A COMPARATIVE ANALYSIS OF
ALUMINUM STEEL AND WOOD
AS STRUCTURAL MATERIALS

By

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A Thesis

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I. INTRODUCTION

It is of prime importance for structural designers to not only understand the mechanics of design; but to also understand the relative merits of the materials which are available for design.

In the literature of today, material is beginning to appear that leads one to believe that all designs are economically feasible with all materials. This, of course, is too broad a statement to be completely true.

The purpose of this thesis is to investigate the relative merits for use in heavy structures of three materials. Namely, wood, steel and aluminum.

Wood is chosen because it is the oldest known structural material. A material which has only recently been rejuvinated in the heavy structural field by the discovery of the high frequency glueing process for laminated members.

Steel is chosen because it is the most commonly and widely used material in the field today as well as being the material which has grown up with civilization. Its development follows very closely that of civilization. Steel is used as the basis for comparing the other materials.

Aluminum is chosen because it is the new, young material in the structural field. Its development has been extremely rapid and it is now bidding for a place in the design of structures with heavy live loads.

The use of all three of these materials in members with simple tension, compression and bending is well known. The choice of material for such a member being based upon availability, cost and weight of member. It being generally understood that the lower specific gravity of aluminum and wood make them very desirable where dead load is a high percentage of the total load.

The desirability of these materials in replacing steel in structures where the live load predominates is in question. It is with this aspect that this thesis treats.

(1) METHOD OF APPROACH

After careful consideration of several posibilities of approach to the problem of comparing these materials deck plate girder bridges were decided upon.

Deck Plate girders were chosen because:

(a) Girders are a well known standard form of heavy structure.

(b) Girders embody the features of crumpling and buckling which are the major features that cause one design to vary from another in a manner other than the variation of allowable stress.

(c) On girders of this type live load is the predominating feature in determining stress; dead load being a very small percentage of the total load.

(d) Aluminum and wooden girders for this type of loading are very rare and little is known of them.

(2) METHOD OF PRESENTATION

The facts concerning Steel, Aluminum and Wood are presented in Chapters I, II and III.

Gider designs of Rivited Steel, Welded Steel, Aluminum and Wood are in Chapters V, VI, VII and VIII.

Supplementary Specifications to the A.R.E.A. specifications for Aluminum are in the appendices.

The conclusions based on this material are in Chapter IV.

This method of presentation was chosen as being the most convenient for the reader.

References are listed by number in the Bibliography and are indicated in the text by use of these numbers.

Standard specifications are assumed available to all interested readers and are referenced accordingly.

II. A BRIEF HISTORY OF THE MATERIALS

(1) Wood

Wood is the oldest structural material known to man. This is true because of two things:

1. Because of its accessibility on the face of the earth.
2. Because of its workability; it could be cut and shaped even by the crude instruments of stone of early man.

The difference in allowable stress with and against the grain combined with its limited size; that of the largest tree; made wood structrually adaptable only to small members.

It is very interesting to note that the method by which wood has finally come into the heavy structural field was used by the Egyptians.

In the tombs of the Pharaoh's have been found examples of laminated wood. This wood was used as an inlay for beautification and down through the centuries cabinet makers have used laminated wood.

This laminated wood gradually became the plywood of today.

Laminated wood was still not ready as a heavy structural material because the glue with which laminates are made was set by exposure to the air. Laminates of over one inch were not practical and even then the glue between each lamination took a long time to set.

Uncertainty as to the efficiency of setting of the glue plus large cost of manufacture made these heavy structural members prohibitive.

During the second world war the high frequency gluing method was discovered whereby laminations of any number and any thickness may be glued at once into a single structural member. Improvement of glues to a point where they are stronger than the wood itself plus judicious positioning of laminations results in a member whose allowable stresses may be made equal in all planes. Wood again has entered the heavy structural field.

(2) STEEL

The use of iron like that of wood is traceable back before the dawn of history. Also like wood, many hundreds of years passed before any significant change in the manufacture of iron occured.

Iron has been used down through the ages from the times of the Bible down to the present day to make cutting instruments and weapons of war. Iron instruments were used in the building of the pyramids. Today iron is the most widely used metal on the face of the earth.

The art of producing crucible steel was known and practiced in ancient India. It was then forgotten for hundreds of years until in 1442. Benjamin Huntman rediscovered it. This process with only minor variations is still used in many countries today.

In 1784 Henry Cort patented a process for converting pig iron into wrought iron. This was the first fundamental change in the iron industry for many centuries[1].

In 1885 Sir Henry Bessemer invented the Bessemer process for making steel. This made possible for the first time the making of steel on a large scale and maybe listed among the outstanding discoveries of the present time insofar as effect on engineering is concerned.

(3) ALUMINUM

Aluminum, the most abundant of all the metallic elements found in the earth's crust[2] unlike steel and wood was not discovered until recent times.

In 1825 Hans Christian Oersted announced to the Royal Danish Academy of Sciences that he had obtained aluminum by gently heating aluminum chloride and potassium amalgam. This was the first time that anyone had succeeded in freeing aluminum from the compounds it occurs in in the earth's crust.

In the next sixty years the discovery of cheaper reducing agents reduced the price of aluminum from $545 a pound to $17 a pound but the production was still only a few tons per year.

In 1886 Charles Martin Hall, at that time only twenty-two years of age, succeeded in separating aluminum by the electrolytic process. Working independently Paul Louis Toussaint Heroult, discovered the same process. He was also only twenty-two years of age.

The discovery of the electrolytic process enabled the production of aluminum on the large scale upon which it is produced today.

III DESIGN CONSIDERATION

(1) Physical Properties

The physical properties of steel are taken where applicable from the American Railway Engineering Association Specifications for Steel Railway Bridge, 1941. Those of aluminum from Specifications for the Design and Fabrication of Structures of Alcoa Aluminum Alloy 61S-T as recommended by the Aluminum Company of America. Those for wood from the National Design Specification for Stress Grade Lumber and its Fastenings 1944 as modified by Bulletin VK-8, Glued Laminated Specifications, January 10, 1949 by Timber Structures Inc.

Weight:

Aluminum = 169 pounds per cubic foot

Steel = 490 pounds per cubic foot

Wood = 60 pounds per cubic foot (Approx)

Modulus of Elasticity:

Aluminum = 10,000,000 lbs. per sq. in.

Steel = 30,000,000 lbs. per sq. in.

Wood = 1,800,000 lbs. per sq. in.

Coefficient of Expansion:

Aluminum = 0.000 013 Per degree F

Steel = 0.000 0065 Per degree F

Poisson's Ratio

Aluminum = 0.33

Steel = 0.30

Allowable Unit Stresses:

Axial Tension (Net Section)

Aluminum = 16,000 p.s.i.

Steel = 18,000 p.s.i.

Tension in extreme fibers of rolled shapes, girders and built sections, subject to bending.

Aluminum = 16,000 p.s.i.

Steel = 18,000 p.s.i.

Stress in extreme fibers of pins

Aluminum = 24,000 p.s.i.

Steel = 27,000 p.s.i.

Shear in power driven rivets

Steel = 13,500 p.s.i.

Aluminum

Cold driven 61S-T = 10,000 p.s.i.

Hot driven S35 = 8,000 p.s.i.

Shear in turned bolts

Aluminum = 10,000 p.s.i.

Steel = 11,000 p.s.i.

(2) Special Considerations

Axial Compression (Gross Section)

Stiffeners in plate girders

Steel = 18,000 p.s.i.

Intermediate stiffeners:

If the depth of the web between the flanges or side plates of a plate girder exceeds 60 times its thickness, it shall be stiffened by pairs of angles welded to the web. The clear distance between stiffeners shall not exceed 72 inches nor that given by the formula:

$$d = \frac{255,000t}{S} \left(\frac{st}{a}\right)^{1/3}$$

d = clear distance between stiffeners in inches

t = thickness of web in inches

a = clear depth of web between flanges or side plates in inches

s = unit shearing stress, gross section, in web at point considered.

Aluminum =

When s/h = or less than 0.4 $\qquad I_s = 6.13\ t^3h$

When s/h = is greater than 0.4 $\qquad I_s = \dfrac{t^3h}{5(5/h4)}$

(s/h)^2 plus 0.625)

Where

I_s = required moment of inertia of stiffener in inches^4

t = thickness of web in inches

s = required stiffener spacing as given by formula for critical shear buckling stress

h = clear height of web in inches

Critical shear buckling stress = $\dfrac{51{,}000{,}000}{\left(\frac{b}{t}\right)^2}$

b = unsupported width of plate in inches

t = thickness of plate in inches

Centrally loaded columns

Steel

$\frac{L}{r}$ not greater than 140

Riveted ends = $15{,}000 - 1/4\ \dfrac{L^2}{r^2}$

Pin ends $= 15,000 - 1/3 \frac{L^2}{r^2}$

L = length of member in inches

r = least radius of gyration of member in inches

Aluminum

$\frac{L}{r}$ equal to or less than 100

$\frac{p}{a}$ $17,000 - 100 \frac{L}{r}$ but not to exceed 15,000 p.s.i.

Where $\frac{L}{r}$ equals the greatest slenderness ratio of the member

Compression in extreme fibers of rolled shapes, girders and built sections, subject to bending.

Steel

For values of L/b not greater than 40

$s = 18,000 - 5 \frac{L^2}{b^2}$

L = length in inches of unsupported flange between lateral connections of knee braces

b = flange width in inches

Aluminum

Basic allowable compressive stress as given alone shall govern except providing that the equivalent radius of gyration of the compression flange is determined in accordance with the following formula:

Equivalent radius of gyration of compression flanges =

$$\frac{0.2}{S_c}\left(I_1\left(J(KL)^2 \text{ plus } 13.1\, I_f\, d^2\right)\right)^{1/2}$$

Where

S_c = section modulus for beam about axis normal to web (compression side) in inches to the third power.

I_1 = moment of inertia for beam about the principal axis parallel to web in inches to the fourth power.

L = laterally unsupported length of compression flange in inches.

K = factor representing end conditions of laterally unsupported length.

I_f = moment of inertia of compression flange of beam about axis parallel to web (may be assumed equal to 1/2 of I_1 in case of I shaped members having both flanges alike) in inches to the fourth power.

d = depth of beam in inches

J = torsion factor in inches to the fourth power.

Where

J = the sum of 1/3 bt^3 (See ALCOA Spec. 3)

b = length of each separate rectangle in member in webs

t = thickness of each separate rectangle.

Horizontal stiffeners

Steel

(Not necessary)

Aluminum

Horizontal stiffeners shall have a radius of gyration not less than that given by the following formula:

$$r = c_r \left(\frac{h}{t}\right)^2 \quad f \times 10\text{-}9$$

r = required radius of gyration of one stiffener in inches.

h = clear height of web in inches

t = web thickness in inches

f = compressive stress at toe of flange angles in p.s.i.

c_r = a coefficient which depends upon the ratio of the spacing of the vertical stiffener, s, to the clear height of the web, h. Values of c_r found in Table IV of Specifications.

Deflection:

Assume an equivalent uniform load such that the moment at the center equals 73,700,000 in. lb.

$$73{,}700{,}000 \quad \frac{wL^2}{8}$$

$$w = \frac{73{,}700{,}000 \times 8}{840 \times 840}$$

$$= 825 \text{ pounds per foot}$$

Steel:

Steel:

Deflection

$$= \frac{5wL^4}{384EI} \quad \text{Average } I = 170{,}400 \text{ in}^4$$

$$= \frac{5(825)\ (840)^4}{384 \times 30 \times 10^6} \times 170{,}400$$

$$= 1.08 \text{ inches}$$

Aluminum:

Deflection

$$= \frac{5wL^4}{384\ EI} \qquad I = 267{,}700 \text{ in.}^4$$

$$= \frac{5(825)\ (840)^4}{384 \times 10 \times 10^6} \times 267{,}700$$

$$= 1.95 \text{ inches}$$

Wind Load:

Steel = 640 pounds per foot

Aluminum = 732 pounds per foot

Cost:

Steel (See Bill of Materials)

Riveted 67,205.8 pounds @ $10\frac{1}{2}$¢ = \$7,056.61

Welded 62,594.3 pounds @ $10\frac{1}{2}$¢ = \$6,572.40

Aluminum (See Bill of Materials)

34,365 pounds @ 37¢	= $12,715.05

Wood

40,320 F.B.M. @ 255.50	= $10,300
Freight to Lansing	= 800
Creosote	= 1,740
Total	$12,840

(3) Maintenance

Steel:

Prime Coat - Red Lead

Cost - $4.00 per gallon

Coverage - 1,000 sq. ft.

Finish Coat - Aluminum Paint

Cost - $6.00 per gallon

Coverage - 1,000 sq. ft.

Aluminum:

Prime Coat - Red Lead

Cost - $4.00 per gallon

Coverage - 1,000 sq. ft.

Finish Coat - Aluminum paint

Cost - $6.00 per gallon

Coverage - 1,000 sq. ft.

IV CONCLUSIONS

The conclusions that follow include a consideration of the past, the comparrisons of the present, and a tentative look into the future. The facts are presented as they exist today and as nearly correct as available information can make them. The suggestions as to the future are the author's own and are presented only as such.

Wood has as yet not reached the stage when it is practical to build structures with heavy, predominamt live loads of this material. Fabrication and cost are still such as to make feasibility of such construction economically unsound.

It is interesting to note, however, that an allowable bending stress of 3000 pounds per square inch is now allowed for laminated members which is about one third greater than that quoted in any hand book published to date.

At the same time, the method of design is the simple method which was probably the first case of bending stress design learned by the student of structures. Namely, a rectangular beam with known bending moment, shear and allowable stress whith a sinple substitution in the flexural formula $S = \frac{Mc.}{I}$

Thus in at least one case progress has brought simplification rather than complication.

The main comparison is left to one between aluminum and steel.

Stress comparison shows aluminum to be eight-ninths as strong as steel except when fully supported in which case it is of the same strength.

Bearing this fact in mind an investigation of other properties gives an indication of the major points of difference between the two materials.

Aluminum weighs approximately one-third as much as steel and has a modulus of elasticity of one-third also while its Poisson's Ratio is very nearly the same.

A modulus of elasticity of one-third means an expansion or contraction due to stress of three times that of steel while its expansion due to temperation is twice as great.

This is the factor which causes aluminum design to be more complicated than steel as well as making it necessary to use much more aluminum, in relation to its allowable stress than steel.

For a given load and the same cross-section of material aluminum will change in length three times as much as steel. This makes aluminum three times as susceptible to crumpling and buckling as steel. In the case of change of length due to temperature the factor is two instead of three but still in favor of steel.

This results in the use of more and heavier vertical stiffeners plus the use of horizontal stiffeners for aluminum design. Steel of course does not require horizontal stiffeners.

In the case of column formulas aluminum in addition to using a lower allowable to start from also uses a much more conservative formula than does steel for the same situation. This also is directly traceable to greater change in length of aluminum than of steel under the same load and cross-section.

Aluminum uses Johnson's straight line formula while steel uses Johnson's formula of the second degree. The steel formula plots much closer to the stress-l/r curve for values of l/r less than 100.

Aluminum uses the formula for equivalent radius of gyration of compression flanges to design these flanges. This formula gives a less conservative cross-section for lighter loads which gradually approaches the cross-section found by conventional methods such as the flange area method and then becomes more conservative as the loads increase.

The deflection of the steel girder is 1.08 inches while the deflection of the aluminum girder is 1.95 inches when the aluminum girder is one foot greater in depth than the steel.

The determination of the deflection of the aluminum girder which is kept within an allowable amount by increasing the depth must be balanced against the increased need for stiffeners with increased depth.

The deflection will be the major control factor but care in keeping it close to the maximum allowable will allow the use of the minimum amount of stiffening.

(The deflection of the wooden girders will be 5 inches for the same case which is again prohibitive.)

The wind load will be slightly greater on the aluminmium girder because of its greater depth. However, minimum angles are used in both cases because the load is relatively light. Therefore, there is no real difference between the two materials.

The first cost of the aluminum girder is twice that of the steel. The unit cost of the aluminum being three and one-half times that of steel.

In spite of the generally accepted opinion that aluminum maintenance has a comparatively low cost because it will not corrode; the facts indicate differently.

The Aluminum Company of America itself in its specifications recommends painting exposed aluminum structures. The recommended primer and finish coats are exactly the same as recommended for steel. One paint company may recommed a different paint than another but for both aluminum and steel. If anything, the aluminum structure is more expensive because of its slightly greater surface area.

It is true that aluminum is nonreactive with water. However, it is just as reactive (some authorities say more reactive) with acids and caustics, as steel. The predominance of manufacturing in all areas of the United States means that there will be more or less caustics and acids in the air at all localities. Therefore protection in the form of paint is necessary.

Anadizing and bonderizing simply make the surfaces of aluminum and steel more acceptible to paint. A locality that requires this treatment for steel also requires it for aluminum.

Fabrication of both materials is done in the same shops. As the shops were originally set up for steel fabrication and the same machinery is used for aluminum there is very little difference in cost.

What of the future of these materials? In order to suggest what might happen in the future it is necessary to consider the past.

Steel was developed over a period of many centuries. Attempts have been made to use it for every conceivable purpose. Its usability has been pretty much standarized. This should not be construed as meaning that there will be no progress in steel but only that it will probably be a steady gradual progress.

Steel has huge facilities for experimentation with a monetary backing that is almost unlimited. This has been true for the last half century.

Aluminum, on the other hand, has had only one century of use. During this century it has made extremely rapid advances. However, there are many fields where its usability has not been thoroughly tried. The finances available for experimentation has been only a fraction of that of steel. It is finding a place in a market already held by other materials. It must not only prove its practicability but also that is has greater practicability than an established material.

Aluminum has approximately the same availability in the earth's crust as steel. It is almost a sure thing that better and cheaper ways of mining and processing it will be developed. It is just as sure that where it is more practicable, the world will accept its use and it will replace the materials now used.

Aluminum has the advantage of being only one-third as heavy as steel while it has about the same strength.

The time will come when progress in mining and processing methods as well as in usability will decrease the cost of aluminum to a point where it will be just as economical to use as steel.

At the present time the conclusion drawn is that aluminum while it is practicable as a material for structures that have a predominance of live loads, it is not as yet more practicable than steel.

V DECK PLATE GIRDER RAILROAD BRIDGE (RIVETED STEEL)

Data and Specifications

Single Track

Span = 70'-0"

According to A.R.E.A. "Specifications for Steel Railway Bridges" 1941

Live Load		Coopers Standard E-72 Loading
Alternate Load	=	2-90,000# Axels
		Spaced 7'-0" c to c

c to c of main girders

(Spec. 103) = 1/15 x 70 = 4.66' less than 6'6"

Use 7'-0"

Design of Ties

Live Load on each tie at each rail = $1/3 \times 90,000/_2$ = 15,000#. No Impact (Spec. 301).

Assume wgt. of floor (Ties, guard rails, steel rails fastenings) = 700# per tie concentrated at rails.

