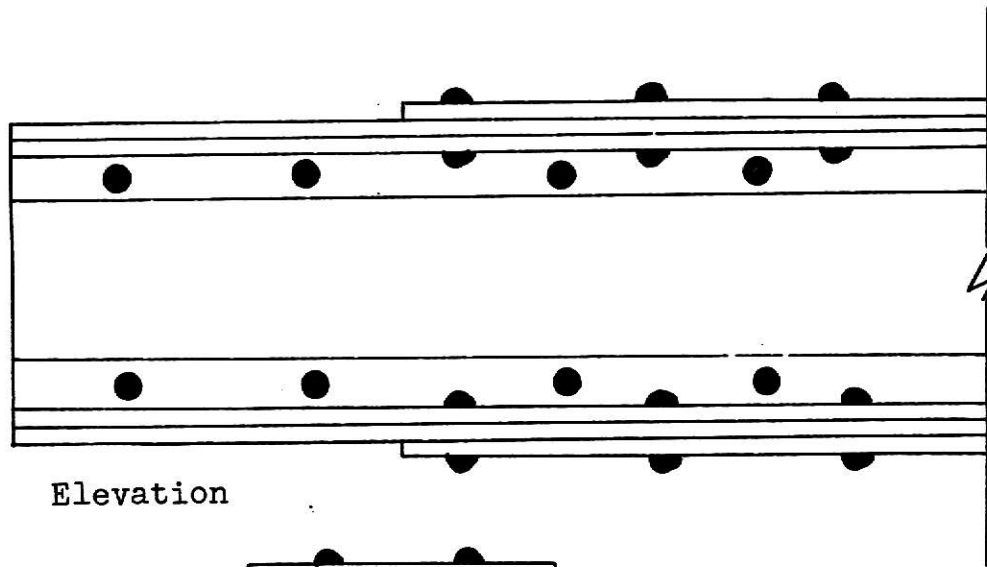
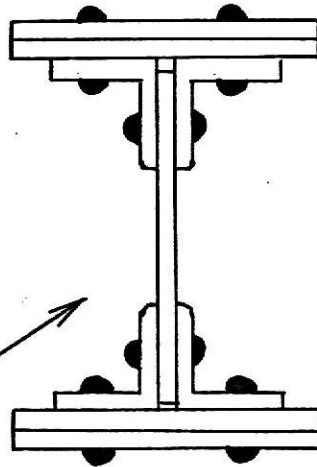
	M188 (A441) Steel	$F_y = 50,000$ psi.
	M183 (A36) Steel	$F_y = 36,000$ psi.

Fig.1.3.b. Hybrid Steel Beam.



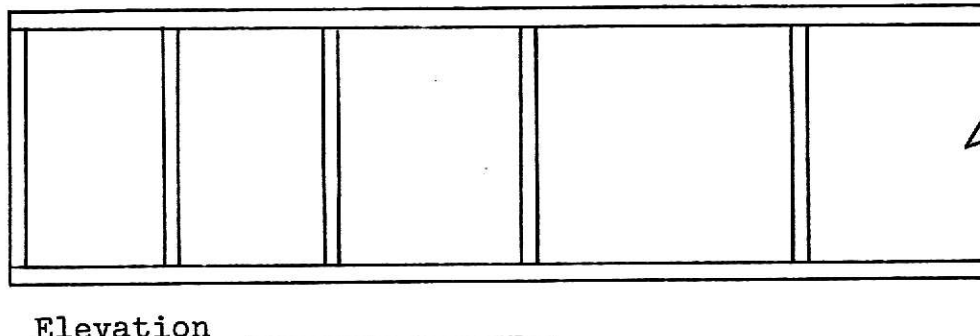
Elevation



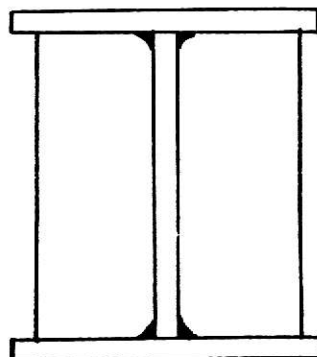
Section

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

Fig.1.3.c Riveted or Bolted Plate Girder.



Elevation



Section

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 torsional 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 (AASHTO specifications)  $F_b = 0.55 F_y$ . (see AASHTO 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

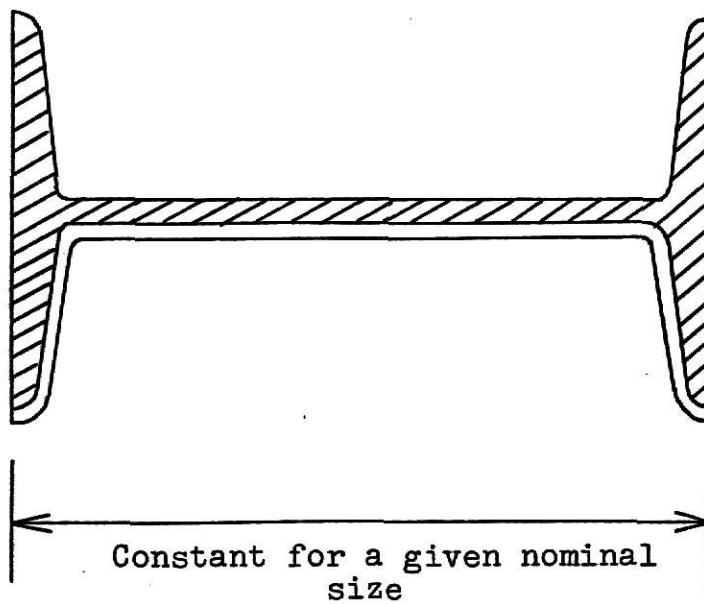


Fig.2.1.a. S Shape Beam.

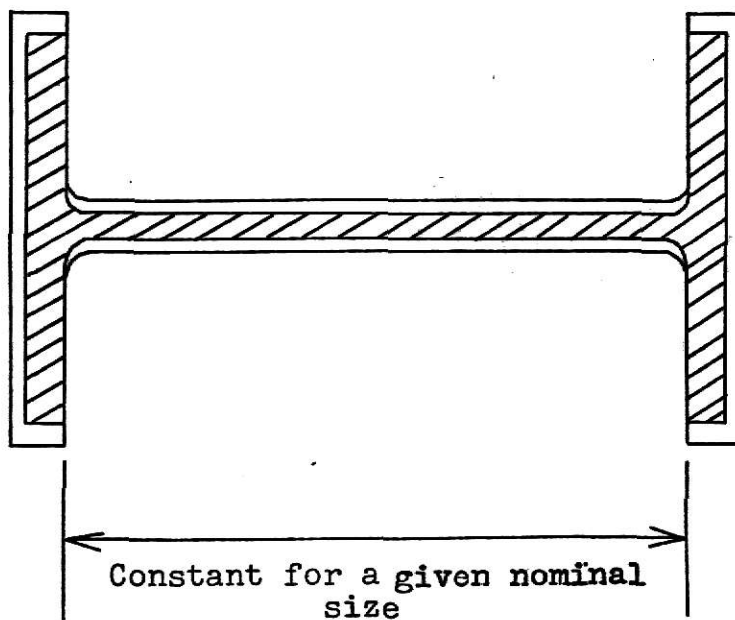


Fig.2.1.b. W Shape Beam.

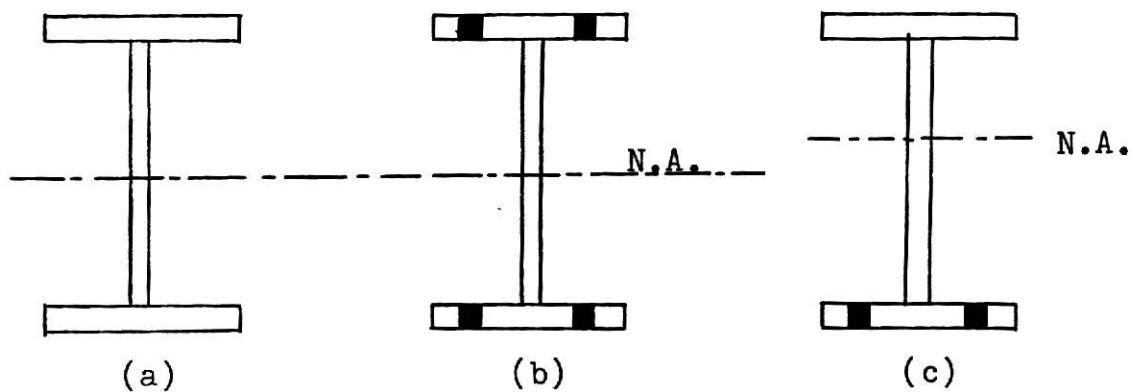


Fig.2.2. Cross-Sectional Area and Neutral Axis.