Total concentration = 15,000 plus 350 = 15,350

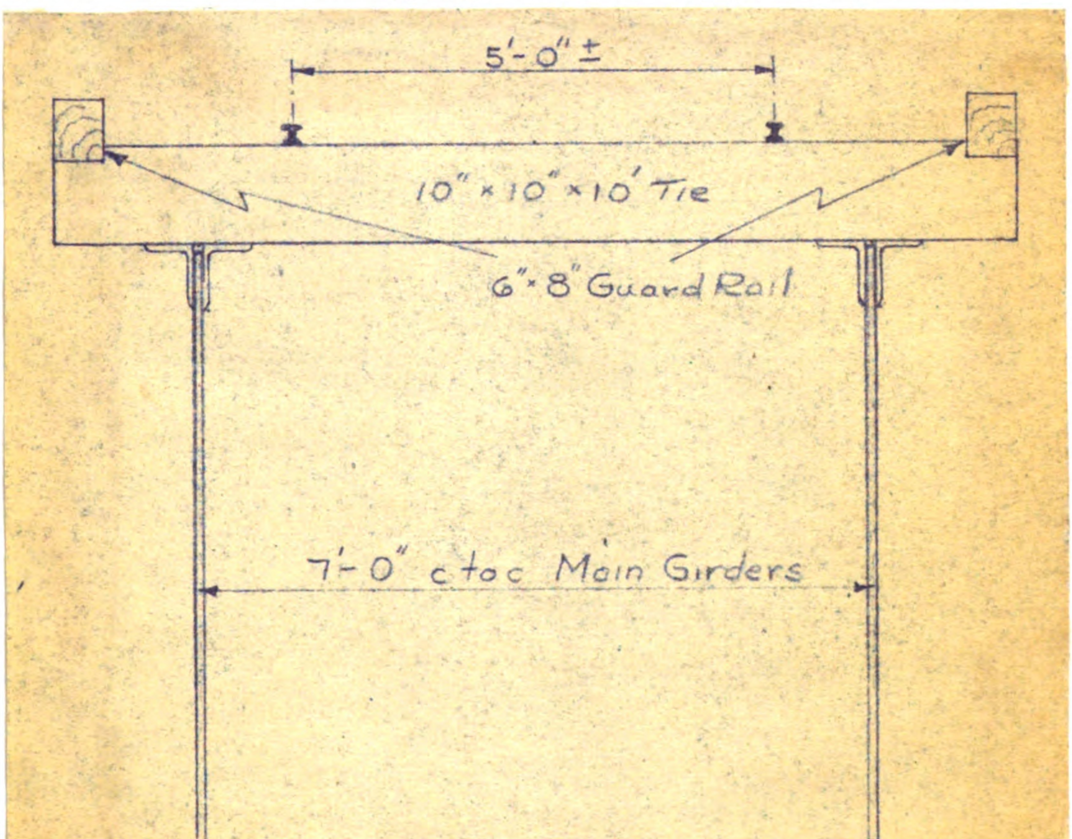


Fig. 1

$$M_{max} = 15,350 \times \frac{(7.0 - 5.0)}{2} \times 12 = 184,000 \text{ in.\#}$$

$$M = S \frac{I}{c} = f \frac{bh^2}{6}$$

S_{w_t} = 1500#/Sq. In. (Spec 301) Yellow pine

$$bh^2 = \frac{184,000 \times 6}{1500} = 738 \text{ in}^3$$

Use a 10" x 10" Tie

$$bh^2 = 10 \times 10^2 = \underline{1000} \text{ in}^3$$

Length Tie = 10'-0") (Spec. 109)
Spacing = 4")

<u>Wgt of F loor per Tie</u>

$$\frac{10 \times 10}{12 \times 12} \times 10 \times 60 = 417\#$$

$$\text{Guard Rails} = 2 \times \frac{6 \times 8}{144} \times 60 \times \frac{14}{12} = 47\#$$

$$\text{Steel Rails \& Fastenings} = 200 \times \frac{14}{12} = 233\#$$

417 plus 47 plus 233 = 697#

Assumed = 700# OK

<u>Design of Main Girder</u>

$$\text{Wgt of floor per lineal foot} = \frac{12}{14} \times 697 = 598\#$$

Formula for assumed wgt of girder and 1/2 bracing exclusive of the floor system =

$$w = \frac{k}{2} \text{ (12.5L plus 100)}$$

$$w = \frac{1.22}{2} \text{ (12.5 x 70 plus 100)} = 615\#$$

L = Span in feet

k = 1.22 for E-72 loading

Total D.L. per foot of girder = 615 plus $\frac{697}{2}$ = 964

Use 1000#/ft.

Max Moments & Shears

(See loading tables in appendix F, Sutherland & Bowman, Structural Design) Cooper E-10 Loading x 7.2 = Cooper E-72 Loading. Max. L. L. Moment under WH 13 at center line of span

Moment about wh 18 = $\frac{2508.5}{2}$ x 7.2 = 9031 Kip ft.

Wgt of wheels = $\frac{84}{2}$ x 7.2 302.4 Kips

c.g. = $\frac{9031}{302.4}$ = 30' = at wheel 13

Place wheel 13 at center line of span

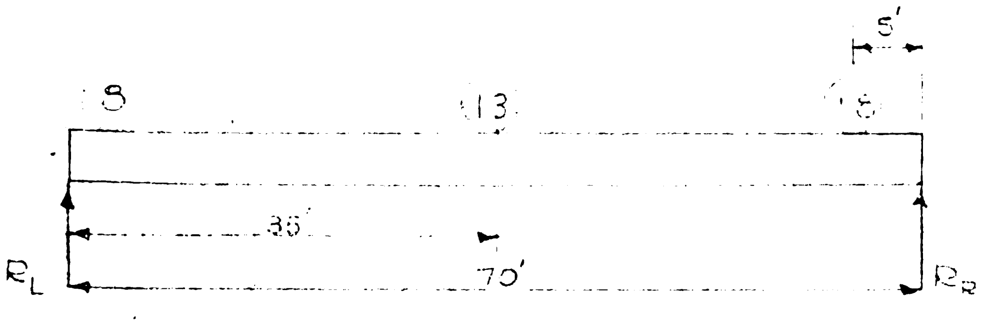


Fig. 2

M_{R_R} = 9031 plus 302.4 x 5

= 9031 plus 1512

= 10,543 Kip ft.

$R_L = \frac{10,543}{70} = 150.6$ Kips

MMax = 150.6 x 35 - 305.25 x 7.2

= 5271 - 2198 = 3073 Kip ft.

MAX MOMENTS IN KIP-FEET

Distance from Support, Ft.	10	20	30	35
Dead Load	300	500	600	612.5
Live Load	1,554	2,528	3,006	3,073.0
Impact	1,212	1,970	2,344	2,398.0
Total	3,066	4,998	5,959	6,083.5
Total In.-Kips	36,792	60,000	71,410	73,000.0

Dead Load Moments:

$R_L = 1000 \times 35 = 35^k$

$M_{@10'} = 35 \times 10 - 10 \times 1 \times 5 = 300$ Kip-ft.

$M_{@20'} = 35 \times 20 - 20 \times 1 \times 10 = 500$ Kip-ft.

$M_{@30'} = 35 \times 30 - 30 \times 1 \times 15 = 600$ Kip-ft.

$M_{@35'} = 35 \times 35 - 35 \times 1 \times 17.5 = 612.5$ Kip-ft.

Live Load Moment

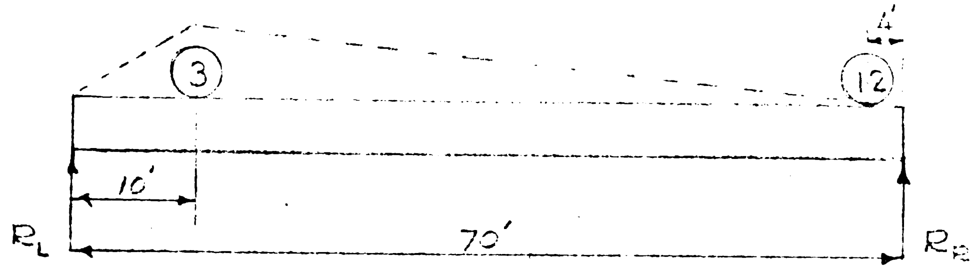


Fig. 3

Max $M_{@10}$ (From Table 2 Max Moment - WH 3)

$$M_{a_{12}} = \frac{3009}{2} \times 7.2 = 10,825 \text{ Kip.-ft.}$$

Sum of the weight of wheels $= \frac{96.0 - 5.0}{2} \times 7.2 = 327.9^k$

$M_{R_R} = M_{12}$ plus 327.9×4

$= 10,825$ plus 1311

$= 12,136$

$$R_L = \frac{12,136}{70} = 173.4^k$$

$$\text{Max } M_{a_{10'}} = 173.4 \times 10 - \frac{50}{2} \times 7.2$$

$$= 1734 - 180$$

$$= 1554 \text{ Kip-ft.}$$

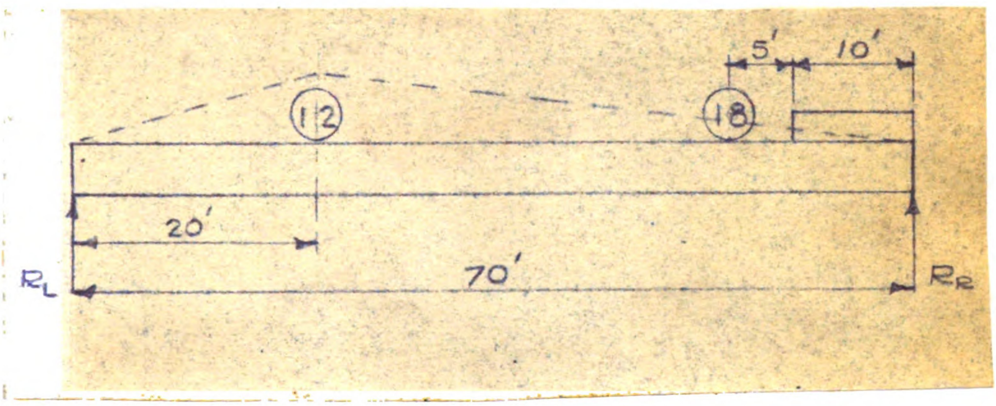


Fig. 4

Max $M_{@_{20'}}$ (From table 2 Max Moment -WH 12)

$$M_{18} = \underline{1748} \times 7.2 = 6,280 \text{ Kip-ft.}$$

$$\text{Sum of Wgt. of wheels} = \frac{142-71}{2} \times 7.2 = 256^k$$

$$M_{R_R} = M_{18} \text{ plus } 256 \times 15 \text{ plus } 10 \times \frac{7.2}{2} \times 5$$

$$= 6280 \text{ plus } 3838 \text{ plus } 180$$

$= 10{,}298$ Kip-ft.

$$R_L = \frac{10{,}298}{70} = 147.1^k$$

$$M_{@20'} = 147.1 \times 20 - M_{12}$$

$$= 2942 - \frac{115}{2} \times 7.2$$

$$= 2942 - 414$$

$$= \underline{\underline{2528}} \text{ Kip-ft.}$$

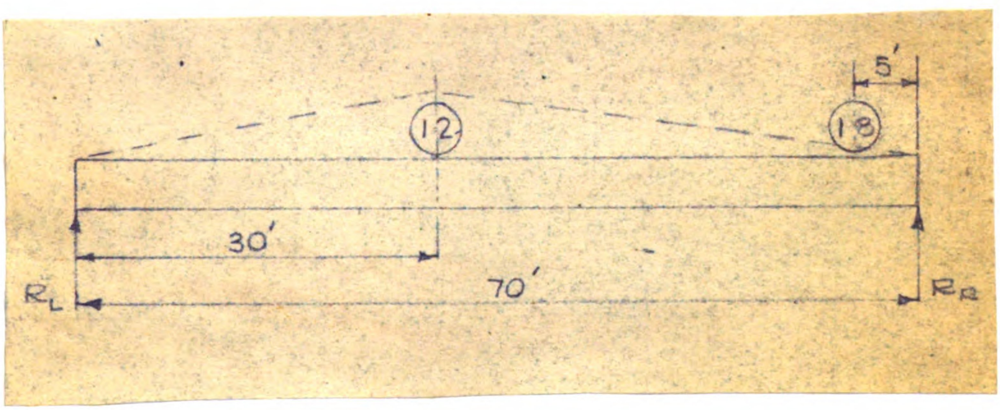


Fig. 5

Max $M_{@30'}$ (From Table 2 Max Moment - WH 12)

$$M_{@18} = \frac{2508.5}{2} \times 7.2 = 9040 \text{ Kip-ft.}$$

Sum of Wgt. of Wheels $= \frac{142-58}{2} \times 7.2 = 302.2^k$

$M_{R_R} = M_{18}$ plus 302.2×5

$= 9040$ plus 1511

$= 10,551^{k1}$

$R_L = \frac{10,551}{70} = 150.7^k$

$M_{@30} = 150.7 \times 30 - M_{12}$

$= 4521 - \frac{420.5}{2} \times 7.2$

$= 4521 - 1515$

$= 3006$ Kip-ft.

Impact: (AREA)

Rolling Effect = 20% = .20

Direct Vertical Effect 100-.6 x 70 = 58% = <u>.58</u>
(See A.R.E.A. Spec. 206(b))

Total .78

Impact:

@ 10' = 1554 x .78 = 1212 Kip-ft.

@ 20' = 2528 x .78 = 1970 Kip-ft.

@ 30' = 3006 x .78 = 2344 Kip-ft.

@ 35' = 3073 x .78 = 2398 Kip-ft.

Max Shears in 1,000 Lb.

Dist. Fm. Support In Feet	0	10	20	30	35
Dead Load	35.0	25.0	15.0	5.0	0
Live Load	199.1	150.1	109.0	71.7	55.3
Impact	155.4	117.2	85.1	55.9	43.0
Total	389.5	292.3	209.1	132.6	98.3

Dead Load (W = 1000#/1)

At 0 $V_0 = \frac{1 \times 70}{} = 35^k$

At 10' V10' = 35 - 10 = 25^k

At 20' V10' = 35 - 20 = 15^k

At 30' V30' = 35 - 30 = 5^k

At 35' V35' = 35 - 35 = 0

Live Load Shears (Max Shear when WH 2 is at Point for all cases)

Vo

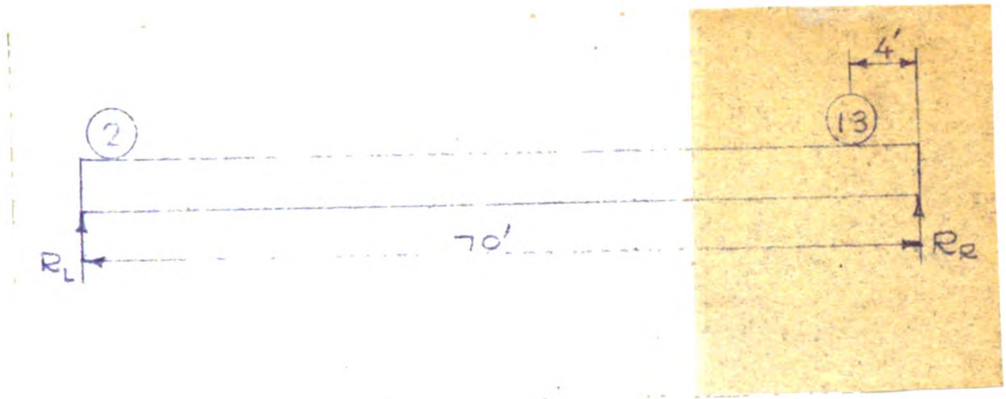


Fig. 6

M_{13} = 3464 x 7.2 = 12,480 Kip-ft.

Sum of Wgt. of Wheels = $\underline{106-5}$ x 7.2 = 363.9^k

M_{R_R} = M_{13} plus 363.9 x 4

= 12,480 plus 1,455

= 13,935 Kip-ft.

Vo = R_L = $\frac{13935}{70}$ = 199.1^k

$V_{10'}$

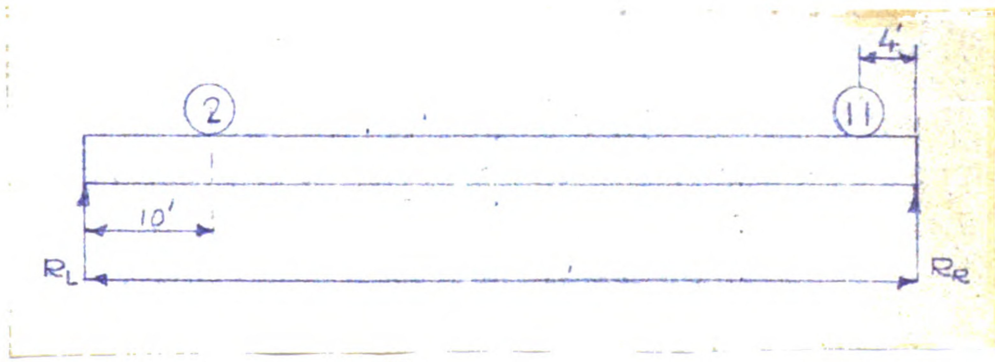


Fig.7

$$M_{11} = \frac{2942}{2} \times 7.2 = 10,530 \text{ Kip-ft.}$$

Sum of wgt of wheels = 309.8

$$M_{R_R} = M_{11} \text{ plus } 309.8 \times 4$$

$$= 10,530 \text{ plus } 1239$$

$$= 11,769 \text{ Kip-ft.}$$

$$Vo = R_L - \frac{5}{2} \times 7.2 = \frac{11,769}{70} - 18.0$$

$$= 168.1 - 18$$

$$= 150.1^k$$

$V_{10'}$

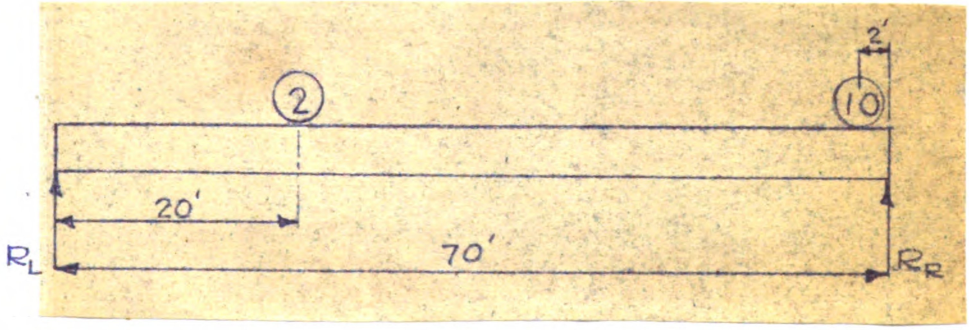


Fig. 8

$$M_{10} = \frac{2316}{2} \times 7.2 = 8340 \text{ Kip-ft.}$$

Sum of Wgt of wheels $= \frac{76}{2} \times 7.2 = 273.6$

$$M_{R_R} = M_{10} \text{ plus } 273.6 \times 2$$

$$= 8340 \text{ plus } 547$$

$$= 8887 \text{ Kip-ft.}$$

$V_{20'} = R_L - 18$

$= \frac{8887}{10} - 18$

$= 127.0 - 18$

$= 109.0^k$

$V_{30'}$

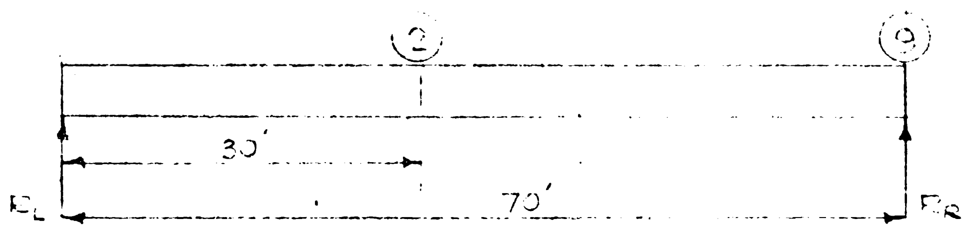


Fig. 9

$M_{R_R} = M_9 = \frac{1748}{2} \times 7.2 = 6{,}280$ Kip-ft.