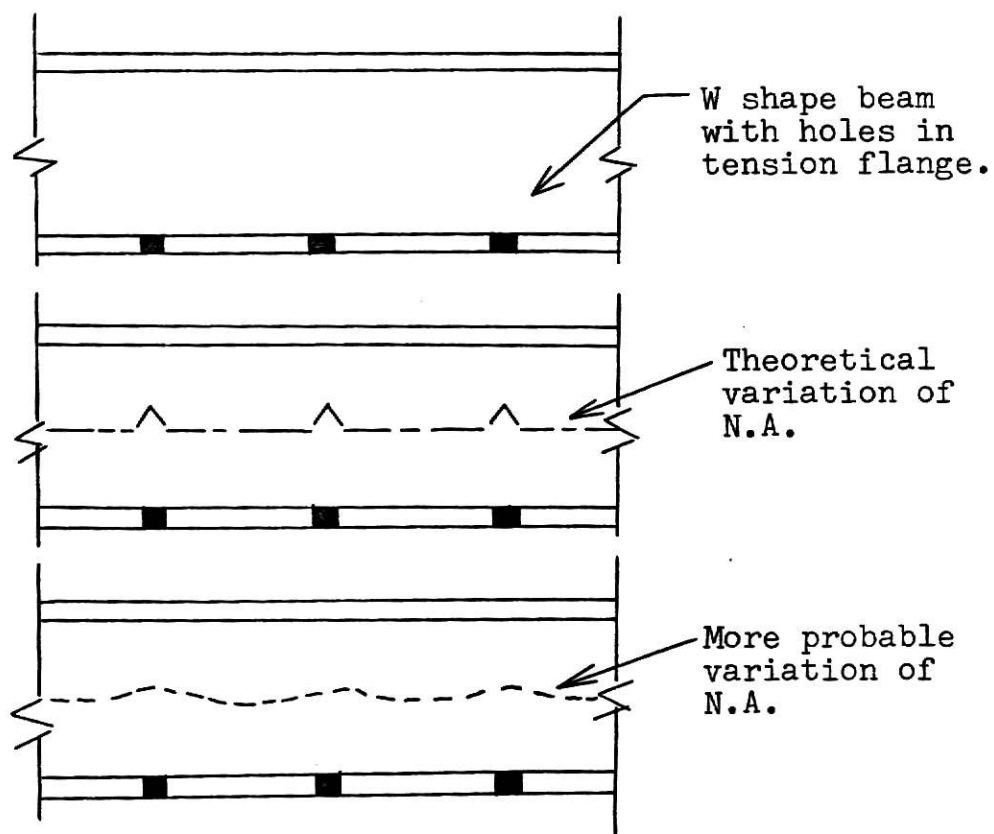


Fig.2.3.

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  $0.33 F_y$ . (AASHTO 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 connector. 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 transfer 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 otherwise 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
<p>Steel section moment of inertia = <math>I</math></p> <p>Composite section moment of inertia = <math>I_c</math></p>	<p>Steel section moment of inertia = <math>I_p</math></p> <p>Composite section moment of inertia = <math>I_{cp}</math></p>

$$n = E_s/E_c = \text{Elastic modular ratio}$$

$$n' = E_s/E'_c = \text{Plastic modular ratio}$$

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 on Composite Section	Location of Stress
	Dead Load	Live Load		
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,

$$f_c = M_d / (n'S'_{cp}) + M_L / (nS_{cp})$$

$$f_t = M_D / S_{st} + M_L / S_{tp} + M_d / S'_{tp}$$

$$f_b = M_D / S_{sb} + M_L / S_{bp} + M_d / S'_{bp}$$



### III-5. Theoretical Length of Cover Plate

The approximate formula used to determine the theoretical length of cover plate is

$$L' = g + (L-g)\sqrt{1-S_b/S_{bp}} \quad (\text{Reference 5. Page 427})$$

where,  $L'$  = the length of the cover plate.

$g$  = the distance from the center of the span to the point of maximum moment.

The approximate formula is ordinarily used since an exact computation of the theoretical length is very tedious.

### III-6. Choice of Cover Plate

For best economy, thicker slabs and cover-plated beams are usually of advantage. An important point to realize is that the major cost of cover plates regardless of their size is in fabrication. The result is that heavy cover plates do not cost proportionately more than thin ones; it is logical, therefore, to use large cover plates or none at all from an economic standpoint. In fact, the use of a cover plate with an area of less than one-half that of the steel section is seldom justified.

### III-7. Simplified Methods of Selecting Composite Sections

Perhaps the greatest difficulty in composite design has been the selection of economical beam size as a lengthy trial-and-error process has been involved. However, a great deal of data has been published which appreciably abbreviates the solution of the problem.

First, the paper written by H. Subkowsky, "Choice of Composite Beams for Highway Bridges" (6) presents a series of design charts for the rapid selection of rolled beams and cover plates based on economic sections of steel.

Second, the book "Composite Construction in Steel and Concrete" by Viest, Fountain and Singleton gives a great deal of information on selection of composite sections(4).

Third, an intelligent method of estimating steel beam size for composite sections is presented on page 104 of "Advanced Design in Structural Steel" by John E. Lothers(16). Lothers suggests that the steel beams be designed as though they alone have to support all of the loads but using a higher allowable stress to account for the composite design. Table 3.2. presents the estimated allowable steel stress increases for varying values of  $n$ .

Table 3.2. Steel Factors For A Simply-Supported  
Composite Beam

$n = E_s/E_c$	*Steel Factor
15	1.20
12	1.24
10	1.28
8	1.33
6	1.37

\*The allowable bending stress in steel is multiplied by this factor for a tentative choice of steel section.

In applying these values it must be remembered that the usual allowable steel stresses may not be exceeded under the action of  $M_D$  alone.

Fourth, additional data is contained in a book entitled "Properties of Composite Sections for Bridges and Buildings" published by the Bethlehem Steel Company (17).

### III-8. Shear Connectors

The AASHO code says that there shall be a minimum of 2 in. of concrete over the tops of shear connectors and the connectors must penetrate a minimum of 2 in. up from the bottom of the slab. Also, the maximum pitch of shear connectors shall not exceed 24 inches.

The AASHO code also requires that shear connectors shall be (A) designed for Fatigue, and (B) checked for Ultimate Strength, and that they may be spaced at regular or variable intervals.

#### (A) Fatigue

The shearing force to be taken per inch is to be calculated by the formula:

$$S_r = (V_r Q) / I$$

where,  $S_r$  = the range of horizontal shear per linear inch at the junction of the slab and girder at the point in the span under consideration.

$V_r$  = the range of shear due to live load and impact.

$Q$  = the statical moment about the neutral axis of the composite section, of the transformed compressive concrete area.

$I$  = the moment of inertia of the transformed composite section.

The required pitch of the connectors for fatigue

at a particular cross-section is determined by dividing the shear to be taken per inch by the strength of the connectors used at one cross-section.

The allowable shear (lbs) for welded studs,  $Z_r$ , is calculated as follows:

$$Z_r = \alpha d^2$$

(when  $H/d$  is equal to or greater than 4)

where,  $H$  = height of stud in inches.

$d$  = diameter in stud in inches.

$\alpha$  = 13,000 for 100,000 cycles.

10,600 for 500,000 cycles.

7,850 for 2,000,000 cycles.

#### (B) Ultimate Strength.

After the connectors are designed for fatigue the code says the number,  $N$ , used between the point of maximum moment and the end points must be checked for ultimate strength by the expression to follow:

$$N = P/(\phi S_u)$$

where,  $S_u$  = the ultimate strength of one shear connector in lbs.

$\phi$  = a reduction factor = 0.85

$P$  = the force in the slab defined as  $P_1$ ,  $P_2$ .

For the maximum positive moment point, the force in the slab is taken as the smaller value of the formulas.

$$P_1 = A_s F_y$$

$$P_2 = 0.85 f'_c b c, \quad \text{or,} = 0.85 f'_c A_c$$

The ultimate strength of the welded stud shear connectors is:

$$S_u = 930 d^2 \sqrt{f'_c} \quad (H/d \geq 4)$$

Only welded stud shear connectors were mentioned above, since they are generally the simplest and most economical type.

### III-9. Deflections

The problem is somewhat reduced in the case of bridges by a provision in the AASHO specifications suggesting that the depth-to-span ratio for the steel section be no less than 1/30 and depth-to-span ratio for the composite section be no less than 1/25. These values were developed as limiting values for structural carbon steel and are not necessarily valid when high-strength steels are used or when hybrid girders are specified.

For H-S trucks, the maximum deflection due to live load plus impact of a simple beam can be calculated as follows:

$$\Delta = [ 324 P_T (L^3 - 555 L + 4780) ] / (E_s I_c) \quad (\text{Ref. 4. p. 39})$$

where,  $\Delta$  = center line deflection, inches.

$P_T$  = weight of one front truck wheel, kips,  
multiplied by live load distribution factor  
plus impact.

$I_c$  = moment of inertia of the composite section  
based on  $n = E_s/E_c$  in<sup>4</sup>

If a cover plate is used, this formula is still valid  
and the error is usually of the order of 1% and is always  
less than 3%.

#### IV. BUILT-UP BEAM AND PLATE GIRDER BRIDGE

##### IV-1. Built-Up Beams

When available rolled-beam sections do not have sufficient strength to resist the external bending moment they may be reinforced along their entire length or any part of it. For flange reinforcement, plates and channels are most conveniently used. If the section has been selected, the moment of inertia of the entire section equals the moment of inertia of the W section plus the moment of inertia of the added plate sizes. In any case, the capacity provided must be larger than the strength required of the beam. Theoretically, a cover plate can be cut off at points where these two capacities are identical, namely, the "theoretical cut-off points" (FIG.4.1). However, the stress transfer from the flange to a cover plate is gradual; adequate extension of the cover plate from a theoretical cut-off point into the region of lower stresses (lower moment) is therefore necessary for the utilization of the full strength of the cover plate. The AASHO code requires that the length of any cover plate added to a rolled beam shall be not less than  $(2D+3)$  feet, where  $(D)$  is the depth of the beam in feet.

Also, welded cover plates preferably shall be limited to one on any flange. The maximum thickness of the cover plate



(or total thickness of all cover plates) on a flange shall not be greater than 1.5 times the thickness of the flange to which the cover plate is attached.

#### IV-2. Plate Girders

Plates and rolled beams can be riveted, welded, or bolted together (FIG.1.3.c.d) to form plate girders of almost any reasonable proportions. This fact may seem to give them a great advantage for all situations, but for the smaller sizes the advantage is usually canceled by the higher fabrication costs. For example, we could replace a W36 with a plate girder roughly twice as deep which would require considerably less steel and would have much smaller deflections; however, the higher fabrication costs will almost always rule out such a possibility.

The AASHO specifications, and design procedure, using web buckling as a design criterion for plate girder bridge members, will be presented in the following.

#### IV-3. Design Procedure

##### (A) Web thickness and depth:

When designing a plate girder to resist a given bending moment,  $M$ , it is desirable to maximize the lever arm of the internal forces, so that the material required for the flanges

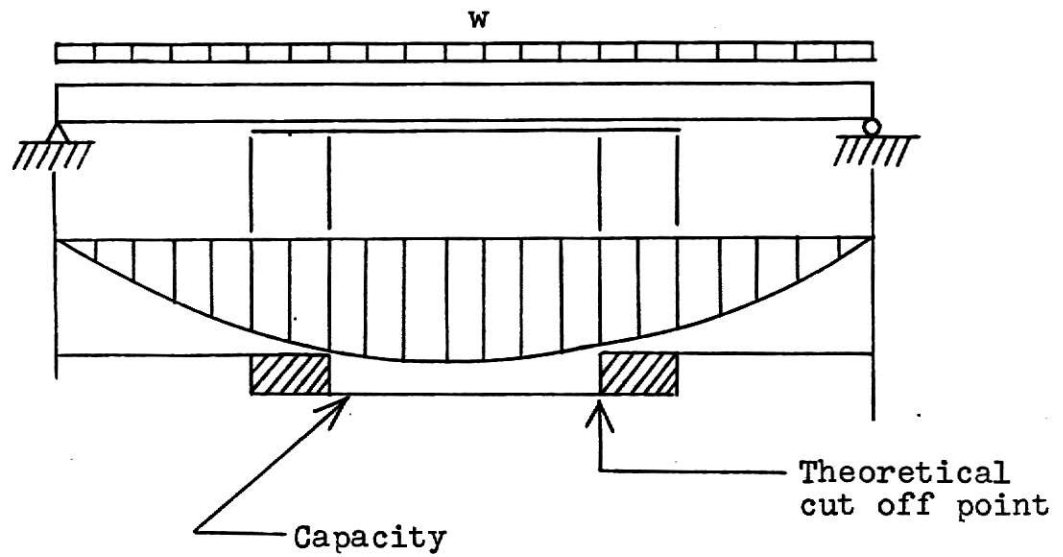


Fig.4.1. Length of Cover Plate

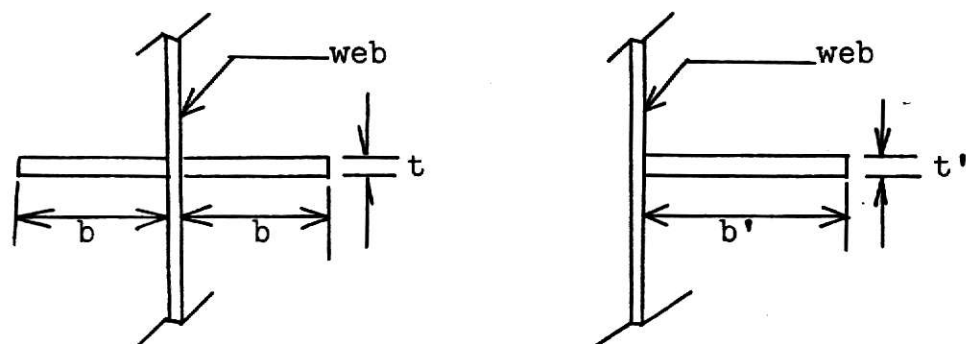


Fig.4.2. (a) Two-Sided and (b) One-Sided  
Stiffener Arrangement.

is minimized. The area of the web ( $=t_w D_w$ ) required to resist a given shear  $V$  depends on the allowable shear stress  $F_v$ . If the depth of the web were not limited by clearance requirements, available plate sizes, aesthetic proportions, or construction conditions, then the lightest girder would be one with a deep thin web. For a short span bridge, to start with, assume a web thickness of  $5/8"$  is very reasonable. Then, the depth of the web may be determined by the following:

$$D_w = (V_{\text{Design}})/(F_v t_w)$$

The web depths are selected to the nearest inch because these plates are not stocked in fractional dimensions. Also, the AASHTO code requires that the web thickness of a plate girder without longitudinal stiffeners, shall not be less than that determined by the formula:

$$t_w = (D \sqrt{f_b})/(23,000) \quad \text{but, in no case shall the thickness be less than } D/170.$$

where,  $D$  = the depth of the beam in feet.

Transverse intermediate stiffeners may also be omitted if the web plate thickness is not less than the thickness determined by the formula:

$$t_w = (D \sqrt{f_v})/(7500) \quad \text{but, in no case shall } t_w \text{ be less than } D/150.$$

Stress analysis shows that tall, thin plate girder webs

may buckle due to a combination of bending and shearing stresses unless stiffeners are used at certain intervals. Tests have shown that for carbon steels there is little danger of buckling if the web thickness is at least  $1/60$  of the unsupported depth of the web. Should a web be selected which is thinner than the value given by this ratio it will be necessary to provide stiffeners spaced no further apart than the clear web depth. It is usually more economical to use thinner webs with stiffeners than to use the thicker ones without stiffeners. From a corrosion standpoint the usual practice is to use some absolute minimum web thickness. For bridges,  $3/8$ " is a common minimum value.

(B) Flange:

After the web dimensions are selected the next step is to select the area of the flange. The goal, of course, is to select a flange of sufficient area such that it will not be overstressed in bending. Bending stress in girders shall be determined using the formula  $f_b = MC/I$ . The tensile stresses shall be computed using  $I_{net}$ , and the compressive stresses using  $I_{gross}$ . Both moments of inertia shall be calculated with respect to the gravity axis of the gross section.

For welded girders each flange may comprise a series of plates joined end to end by full penetration butt welds. Changes in flange area may be accomplished by varying the

thickness or width of the flange plate, or by adding cover plates. The full width-to-thickness ratio ( $b/t$ ) of compression flange plates shall not exceed  $(3250)/(\sqrt{f_b})$ , and shall in no case be greater than 24. For riveted or bolted girders, flange angles shall form as large a part of the area of the flange as practicable. For flange angles in compression the leg width-to-thickness ratio ( $b'/t$ ) shall not exceed  $(1625)/(\sqrt{f_b})$ , and in no case shall it be greater than 12. The gross area of the compression flange, except for composite design, shall not be less than the gross area of the tension flange.

#### IV-4. Transverse Stiffeners

It is sometimes necessary to stiffen the tall thin webs of plate girders to keep them from buckling. Transverse stiffeners are divided into two types; bearing stiffeners which transfer heavy reactions to the full depth of the web, and nonbearing stiffeners which are placed at various intervals along the web to prevent buckling due to diagonal compression.

##### (A) Bearing stiffeners:

Over the end bearing of welded plate girders there shall be stiffeners. They should extend as nearly as practicable to the outer edges of the flange plates and fit tightly

against the flanges. Bearing stiffeners shall be designed as columns and the allowable bearing stress shall not exceed  $0.55 F_y$  nor shall the compressive stress on the effective column section exceed the allowable value corresponding to the slenderness ratio of the bearing stiffeners including the effective web area.

(B) Nonbearing stiffeners:

Tests have shown that if the ratio of the thickness of the web to its unsupported height is less than  $1/60$  buckling due to diagonal compression is possible and intermediate stiffeners are required.

The spacing 'd' of transverse stiffeners shall not be greater than  $(11000t_w)/(\sqrt{f_v})$ , and in no case shall it be greater than the clear unsupported depth of the girder web between the flanges. Thus, the shearing stress  $f_v$  shall not be greater than  $[11000(t_w/d)]^2$ . The spacing of the first two stiffeners at the ends of simply supported girders shall be  $1/2$  of the foregoing limit. Intermediate transverse stiffeners shall be made of plates for welded girders and shall be made of angles for riveted plate girders. When stiffeners are used in pairs, one fastened on each side of the web plate, they should have a tight fit at the compression flange. A single stiffener fastened on one side of the web plate shall be fastened to the compression

flange. Some clearance between the stiffeners and the tension flange is usually provided. The width of a plate stiffener or the outstanding leg of an angle stiffener shall not be less than 2 in. plus  $1/30$  of the depth of the girder, and preferably it shall not be less than  $1/4$  of the flange width. The thickness of the stiffener shall not be less than  $1/16$  of the width.

#### IV-5. Longitudinal Stiffeners

The longitudinal stiffener, if used, shall be located at  $1/5$  of the web depth from the inner surface or leg of the compression flange component. The thickness of the longitudinal stiffener shall not be less than  $(b'/f_b)/(2250)$ . where,  $b'$  is the width of stiffeners. and  $f_b$  is the calculated compressive bending stress in the flange. Also, the stiffener will be so proportioned that its moment of inertia,  $I$ , taken about its edge in contact with the web plate shall be not less than

$$I = D t_w^3 (2.4 d_o^2/D^2 - 0.13)$$

where,  $D$  = the unsupported distance between flange components,  
in inches.

$t_w$  = the thickness of the web plate, in inches.

$d_o$  = the actual distance between transverse stiffeners,  
in inches.

It is of interest to compare the economics of two-sided and one-sided stiffeners. In FIG.4.2.a, neglecting the thickness of the web, the moment of inertia of the two-sided arrangement is  $I = [(2b)^3 t]/12 = (2b^3 t)/3$ . In FIG.4.2.b, the moment of inertia of the one-sided stiffener about the edge in contact with the web plate is  $I' = [(b')^3 t']/3$ . Letting  $t' = t$  and  $I' = I$ ,  $b'$  is found to be  $1.26b$ . Thus, the area of the first arrangement is  $2bt$  and that of the second arrangement is  $1.26bt$ , indicating that the use of a one-sided stiffener requires only 63% of the area required for a two-sided stiffener when only stiffener moment of inertia is required.

The longitudinal stiffeners need not be continuous and may be cut at their intersections with the transverse stiffeners.



## V. DESIGN EXAMPLES

### V-1. Specification and Data

Using the 1969 AASHTO specification and M183 (A36) steel design an interior stringer for a 70-foot span simply supported highway bridge. (FIG.1.1)

Using:

- (A) Beam Bridge. (Lateral support provided at 25 foot intervals)
- (B) Beam Bridge. (Continuous lateral support to compression flange)
- (C) Composite Bridge. (Rolled beams; no shoring)
- (D) Composite Bridge. (Rolled beams; light cover plates; no shoring)
- (E) Composite Bridge. (Rolled beams; heavy cover plates; no shoring)
- (F) Built-Up Beam Bridge. (Rolled beams; light cover plates)
- (G) Built-Up Beam Bridge. (Rolled beams; heavy cover plates)
- (H) Plate Girder Bridge. (Welded plate girders)

Design data:

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,  $f'_c$ ..... 3,000 psi.  
 M183 (A36) steel,  $F_y$ ..... 36,000 psi.  
 6" slab.  
 6' - 6" center to center stringers.