$V_{30'} = R_L - 18$

$= \frac{6280}{70} - 18$

$= 89.7 - 18$

$= 71.7^k$

$V_{35'}$

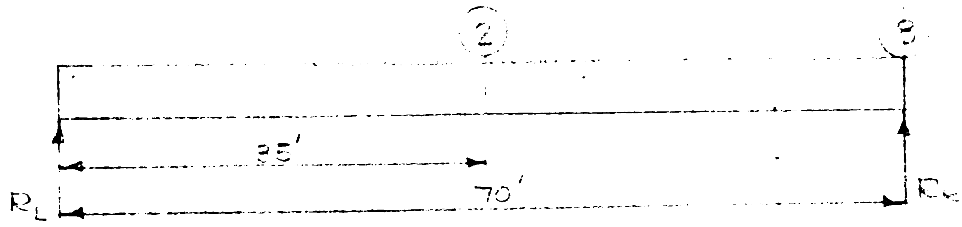


Fig. 10

$$M_{R_R} = M_8 = \frac{1425.5}{2} \times 7.2 = 5,130^{k1}$$

$$V_{35'} = R_L - 18$$

$$= \frac{5130}{70} - 18$$

$$= 73.3 - 18$$

$$= 55.3^k$$

Impact

At 0' = 199.1 x .78 = 155.4^k

At 10' = 150.1 x .78 = 117.2^k

At 20' = 109.0 x .78 = 85.1^k

At 30' = 71.7 x .78 = 55.9^k

At 35' = 55.3 x .78 = 43.0^k

Design of Web

(AREA - Depth not less than $\frac{1}{12}$ L)

$$D = \frac{1}{12} \times 70' = 70''$$

For economy use

$$D = \frac{1}{10} \times 20 \times 12 = 84''$$

Req. Web Area $= \frac{389,500}{11,000} = 35.4$ Sq. In. (Spec 301)

Thickness Req. $= \frac{35.4}{84} = 0.422''$

(t not less than $\frac{1}{170}$ Clear distance between flanges)

(Spec. 431)

Assume vertical legs of flange angles = 6" and distance B to B of flange angles $\frac{1}{2}''$ greater than depth of web.

Therefore: $t = \frac{84.5}{170} = .497''$ <u>Use 1/2"</u>

Design Flanges:

Assume effective depth = 84.5

Max Flange Stress $= \frac{73,000,000}{84.5} = 864,000\#$

Total effective net flange area req. $= \frac{864,000}{18,000} =$

48 Sq. In.

Equivalent flange area of web $= (\frac{1}{8} \times \text{gross area})$

$$= \frac{1}{8} \times 84 \times \frac{1}{2} = 5.24 \text{ Sq. In.}$$

Net area req. in flanges and cover plates = 48-5.24

= 42.76 Sq. In.

Assumed Section:

Section	Gross Area	No. of Rivet Holes	Area of Rivet Holes	Net Area
2 Angles $6 \times 6 \times \frac{7}{8}$	19.46 Sq In	4	3.00 Sq In	16.46 Sq In
2 Cover Plates $15 \times \frac{3}{4}$	22.50 Sq In	4	3.00 Sq In	19.50 Sq In
1 Cover Plate $15 \times \frac{9}{16}$	8.43 Sq In	2	1.13 Sq In	7.30 Sq In
Total	50.39 Sq In			43.26 Sq In

Net area cover plates $= \frac{26.80}{43.26} = .598$ less than $\frac{2}{3}$ OK

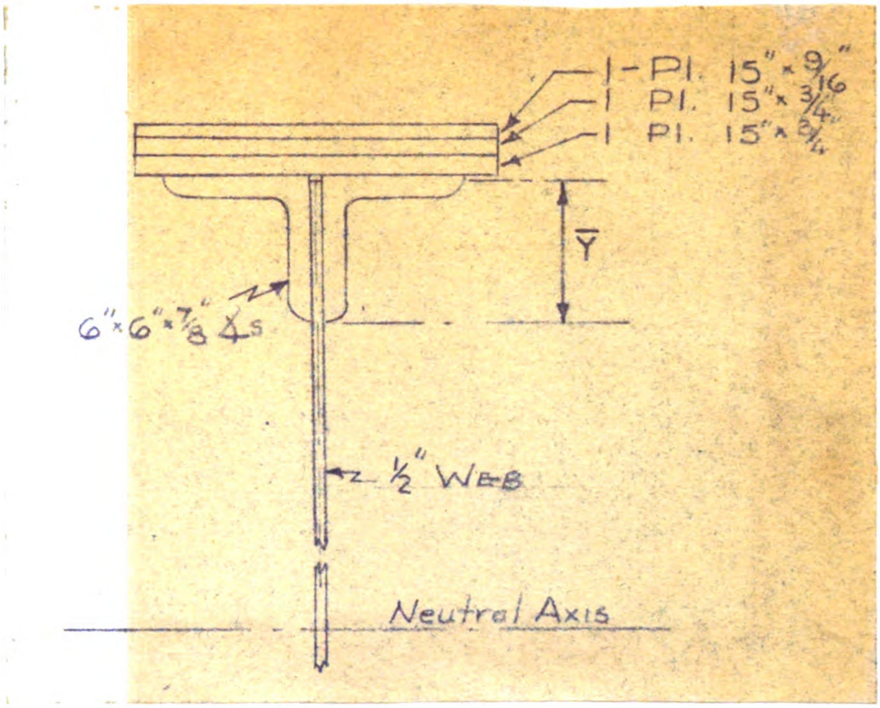


Fig. 11

$$\bar{y} = 7.03 \times 30.93 \text{ plus } 19.46 \times 4.18 = \frac{217.5 \text{ plus } 81.5}{50.39} =$$

$$\frac{299.0}{50.39} = 5.95$$

OK

Use h = 84.4" = 84.5-.05 x 2 = 84.4"

Lengths of Cover Plates

Resisting Moments = M = (A_f plus $1/8A_w$) x h x 18,000

	Net Area s& Covers Sq. In.	$1/8A_W$ Sq In	Total Eff Net Flange Area Sq In	h In	M (In-lb)
2 angles plus 3 Covers	43.26	5.24	48.50	84.4	73,700,000
2 angles plus 2-3/4" Covers	35.96	5.24	41.20	83.7	62,100,000
2 angles plus 1-3/4" Covers	26.21	5.24	31.45	82.5	46,600,000
2 angles	16.46	5.24	21.70	80.9	31,600,000

2 angles plus 3 covers

M 48.50 x 84.4 x 18,000 = 73,700,000 in/lb

2 angles plus 2 - 3/4" covers

$\bar{y} = \dfrac{6.75 \times 22.50 \text{ plus } 81.5}{41.96}$

Base angles

$= \dfrac{153.75 \text{ plus } 81.5}{41.96}$

$= \dfrac{235.25}{41.96} = 5.61"$

Therefore D = 84.5 - 2 x . 39 D = 83.7 Use

= 84.5 - .78

2 angles plus 2 - 3/4" Covers

$M = 41.20 \times 83.7 \times 18{,}000 = \underline{62{,}100{,}000}$ in.-lb.

2 angles plus 1 - 3/4" Cover

$\bar{y}$ Base of angles $= \frac{6.375 \times 11.25 \text{ plus } 81.5}{30.71}$

$= \frac{71.8 \text{ plus } 81.5}{30.71}$

$= \frac{153.3}{30.71}$

$= 5.0"$

$D = 84.5 - 2 \times 1.0$

$= 82.5"$

$M = 31.45 \times 82.5 \times 18{,}000 = \underline{46{,}600{,}000}$ in.-lb.

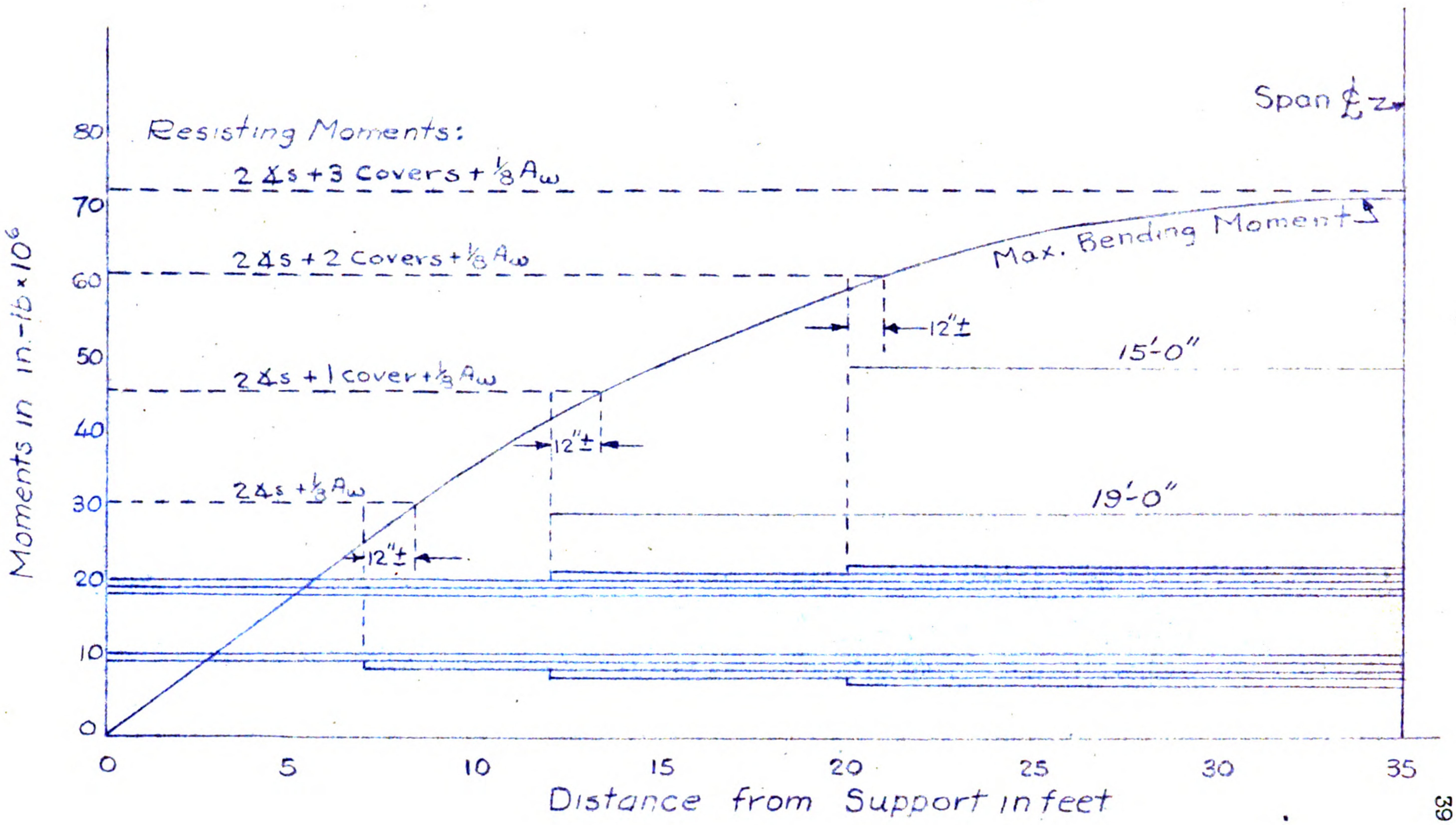
2 angles

D $84.5 - 1.82 \times 2$

$= 84.5 - 3.64$

$= 80.86"$

$M = 21.70 \times 80.9 \times 18{,}000 = \underline{31{,}600{,}000}$ in.-lb.



Span ℄
Resisting Moments:
2 ∡s + 3 Covers + 1/8 Aω
2 ∡s + 2 Covers + 1/8 Aω
2 ∡s + 1 cover + 1/8 Aω
2 ∡s + 1/8 Aω
Max. Bending Moment
12"±
12"±
12"±
15'-0"
19'-0"
Moments in in.-lb × 10^6
0
10
20
30
40
50
60
70
80
0
5
10
15
20
25
30
35
Distance from Support in feet

Summary:

<u>Top</u>

1 - Cover 15" x 9/16" x 30'-0" plusor minus

1-- Cover 15" x 3/4" x 38'-0" plus or minus

1 - Cover 15" x 3/4" x 70'-0" plus or minus

<u>Bottom</u>

1 - Cover 15" x 9/16" x 30'-0" Plus or minus

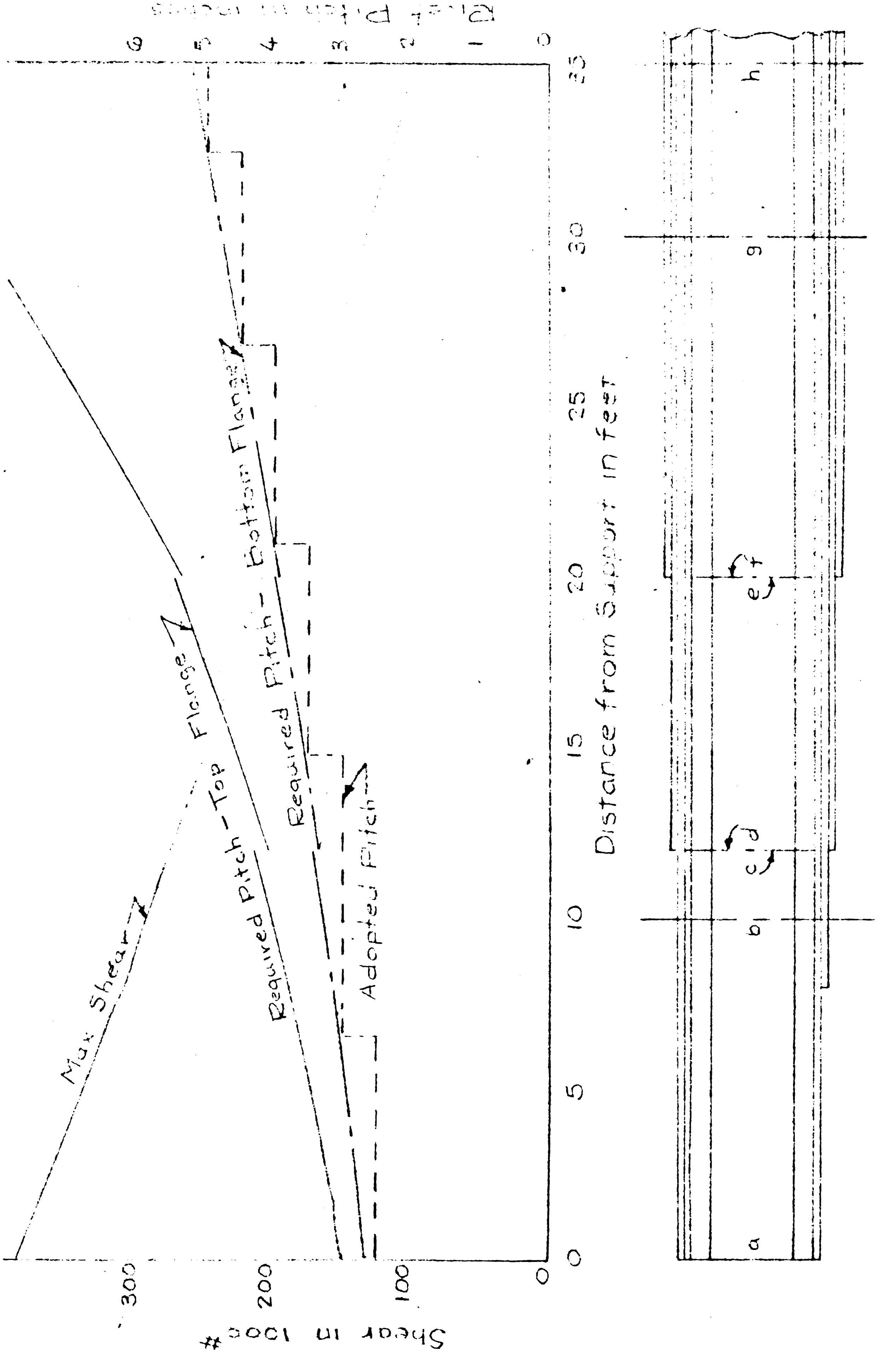
1 - Cover 15" x 3/4" x 38'-0" plus or minus

1 - Cover 15" x 3/4" x 56'-0" plus or minus

<u>Pitch of Rivets in Verticle Legs of Angles</u>

PITCH OF RIVETS JOINING WEB AND UPPER FLANGE

Point 5 (See Fig.)	Dist. from End, Ft.	Shear, V, 1000#	Eff. Depth, h, in.	Gross Area angles plus Covers = A_f Sq. In.	Gross Area angles plus Covers plus 1/8" A_W = A_f plus 1/8 A_W Sq. In.	H.I. = $\frac{V}{h}$ x $\frac{A_f}{A_f \text{ plus } 1/8\ A_W}$ #1 Sq. In.	Vert Load, W #1"	R.I. = $((H.I.)^2 \text{ plus } W^2)^{\frac{1}{2}}$	Pitch p = $\frac{11,810 \text{ in.}}{R.I.}$
a	0	389.5	82.5	30.71	35.95	4030	2025	4475	2.64"
b	10	292.3	82.5	30.71	35.95	3030	2025	3590	3.29"
c	12	275.0	82.5	30.71	35.95	2850	2025 2025	3480	3.40"
d	12	275.0	83.7	41.96	47.20	2920	2025	3540	3.34"
e	20	209.1	83.7	41.96	47.20	2224	2025	2995	3.94"
f	20	209.1	84.4	50.39	55.63	2241	2025	3008	3.93"
g	30	132.6	84.4	50.39	55.63	1424	2025	2460	4.80"
h	35	98.3	84.4	50.39	55.63	1053	2025	2261	5.22"



Rivet Pitch in Inches
Shear in 1000 #
Distance from Support in feet
Max. Shear
Required Pitch - Top Flange
Required Pitch - Bottom Flange
Adopted Pitch
0
100
200
300
0
1
2
3
4
5
6
0
5
10
15
20
25
30
35
a
b
c d
e f
g
h

Horizontal Increment of Flange Stress (Upper Flange)

$$H.I. \frac{V}{h} \times \frac{A_f}{A_f \text{ plus } 1/8\ A_w}$$

$$Pt(a) = \frac{389.5}{82.5} \times \frac{30.71}{35.95} \times 1000 = 4{,}030\#$$

$$Pt(b) = \frac{292.3}{82.5} \times \frac{30.71}{35.95} \times 1000 = 3.030\#$$

$$Pt(c) = \frac{275.0}{82.5} \times \frac{30.71}{35.95} \times 1000 = 2{,}850\#$$

$$Pt(d) = \frac{275.0}{83.7} \times \frac{41.96}{42.20} \times 1000 = 2{,}920\#$$

$$Pt(e) = \frac{209.1}{83.7} \times \frac{41.96}{47.20} \times 1000 = 2{,}224\#$$

$$Pt\ (f) = \frac{209.1}{84.4} \times \frac{50.39}{56.63} \times 1000 = 2.241\#$$

$$Pt(g) = \frac{132.6}{84.4} \times \frac{54.39}{55.63} \times 1000 = 1{,}424\#$$

$$Pt(h) = \frac{98.3}{84.4} \times \frac{50.39}{55.63} \times 1000 = 1{,}053\#$$

Vertical Increment of Flange Stress

(Use one heavy driver over 3 feet plus floor load) (Spec. 428)

$W = 598$ plus $72{,}000 \times 2 = 25$ plus $2000 = 2025\#$

Resultant Increment of Flange Stress

R.I. $= ((H.I.)^2 \text{ plus } W^2)^{\frac{1}{2}}$

Pt(a) $((4030)^2 \quad (2025)^2)^{\frac{1}{2}} = (16{,}000{,}900 \quad 4{,}000{,}625)^{\frac{1}{2}} =$

$(20{,}001{,}525)^{\frac{1}{2}} = 4{,}475\#$

Pt(b) $((3{,}030)^2 \text{ plus } (2{,}025)^2)^{\frac{1}{2}} = (9{,}000{,}900 \text{ plus } 4{,}000{,}625)^{\frac{1}{2}} =$ $3{,}590\#$

Pt(c) $((2{,}850)^2 \text{ plus } (2{,}025)^2)^{\frac{1}{2}} = (8{,}125{,}000 \text{ plus } 4{,}000{,}000)^{\frac{1}{2}} =$ $3{,}480\#$

Pt(d) $((2{,}920)^2 \text{ plus } (2{,}025)^2)^{\frac{1}{2}} = (8{,}525{,}000 \text{ plus } 4{,}000{,}000)^{\frac{1}{2}} =$ $3{,}540\#$

Pt(e) $((2{,}222)^2 \text{ plus } (2{,}025)^2)^{\frac{1}{2}} = (4{,}950{,}000 \text{ plus } 4{,}000{,}000)^{\frac{1}{2}} =$ $2{,}995\#$

Pt(f) $((2{,}241)^2 \text{ plus } (20.5)^2)^{\frac{1}{2}} = (5{,}040{,}000 \text{ plus } 4{,}000{,}000)^{\frac{1}{2}} =$ $3{,}008\#$

Pt(g) $((1{,}424)^2 \text{ plus } (2{,}025)^2)^{\frac{1}{2}} = (2{,}040{,}000 \text{ plus } 4{,}000{,}000)^{\frac{1}{2}} =$ $2{,}460\#$

Pt(h) $((1{,}055)^2 \text{ plus } (2{,}025)^2)^{\frac{1}{2}} = (1{,}111{,}000 \text{ plus } 4{,}000{,}000)^{\frac{1}{2}} =$ $2{,}261\#$

Bearing per Rivet

Rivet Bearing Allowable = 27,000 lb./Sq.In. (Spec 301)

Bearing per Rivet = $27,000 \times \frac{1}{2} \times \frac{7}{8}$ = 11,810 lb./Sq.In.