The drawing (FIG.1.1) indicates a construction joint between the slab and curb; the curb and the sidewalk are cast after the slab is cured.

V-2. Case 1. Beam Bridge.(Lateral support provided at 25 feet intervals)

(The AASHO code section 1.7.22. requires that lateral bracing be spaced at intervals not to exceed 25 feet)

Design of an interior stringer

Estimated loads:

Dead loads applied after slab has hardened

Two rails = 20 psf x 2/5 = 8.0 plf

Two sidewalks and curbs = 154.0 plf

Future paving 26 x 15 x 1/5 = 78.0 plf

---

Total = 240.0 plf

Dead loads applied before slab has hardened

$$\text{Slab weight } 6.5 \times 0.5 \times 150 = 490 \text{ plf}$$

$$\text{Transverse diaphragms} = 30 \text{ plf}$$

$$\text{Slab coping, or haunch} = 20 \text{ plf}$$

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$$\text{Total} = 540 \text{ plf}$$

$$\text{Estimated stringer weight} = 260 \text{ plf}$$

---


$$\text{Total} = 800 \text{ plf}$$

Dead load moments

$$M_d = (1/8) \times 240 \times 70^2 = 147000 \text{ ft-lb}$$

$$M_D = (1/8) \times 800 \times 70^2 = 490000 \text{ ft-lb}$$

Live loads

The live load distribution factor in this case is

$$(6.5)/(5.5) = 1.18 \quad (\text{AASHO code section 1.3.1.(B)})$$

The live load moment

$$M_L = 492800 \times 1.18 = 582000 \text{ ft-lb} \quad (\text{AASHO Appendix A})$$

Impact coefficient

$$I = (50)/(L+125) = (50)/(70+125) = 0.256 < 30\% \quad (\text{AASHO 1.2.12c})$$

The maximum impact moment is

$$M_I = 582000 \times 0.256 = 149000 \text{ ft-lb}$$

$$\begin{aligned} \text{Total moment} &= 147000 + 490000 + 582000 + 149000 \\ &= 1368000 \text{ ft-lb} \end{aligned}$$

Allowable compressive stress

$$\begin{aligned} F_b &= 20000 - 7.5 \times (L/b)^2 = 20000 - 7.5 \times (25 \times 12 / 16.551)^2 \\ &= 17535.9 \text{ psi.} \end{aligned}$$

Where,  $b$  = flange width, in inches for a W36x260.

Section modulus required

$$S_{\text{req'd}} = (1368000 \times 12) / (17535.9) = 936.0 \text{ in}^3$$

Try W36x260 (AISC page 2-7)

Which has a  $S = 952.0 > 936.0 \text{ in}^3$

Check shear

$$\text{D.L. shear} = wL/2 = (240 + 800) \times 70 / 2 = 36400 \text{ lb.}$$

$$\text{L.L. shear} = (62400/2) \times 1.18 = 36800 \text{ lb.}$$

$$\text{Impact} = 36800 \times 0.256 = 9420 \text{ lb.}$$

$$\text{Total shear} = 82620 \text{ lb.}$$

$$f_v = (V_{\text{Design}}) / (t_w D) = (82620) / (0.841 \times 36.24) \\ = 2710.8 < 12000 \text{ psi.} \quad (\text{O.K.})$$

Check minimum depth-to-span ratio

$$(36.24) / (12 \times 70) = 0.0431 > 0.04 (=1/25) \quad (\text{O.K.})$$

Use W36x260

V-3. Case 2. Beam Bridge. (Continuous lateral support to the compression flange)

Design of an interior stringer.

All the loads are the same as previous case, except that the estimated stringer weight = 230 plf. This gives a

$$M_D = (1/8) \times 770 \times 70^2 = 472000 \text{ ft-lb.}$$

$$\begin{aligned} \text{Total moment} &= 147000 + 472000 + 582000 + 149000 \\ &= 1350000 \text{ ft-lb.} \end{aligned}$$

Allowable stress = 20000 lb.

Section modulus required

$$S_{\text{req'd}} = (1350000 \times 12) / (20000) = 810.0 \text{ in}^3$$

Try W36x230 (AISC page 2-7)

Which has a  $S = 837.0 > 810.0 \text{ in}^3$

Check shear

$$\text{D.L. shear} = (240 + 770) \times 70 / 2 = 35350 \text{ lb.}$$

$$\text{L.L. + I shear as previous case} = 46220 \text{ lb.}$$

$$\text{Total shear} = 35350 + 46220 = 81570 \text{ lb.}$$

$$f_v = (81570) / (0.761 \times 35.88) = 2987.4 < 12000 \text{ psi. (O.K.)}$$

Check minimum depth-to-span ratio

$$(35.88) / (12 \times 70) = 0.0427 > 0.04 (=1/25) \quad (\text{O.K.})$$

Use W36x230

V-4. Case 3. Composite Bridge. (Rolled beams; no shoring)

Design of an interior stringer.

Effective flange width

$$(a) L/4 = (70 \times 12)/4 = 210.0 \text{ in.}$$

$$(b) \text{ Center to center of stringers} = 6' - 6" = 78 \text{ in.}$$

$$(c) 12 \times 6" = 72 \text{ in.} \quad (\text{Control})$$

All the loads are the same as case 2, except that the estimated stringer weight = 200 plf. This gives a

$$M_D = (1/8) \times 740 \times 70^2 = 454000 \text{ ft-lb.}$$

$$\begin{aligned} \text{Total moment} &= 147000 + 454000 + 582000 + 149000 \\ &= 1332000 \text{ ft-lb.} \end{aligned}$$

Using Lothers' increased allowable stress method (28% increase for  $n = 10$  )

Section modulus required

$$S_{\text{req'd}} = (1332000 \times 12) / (1.28 \times 20000) = 625.0 \text{ in}^3$$

Try W36x194

Section properties. (FIG.5.1)

Steel section only.

$$A = 57.2 \text{ in}^2$$

$$d = 36.48 \text{ in.}$$

$$I = 12100 \text{ in}^4$$

$$\bar{y} = d/2 = 18.24 \text{ in.}$$

Composite section. ( $n = 10$ )

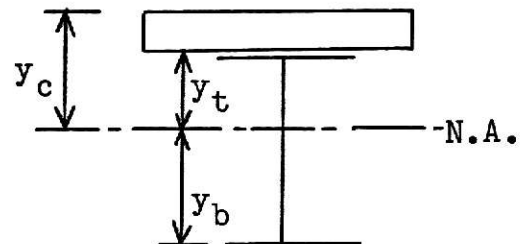


Fig.5.1.

$$A = 6 \times 72 / 10 + 57.2 = 100.4 \text{ in}^2$$

$$y_b = (57.2 \times 18.24 + 43.2 \times 39.48) / (100.4) = 27.38 \text{ in.}$$

$$y_t = d - y_b = 36.48 - 27.38 = 9.1 \text{ in.}$$

$$I_c = 12100 + (57.2) \times (27.38 - 18.24)^2 + (1/12) \times (7.2) \times (6)^3 \\ + (43.2) \times (12.1)^2 = 23333.0 \text{ in}^4$$

$$y_c = y_t + 6 = 9.1 + 6 = 15.1 \text{ in.}$$

Composite section. ( $n' = 30$ )

$$A' = 6 \times 72 / 30 + 57.2 = 71.6 \text{ in}^2$$

$$y'_b = (57.2 \times 18.24 + 14.4 \times 39.48) / (71.6) = 22.51 \text{ in.}$$

$$y'_t = d - y'_b = 36.48 - 22.51 = 13.97 \text{ in.}$$

$$I'_c = 12100 + (57.2) \times (22.51 - 18.24)^2 + (1/12) \times (72/30) \times (6)^3 \\ + (14.4) \times (16.97)^2 = 17333.04 \text{ in}^4$$

$$y'_c = y'_t + 6 = 13.97 + 6 = 19.97 \text{ in.}$$

$$S = I/\bar{y} = 12100/18.24 = 665.0 \text{ in}^3$$

$$S_c = I_c/y_c = 23333.0/15.1 = 1545.0 \text{ in}^3$$

$$S_t = I_c/y_t = 23333.0/9.1 = 2564.0 \text{ in}^3$$

$$S_b = I_c/y_b = 23333.0/27.38 = 852.2 \text{ in}^3$$

$$S'_c = I'_c/y'_c = 17333.04/19.97 = 868.0 \text{ in}^3$$

$$S'_t = I'_c/y'_t = 17333.04/13.97 = 1240.7 \text{ in}^3$$

$$S'_b = I'_c/y'_b = 17333.04/22.51 = 770.0 \text{ in}^3$$

Review of stresses

$$f_c = (M_d)/(n'S'_c) + (M_L)/(nS_c) = (147000 \times 12)/(30 \times 868.0) \\ + (731000 \times 12)/(10 \times 1545.0) = 635.5 < 1200 \text{ psi. (O.K.)}$$

$$f_t = M_D/S + M_L/S_t + M_d/S'_t = (454000 \times 12)/(665.0) + (731000 \times 12)/(2564.0) + (147000 \times 12)/(1240.7) = 13035.4 < 20000 \text{ psi. (O.K.)}$$

$$f_b = M_D/S + M_L/S_b + M_d/S'_b = (454000 \times 12)/(665.0) + (731000 \times 12)/(852.2) + (147000 \times 12)/(770.0) = 20775.7 > 20000 \text{ psi. (O.K.)}$$

(Over stressed 3.87%. It is still acceptable. If we go to the next **heavier** section, it will not be economical.)