Pitch of Rivets Joining Web & Lower Flange

Point	Distance from end (feet)	Shear (1000 lb)	Eff. Depth h (inches)	Net Area A_f (Square inches)	A_f plus $\frac{1}{8} A_w$ (Square inches)	H.I. = v/h x A_f/A_f plus $\frac{1}{8} A_w$	Pitch, p, = 11,810/ R.I. in.
a	0	389.5	82.5	26.21	31.45	3,939	3.01
b	10	202.3	82.5	26.21	31.45	2,950	4.02
c	12	275.0	82.5	26.21	31.45	2,758	4.28
d	12	275.0	83.7	35.96	41.20	2,868	4.12
e	20	209.1	83.7	35.96	41.20	2,179	5.42
f	20	209.1	84.4	43.26	48.50	2,210	5.35
g	30	132.6	84.4	43.26	48.50	1,402	8.42
h	35	98.3	84.4	43.26	48.50	1,036	11.41

Horizontal Increment of Flange Stress (Lower Flange)

$$\text{H.I.} = \frac{V}{h} \times \frac{A_f}{A_f\ 1/8\ A_w}$$

$$Pt(a) = \frac{389.5}{82.5} \times \frac{26.21}{31.45} \times 1{,}000 = 3{,}930\#$$

$$Pt(b) = \frac{292.3}{82.5} \times \frac{26.21}{31.45} \times 1{,}000 = 2{,}950\#$$

$$Pt(c) = \frac{275.0}{82.5} \times \frac{26.21}{31.45} \times 1{,}000 = 2{,}758\#$$

$$Pt(d) = \frac{275.0}{83.7} \times \frac{35.96}{41.20} \times 1{,}000 = 2{,}868\#$$

$$Pt(e) = \frac{209.1}{83.7} \times \frac{25.96}{41.20} \times 1{,}000 = 2{,}179\#$$

$$Pt(f) = \frac{209.1}{84.4} \times \frac{43.26}{48.50} \times 1{,}000 = 2{,}210$$

$$Pt(g) = \frac{132.6}{84.4} \times \frac{43.26}{48.50} \times 1{,}000 = 1{,}402\#$$

$$Pt(k) = \frac{98.3}{84.4} \times \frac{43.26}{48.50} \times 1{,}000 = 1{,}036\#$$

Rivet Pitch of Horizontal Legs of Angles (p')

Dependent on H.I. only. Critical Valves of (p') occur just before cutoffs. i.e., points (a), (d) & (f)

$\therefore$

$$(HI)' = \frac{V}{h} \times \frac{Ac}{Af \text{ plus } 1/8\ Aw}$$

Ac = Area of Covers

Af = Gross Area

$$p' = \frac{Vc}{(H.I.)}$$

Vc = Stress of 1 Cover Plate Rivet (Single Shear)

Vc = 13,500 x 3.1416 $(7/8)^2$ = 8,100 lb./sq/ in.

$$\text{Point (a)} = \frac{389.5}{82.5} \times \frac{11.25}{35.95} = 1{,}478\#$$

$$p' = \frac{8{,}100}{1{,}478} = \underline{\underline{5.48''}}$$

$$\text{Point (d)} = \frac{275.0}{83.7} \times \frac{22.50}{47.20} = 1{,}565\#$$

$$p' = \frac{8{,}100}{1{,}565} = \underline{\underline{5.18''}}$$

$$\text{Point (f)} = \frac{209.1}{84.4} \times \frac{30.93}{55.63} = 1{,}378\#$$

$$p' = \frac{8{,}100}{1{,}378} = \underline{\underline{5.87''}}$$

Will be doubled giving 10.36" for Min. Use Min. Spacing

$7 \times 7/8 = \frac{49}{8} = 6\ 1/8$" Use 6"

End Stiffener Angles

Assume 4 angles 6" x 4" x 3/4"

Effective width = 5 1/4 - 1/2 = 4 3/4" (Spec. 432)

Bearing Area = 4 x 4 3/4 x 13/16 = 15.4 Sq. In.

Bearing Reg. $= \frac{389.5}{27,000} = 14.4$ Sq. In. USE

No. of **Rivets** $= \frac{389.5}{11,810} = 33$ Rivets Use 34 Rivets

Intermediate Stiffener Angles (Spec. 433)

60 x 1/2 = 30" less than 84" Use Stiffeners

Clear Distance

$$d = \frac{255,000f}{S} \left(\frac{Sxt}{a} \right)^{1/3}$$

f = 1/2"

a = 72.5"

S = Unit Shear

At Support

$$S = \frac{389,500}{84 \times 1/2} = 9,270 \qquad d = \frac{255,000x1/2}{9,270} \qquad \left(\frac{9279x1/2}{72.5}\right)^{1/3} = 55"$$

Outstanding Leg = Not less than 2 plus $(\frac{1}{30} \times 84) = 5"$

Thickness = 5/16"   Use 3/8" Minimum

Ust 2 angles 5" x 3 1/2" x 3/8"

19' from Support

$$S = \frac{292.3}{84x1/2} = 6950 \qquad d = \frac{255,000x1/2}{6,950} \qquad \left(\frac{6950x1/2}{72.5}\right)^{1/3} = 67"$$

20' from Support

$$S = \frac{209.1}{84x1/2} = 4,970 \qquad d = 255,000x1/2 \qquad \left(\frac{4,970x1/2}{73.5}\right)^{1/3} = 83.5"$$

<u>Greater than 72"</u>   <u>Use 72"</u>   (Spec. 433)

<u>Web Splice</u>   (Spec. 430)   (Design for Shear & Moment)

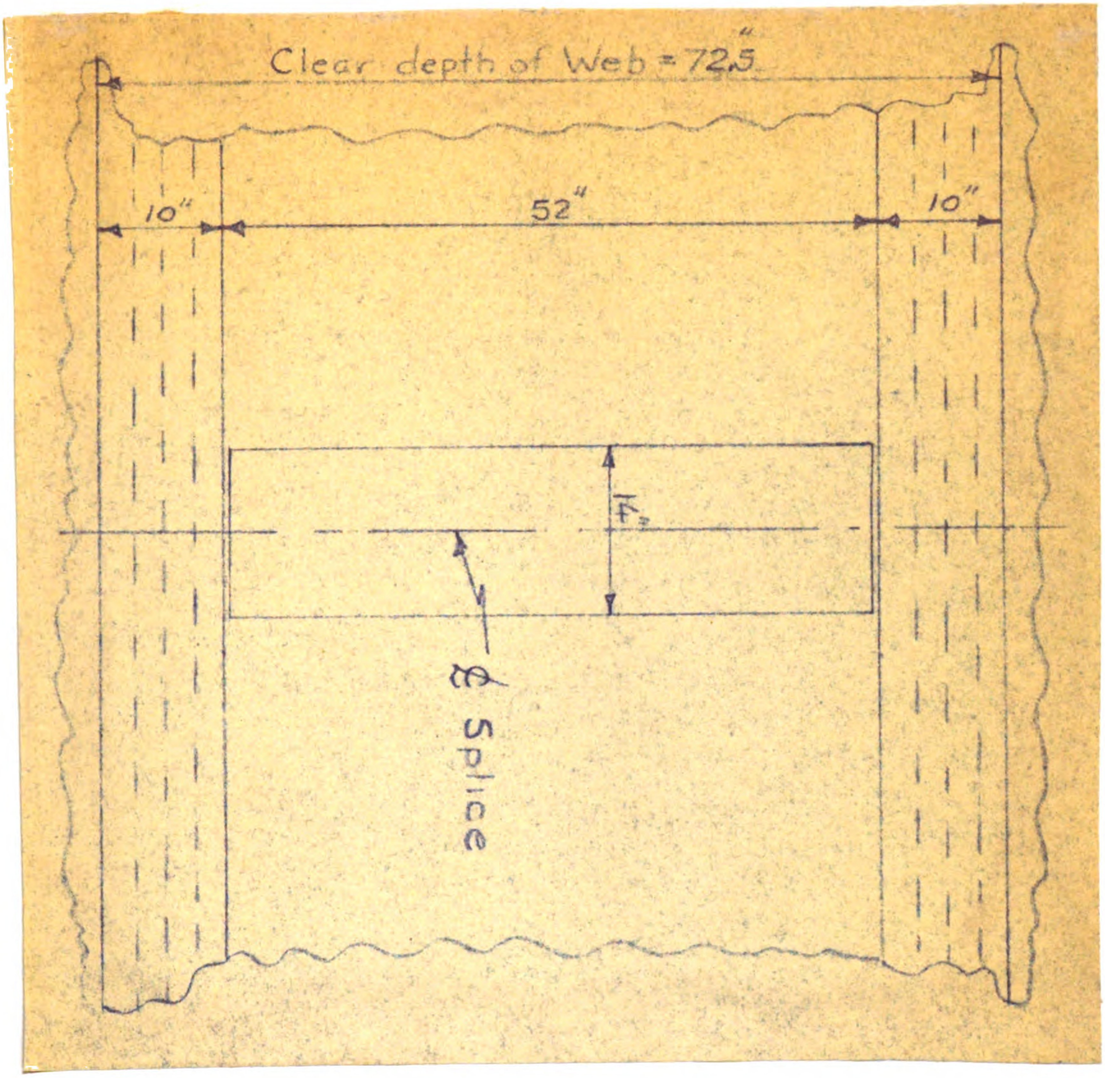


Fig. 12

Assume Plates and Rivet Rows as shown all clearances =

1/8" (10 plus 10 plus 52 plus 1/8 x 4 = 72.5")

Net Cross Section of one pair Splice Plates x c to c distance between splice plates = 1/8 A_W x Eff. Depth

$$\text{Net Area Required} = \frac{5.24 \times 84.4}{62.25} = 7.12 \text{ Sq. In.}$$

Assume Rivets placed opposite

Then net width plate = 10" - 3" = 7"

Thickness of each plate = $\frac{1/2 \times 7.12}{7}$ = .50" <u>Use 1/2" Plate</u>

No. of Rivets = (Assume no eccentricity as rivets are grouped close together)

Strength of 2 plates (t) = 2 x 7 1/2 x 18,000 = 126,000#

One Rivet Bearing = 11,810 lb/sq.in.

Rivets Req. $\frac{126,000}{11,810}$ = 11 Rivets

4 in each of outer rows

3 in inner row

Gross section shear plates = Total Web Area

Plate thickness = $\frac{1/2 \times 84 \times 1/2}{52}$ = .403"

<u>Use 1/2"</u> Moment Plates

<u>No. Rivets</u> (Use max shear in web) = 389.5^{k}

$\frac{389,500}{11,810}$ = 33 Rivets   Use 34 (17 in each row)

Lateral Bracing (Use Warren type) (Upper flanges only)

Max Allowance Stress in (C) $= 18,000-5\ \frac{L^2}{b^2}$

b = Flange width

l = Length between lateral bracing

(Not to exceed 18 feet)

Unit Compressive Flange Stress = Max Moment/Eff. Depth x (Gross A_f plus 1/8 A_w)

Unit Compressive Flange Stress $= \frac{73,000,000}{84.4(55.63)} = 15,590$ lb/sq in

$$15,590 = 18,000 - 5\ \frac{L^2}{(15)^2}$$

$$L^2 = 45\ (18,000 - 15,590)$$

$$L = (108,500)^{\frac{1}{2}}$$

$$= 330''$$

$$= 27.5'$$

<u>Use 18' Min</u> (Spec 438)

Use 10 Panels @ 7' = 70'

Gross Frames at 14'

<u>Wind Loading</u> (Spec 209)

(Bridge) Wind load = 1 1/2 x 84 x 30 - 315#/Ft. less than 200 plus

150 = 350#/Ft. Use 350#/Ft.

(Train)Wind load = 300#/Ft.

650#/Ft.

Panel Load = 650 x 7 - 4,550

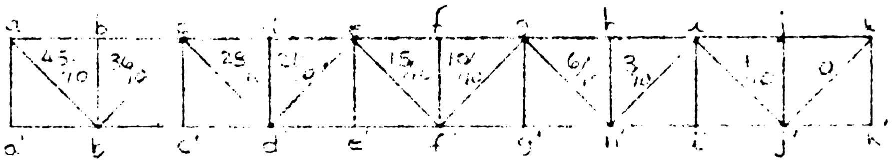


Fig. 13

Unit shear per panel (When all panels to right loaded)

$$ab = \frac{9}{10} \text{ plus } \frac{8}{10} \text{ plus } \frac{7}{10} \text{ plus } \frac{6}{10} \text{ plus } \frac{5}{10} \text{ plus } \frac{4}{10} \text{ plus } \frac{3}{10} = \frac{45}{10}$$

Stress ab' = $\frac{45}{10}$ x 4,550 x 1.414 = 29,000#

b'c = $\frac{36}{10}$ x 4,550 x 1.414 = 23,200#

cd' = $\frac{28}{10}$ x 4,550 x 1.414 = 18,050#

d'e = $\frac{21}{10}$ x 4,550 x 1.414 = 13,520#

e'f = $\frac{15}{10}$ x 4,550 x 1.414 = 9,660#

Allowable stress (c) = $15,000 - 1/4 \frac{L^2}{r^2}$

L (1.41 x 7 x 12) - 18 = 99 in.

Try 5 x 3 x 1/2 angle (r = .65)

Stress $15,000 - \frac{1}{4} \times \frac{99^2}{.65^2} = 9,200$ lb/sq. in.

Area Req = $\frac{29,000}{9,200}$ = 3.15 sq. in.

Furnished = 3.75 sq. in. OK Use throughout

Check for tension: (Spec 410)

Connect 5" leg

Net Section Connected Leg = 5 x 1/2 = (1,725 x 1 x 1/2)

1 plus $\frac{S^2}{4g}$ 2.5 - . 862 = sq. in.

1 plus $\frac{(2.25)^2}{4 \times 1.75}$ S = Pitch = 2.25"

1 x .725 = 1.725 g = guage = 1 3/4"

Net Section unconnected leg = 1/ x 2.5 x 1/2 = <u>.625</u> Sq. In.

Net area supplied = Sum = 2.265 sq. in.

Net Area required = $\frac{29,000}{18,000}$ = 1.61 sq. in (Spec. 310) <u>OK</u>

Max stress Developed = 9.200 x 3.75 = 34,500#

Rivets needed = $\frac{34,500}{8,120}$ = $\underline{\underline{4}}$ Rivets (Single shear)

Least (r) for struts = $\frac{6 \times 12}{140}$ = .515"

Unsupported length = 7'-0"-1-0" = 6'-0" (Assumed)

<u>Use 3 1/2" x 3 1/2" x 3/8" angle</u>

Panel Load = 4,550#

$\frac{4,550}{2} = 2,275$ (Taken by each strut)

Gross area = 2.48 sq. in. (Easily sufficient)

Cross Frames

Lateral force = 29,000#

Shear in each diagonal = $\frac{29,000}{2} = 14,500\#$

Stress = 14,500 x 1.414 = 20,500#

Assume 3 1/2" x 3 1/2" x 3/8" angle (r) = (.62)

Min (r) Allowable = $\frac{L}{140}$ $L = 1/2 \; (84^2 \text{ plus } 84^2)^{\frac{1}{2}}$

$= \frac{60}{140} = .43$ $= 60"$

OK

Allowable stress = $15,000 - 1/4 \; \frac{3,600}{.62} = 12,850$ lb/sq.in.

Area req = $\frac{20,500}{12,850} = 1.6$ sq. in.

Area supplied = 2.48 sq. in. OK

Use 5" x 3 1/2" x 3/8" Throughout

<u>Tension</u>

Req-net area in tension = $\underline{20,500}$ = 1.14 sq. in.

Supplied = 1.52 sq. in.   <u>OK</u>

Max stress developed = 1.52 x 18,000 = 27,400#

Rivets = $\frac{27,400}{8,120}$ = <u>4 Rivets</u>

<u>BILL OF MATERIALS</u>

Item	No. of Pieces	Section	Wgt Per Foot	Total Ft.	Total Wgt.
Flange angles	8	6"x6"x$\frac{7}{8}$x70'-0"	33.1#	576-0"	19,065.6#
Web Plates	4	84"x$\frac{1}{2}$"x36'-0"	143.0#	144'-0"	20,592.0#
Cover Plates	1	15"x$\frac{3}{4}$x72'-0"	38.3#	72'-0"	2,757.6#
	1	15"x$\frac{3}{4}$x56'-0"	38.3#	56'-0"	2,144.8#
	2	15"x$\frac{3}{4}$x38'-0"	38.3#	76'-0"	2,910.8#
	2	15"x$\frac{9}{16}$x30'-0"	28.7#	60'-0"	1,722.0#

Stiffeners

End Angles	16	6"x4"x$\frac{3}{4}$x6'-11"	23.6#	110'-8"	2,611.1#
End Fillers	8	14"x$\frac{7}{8}$x6'-01/4"	41.7#	48'-2"	2,008.7#
Int. Angles	56	5"x3$\frac{1}{2}$x$\frac{3}{8}$x6'-11"	10.4#	387'-4"	4,028.2#
Int. Fillers	56	3$\frac{1}{2}$"x$\frac{7}{8}$x6'-0$\frac{1}{4}$"	10.4#	337'-2"	3,506.6#
Splice Plates	4	14"x$\frac{1}{2}$"x4'-4"	23.8#	17'-4"	412.5#
	8	10"x$\frac{1}{2}$"x2'-0"	17.0#	16'-0"	272.0#
Lateral Bracing	10	5"x3"x$\frac{1}{2}$"x8'-3"	12.8#	82'-6"	1,055.8#
	5	3$\frac{1}{2}$"x3$\frac{1}{2}$"x$\frac{3}{8}$x5'-7"	8.5#	27'-11"	237.3#
	2	20"x$\frac{7}{16}$x1'-6"	29.8#	3'-0"	89.4#
	2	14"x$\frac{7}{16}$x1'-2"	20.8#	2'-4"	48.5#
	9	15"x$\frac{7}{16}$x3'-3"	20.3#	29'-3"	652.3#
	9	14"x$\frac{7}{16}$x1'-2"	20.8#	10'-6"	218.4#

Cross Frames

End Frames	4	5"x3$\frac{1}{2}$"x$\frac{3}{8}$x7'-5"	10.4#	29'-8"	308.3#
	4	5"x3$\frac{1}{2}$"x$\frac{3}{8}$x6'-5"	10.4#	25'-8"	266.3#
	8	19"x$\frac{7}{16}$x1'-7"	28.3#	12'-8"	357.8#
	2	10"x$\frac{3}{8}$x10"	12.8#	1'.8"	21.0#
Int. Frames	8	3$\frac{1}{2}$"x3$\frac{1}{2}$"x$\frac{3}{8}$x6'5"	8.5#	51'4"	435.6#
	8	3$\frac{1}{2}$"x3$\frac{1}{2}$"x$\frac{3}{8}$x8'-2"	8.5#	65'-4"	555.9#
	16	15"x$\frac{3}{8}$x1'-3"	19.1#	20'-0"	382.0#
	8	9"x$\frac{3}{8}$x9"	11.5#	6'-0"	69.0#
Rivet Heads	8200	18# per 100 =			1,476.0#
				Total	67,205.8#

VI DECK PLATE GIRDER R.R. BRIDGE (WELDED STEEL)

Data and Specifications

Single Track

Span = 70'-0"

Loading Cooper E-72

Working Stresses

Flexure 20,000#/Sq." Where flanges are supported laterally

Fillet Welds 13,600"/Sq." Shear in throat.