Check shear

$$\text{D.L. shear} = (240 + 740) \times 70 / 2 = 34300 \text{ lb.}$$

$$\text{L.L. + I shear as previous case} = 46220 \text{ lb.}$$

$$\text{Total shear} = 80520 \text{ lb.}$$

$$f_v = (80520)/(36.48 \times 0.770) = 2866.54 < 12000 \text{ psi. (O.K.)}$$

Check minimum depth-to-span ratio

$$(36.48 + 6)/(12 \times 70) = 0.0505 > 0.040 (=1/25) \quad (\text{O.K.})$$

$$(36.48)/(12 \times 70) = 0.0434 > 0.0333 (=1/30) \quad (\text{O.K.})$$

Use W36x194

Design of shear connectors.

An estimated 500,000 cycles of maximum stress is assumed to be applied to the shear connectors. At the center line of bearing, the distribution of load to girder G2 is

$$\begin{aligned} & 3.5/6.5 + 3.5/6.5 \\ & = 0.538 + 0.538 \\ & = 1.076 \text{ (FIG.5.2)} \end{aligned}$$

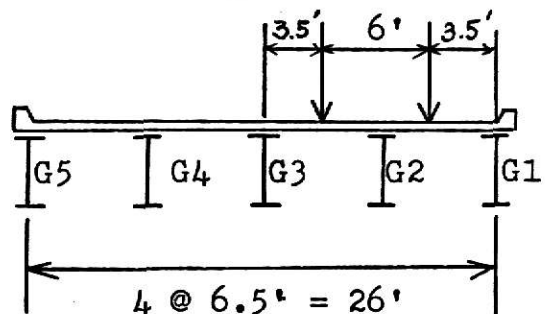


Fig.5.2.



## Vertical shear

At the center line of bearing, with a 16-kip wheel over the bearing, a 16-kip wheel 14 feet on the span and a 4-kip wheel another 14 feet away.

$$V_{LL} = 16 \times 1.076 + [16 \times (70 - 14) + 4 \times (70 - 28)] \times (1.18 / 70) \\ = 35.16 \text{ kips}$$

$$\text{Impact} = [(50) / (70 + 125)] \times 35.16 = 9.02 \text{ kips}$$

---


$$\text{Max } V_{L+I} = 44.18 \text{ kips}$$

At 10 ft from the center line of bearing,

$$V_{LL} = (16 \times 60 + 16 \times 46 + 4 \times 32) \times (1.18) / (70) = 30.75 \text{ kips}$$

$$\text{Impact} = [(50) / (60 + 125)] \times 30.75 = 8.31 \text{ kips}$$

---


$$\text{Max } V_{L+I} = 39.06 \text{ kips}$$

For minimum shear at this section,

$$V_{LL} = -(16 \times 10) \times (1.18) / (70) = -2.70 \text{ kips}$$

$$\text{Impact} = (0.3) \times (-2.70) = -0.81 \text{ kips}$$

---


$$\text{Min } V_{L+I} = -3.51 \text{ kips}$$

At 20 ft from the center line of bearing,

$$V_{LL} = (16 \times 50 + 16 \times 36 + 4 \times 22) \times (1.18) / (70) = 24.68 \text{ kips}$$

$$\text{Impact} = [(50) / (50 + 125)] \times (24.68) = 7.05 \text{ kips}$$

---


$$\text{Max } V_{L+I} = 31.73 \text{ kips}$$

For minimum shear at this section,

$$V_{LL} = -(16 \times 20 + 16 \times 6) \times (1.18) / (70) = -7.01 \text{ kips}$$

$$\text{Impact} = (0.30) \times (-7.01) = -2.01 \text{ kips}$$


---

$$\text{Min } V_{L+I} = -9.11 \text{ kips}$$

At 30 ft from the center line of bearing,

$$V_{LL} = (16 \times 40 + 16 \times 26 + 4 \times 12) \times (1.18) / (70) = 18.61 \text{ kips}$$

$$\text{Impact} = (0.30) \times (18.61) = 5.58 \text{ kips}$$


---

$$\text{Max } V_{L+I} = 24.19 \text{ kips}$$

For minimum shear at this section,

$$V_{LL} = -(16 \times 30 + 16 \times 16 + 4 \times 2) \times (1.18) / (70) = -12.54 \text{ kips}$$

$$\text{Impact} = (0.3) \times (-12.54) = -3.76 \text{ kips}$$


---

$$\text{Min } V_{L+I} = -16.30 \text{ kips}$$

The range of shear at ends = 44.18 kips

The range of shear at 10 ft = 39.06 - 3.51 = 35.55 kips

The range of shear at 20 ft = 31.73 - 9.11 = 22.62 kips

The range of shear at 30 ft = 24.19 - 16.30 = 7.89 kips

Assume 3 - 3/4 x 3 headed studs (H/d=4.0) at each section.

$$Z_r = \alpha d^2 = (10600) \times (3/4)^2 = 5960 \text{ lb/stud.}$$

$$S_u = 930 d^2 \sqrt{f'_c} = (930) \times (3/4)^2 \sqrt{3000} = 28700 \text{ lb/stud.}$$

Ultimate strength design

$$P_1 = A_s F_y = 57.2 \times 36 = 2059 \text{ kips.}$$

$$P_2 = 0.85 f'_c b c = 0.85 \times 3.0 \times 72 \times 6 = 1101.6 \text{ kips. (Control)}$$

$$N = (P) / (\phi S_u) = (1101.6) / (0.85 \times 28.7) = 45.16 \text{ (say 46}$$

each side of maximum moment point)

$$Q = 7.2 \times 6 \times 12.1 = 522.72 \text{ in}^3$$

$$v \text{ at ends} = (VQ)/(I) = (44.18 \times 522.72)/(23333.0) = 0.989 \text{ k/in.}$$

$$\text{Spacing required} = (3 \times 5.96)/(0.989) = 18.07 \text{ in.}$$

$$v \text{ at 10 ft} = (35.55 \times 522.72)/(23333.0) = 0.796 \text{ k/in.}$$

$$\text{Spacing required} = (3 \times 5.96)/(0.796) = 22.46 \text{ in.}$$

$$v \text{ at 20 ft} = (22.62 \times 522.72)/(23333.0) = 0.506 \text{ k/in.}$$

$$\text{Spacing required} = (3 \times 5.96)/(0.506) = 35.33 \text{ in.}$$

$$v \text{ at 30 ft} = (7.89 \times 522.72)/(23333.0) = 0.177 \text{ k/in.}$$

$$\text{Spacing required} = (3 \times 5.96)/(0.177) = 101.0 \text{ in.}$$

Assuming the following spacing diagram. (FIG.5.3)

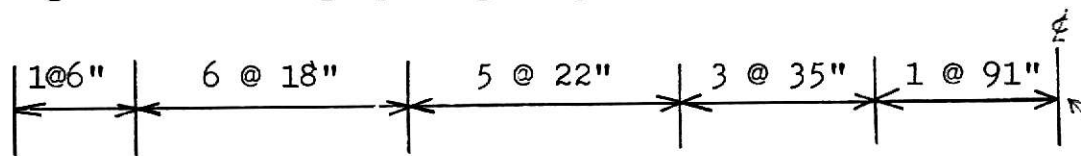
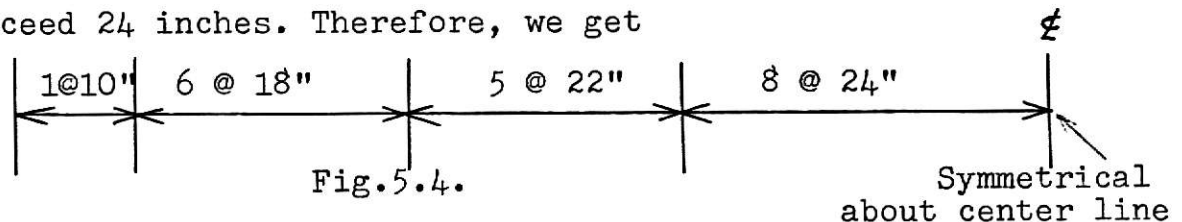


Fig.5.3. Symmetrical about center line

Total number of shear connectors =  $3 \times 16 = 48$  on each side of  $\phi$   $> 46$  required by ultimate design. The AASHO code also requires that the maximum pitch of shear connectors shall not be exceed 24 inches. Therefore, we get



Use  $3 \times 20 = 60$  connectors each side of  $\phi$  with one placed at beam  $\phi$  .(FIG.5.4) The AASHO code would also allow these to be spaced uniformly.

V-5. Case 4. Composite Bridge. (Rolled beams; light cover plate; no shoring)

(The area of a light cover plate is approximately equal to one half of the area of the tension flange.)

Design of an interior stringer.

Loads are the same as previous case.

$$M_d = 147000 \text{ ft-lb.}$$

$$M_D = 454000 \text{ ft-lb.}$$

$$M_L = 582000 + 149000 = 731000 \text{ ft-lb.}$$

Try W36x150 with cover plate 11.0" x 0.75"

Properties of the section. (FIG.5.5)

Rolled beam only.

$$A = 44.2 \text{ in}^2$$

$$d = 35.84 \text{ in.}$$

$$I = 9030.0 \text{ in}^4$$

Steel section.

$$A = 11.0 \times 0.75 + 44.2 = 52.45 \text{ in}^2$$

$$y' = [11.0 \times 0.75 \times 0.75 / 2 + 44.2 \times (35.84 / 2 + 0.75)] / 52.45$$

$$= 15.79 \text{ in.}$$

$$y = 35.84 + 0.75 - 15.79 = 20.80 \text{ in.}$$

$$I_p = (1/12) \times (11.0) \times (0.75)^3 + (11.0 \times 0.75) \times (15.79 - 0.75/2)^2$$

$$+ 9030 + (44.2) \times (35.84/2 - 15.79 + 0.75)^2 = 11357.38 \text{ in}^4$$

$$S_{st} = I_p / y = 11357.38 / 20.80 = 546.03 \text{ in}^3$$

$$S_{sb} = I_p / y' = 11357.38 / 15.79 = 719.28 \text{ in}^3$$

Composite section. (n=10)

$$A = 72 \times 6 / 10 + 52.45 = 95.65 \text{ in}^2$$

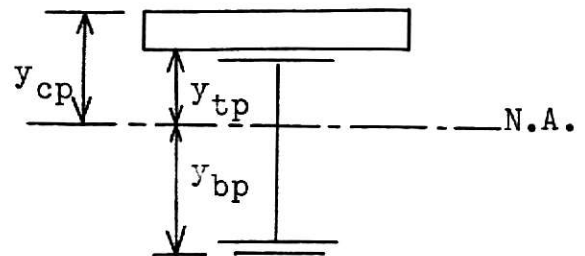


Fig.5.5.