Specifications

American Institute of Steel Construction

American Welding Society

Design of Web and Stiffeners

Depth of girder $= \frac{1}{10} \times 70 \times 12 = 84"$

Assume dead load = 20% less than for riveted case.

MAX SHEARS IN 1,000 LB.

Distance fm. Support in Ft.	0	10	20	30	35
Dead Load in Kips	29.0	20	12	4	0
Live Load in Kips	199.1	150.1	109.0	71.7	55.3
Impact	155.4	117.2	85.1	55.9	43.0
Total Kips	383.5	287.3	206.1	131.6	98.3

MAX MOMENTS IN KIP FEET

Dist. fm. Support in Ft.	10	20	30	35
Dead Load	240	400	480	490
Live Load	1,554	2,528	3,006	3,073
Impace	1,212	1,970	2,344	2,398
Total	3,006	4,898	5,830	5,951

Total in K-in = 36,072   58,800   69,960   72,412

End Shear = 383,000#

Web Thickness Allowable = 13,000 #/Sq." AISC (Spec 10)

$$\frac{383{,}000}{84 \times 13{,}000} = .350$$

Min. Web Thickness $= \frac{84}{170} = .484$ (Spec 42) AISC

Use $\frac{1}{2}$" web

Unit Web Shear $= \frac{383{,}000}{84 \times 1/2} = 9{,}130$#/Sq."

Need for Stiffeners:

If $\frac{h}{t}$ = or greater than 70 Intermediate stiffeners are required wherever $\frac{h}{t}$ exceeds $8000/(S_s)^{\frac{1}{2}}$

Near the Reaction Point $8{,}000/(S_s)^{\frac{1}{2}} = 8{,}000/(9130)^{\frac{1}{2}} =$ 83.7" (Spec. 45)

Value of $h/_t = \frac{84}{.50} = 168$ <u>Stiffeners Needed</u>

Stiffener Spacing $= d = \frac{270{,}000t}{S_s} \left(\frac{S_s t}{h}\right)^{1/3}$ (Spec. 45)

$$= \frac{270{,}000 \times .5}{9130} \left(\frac{9130}{168}\right)$$

* 56" <u>Use</u>

There will be 14 stiffeners spaced at 56" throughout length of span.

Stiffener Plates $= \frac{84}{12} = 7'$

Good practice is to make stiffener width in inches equal to depth in feet (Use 7")

$$t = \frac{1}{12} \text{ x width} = \text{(Spec 17)} = \frac{7}{12} = .583"$$

Use 7' x 8/16" plates placed in pairs on opposite sides of web.

<u>Strength of Stiffener Welds</u>

Welding Stiffeners to Web

Common practice is to use 3/4" rivets on 5" centers.

Double bearing strength = 11,200#

$\frac{1}{4}$" fillet welds = 2,400# per lineal inch

Use four welds on two stiffeners = 4 x 2,400 = 9,600# per lineal inch.

Equivalent of rivet per 6" = $\frac{6/5 \text{ x } 11,200}{9,600}$ =

1.4" of weld

However min. intermittent weld = 2" Spaced at 6"

<u>Use</u>

End Stiffeners

Reaction transferred to web = 383,000#

Stiffeners act as column whose height = $\frac{1}{2}$ web depth =

1/2 x 84" = 42"

This is stiff short column

Allowable = 17,000 lb. per sq. in.

$$\frac{383,000}{17,000} = 22.5 \text{ sq. in.}$$

Use two pair of end stiffeners each to take 2/3 of total load as one will probably receive more than 1/2 total reaction.

2/3 x 22.5 = 15.0 sq. in.

One plate = 7.0 sq. in.

Use 8" x 7/8" Stiffeners (2 Pairs)

Min = 6" on centers

Welding and stiffeners

Reaction transferred per pair of stiffeners =

2/3 x 383,000 = 255,000#

$$\text{Length } 1/4\text{" weld required} = \frac{255,000}{2,400} = 107\text{"}$$

$\frac{107"}{4}$ = 27" in 84" of web

Use 9 3" welds = 27"

Check intermediate stiffeners for max concentration load.

Max load = 72,000# plus 72,000 x .78 =

72,000 plus 56,000 = 128,000#

$\frac{128,000}{17,000}$ = 7.53 Sq. In. Needed

7 x 9/16 = 7.88 Sq. In. Supplied

OK

Design of flange (with lateral support)

Max moment = 71,412,000 in.#

Approximate eff. depth = 84" plus 1" = 85"

Flange area required = $\frac{71,412,000}{18,000 \times 85}$ = 46.7 Sq. In.

Effective area of web = 1/6 x 1/2 x 84 = 7.0 Sq. In.

Area in flange plates = 46.7 - 7 = 39.7 Sq. In.

Flange section: Use 1 - 20" x 7/8"

1 - 18" x 3/4"

1 - 16" x 5/8"

Gross moment of inertia

Web = 1/2 x 1/2 x 84^3 = 24,700 in.^{4}

20 x 7/8" plates = 2 x 17.50 x $\overline{42.44}^2$ = 63,000 in.^{4}

18 x 3/4" plates 2 x 13.50 x $\overline{43.25}^2$ = 50,500 in.^{4}

16 x 5/8" plates 2 x 10.00 x $\overline{43.9375}$ = 38,600

Total I 176,900 in.^{4}

True fiber stress = $\dfrac{71,412,000 \times 44.250}{176,900}$ = 17,830 Lb. Per Sq. In.

Use

Cut off covers

Net I of web plus 20" x 7/8" plus 18" x 2/4" plates = 138,000 in.^{4}

Allowable moment = $\dfrac{18,000 \times 138,300}{43,625}$ = 57,000,000 In.#

Bending moment at 18' point = 54,800,000 in.# .

Cut off top plate.

Net I of web plus 20" x 7/8" plate = 87,700 in. lb.

Allowable moment = $\frac{87,700 \times 18,000}{42,875}$ = 36,800,000 in.#

Moment at 10' point = 36,072,000 in.#

Cut off 18" x 3/4" plate at this point.

Flange Welds

Weld between web and flange

$$S_s = \frac{VQ}{I} = \frac{383,000}{176,900} (41.00 \times 43.1) = 3,830\# \text{ per lineal inch}$$

$$\bar{y} = \frac{17.50 \times 42.44 \text{ plus } 13.50 \times 43.25 \text{ plus } 10.00 \times 43.9375}{41.00} =$$

$$= \frac{743 \text{ plus } 585 \text{ plus } 439}{41.00} = \frac{1,767}{41} = 43.1"$$

A 7/16" fillet weld gives a value of

.4375 x .707 x 13,600 = 4,200# per lineal in.

<u>Use</u>

Weld between flange plate and first cover

$$S_s = \frac{VQ}{I} = \frac{383{,}000 \times (23.50 \times 43.5)}{176{,}900} = 2200 \text{ \# per inch.}$$

$$\bar{y} = \frac{585 \text{ plus } 439}{23.50} = \frac{1024}{23.50} = 43.5''$$

a 3/8" intermittent weld 2" long spaced at 4" in the clear

produces shear = 2,600# per inch.

Weld between inner and outer cover plates

$$S_s = \frac{383{,}000}{176{,}900} \times (\underline{10.00 \times 43.9375}) = 950\text{\# per in.}$$

Use Minimum 1/4" intermittent weld 2" long spaced 4" in clear.

Check web splice

Fibre stress in web = $\frac{42}{44.25} \times 19{,}600 = 18{,}600$# per Sq. in.

Tensile value of plate = 1/2 x 18,600 = 9,300# per in.

Tensile value of 1/2" butt weld =

1/2 x 13.00 = 6,500# per inch.

Total = 2,850# per inch.

Area reinforced plates

$$\frac{\text{Reinforced plate area}}{\text{Original plate area}} = \frac{2{,}850}{6{,}500} = .438$$

Area needed = .438 x 12 x 1/2 = 2.63 Sq. In. per foot.

at 12" below top of web

Fibre stress = $\frac{30}{44}$ x 19,600 = 13,300# per inch.

Use two 6" x 1/4" plates

Area supplied = 3.00 Sq. inches

Weld on reinforcing plate

Use 1/4" fillets weld shear = 2,000# per in.

Weld length = $\frac{2.850 \times 12}{2 \times 2000}$ = 8.55"

Use 8" min plate to equalize butt weld

Plates = 2 - 8" x 6" x 1/4"

Lateral bracing (use Warren type) (Upper flange only)

Max allowable stress in (c) = 18,000 - 5 $\frac{l^2}{b^2}$

b = Flange width

l = length between lateral bracing

Not to exceed

Unit (c) flange stress = $\frac{\text{Max moment}}{\text{Eff depth}\,(A_f \text{ plus } \frac{1}{6} A_w)}$ =

$$\frac{71,412,000}{85\ (48.00)} = 17,400\#$$ per sq. inch

$$17,400 = 18,000 - 5\ \frac{1^2}{(20)^2}$$

$$1^2 = 80\ (18,000 - 17,400)$$

$$= 48,000$$

1 219" = 18 plus feet Use 18'

Use same bracing as in riveted truss

Diagonals = 5" x3" x 1/2" angles

Max stress developed = 9,200 x 3.75 = 34,500#

1⁄2" fillet weld = 5000# per lineal inch

$$\frac{34,500}{5,000} = 7"$$ Use 3.5" on each side

Struts: $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{3}{8}$" angles

$$\text{Stress} = 15,000 - 1/4 \times \frac{(64)^2}{(1.06)^2}$$

$$= 15,000 - 910$$

$$= 14,090\#$$ per sq. in.

14,090 x 2.49 = 35,000#

3/8" welds = 3750 per lineal inch

35,000 = 9.3"

Use 4.5" on each side. Use 9" of weld per intermediate frames.

End Frames:

$3.05 \times 14{,}000 = 42{,}000\#$

$\frac{42{,}000}{3{,}750} = 11.$ Use 11" 5.5 on each side.

BILL OF MATERIALS

ITEM	# OF PIECES	SECTION	WGT/FT	TOTAL	TOTAL WEIGHT
Flange Plates	4	16"x5/8"x 30'-0"	34.0#	120'-0"	4,080.0#
	4	18"x3/4"x 48'-0"	45.9#	192'-0"	8,812.8#
	4	20"x7/8"x 72'-0"	59.5#	288'-0"	17,136.0#
Web Plates	4	84'x½"x36' -0"	143.0#	144'-0"	20,592.0#
End Stiffeners	16	8"x7/8"x7' -0"	23.8#	112'-0"	2,665.6#
Int Stiffeners	56	7"x9/16x7' -0"	13.4#	392'-0"	5,252.8#
Splice Plates	4	6"x½"x8"	5.1#	2'-0"	13.6#
Lateral Bracing	10	5"x3"x½"x 9'-6"	12.8#	95'-0"	1,216.0#
	5	3½"x3½"x 3/8"x6'-2"	8.5#	30'-10"	262.1#
End Frames	4	5"x3½"x3/8" x7'-5"	10.4#	29'-8"	308.3#
	4	5"x3½"x3/8" x6'-5"	10.4#	25'-8"	266.7#
	8	19"x7/16"x 1'-7"	28.3	12'-8"	357.7#

	2	10"x7/8"x10"	12.8#	1'-8"	21.0#
Int Frames	8	$3\frac{1}{2}$"x$3\frac{1}{2}$"x3/8" x6'-5"	8.5#	51'-4"	435.6#
	8	$3\frac{1}{2}$"x$3\frac{1}{2}$"x3/8" x8'-2"	8.5#	65'-4"	555.9#
	16	15"x3/8"x1'-3"	19.1#	20'-0"	382.0#
	8	9"x3/8"x9"	11.5#	6'-0"	69.0#
Welds					168.0#
			Total Weight		62,594.3#

VII DECK PLATE GIRDER RAILROAD BRIDGE (ALUMINUM)

DATA AND SPECIFICATIONS

Single Track

Span = 70'-0"

According to American Railroad Engineering Association "Specifications for Steel Railways Bridges, 1941, as "Supplemented by "Specifications for the Design and Fabrication of Structures of Alcoa Aluminum Alloy" 615-t." As revised September 15, 1947.

As the dead load stresses are 10% or less of the total and the weight of aluminum spans rum about 40% of corresponding steel the shears and moments will be considered as the same as for the steel spans which is on the safe side.

Because the live load deflection of aluminum is greater than that for steel and because the web depths for steel are arbitrarily chosen the depth of webs for aluminum will be chosen one foot greater than that for steel. This will change the weight only a small amount.

D = 96"

Assume:

Flange angles = 8" x6" x 5/8"

2 Plates = 14" x 5/8"

1 Plates = 14" x $\frac{1}{2}$"

h = 80"

Assume t = 3/4"

$\frac{h}{t}$ = less than 160

Use Horizontal Stiffener

S_a = 16,000 pounds per sq. in. (See Specs,)

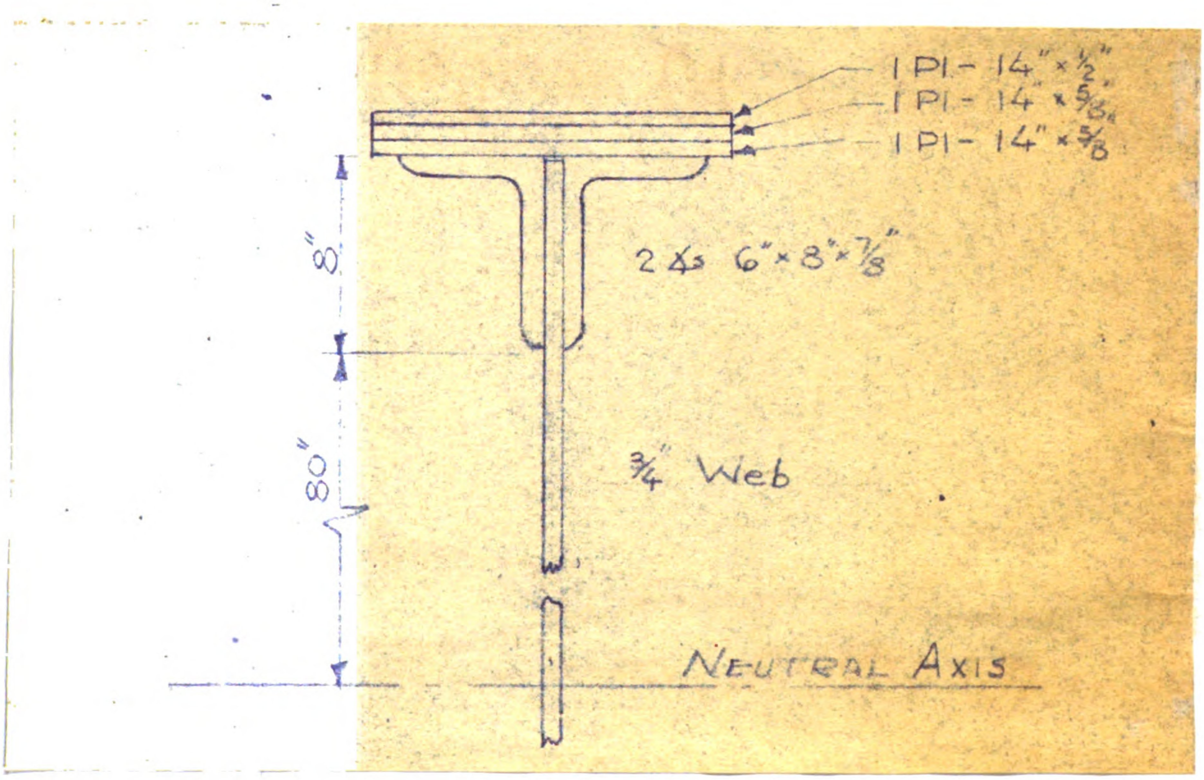


Fig. 14

Try: 3/4" Web

2-14" x 5/8" Plates

1-14" x 1/2" Plate

2-6" x 8" x7/8" Angles

$$I = \frac{3/4 \times (96)^3}{12} \text{ plus } 2\ \frac{(14x(1.75)^3}{12} \text{ plus } 24.5 \times (48.875)^2$$

$$\text{plus } 4(72.3 \text{ plus } 11.48\ (45.39)^2\ )$$

$$= 55{,}300 \text{ plus } 117{,}500 \text{ plus } 300 \text{ plus } 94{,}600$$

$$= 267{,}700 \text{ in.}^4$$

$$S = \frac{73{,}000{,}000 \times 49.75}{267{,}700} = 13{,}600\#$$

M = 73,000,000 in.-16(See riveted girder)

S_a= 16,000# per sq. in. (Figure 5A of Specifications)

Check against sidewise buckling (Figure 2, Page 12, Specifications)

I_1 = Moment of Inertia for beam about principal axis parallel to web.

$$I_1 = \frac{96 \times (3/4)^3}{12} \text{ plus } 2\ \frac{(1.75 \times 14^3)}{12} \text{ plus } 4\ (34.9 \text{ plus } 11.48\ (1.985)^2)$$

$$= 3 \text{ plus } 800 \text{ plus } 140 \text{ plus } 181 = \underline{1124 \text{ inches}^4}$$

I_f = Moment of Inertia for compression flange about axis parallel to web

$$I_f = 1/2\ I_1 = 562 \text{ inches}^4$$

J = Torsion Factor = Sum of $1/3\ bt^3$ (See Specs)

$$= \frac{96 \times 3/4^3}{3} \text{ plus } 4\ \frac{(8 \text{ plus } 5.375) \times (7/8)^3}{3} \text{ plus } (14 \times (1.75)^3\ )$$

$$= 13.5 \text{ plus } 11.9 \text{ plus } 50$$

$$= \underline{85.4} \text{ inches}^3$$

S_c = Section modulus of compression flange

$$I_c = \frac{3/4 \times (48)^3}{3} \text{ plus } \frac{(14 \times 1.75)^3}{3} \text{ plus } 24.5 \times (48.875)^2)$$

$$\text{plus } 2(72.3 \text{ plus } 11.48\ (45.39)^2)$$

$$= 27{,}700 \text{ plus } 58{,}750 \text{ plus } 150 \text{ plus } 47{,}300$$

$$= 133{,}900 \text{ inches}^4$$

$$S_c = \frac{133{,}900}{49.75} = 2{,}690 \text{ inches}^3$$

L = Laterally unsupported length of compression flange in inches

Assume L = 10' = 168"

$B = (I_1 (J L^2 \text{ plus } 23 I_f d^2))^{\frac{1}{2}}$

$= (1124 (85.4 \times 120^2 \text{ plus } 23 \times 562 \times (99.5^2))^{\frac{1}{2}}$

$= (144,500,000,000)^{\frac{1}{2}}$

$= 379,500$

$$\frac{L}{\left(\frac{B}{S_c}\right)^{\frac{1}{2}}} = \frac{120}{\left(\frac{379,500}{2,690}\right)^{\frac{1}{2}}} = \frac{10.05}{11.90}$$

S_a = 13,900 lb/sq. in. <u>OK</u>

<u>Horizontal Stiffeners</u> (See Spec. 5)

Radius of gyration shall not be less than

$$r = C_r \left(\frac{h}{t}\right) f\ 10 = 9$$

r = Required radius of gyration of one stiffener in inches

h = Clear height of web in inches

t = Web thickness

f = Compressive stress at toe of flange angles in # (Sq. In.)

c_r = A coefficient which defends upon the ratio of the spacing of the vertical stiffeners, S, to the clear height of the web, h. (See Table IV, Specifications)

Assume S = h $\quad : \frac{s}{h} = \frac{70}{80} = .875$

Where S = Stiffener Spacing

h = clear depth of web.

$$f = 13,600 \times \frac{40}{49.75} = 11,000 \text{ \#/Sq. In.}$$

$$r = 6.60 \left(\frac{80}{3/4}\right) \quad 11,000 \times 10^{-9}$$

$$= .825$$

Try a 3" x 3" x 1/4" angle

$$I = 1.18 \times 1.43 \times (82)^2$$

$= 1.18 \times .96$

$= 2.14$ inches^4

$$r = \left(\frac{I}{A}\right)^{\frac{1}{2}} = \left(\frac{2.14}{1.43}\right)^{\frac{1}{2}} = 1.23''$$

Horizontal stiffeners **shall** be cut off at the vertical stiffeners and shall be spaced 16.21" below the toe of the top flange angles.

$$a = 1/2\ h = \frac{80}{2} = 40''$$

$$40 \times .4 = 16'' \text{ plus } 0.21'' = 16.21''$$

Stiffener Spacing: (Figure 6A (Specifications)

$$\text{Shear on Web (Gross Section)} = \frac{389,500}{96 \times 3/4} = 5,400 \text{ lb/sq.in.}$$

$$\frac{h}{t} \text{ Ratio} = \frac{80}{3/4} = 107$$

Use Stiffener Spacing = h = 70"

(From Figure 6A) Allowance Shear = 6,250 lb/sq. in.

Size of Stiffener:

When $\frac{s}{h}$ is greater than 0.4

$$I_s = \frac{t^3 h}{5\left(\frac{s}{h}\right)} 4\left(\left(\frac{s}{h}\right)^2 \text{ plus } 0.625 \right)$$

I_s = Required I of stiffener in inches^4

t = Web thickness in inches

s = Stiffener spacing = 70"

h = Clear height of web = 80"

Substituting:

$$I_s = \frac{3/4^3 \ \frac{16}{80}}{5x\ (.875)^4} \left((.875)^2 \text{ plus } 0.625 \right)$$

$$I_s = 11.5\ (.765 \text{ plus } .625)$$

$$11.5\quad(1.390)$$

$$16.0$$

Try Two Stiffeners 4" x 3" x 5/16"

$$I = 2\ (3.29 \text{ plus } 2.09\ (1.615)^2$$

$$= 2\ (3.29 \text{ plus } 5.45$$

$$= 2\ (8.74)$$

$$= 17.481$$

PITCH OF RIVETS IN VERTICAL LEGS OF ANGLES

(USE 7/8" COLD DRIVEN)

Point	Distance From End	Shear, V, (1000#)	Eff. Depth,(In)	Gross Area, Angles plus Covers, A_f (Sq.In.)	Gross Area, A_f plus 1/8 A_w, (Sq. In.)	H.I.= V/hxA_f / A_f plus 1/8 A_w (lb/sq.in.)	Vert. Load, (lb/sq.in.)	RI= $(HI^2 \text{plus} W^2)^{\frac{1}{2}}$	Pitch,P,= 12,470/R.I.
a	0	389.5	94.4	42.22	51.22	3460	2025	3990	3.12
b	10	292.3	94.4	42.22	51.22	2560	2025	3260	3.82
c	20	209.1	94.4	42.22	51.22	1830	2025	2725	4.50
d	30	132.6	94.4	42.22	51.22	1160	2025	2350	5.30
e	35	98.3	94.4	42.22	51.22	860	2025	2180	5.70

$$\text{EFF. DEPTH} = \frac{24.5 \text{ x } 48.875 \text{ plus } 22.96 \text{ x } 45.39}{24.5 \text{ plus } 22.96}$$

$$= \frac{1200 \text{ plus } 1040}{47.46}$$

$$= 47.2$$

$$47.2 \text{ x } 2 = 94.4$$

PITCH OF RIVETS IN VERTICAL LEGS OF ANGLES

(USE 7/8" COLD DRIVEN)

Point	Distance From End	Shear, V, (1000#)	Eff. Depth, (In)	Gross Area, Angles plus Covers, A_f (Sq.In.)	Gross Area, A_f plus 1/8 A_w, (Sq. In.)	H.I. = V/hxA_f / A_f plus 1/8A_w (lb/sq.in.)	Vert. Load, (lb/sq.in.)	RI= $(HI^2 plus W^2)^{\frac{1}{2}}$	Pitch, P, = 12,470/R.I.
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b	10	292.3	94.4	42.22	51.22	2560	2025	3260	3.82
c	20	209.1	94.4	42.22	51.22	1830	2025	2725	4.50
d	30	132.6	94.4	42.22	51.22	1160	2025	2350	5.30
e	35	98.3	94.4	42.22	51.22	860	2025	2180	5.70

$$\text{EFF. DEPTH} = \frac{24.5 \times 48.875 \text{ plus } 22.96 \times 45.39}{24.5 \text{ plus } 22.96}$$

$$= \frac{1200 \text{ plus } 1040}{47.46}$$

$$= 47.2$$

$$47.2 \times 2 = 94.4$$

PITCH OF RIVETS IN VERTICAL LEGS OF ANGLES

(USE 7/8" COLD DRIVEN)

Point	Distance From End	Shear, V, (1000#)	Eff. Depth,(In)	Gross Area, Angles plus Covers, A_f (Sq.In.)	Gross Area, A_f plus 1/8 A_w, (Sq. In.)	H.I.= V/hxA_f / A_f plus 1/8 A_w (lb/sq.in.)	Vert. Load, (lb/sq.in.)	RI= $(HI^2 \text{plus} W^2)^{\frac{1}{2}}$	Pitch,P,= 12,470/R.I.
a	0	389.5	94.4	42.22	51.22	3460	2025	3990	3.12
b	10	292.3	94.4	42.22	51.22	2560	2025	3260	3.82
c	20	209.1	94.4	42.22	51.22	1830	2025	2725	4.50
d	30	132.6	94.4	42.22	51.22	1160	2025	2350	5.30
e	35	98.3	94.4	42.22	51.22	860	2025	2180	5.70

$$\text{EFF. DEPTH} = \frac{24.5 \times 48.875 \text{ plus } 22.96 \times 45.39}{24.5 \text{ plus } 22.96}$$

$$= \frac{1200 \text{ plus } 1040}{47.46}$$

$$= 47.2$$

$$47.2 \times 2 = 94.4$$

Horizontal Increment of flange Stress (Upper Flange)

$$HI = \frac{V}{h} \times \frac{A_f}{A_f \times 1/8\ A_w}$$

Where HI = Horizontal Increment

V = Shear at section

h = Effective depth

A_f = Area of flange

A_w = Area of web

$P_t(a)$ $\frac{389.5}{94.4} \times \frac{42.22}{51.22} \times 1000 = 3.460\#$

$P^t(b)$ $292.3 \times 8.75 = 2.560\#$

$P_t(c)$ $209.1 \times 8.75 = 1830\#$

$P_t(d)$ $132.6 \times 8.75 = 1160\#$

$P_t(e)$ $98.3 \times 8.75 = 860\#$

Vertical Increment of Flange Stress

(Use one heavy driver over 3' plus floor load

(Spec 428 Area)

$$W = \frac{598}{2 \times 12} \text{ plus } \frac{72,000 \times 2}{2 \times 36} = 25 \times 2000 = 2025\#$$

Resultant Increment of Flange Stress

$$R.I. = \left((HI)^2 \times W^2 \right)^{\frac{1}{2}}$$

$$(3460^2 \text{ plus } 2025^2)^{\frac{1}{2}}$$

Where H.I = Horizontal Increment

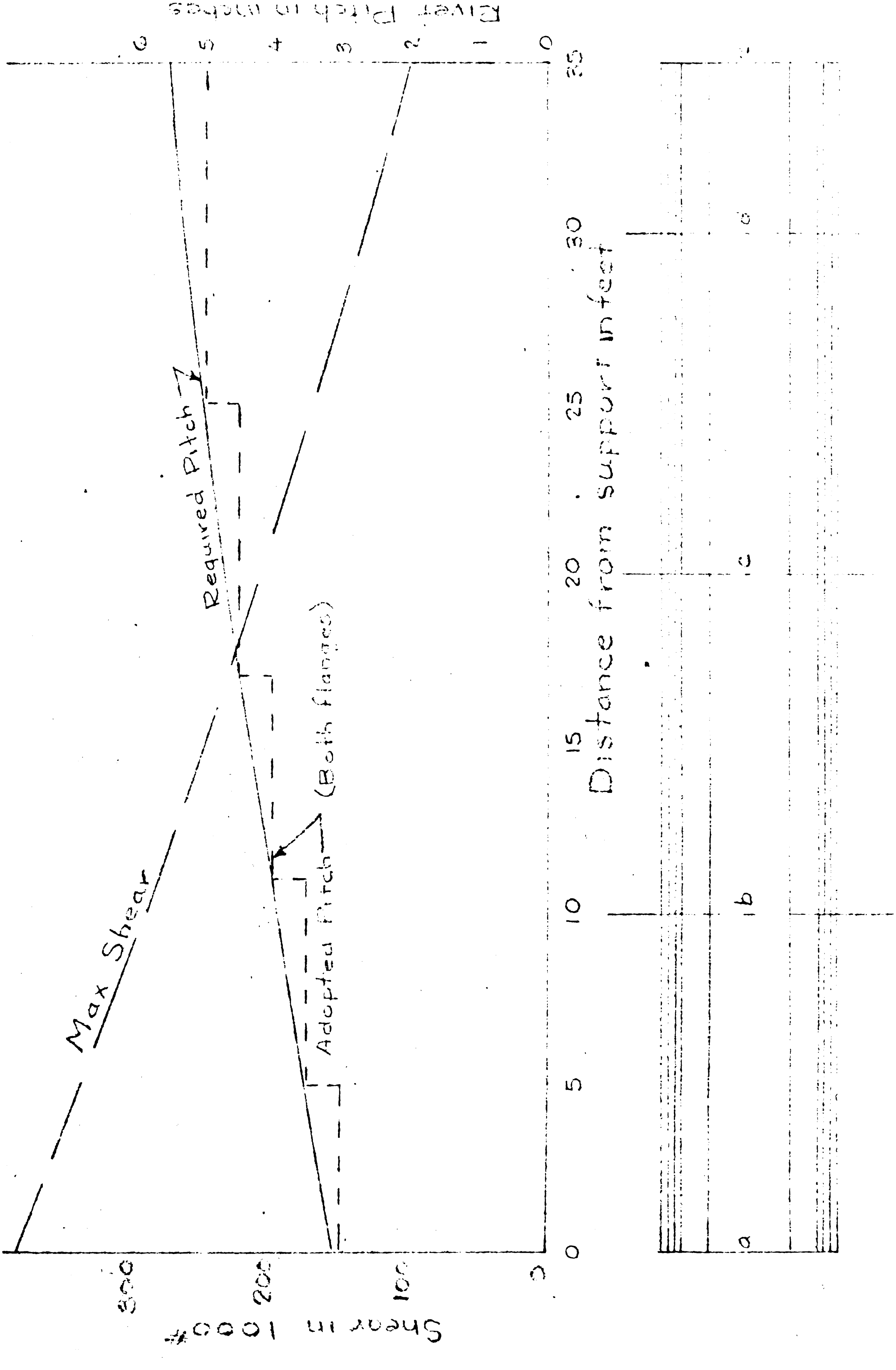
W = Vertical load

$$Pt(a) = (11,950,000 \text{ plus } 4,000,625)^{\frac{1}{2}}$$

$$= (15,950,625)^{\frac{1}{2}} = \underline{3,970\#}$$

$$Pt(b) = (2560^2 \text{ plus } 2025^2)^{\frac{1}{2}} = (6,650,000 \text{ plus } 4,000,625)^{\frac{1}{2}}$$

$$(10,650,625)^{\frac{1}{2}} \qquad = \underline{3,260\#}$$



Rivet Pitch in inches
0
1
2
3
4
5
6
Max Shear
Required Pitch
Adopted Pitch
(Both flanges)
Distance from support in feet
0
5
10
15
20
25
30
35
Shear in 1000#
0
100
200
300
a
b
c
d
e

Pt(c) $(1850^2 \text{ plus } 2025^2)^{\frac{1}{2}} = (3,350,000 \text{ plus } 4,000,625)^{\frac{1}{2}}$

$= (7,350,625)^{\frac{1}{2}} = \underline{2,775\#}$

Pt(d) $(1160^2 \quad 2025^2)^{\frac{1}{2}} = (1,350,000 \text{ plus } 4,000,625)^{\frac{1}{2}}$

$= (5,350,625)^{\frac{1}{2}} = \underline{2,350\#}$

Pt(e) $(860^2 \text{ plus } 2025^2)^{\frac{1}{2}} = (740,000 \text{ plus } 4,000,625)^{\frac{1}{2}}$

$= (4,740,625)^{\frac{1}{2}} = \underline{2,180\#}$

(Use same pitch on bottom Flange)

Rivet Pitch of horizontal legs or angles (P'). Dependent on H. I. only.

$HI = \frac{V}{h} \times \frac{Ac}{Af} \text{ plus } \quad 1/8\ A_w$

A_c = Area of covers

A_f = Gross Area

$P' = \frac{Vc}{H.I.}$

r_c = Stress of 1 cover Plate Rivet (Single shear)

$r_c = 10,000 \times \frac{3.14\ 7/8^2}{4} =$

6,030 lb/sq/in

$$HI = \frac{389.5}{94.4} \times \frac{24.5}{51.22} = 1{,}970\#$$

$$P' = \frac{6030}{1970} = 3.06''$$

Will be dou bled (Rivet on each side) = 6.12" Use $\underline{\underline{6}}$"

Check for buckling:

$$\frac{P}{A} = 46{,}600 - 384\ \frac{KL}{r} = 46{,}600 - \frac{384 \times .5 \times 7}{362}$$

$$= 46{,}000 - 3{,}720 - 42.880$$ Ultimate stresses

$r = \underline{.29 \times 1.25 = .362}$ (Page 71 Alcoa Structure Handbook)

Allowable stress (Specifications) = $17{,}000 - 100 \times \frac{7}{.362}$

$= 15{,}050$ lb/sq in

<u>Use</u> 15,000 lb/sq in

$\frac{42{,}880}{15{,}000} = 2.86$ = Factor of Safety <u>OK</u>

<u>Use</u> 6" Spacing

<u>Web Splice</u> (Spec. 430 A.R.E.A.) (Design for shear moment)

Clear Depth of Web = 80"

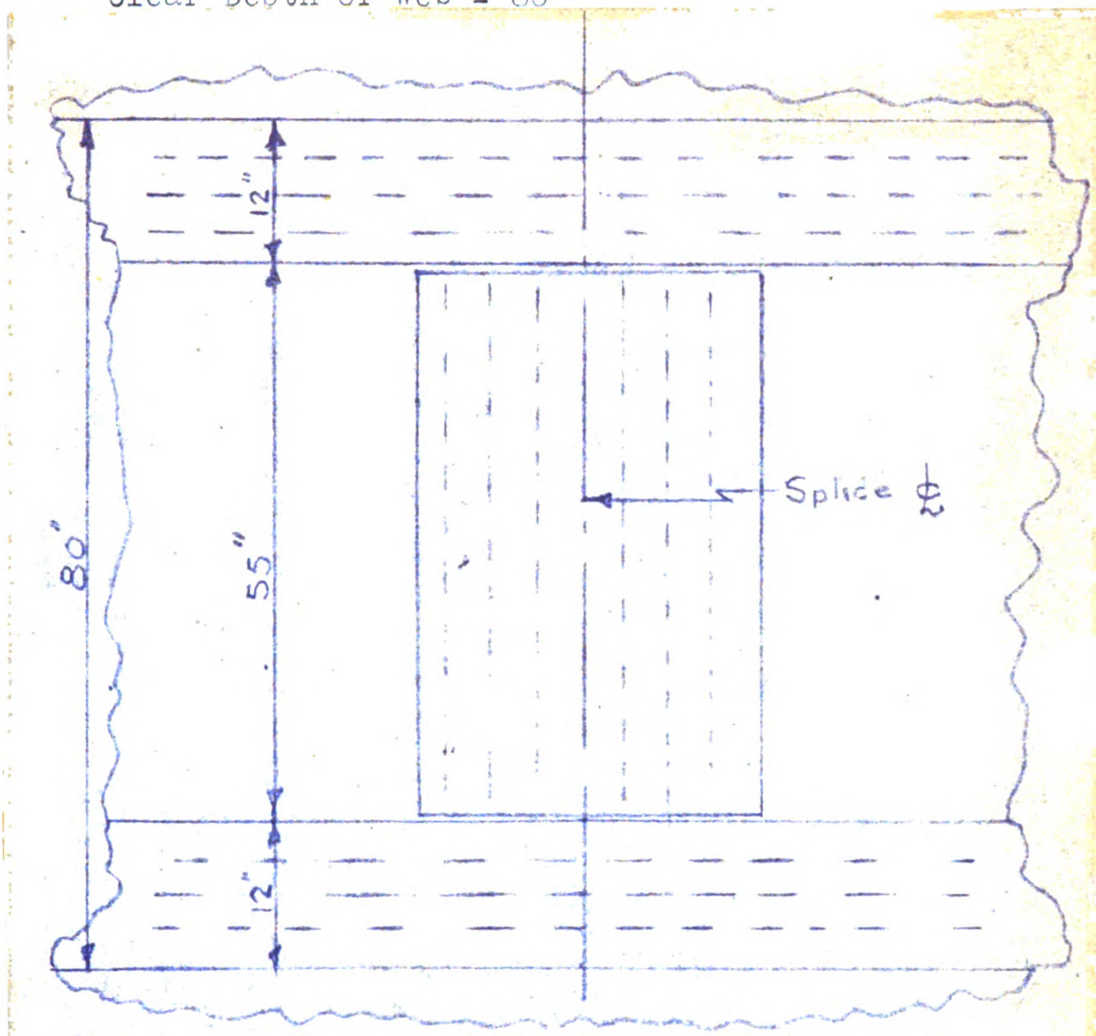


Fig. 15

Assume all clearances = 1/8", Plates and rivet rows as shown.

Net cross section of one pair of Splice Plates x c to c distance between splice plates = 1/8 Aw x Eff Depth.

$$\text{Net Area Req} = \frac{9 \times 92.4}{67.5} = 12.3 \text{ sq. in.}$$

Assume Rivets placed opposite

then net width = 12" -3" = 9"

Thickness of each plate $= \frac{1}{2} \times \underline{12.3} = .685$ (Use 3/4" Plate)

Number of rivets (Assume no eccentricity)

Strength of two plates $= 2 \times 9 \times 16{,}000 = 216{,}000$#

No. rivets (bearing) $= \frac{216{,}000}{12{,}470} = 17.3 = 18$ rivets 6 in each row

Gross Section shear plate = Web

Plate thickness $= 1/2 \times \frac{96 \times 1/2}{55} = .46$

Use 1/2" Moment Plates

No rivets (Use Max shear in web) = 389.5 Kips

$\frac{389{,}500}{12{,}470} = 32$ Rivets

Use 32 Rivets 16 in each row.

End Stiffeners

Assume 4 angles 6" x 4" x 3/4"

Effective width $= 51/4 - 1/2 = 4\ 3/4$"

Bearing area = $4 \times 4\ 3/4 \times \frac{13}{16}$ = 15.4 sq. in.