$$y_{bp} = [ 11.0 \times 0.75 \times 0.75 / 2 + (44.2) \times (35.84 / 2 + 0.75) + (43.2) \times (0.75 + 35.84 + 3) ] / (95.65) = 26.54 \text{ in.}$$

$$y_{tp} = 35.84 + 0.75 - 26.54 = 10.05 \text{ in.}$$

$$y_{cp} = 10.05 + 6 = 16.05 \text{ in.}$$

$$I_{cp} = (1/12) \times (11.0) \times (0.75)^3 + (11.0) \times (0.75) \times (26.54 - 0.75/2)^2 + 9030 + (44.2) \times (35.84/2 - 10.05)^2 + (1/12) \times (72/10) \times (6)^3 + (43.2) \times (10.05 + 3)^2 = 24902.67 \text{ in}^4$$

$$S_{cp} = I_{cp} / y_{cp} = 24902.67 / 16.05 = 1551.57 \text{ in}^3$$

$$S_{tp} = I_{cp} / y_{tp} = 24902.67 / 10.05 = 2477.88 \text{ in}^3$$

$$S_{bp} = I_{cp} / y_{bp} = 24902.67 / 26.54 = 938.31 \text{ in}^3$$

Composite section. (n=30)

$$A = 72 \times 6 / 30 + 52.45 = 66.85 \text{ in}^2$$

$$y'_{bp} = [ 11.0 \times 0.75 \times 0.75 / 2 + 44.2 \times (35.84 / 2 + 0.75) + 14.4 \times (0.75 + 35.84 + 3) ] / 66.85 = 20.92 \text{ in.}$$

$$y'_{tp} = 35.84 + 0.75 - 20.92 = 15.67 \text{ in.}$$

$$y'_{cp} = 15.67 + 6 = 21.67 \text{ in.}$$

$$I'_{cp} = (1/12) \times (11.0) \times (0.75)^3 + (11.0) \times (0.75) \times (20.92 - 0.75/2)^2 + 9030 + 44.2 \times (35.84/2 - 15.67)^2 + (1/12) \times (72/30) \times (6)^3 + (14.4) \times (15.67 + 3)^2 = 17799.04 \text{ in}^4$$

$$S'_{cp} = I'_{cp} / y'_{cp} = 17799.04 / 21.67 = 821.37 \text{ in}^3$$

$$S'_{tp} = I'_{cp} / y'_{tp} = 17799.04 / 15.67 = 1135.87 \text{ in}^3$$

$$S'_{bp} = I'_{cp} / y'_{bp} = 17799.04 / 20.92 = 850.81 \text{ in}^3$$

Check stresses.

$$f_c = (M_d) / (n' S'_{cp}) + (M_L) / (n S_{cp}) = (147000 \times 12) / (30 \times 821.37) + (731000 \times 12) / (10 \times 1551.57) = 636.95 < 1200 \text{ psi. (O.K.)}$$

$$\begin{aligned}
 f_t &= M_D/S_{st} + M_L/S_{tp} + M_d/S'_{tp} = (454000 \times 12)/(546.03) \\
 &\quad + (731000 \times 12)/(2477.88) + (147000 \times 12)/(1135.87) \\
 &= 15070.58 < 20000 \text{ psi.} \quad (O.K)
 \end{aligned}$$

$$\begin{aligned}
 f_b &= M_D/S_{sb} + M_L/S_{bp} + M_d/S'_{bp} = (454000 \times 12)/(719.28) \\
 &\quad + (731000 \times 12)/(938.31) + (147000 \times 12)/(850.81) \\
 &= 18996.28 < 20000 \text{ psi.} \quad (O.K)
 \end{aligned}$$

Check shear.

Total shear as previous case = 80520 lb.

$$f_v = (80520)/(35.84 \times 0.625) = 3594.64 < 12000 \text{ psi.} \quad (O.K)$$

Check minimum depth-to-span ratio.

$$(35.84 + 6)/(12 \times 70) = 0.0498 > 0.04 \quad (=1/25) \quad (O.K)$$

$$(35.84)/(12 \times 70) = 0.0427 > 0.0333 \quad (=1/30) \quad (O.K)$$

Check maximum cover plate thickness

$$(1.5) \times (0.940) = 1.41 > 0.75 \text{ in.} \quad (O.K)$$

Length of cover plate.

The length of the cover plate is determined from equation

$$L' = g + (L - g) \sqrt{1 - S_b/S_{bp}}$$

where,  $g$  is the distance from the center of the span to the point of maximum moment,

$$g = (1/2) \times [(14 \times 32 + 28 \times 32)/(72) - 14] = 2.33 \text{ ft.}$$

$$\text{then, } L' = 2.33 + (70 - 2.33) \sqrt{1 - 662/938.31} = 39.0 \text{ ft.}$$

Allowing about 1 ft extra at each end, the actual length of the cover plate becomes 41 ft. The distance from the center of bearing to the theoretical edge of the cover plate is 14.5 ft.

Review of stresses. (FIG.5.6)

$$0.5 \times 4 / 70 = 0.029$$

$$14.5 \times 16 / 70 = 3.314$$

$$28.5 \times 16 / 70 = 6.514$$

---


$$9.857$$

$$(16 \times 2 + 4) - 9.857 = 26.143$$

$$M_L = (26.143 \times 14.5 - 4 \times 14) \times (1.18) = 381.23 \text{ k-ft.}$$

$$\text{Impact} = 381.23 \times 0.256 = 97.59 \text{ k-ft.}$$

---


$$M_{L+I} = 478.82 \text{ k-ft.}$$

$$M_D = (1 - x^2/L^2) M_{D \text{ max}} = (1 - 41^2/70^2) \times (454.0) = 298.28 \text{ k-ft.}$$

$$M_d = (0.657) \times (147) = 96.58 \text{ k-ft.}$$

Omit the calculation, we get the section properties (no cover plate)

$$S = 504.0 \text{ in}^3$$

$$S_b = 662.0 \text{ in}^3$$

$$S'_b = 599.0 \text{ in}^3$$

Bottom flange stress without cover plate.

$$f_s = M_D/S + M_L/S_b + M_d/S'_b = (298280 \times 12)/(504) + (478820 \times 12)/(662) + (96580 \times 12)/(599) = 17716.3 < 20000 \text{ psi. (O.K.)}$$

Design of shear connectors will be the same as previous case, therefore will not be repeated here.

Use W36x150 with cover plate 11.0" x 0.75"

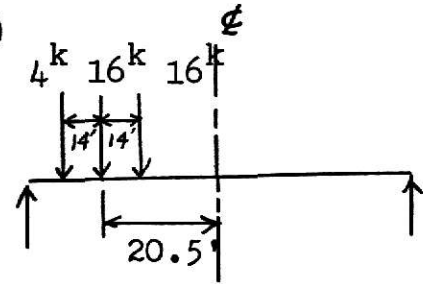


Fig.5.6.

V-6. Case 5. Composite Bridge. (Rolled beams; heavy cover plates; no shoring)

(The area of a heavy cover plate is approximately equal to the full area of the tension flange.)

Design of an interior stringer.

Loads are the same as previous case.

$$M_d = 147000 \text{ lb-ft.}$$

$$M_D = 454000 \text{ lb-ft.}$$

$$M_L = 731000 \text{ lb-ft.}$$

Try W36x135 with cover plate 10.5" x 1.0"

Omit the calculation, we get

$$S_{st} = 489.35 \text{ in}^3$$

$$S_{sb} = 707.02 \text{ in}^3$$

$$S_{cp} = 1523.8 \text{ in}^3$$

$$S_{tp} = 2426.6 \text{ in}^3$$

$$S_{bp} = 938.6 \text{ in}^3$$

$$S'_{cp} = 784.4 \text{ in}^3$$

$$S'_{tp} = 1081.6 \text{ in}^3$$

$$S'_{bp} = 846.6 \text{ in}^3$$

Check stresses.

$$f_c = (M_d)/(n'S'_{cp}) + (M_L)/(nS_{cp}) = (147000 \times 12)/(30 \times 784.4) + (731000 \times 12)/(10 \times 1523.8) = 650.63 < 1200 \text{ psi. (O.K)}$$

$$f_t = M_D/S_{st} + M_L/S_{tp} + M_d/S'_{tp} = (454000 \times 12)/(489.35) + (731000 \times 12)/(2426.6) + (147000 \times 12)/(1081.6) = 16378.99 < 20000 \text{ psi. (O.K)}$$

$$f_b = M_D/S_{sb} + M_L/S_{bp} + M_d/S'_{bp} = (454000 \times 12)/(707.02) + (731000 \times 12)/(938.6) + (147000 \times 12)/(846.6) = 19135.04 < 20000 \text{ psi. (O.K)}$$



Check shear.

Total shear as previous case = 80520 lb.

$$f_v = (80520)/(35.55 \times 0.598) = 3787.59 < 12000 \text{ psi. (O.K)}$$

Check minimum depth-to-span ratio.

$$(35.55 + 6)/(12 \times 70) = 0.0495 > 0.040 (=1/25) \quad (\text{O.K})$$

$$(35.55)/(12 \times 70) = 0.0423 > 0.0333 (=1/30) \quad (\text{O.K})$$

Check maximum cover plate thickness.

$$(1.5) \times (0.794) = 1.191 > 1.0 \text{ in. (O.K)}$$

Use W36x135 with cover plate 10.5" x 1.0"

V-7. Case 6. Built-Up Beam Bridge. (Rolled beams; light cover plates)

(Assume continuous lateral support to the compression flange.)

Design of an interior stringer.

Loads are the same as previous case, except that the estimated stringer weight = 230 plf. this gives a

Total moment  $M_T = 1350000$  ft-lb.

Allowable stress = 20000 psi.

Section modulus required,

$$S_{\text{req'd}} = (1350000 \times 12) / (20000) = 810.0 \text{ in}^3$$

Try W33x152 with 1 - 14.0" x 0.75" cover plate each flange.

$$I = 8160 + (2) \times (1/12) \times (14) \times (0.75)^3 + (2) \times (14) \times (0.75) \times (33.5/2 + 0.75/2)^2 = 14319.56 \text{ in}^4 \quad (\text{FIG. 5.7})$$

$$S = (14319.56) / (33.5/2 + 0.75) = 818.26 > 810.0 \text{ in}^3 \quad (\text{O.K.})$$

Check shear.

$$\text{D.L. shear} = (240 + 770) \times (70/2) = 35350.0 \text{ lb.}$$

$$\text{L.L. + I. shear as case 1} = 46220.0 \text{ lb.}$$

$$\text{Total shear} = 81570.0 \text{ lb.}$$

$$f_v = (81570) / (33.5 \times 0.635) = 3834.53 < 12000 \text{ psi.} \quad (\text{O.K.})$$

Check minimum depth-to-span ratio.

$$(35.0) / (12 \times 70) = 0.0417 > 0.04 \quad (=1/25) \quad (\text{O.K.})$$

Check maximum cover plate thickness.

$$(1.5) \times (1.055) = 1.583 > 0.75 \text{ in.} \quad (\text{O.K.})$$

Use W33x152 with 1 - 14.0" x 0.75" cover plate each flange.

Weight = 152 + 71.4 = 223.4 plf. (AISC page 1-113)

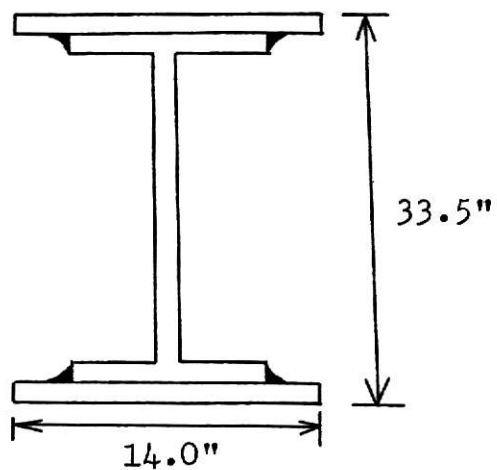


Fig.5.7.

V-8. Case 7. Built-Up Beam Bridge. (Rolled beams; heavy cover plates)

(Assume continuous lateral support to the compression flange.)

Design of an interior stringer.

Loads are the same as previous case, except that the estimated stringer weight = 240 plf. This gives a total moment  $M_T = 1356000$  ft-lb.

Allowable stress = 20000 psi.

Section modulus required,

$$S_{\text{req'd}} = (1356000 \times 12) / (20000) = 814.0 \text{ in}^3$$

Try W33x130 with 1 - 14.0" x 1.0" cover plate each flange.

$$I = 6710 + (2) \times (1/12) \times (14) \times (1.0)^3 + (2) \times (14) \times (1.0) \times (33.10/2 + 1.0/2)^2 = 14852.0 \text{ in}^4$$

$$S = (14852.0) / (33.1/2 + 1.0) = 846.26 > 814.0 \text{ in}^3 \quad (O.K)$$

Check shear.

$$\text{D.L. shear} = (240 + 780) \times (70/2) = 35700.0 \text{ lb.}$$

$$\text{L.L. +I. shear as case 1} = 46220.0 \text{ lb.}$$

$$\text{Total shear} = 81920.0 \text{ lb.}$$

$$f_v = (81920.0) / (33.1 \times 0.580) = 4267.1 < 12000 \text{ psi.} \quad (O.K)$$

Check minimum depth-to-span ratio.

$$(35.10) / (12 \times 70) = 0.0418 > 0.040 \quad (=1/25) \quad (O.K)$$

Check maximum cover plate thickness.

$$(1.5) \times (0.855) = 1.283 > 1.0 \text{ in.} \quad (O.K)$$

Use W33x130 with 1 - 14.0" x 1.0" cover plate each flange.

Weight = 130 + 95.2 = 225.2 plf. (AISC page 1-113)

V-9. Case 8. Plate Girder Bridge. (Welded plate Girder)

(Continuous lateral support to the compression flange.)

Use cross frames at 25' intervals. In order to make a fair comparison with other cases, no transverse and longitudinal stiffeners are used.

Design of an interior stringer.

Loads are the same as before, except the estimated stringer weight = 170 plf. This gives a total moment

$$M_T = 1313000 \text{ ft-lb.}$$

Shear.

$$\text{D.L. shear} = wL/2 = (240 + 710) \times (70/2) = 33250 \text{ lb.}$$

$$\text{L.L. + I. shear as case 1} = 46220 \text{ lb.}$$

$$\text{Total shear} = 80000 \text{ lb.}$$

$$\text{Allowable bending stress} = 20000 \text{ psi.}$$

$$\text{Allowable shearing stress} = 12000 \text{ psi.}$$

$$S_{\text{req'd}} = (1313000 \times 12) / (20000) = 788.0 \text{ in}^3$$

(A) Assume  $t_w = 5/8"$ , then

$$\begin{aligned} t_w &= D \sqrt{f_v} / (7500) = (D \sqrt{V / (D t_w)}) / (7500) \\ &= (D \sqrt{80000 / (t_w D)}) / (7500) \quad (\text{AASHTO Sect. 1.7.72}) \end{aligned}$$

$$D = (7500)^2 t_w / (80000)$$

$$D = 703 t_w^3 = 703 \times (5/8)^3 = 171 \text{ in.}$$

$$\text{Also, } t_w = 150 \times 5/8 = 93.75" \quad (93" \text{ used})$$

Check shear.

$$f_v = (80000)/(93 \times 5/8) = 1376.3 < 12000 \text{ psi. (O.K.)}$$

$$\text{Assume } t_f = 1''$$

$$C = 93/2 + 1 = 47.5''$$

$$I_{\text{req'd}} = S_{\text{req'd}} \times C = 788 \times 47.5 = 37430.0 \text{ in}^4$$

$$I_{\text{web}} = bh^3/12 = (5/8) \times (93)^3 \times (1/12) = 41893.5 > 37430.0 \text{ in}^4$$

So, we know that  $t_w$  must be less than  $5/8''$

$$(B) \text{ Assume } t_w = 9/16'' \quad (\text{FIG. 5.8})$$

$$D = 703 \ t_w^3 = 703 \times (9/16)^3 = 125''$$

$$\text{Also, } D = 150 \ t_w = 84.375'' \quad (\text{Use } 84'')$$

Check shear.

$$f_v = (80000)/(84 \times 9/16) = 1693.1 < 12000 \text{ psi. (O.K.)}$$

$$\text{Assume } t_f = 1''$$

$$C = 84/2 + 1 = 43''$$

$$I_{\text{req'd}} = 788 \times 43 = 33884.0 \text{ in}^4$$

$$I_{\text{web}} = bh^3/12 = (1/12) \times (9/16) \times (84)^3 = 27783 \text{ in}^4$$

$$I_{\text{flange}} = I_{\text{req'd}} - I_{\text{web}} = 33884.0 - 27783.0 = 6101.0 \text{ in}^4$$

$$2y = 84 + (1+1)/2 = 85''$$

$$A_f = 6101/3613 = 1.69 \text{ in}^2 \quad (\text{AISC page 2-127})$$

$$b/t_f = 23 \quad (\text{AASHTO 1.7.70 page 112})$$

$$A_f = b t_f$$

$$t_f = A_f/b = A_f/23 t_f$$

$$t_f^2 = A/23$$

$$t_f = \sqrt{A/23} = \sqrt{1.69/23} = 0.271'' \quad (0.5'' \text{ used})$$

$$b = 1.69/0.5 = 3.38'' \quad (4'' \text{ used})$$

$$\begin{aligned}
 \text{Weight} &= W_{\text{web}} + W_{\text{flange}} \\
 &= 161 + 2 \times 6.8 \\
 &= 174.6 \text{ plf.}
 \end{aligned}$$

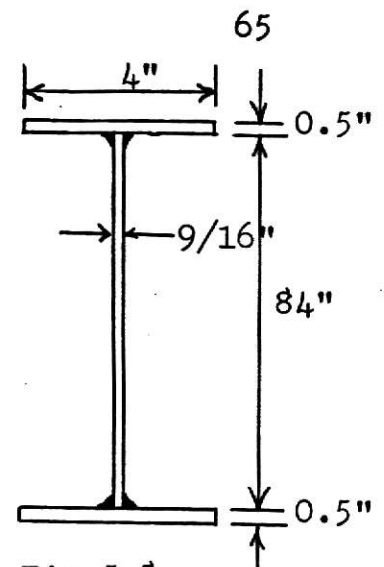


Fig. 5.8.

(C) Assume  $t_w = 0.5"$  (FIG. 5.9)

$$D = 703 \times (0.5)^3 = 87.875"$$

$$\text{Also, } D = 150 \times 0.5 = 75" \quad (\text{used})$$

Check shear.

$$f_v = (80000) / (0.5 \times 7.5) = 2133.3 < 12000 \text{ psi.} \quad (\text{O.K.})$$

$$\text{Assume } t_f = 1"$$

$$C = 75/2 + 1 = 38.5"$$

$$I_{\text{req'd}} = 788 \times 38.5 = 30338.0 \text{ in}^4$$

$$I_{\text{web}} = bh^3/12 = (1/12) \times (0.5) \times (75)^3 = 17578.0 \text{ in}^4$$

$$I_{\text{flange}} = I_{\text{req'd}} - I_{\text{web}} = 30338 - 17578 = 12760.0 \text{ in}^4$$

$$2y = 75 + (1+1)/2 = 76"$$

$$A_f = 12760 / 2888 = 4.42 \text{ in}^2 \quad (\text{AISC page. 2-126})$$

$$t_f = \sqrt{A_f / 23} = \sqrt{4.42 / 23} = 0.44" \quad (0.5" \text{ used})$$

$$b = 4.42 / 0.5 = 8.84" \quad (9.0" \text{ used})$$

$$\begin{aligned}
 \text{Weight} &= 128 + 30.6 \\
 &= 158.6 \text{ plf.}
 \end{aligned}$$

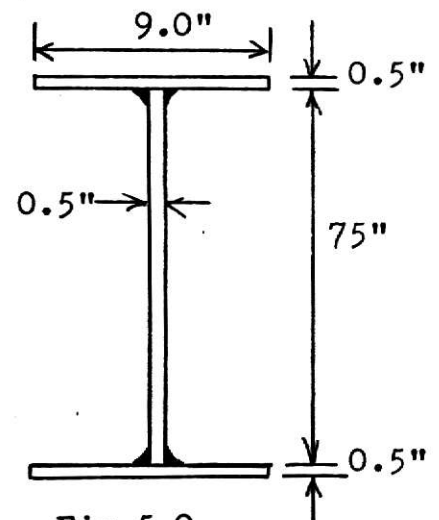


Fig. 5.9.