Bearing Req. = $\frac{389,500}{27,000}$ = 14.4 sq. in.

Number of rivets = $\frac{389.5}{12,470}$ = 31.2 <u>Use 32 rivets</u>

<u>Lateral Bracing</u>

Use 7 panela @ 10' = 70'

Wind Loading (A.R.E.A. Spec. 209)

Bridge = $1\frac{1}{2} \times 96 \times 39$ = 432 lb/ft. greater than 350 lb/ft

Train = 300 lb/ft.

Top Bracing

Train = 300 #/1

Bridge= 216 #/1

516 #/1

516 #/1 x 10' = 5,160#

(Use Warren type bracing)

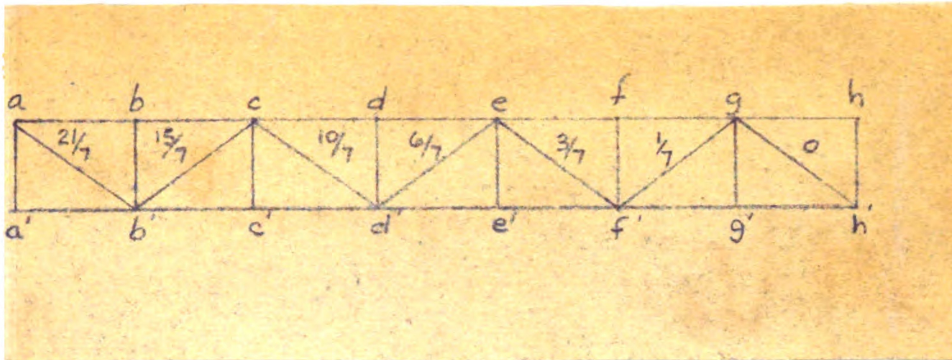


Fig. 16

Unit shear per panel (When all panels to right loaded)

$\frac{6}{7}$ plus $\frac{5}{7}$ plus $\frac{4}{7}$ plus $\frac{3}{7}$ plus $\frac{2}{7}$ plus $\frac{1}{7}$ = $\frac{21}{7}$

Stress ab' = 3 x 5160 x 1.75 = 27,100#

Allowable stress (c) = 17,000 - 100 $\frac{l}{r}$

l = (12.2 x 12) - 30" = 116.5"

Try Two 3 ½" x 3 ½" x 3/8" angles r = 1.07"

S_a = 17,000 - 100 $\frac{116.5}{1.06}$ = 6,000#

Area Required = $\frac{27,100}{6,000}$ = 4.5 sq. in.

Area Supplied = 4.96 sq. in. <u>OK</u> <u>Use throughout</u>

<u>Check for Tension</u> (Spec. 410 Area)

Net Area Required = $\frac{27,100}{16,000}$ = 1.69 sq. in.

Net Section Connected legs = 2 (3 ½ x ½ -

(1.725 x 1 x ½) =

3.5 - .862 = 2.64 sq. in.

<u>OK</u>

Max Stress Developed = 6,000 x 4.96 = 29,800#

Rivets Needed = $\frac{29,800}{6,240}$ = 6 Rivets (Single Shear)

(3 in each leg)

Least r for struts $\frac{(6 \times 12-4}{100}$ = .68 OK

Use 3/12" x 3/12" x 3/8"

Panel Load = 5,160#

$\frac{5,160}{2}$ = 2,580 Taken by Strut

Gross Area = 2.49 sq. in. <u>OK</u>

<u>Bottom Bracing</u> (Use Warren type)

Struts   Use 3" x 3" x 7/8"

Panel Load = 10 x 216 = 2,160#

Stress in a b' = 3 x 2,160 x 1.75 = 11,300#

l = 116.5"

Try 2   3" x 3" x 3/8"   r = .91

$$S_a = 17,000 - 100 \frac{116.5}{.91} = 4,200 \text{ lb/sq.in.}$$

$$\text{Area Required} = \frac{11,300}{4,200} = 2.69 \text{ sq. in.}$$

Area Supplied = 4.22 sq. in.   OK

Tension less than 1 sq. in.   OK

Stress Developed = 4,200 x 4.22 = 17,700#

$$\text{Rivets Needed} = \frac{17,700}{6,240} = 2.9$$ Use 4 Rivets

Cross Frames

Intermediate Cross Frames - Use 3" x 5" x 3/8" angles

End Frames

Lateral Force = 27,700#

Shear in Back Diagonal = $\frac{27,700 - 11,300}{2}$ = 8,200#

Stress in Back Diagonal = 8,200 x $\frac{10.6}{7}$ = 12,400#

Assume 3 ½" x 3 ½" x 3.8" angles r = (.62)

Use 5" x 3 ½" x 3/8" angles

Min (r) Allowable = $\frac{1}{100}$ = $\frac{52}{100}$ = (52) OK

1 = ½ x 10.6 x 12 - 12 = 52"

Use 2 Rivets = 6,240 x 2 = 12,480#

BILL OF MATERIALS

ITEM	# OF PIECES	SECTION	WGT/FT	TOTAL FEET	TOTAL WEIGHT
Flange Angles	8	8"x6"x5/8"x 72'-0"	9.84#	560.0"	5,667.8#
Web Angles	4	96"x3/4"x 36'-0"	87.12#	140.'0"	12,545.3#
Cover Plates	8	14"x4/8"x72' -0"	10.59#	560.'0"	6,049.8#
Cover Plates	4	14"x½"x 72'-0"	8.47#	280.'0"	2,439.4#
End Stiffeners	16	6"x4"x3/4"x 7'-10"	8.39#	125'-4"	1,051.5#
End Stif-feners Fill	8	10"x5/8"x6' -8"	7.563#	53'-4"	403.3#
Hor Stiffener	12	3"x3"x¼"x 5'-7"	1.73#	67'-0"	115.9#
Int. Stiffener	44	4"x3"x5/16"x 7'-10"	2.53#	344'-8"	872.0#
Splice Plates	8	12"x3/4"x 1'-6"	10.89#	12'-0"	130.7#
Splice Plates	4	14"x½"x 4'-7"	8.47#	18'-4"	155.3#
Lateral Bracing	28	3½"x3½"x3/8" x9'8½"	3.01#	271'-0"	818.2#
	16	3½"x3½"x 3/8"x5'-8"	3.01#	90'-8"	272.9#
	28	3"x3"x3/8"x 9'-8½"	2.55#	271'-10"	693.2#
	16	3"x3"x3/8"x 5'-8"	2.55#	90'-8"	231.2#
	24	15"x7/16"x 3'-5"	9.941#	82'-0"	651.2#
	32	14"x7/16"x 1'2"	7.411	37'-4"	276.7#

	32	14"x7/16"x 1'-2"	7.411#	37'-4"	276.7#
End Frames	4	5"x3½"x3/8" x8'-0"x3½"	3.69#	33'-2"	122.3#
	8	15"x3/8"x 1'-3"	6.806#	10'-0"	68.1#
	2	9"x3/8"x9"	4.084#	1'-6"	6.1#
Int.Frames	12	3"x3"x3/8"x 8'-4½"	2.55#	100'-6"	256.3#
	24	15"x3/8"x 1'-3"	6.806#	30'-0"	204.2#
	6	9"x3/8"x9"	4.084#	4'-6"	18.4#
Rivet Heads	8600		7.6#/100		653.6#
			Total		34,365.0#

VII WOODEN GIRDER RAILROAD BRIDGE

Data and Specifications

Single track

Span = 70'-0"

Depth = 6'-0" Limit

Loading = Cooper E-72

Walking Stress

Flexure = 3,000 p.s.i.

Moment = 70,000,000 inch pounds

Shear = 380,000 pounds

Try 4 - 11" x 56 7/8" glued laminated woods girders.
72'-8" long

$$S = \frac{Mc}{I}$$

$$= \frac{70,000,000 \times 28.4375}{\frac{4 \times 11 \times (56.875)^3}{12}}$$

$$= 2900 \text{ p.s.i.} \qquad \text{OK}$$

This would require 8 girders of this size which is impractical.

APPENDIXES

I.

SPECIFICATIONS

for the

DESIGN AND FABRICATION

of

STRUCTURES

of

ALCOA ALUMINUM ALLOY 61S-T

Aluminum Company of America
New Kensington, Penna.
Revised- September 15, 1947

DESIGN

These specifications are intended to supplement standard specifications prescribed for the design of steel structures. Nothing in these specifications is to be construed as permitting any radical departure from accepted good design practice. These specifications are comparable to those standard specifications for ordinary grade carbon steel structures in which the basic tensile design stress is 18,000 p.s.i.

Alcoa aluminum alloy 61S-T is a heat-treated material and has the following nominal chemical composition:

Al	Cu	Si	Mg	Cr
97.9%	0.25%	0.6%	1.0%	0.25%

The following are the typical physical properties of 61S-T alloy:

Weight per cubic inch	0.098 lb.
Tensile strength	45,000 psi.
Tensile yield strength (0.2 percent set)	40,000 psi.
Elongation in 2 in. (1/2-in. Diameter round specimen)	17 percent
Compressive yield strength (0.2 percent set)	40,000 psi.
Ultimate shear strength	30,000 psi.
Shear yield strength (0.2 percent set)	26,000 psi.
Ultimate bearing strength (edge distance = twice rivet diameter)	94,000 psi.
Modulus of elasticity in tension and compression	10,000,000 psi.
Modulus of elasticity in shear	3,800,000 psi.
Poisson's ratio	0.33

Brinell hardness, 500 kg load, 10 mm ball	95
Coefficient of expansion per 1° F	0.000013

The following material specifications apply to this alloy:

Sheet and plate:	Navy 47-A-12b, Federal QQ-A-327
Tubing:	Navy 44-T-30b, Federal WW-T-789
Shapes:	Navy 46-A-10d, Federal QQ-A-325

Alloy 61S-T is produced in the form of sheet, plate, extruded shapes, rolled shapes, tubing, rod, bar, wire, and rivets. It combines good strength characteristics with the best cold workability of any of the heat-treated aluminum alloys. Because of its excellent resistance to corrosion, it is widely used in marine structures and in other locations where conditions of exposure are severe.

Reference: Alcoa Structural Handbook; Aluminum Research Laboratories Technical Paper No. 1, Column Strength of Various Aluminum Alloys".

1. Allowable Unit Stresses:

Axial tension, net section	16,000 psi.
Tension in extreme fibers of rolled shapes, extruded shapes, girders, and built sections subjected to bending	16,000 psi.
Stress in extreme fibers of pins	24,000 psi.
Shear in power-driven rivets, cold-driven 61S-T	10,000 psi.
Shear in power-driven rivets, hot-driven 53S (1030 to 1050°F)	8,000 psi.
Shear in pins and in turned bolts	10,000 psi.
Bearing on pins	24,000 psi.

Bearing on power-driven rivets, turned bolts in reamed holes, milled stiffeners, and other parts in contact 27,000 psi

Bearing on unfinished bolts 18,000 psi

2. <u>Allowable Compressive Stresses in Columns</u>:

For columns centrically loaded the allowable compressive stress on the gross section shall be found using the following formulas:

For $\frac{L}{r}$ less than or equal to 100 $\frac{P}{A} - 17{,}000 - 100\,\frac{L}{r}$ but not to exceed 15,000 psi

For $\frac{L}{r}$ greater than 100

$$\frac{P}{A} = \frac{70{,}000{,}000}{(\frac{L}{r})^2}$$

Where $\frac{L}{r}$ = greatest slenderness ratio of member.

These column formulas are based on partial fixation of ends. (K = 0.75). A plot of these column formulas is given in Fig. 1, page xiii.

3. <u>Allowable Compressive Stresses in Flanges of Beams</u>:

The compressive stress in the extreme fiber of rolled shapes, extruded shapes, and single-web girders and built sections, subject to bending, gross section, shall not exceed the values given by the curve in the attached Fig. 2, page xiv.

Values of torsion factor, J to be used in connection with Fig. 2 are given in the Alcoa Structural Handbook for many standard shapes. Values for plates and shapes not shown may be calculated by assuming the sections to be composed of rectangles and taking the sum of the terms $\frac{1}{3}\,bt^3$

for each rectangle where b equals the length and t the thickness of the rectangle. The value of J for a built member is the sum of the individual J values of the sections of which it is composed.

The term $\left(\frac{B}{S_c}\right)^{\frac{1}{2}}$ used in Fig. 2 is rarely less than one-half width of the compression flange for a plate girder. This fact is useful in preliminary design.

Double-web box girders, because of their tube-like cross section, are very much stiffer in torsion than single-web girders of comparable size. For the depth-width ratios ordinarily encountered is design, double-web box girders are so stiff in torsion that lateral buckling failures of the compression flange are of no importance in structural design, and therefore it is not necessary to make any reduction in allowable stress because of the slenderness ratio or length-width ratio of the flange. The allowable stress on the compression flange of such members is usually restricted by the possibility of local buckling of the compression cover plate.

4. Allowable Compressive Stresses for Flat Plates, Legs, Webs and Flanges:

The compressive stress on the gross area of flat plates, legs and flanges shall not exceed the values given by the curves in the attached Fig. 3, page xv, and Fig. 4, page xvi.

The compressive stress at the toes of the flange angles in the web of a girder or built member subject to bending, gross section, shall not exceed the values given by the curve in Fig. 5.

5. Size of Horizontal stiffeners on the Webs of Plate Girders:

A horizontal stiffener of the type covered by Fig. 5A shall have a radius of gyration not less than that given by the following formula:

$$r = c_r \left(\frac{h}{t}\right)^2 f\, 10^{-9}$$

r = required radius of gyration of one stiffener in in.,

h = clear height of web in in.,

t = web thickness in in.,

f = compressive stress at toe of flange angles in psi,

c_r = a coefficient which depends upon the ratio of the spacing of the vertical stiffeners, s, to the clear height of the web, h. Values of c_r are given in Table IV, page 10.

For a stiffener composed of equal size members on both sides of the web, the radius of gyration shall be taken about the center line of the web. For a stiffener composed of a member on one side only, the radius of gyration shall be taken about the face of the web in contact with the stiffener.

6. Allowable Shear Stresses for Flat Plates and Webs:

The shear stress on flat webs shall not exceed the values given by the curve in the attached Fig. 6, page xx. The values in Fig. 6 apply to the gross area of the web, but the shear on the net area shall not exceed 12,000 psi.

7. <u>Spacing of Vertical Stiffeners on the Webs of Plate Girders</u>

The distance between vertical stiffeners applied to the web of a plate girder to resist shear buckling of the web shall not exceed the value indicated by Fig. 6A, pagexxi. Fig. 6A is merely a replot of the data in Fig. 6 rearranged for convenience in establishing stiffener spacings. Where a stiffener is composed of a pair of members, one on each side of the web only, the distance s shall be the distance measured from the rivet line. In determining the spacing of vertical stiffeners to resist shear buckling in panels containing a horizontal stiffener located as shown in Fig. 5A, the distance h in Fig. 6A may be taken as 90 percent of the clear distance between flanges.

8. <u>Size of Vertical Stiffeners on the Webs of Plate Girde rs:</u>

Stiffeners applied to plate girder webs to resist shear buckling shall have a moment of inertia, I_s, not less than given by the following formulas:

when $\frac{s}{h}$ less than or equal to 100. 0.4, I_s $6.13t^3h$

when $\frac{s}{h}$ greater than 0.4, $I_s = \frac{t^3h}{5(\frac{s}{h})} \left((\frac{s}{h})^2 \text{ plus} 0.625 \right)$

I_s = required moment of inertia of stiffener in in.^{4},

t = thickness of web in in.,

s = required stiffener spacing from Fig. 6A in in.,

h = clear height of web in in.

For a stiffener composed of equal size members on both sides of the web, the moment of inertia shall be taken about the center line of the web. For a stiffener composed of a member on one side only, the moment of inertia shall be taken about the face of the web in contact with the stiffener. In determining moment of inertia of stiffeners the term "h" shall always be taken as the full height between flanges regardless of whether or not a horizontal stiffener is present.

9. Reversal of Stress:

Members subject to reversal of stress under the passage of the live load shall be proportioned as follows:

Determine the tensile stress and the compressive stress and increase each by 50 percent of the smaller; then proportion the member so that it will be capable of resisting either increased stress. The connections shall be proportioned for the sum of the stresses.

10. Allowable Loads for Rivets and Bolts:

The allowable loads on cold-driven 61S-T rivets are given in the attached Table I, page , and those for hot-driven 53S rivets in the attached Table II, pagexi The allowable loads on unfinished 24S-T bolts are given in the attached table III, page .

These allowable loads are computed on the basis of the allowable shear and bearing stresses given in Paragraph 1 and represent in each case the controlling value, shear or bearing, whichever is lower. In computing the shear values, a correction was applied where necessary to take into account the reduction resulting from the cutting action of thin plates. (See Table 4, "Riveting Alcoa Aluminum" - 1946). All rivet values are based on the bolt diameters.

TABLE I

ALLOWABLE DESIGN LOAD PER RIVET

FOR COLD-DRIVEN 61S-T RIVETS IN 61S-T STRUCTURES

Shear - 10,000 psi
Bearing - 27,000 psi

Allowable loads given in pounds.