(D) Assume  $t_w = 7/16"$  (FIG. 5.10)

$$D = 703 \times (7/16)^3 = 58.87" \quad (58" \text{ used})$$

$$\text{Also, } D = 150 \times 7/16 = 65"$$

Check shear.

$$f_v = (80000)/(58 \times 7/16) = 3152.7 < 12000 \text{ psi. (O.K.)}$$

$$\text{Assume } t_f = 1"$$

$$C = 58/2 + 1.0 = 30"$$

$$I_{req'd} = 788 \times 30 = 23640.0 \text{ in}^4$$

$$I_{web} = bh^3/12 = (7/16) \times (58)^3 / (12) = 7113.5 \text{ in}^4$$

$$I_{flange} = 23640 - 7113.5 = 16526.5 \text{ in}^4$$

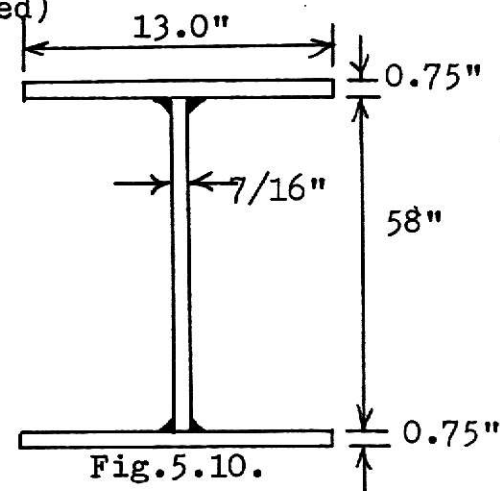
$$2y = 58 + (1+1)/2 = 59"$$

$$A_f = 16526.5 / 1741.0 = 9.49 \text{ in}^2 \quad (\text{AISC page. 2-127})$$

$$t_f = \sqrt{A_f / 23} = \sqrt{9.49 / 23} = 0.64" \quad (0.75" \text{ used})$$

$$b = 9.49 / 0.75 = 12.65" \quad (13" \text{ used})$$

$$\begin{aligned} \text{Weight} &= 86.3 + 66.4 \\ &= 152.7 \text{ plf.} \end{aligned}$$



(E) Assume  $t_w = 3/8"$  (FIG. 5.11)

$$D = 703 \times (3/8)^3 = 37" \quad (\text{used})$$

$$\text{Also, } D = 150 \times 3/8 = 56.2"$$

Check shear.

$$f_v = (80000)/(37 \times 3/8) = 5765.7 < 12000 \text{ psi. (O.K.)}$$

$$\text{Assume, } t_f = 1"$$

$$C = 37/2 + 1 = 19.5"$$

$$I_{req'd} = 788 \times 19.5 = 15366.0 \text{ in}^4$$

$$I_{web} = bh^3/12 = (3/8) \times (37)^3 / (12) = 1582.9 \text{ in}^4$$

$$I_{flange} = 15366.0 - 1582.9 = 13783.1 \text{ in}^4$$

$$2y = 37 + (1+1)/2 = 38"$$

$$A_f = 13783.1 / 722 = 19.09 \text{ in}^2 \quad (\text{AISC page. 2-126})$$

$$t_f = \sqrt{A_f / 23} = \sqrt{19.09 / 23} = 0.911" \quad (1" \text{ used})$$

$$b = 19.09 / 1.0 = 19.09" \quad (20.0" \text{ used})$$

$$\text{Weight} = 47.2 + 2 \times 68.0$$

$$= 183.2 \text{ plf.}$$

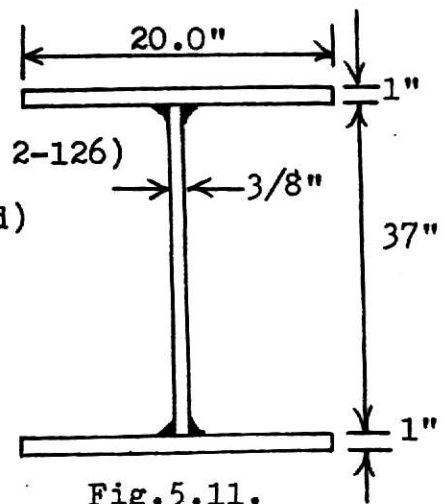


Fig. 5.11.

Compare:

$t_w$	$D_w$	$t_f$	$b_f$	Weight
9/16"	84"	0.5"	4.0"	174.6 plf.
1/2"	75"	0.5"	9.0"	158.6 plf.
7/16"	58"	0.75"	13.0"	152.7 plf.
3/8"	37"	1.0"	20.0"	183.2 plf.

The lightest section  
will be used.

Review of stresses:

$$\text{Dead load moment} = (1/8) \times (540 + 160) \times (70)^2 = 429000 \text{ ft-lb.}$$

$$\text{Total moment} = 147000 + 731000 + 429000 = 1307000 \text{ ft-lb.}$$

$$I = (7/16) \times (58)^3 / (12) + (2) \times (13) \times (0.75) \times (29.375)^2 \\ + (2) \times (13) \times (0.75)^3 / (12) = 23940.74 \text{ in}^4$$

$$S = 23940.74/29.75 = 804.7 \text{ in}^3$$

$$f_b = (1307000 \times 12)/(804.7) = 19490.5 < 20000 \text{ psi. (O.K)}$$

$$f_v = (80000)/(58 \times 7/16) = 3152.7 < 12000 \text{ psi. (O.K)}$$

Check minimum depth-to-span ratio

$$(59.5)/(12 \times 70) = 0.0708 > 0.040 (=1/25) \quad (\text{O.K})$$

#### V-10. Design of Exterior Stringers

The design of the exterior stringers does not differ materially from that for the interior stringers, and details will not be presented here.

#### V-11. Conclusions:

Using the simple beam bridge (case 1) as a reference structure, compare the results.

Section	Depth.(in)	Percent.	Weight.(plf)	percent
Case 1. W36x260	36.24	100.00%	260.0	100.00%
Case 2. W36x230	35.88	99.01%	230.0	88.46%
Case 3. W36x194	36.48	100.66%	194.0	74.62%
Case 4. W36x150 cp.11.0"x0.75"	36.59	100.97%	178.1	68.50%
Case 5. W36x135 cp.10.5"x1.0"	36.55	100.86%	170.7	65.65%
Case 6. W33x152 cp.14.0"x0.75"	35.00	96.58%	223.4	85.92%
Case 7. W33x130 cp.14.0"x1.0"	35.10	96.85%	225.2	86.62%
Case 8. Plate Girder	59.50	164.18%	152.7	58.73%

From the comparison, we see that

(A) For the simple beam bridge (case 1 and 2), the design is relatively easy and so is the construction. If continuous lateral support is provided, then the stringer would weigh less than it would with little lateral support. The amount of steel saving is 11%.

(B) For the composite beam design (case 3,4 and 5), with or without cover plate, the amount of weight saving may vary from 25% to 34%. Saving in material, however, does not necessarily mean saving in total cost, if the fabrication cost is high. With the advent of automatic welding and development of ASTM A36 steel which permits use of normal procedures for welding of thick cover plates to the beam, the unit cost of fabrication has been reduced considerably. Stud shear connectors can also be easily and swiftly welded in place in shop or field. This is another reason that the composite design becomes more and more popular.

(C) In this problem, due to the limitations of depth-to-span (no less than  $1/25$ ) and the maximum thickness of cover plate (no greater than 1.5 times the thickness of the flange to which the cover plate is attached), the built-up beams have gained only little advantage. The cost of welding cover plates to flanges would possibly rule out such solutions.

(D) In order to make a fair comparison with other cases, no transverse and longitudinal stiffeners are used. Even so, the plate girder bridge saves 41% of the steel. Meanwhile, as the result of steel saving, the depth is increased by 64%.

When designing bridges, designers generally follow the age old axiom: least weight-least cost. With today's spiraling labor rates, the axiom is not necessarily true. Indeed, for example, the plate girder bridge would save 41% of the steel, but would the saving in material balance the high shop fabrication costs? Also, if the plate girder bridge is used, the depth is going to increase by 64%. A good bridge layout must take into account the geographical and geological conditions of the site, also, the clearance requirements and the aesthetic proportions. Therefore, a good designer should look into all these influential factors before he makes any decision.

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Robert T. Lee

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

by

ROBERT TEH HSIN LEE

Diploma, Taiwan Provincial Taipei Institute of Technology  
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AN ABSTRACT OF A MASTER'S REPORT

submitted in partial fulfillment of the  
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A relatively small proportion of all bridges built today 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 civil engineers.

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.

The beam bridge 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.

The composite bridge derives its popularity from its relatively low cost, ease and speed of construction, clean appearance, and simplicity of design. Simple span composite highway bridges have been economically used for spans from 50 feet to 120 feet.

The plate girder bridge is suitable for spans from 80 feet and up. For longer spans the plate girder bridge begins to compete very well economically.

Each of these different types of bridges have its own advantages and disadvantages, therefore, a good designer should take into account all the factors which influence a particular bridge construction, such as the geographical and geological conditions of the site, the clearance requirements and the aesthetic proportions, before he makes any decision.