Rivet Diameter In.	1/2		5/8		3/4		7/8		1	
Hole Diameter In.	0.516		0.641		0.766		0.891		1.016	
Drill Size	33/64		41/64		49/64		57/64		1-1/64	
Rivet in Single or Double Shear / Thickness of Plate or Shape, In.	ss	ds	ss	ds	ss	ds	ss	ds	ss	ds
3/8	2090	4180	3230	6310	4610	7760*	6240	9 020*	8110	10 290*
7/16	2090	4180	3230	6450	4610	8970	6240	10 520*	8110	12 000*
1/2	2090	4180	3230	6450	4610	9220	6240	12 030*	8110	13 720*
9/16	---	---	3230	6450	4610	9220	6240	12 390	8110	15 430*
5/8	---	---	3230	6450	4610	9220	6240	12 470	8110	16 000
3/4	---	---	---	---	4610	9220	6240	12 470	8110	16 220
7/8	---	---	---	---	---	---	6240	12 470	8110	16 220
1	---	---	---	---	---	---	---	---	8110	16 220

* These values governed by bearing, all others by shear

TABLE II

ALLOWABLE DESIGN LOAD PER RIVET

FOR HOT-DRIVEN 53S RIVETS IN 61S-T STRUCTURES**

Shear - 8,000 psi
Bearing - 27,000 psi

Allowable loads given in pounds

Rivet Diameter in.	1/2		5/8		3/4		7/8		1	
Hole Diameter in.	0.531		0.656		0.781		0.922		1.063	
Drill Size	17/32		21/32		25/32		59/64		1-1/16	
Rivet Single or Double Shear	ss	ds	ss	ds	ss	ds	ss	ds	ss	ds
Thickness of Plate or Shape, in. 3/8	1770	3540	2700	5290	3830	7170	5340	9 340*	7100	10 760*
7/16	1770	3540	2700	5410	3830	7460	5340	9 990	7100	12 560*
1/2	1770	3540	2700	5410	3830	7670	5340	10 610	7100	13 690
9/16	--	--	2700	5410	3830	7670	5340	10 340	7100	13 280
5/8	--	--	2700	5410	3830	7670	5340	10 680	7100	14 020
3/4	--	--	--	---	3830	7670	5340	10 680	7100	14 200
7/8	--	--	--	--	--	--	5340	10 680	7100	14 200
1	--	--	--	--	--	--	--	--	7100	14 200

* These values governed by bearing, all others by shear

TABLE IV

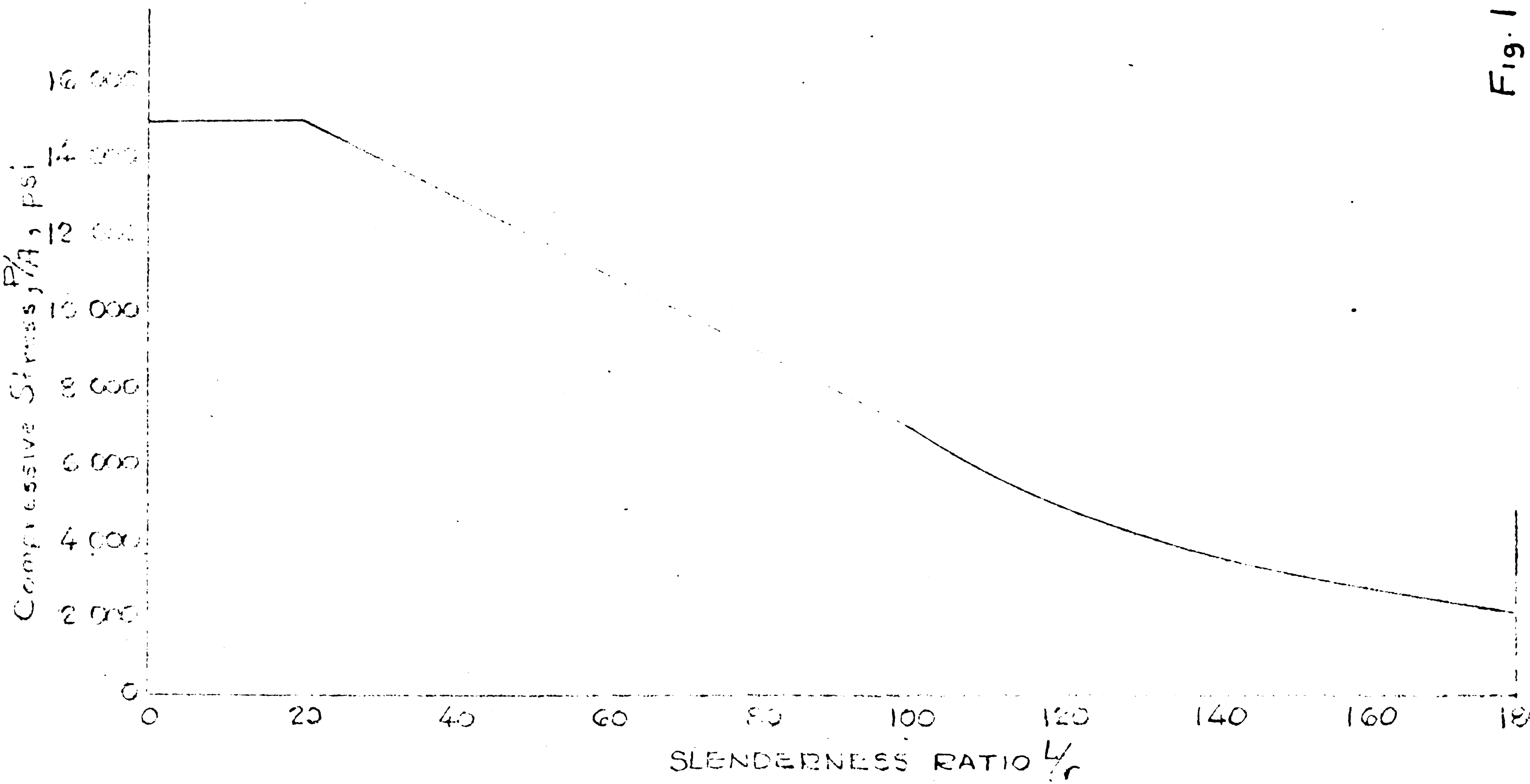
VALUES OF STIFFENER COEFFICIENT C_r

OF HORIZONTAL STIFFENERS FOR WEBS OF PLATE GIRDERS

REINFORCED BY ONE HORIZONTAL STIFFENER

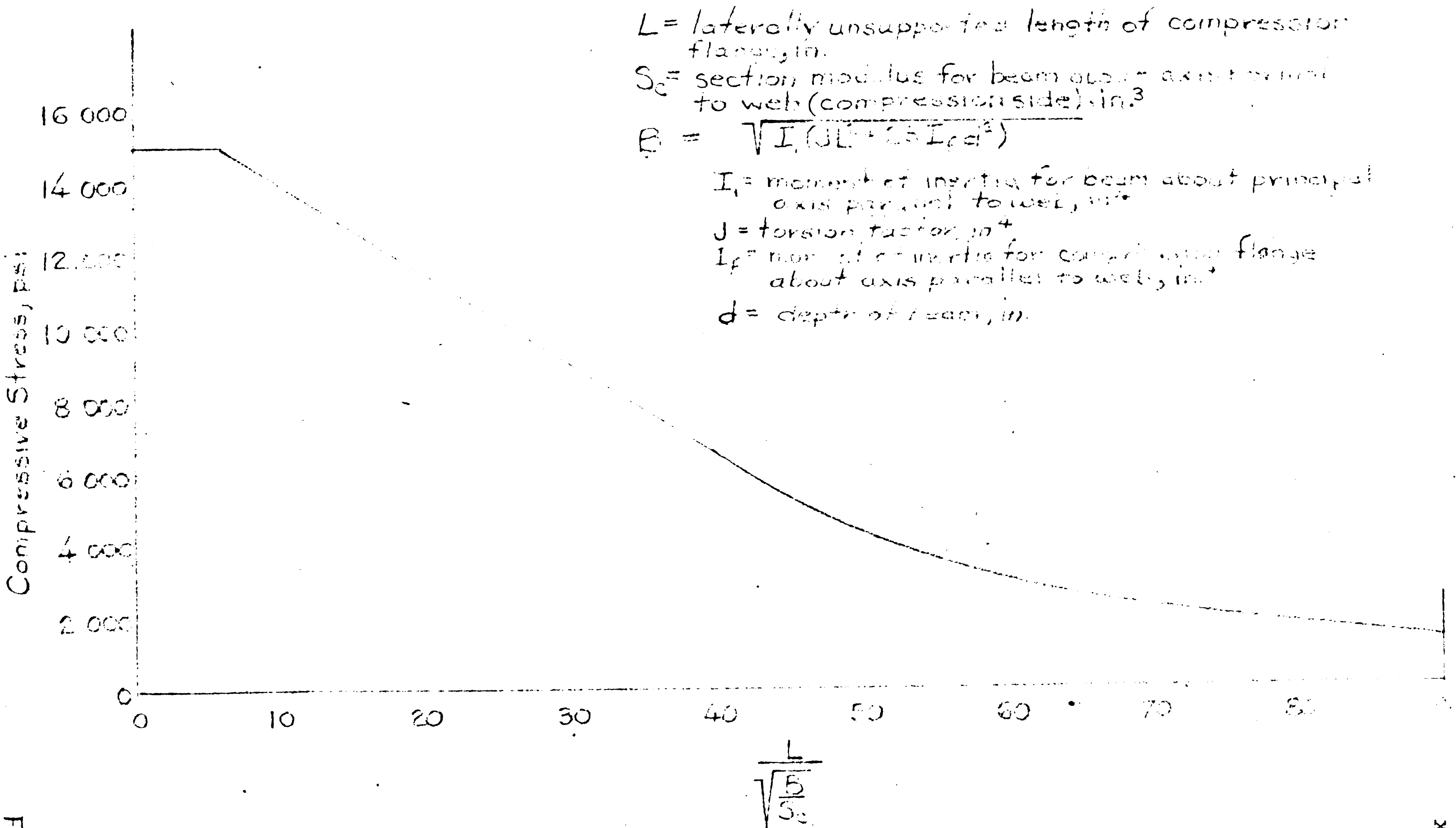
Values of Coefficient C_r for Various Web Thicknesses				
s/h	3/8"	1/2"	5/8"	3/4"
0.60	2.16	3.16	4.19	5.28
0.65	2.29	3.35	4.43	5.57
0.70	2.42	3.53	4.66	5.83
0.75	2.55	3.71	4.89	6.08
0.80	2.68	3.89	5.11	6.32
0.85	2.80	4.07	5.32	6.55
0.90	2.92	4.24	5.55	6.76
0.95	3.04	4.40	5.71	6.96
1.00	3.15	4.55	5.90	7.15
1.05	3.26	4.69	6.08	7.33
1.10	3.36	4.83	6.25	7.51
1.15	3.46	4.96	6.41	7.68
1.20	3.56	5.09	6.56	7.85

Fig. 1



Allowable Compressive Sresses for Axially Loaded Columns (Gross Section)
(Partial restraint assumed at ends of unsupported length)

Fig. 1



Allowable Compressive Stress in Beam and Girder Flanges (Gross Section)
(Partial restraint assumed at ends of laterally unsupported length)

Fig 2

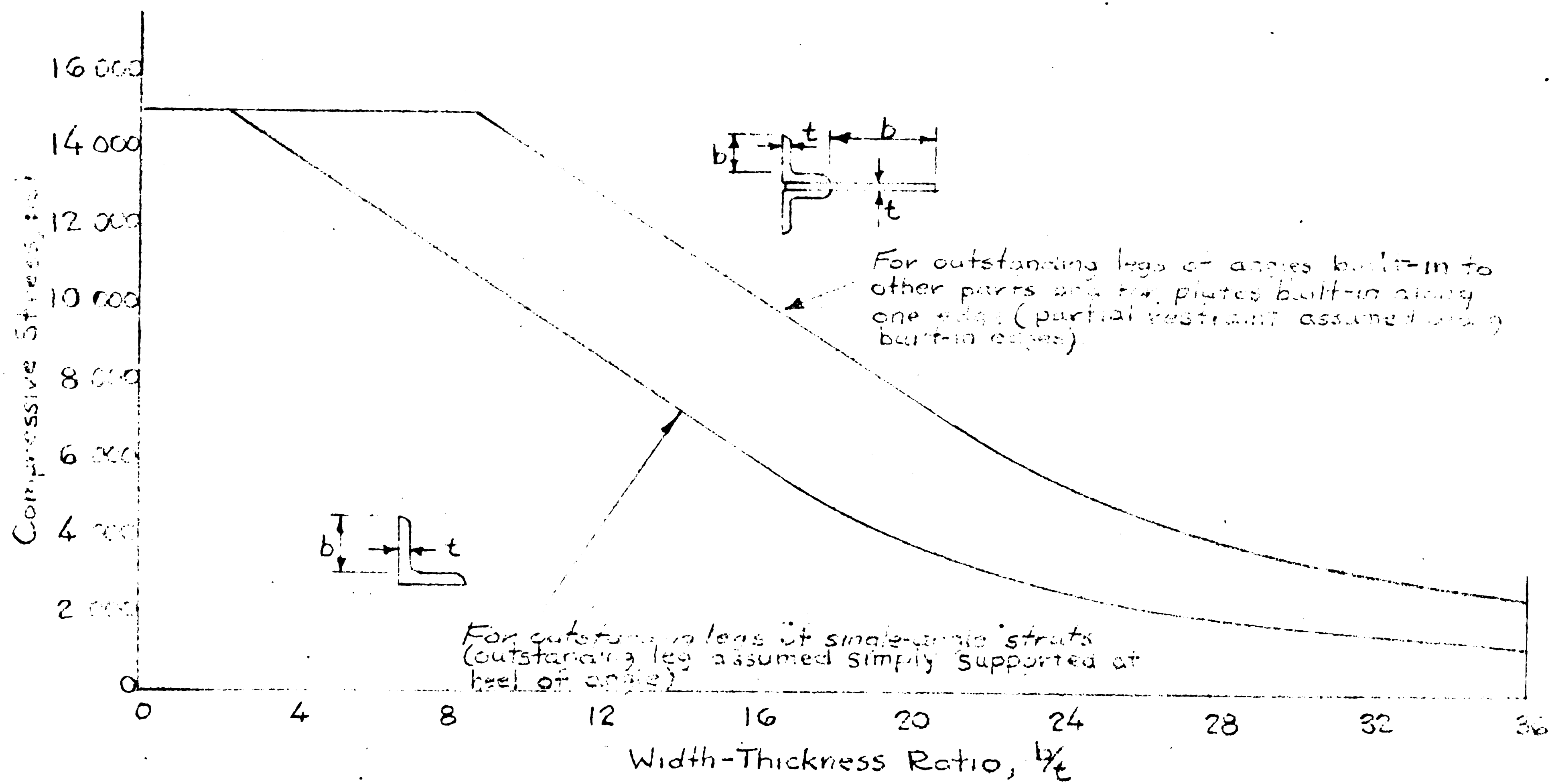
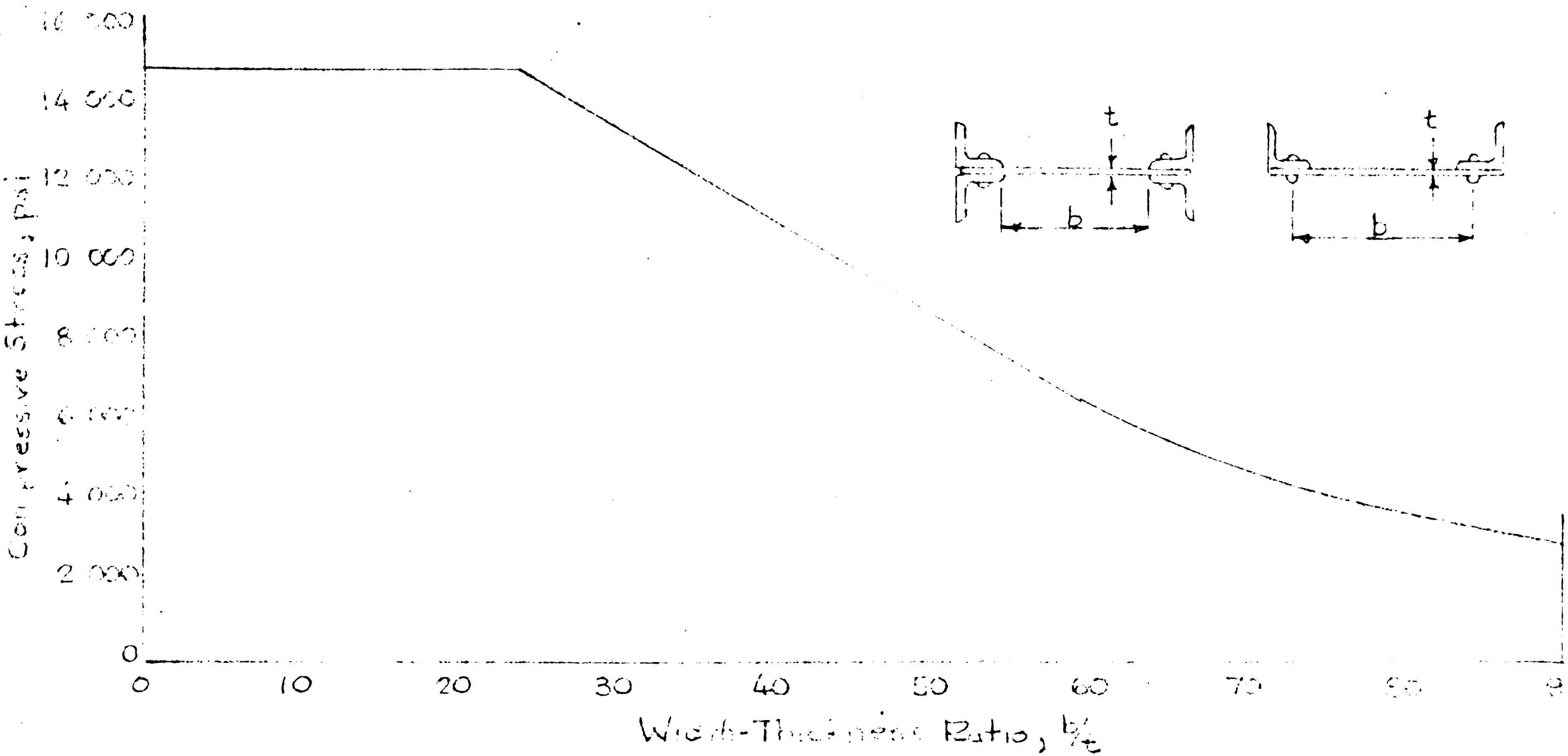


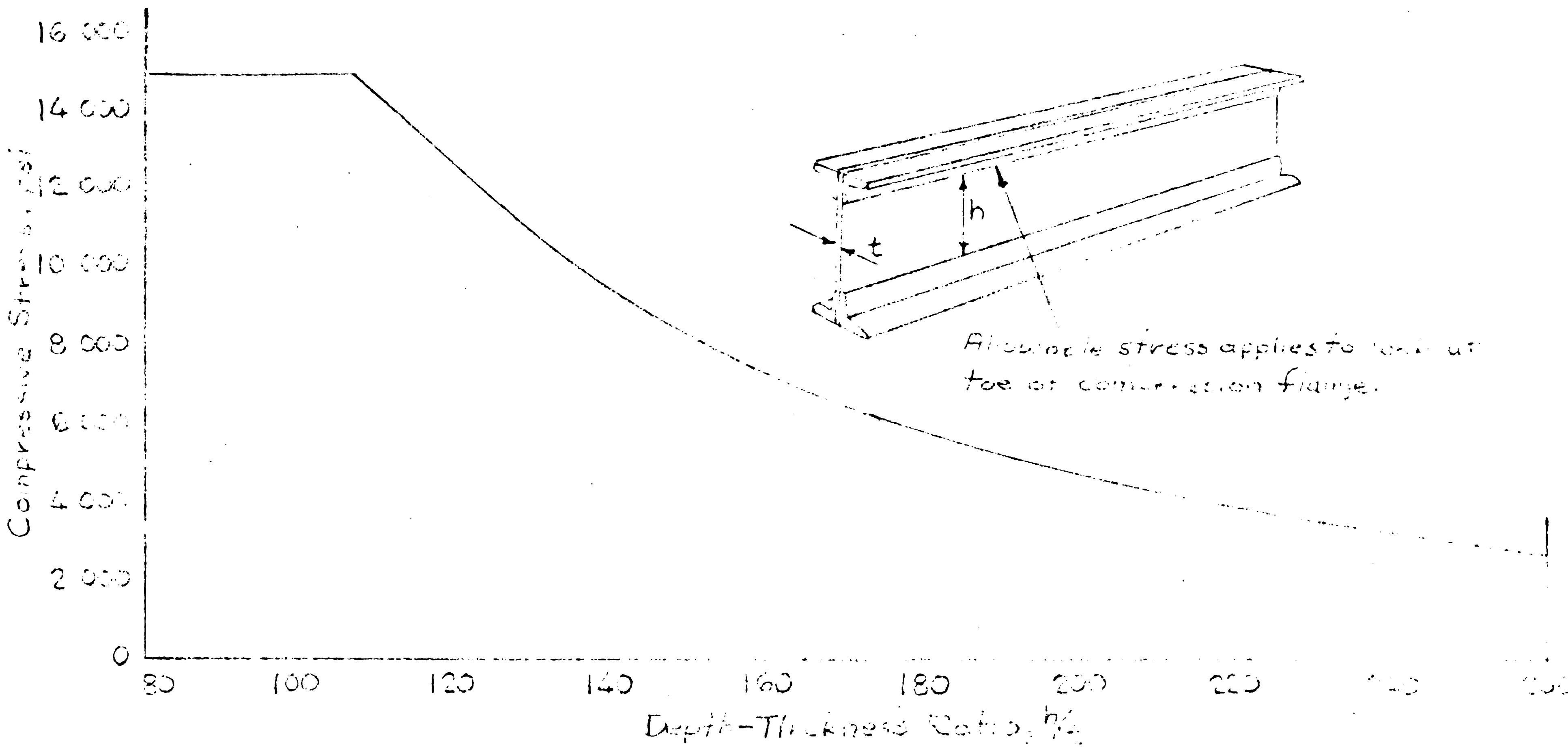
Fig. 3

Allowable Compressive Stress on Outstanding Flat Plates and Legs (Gross Section)



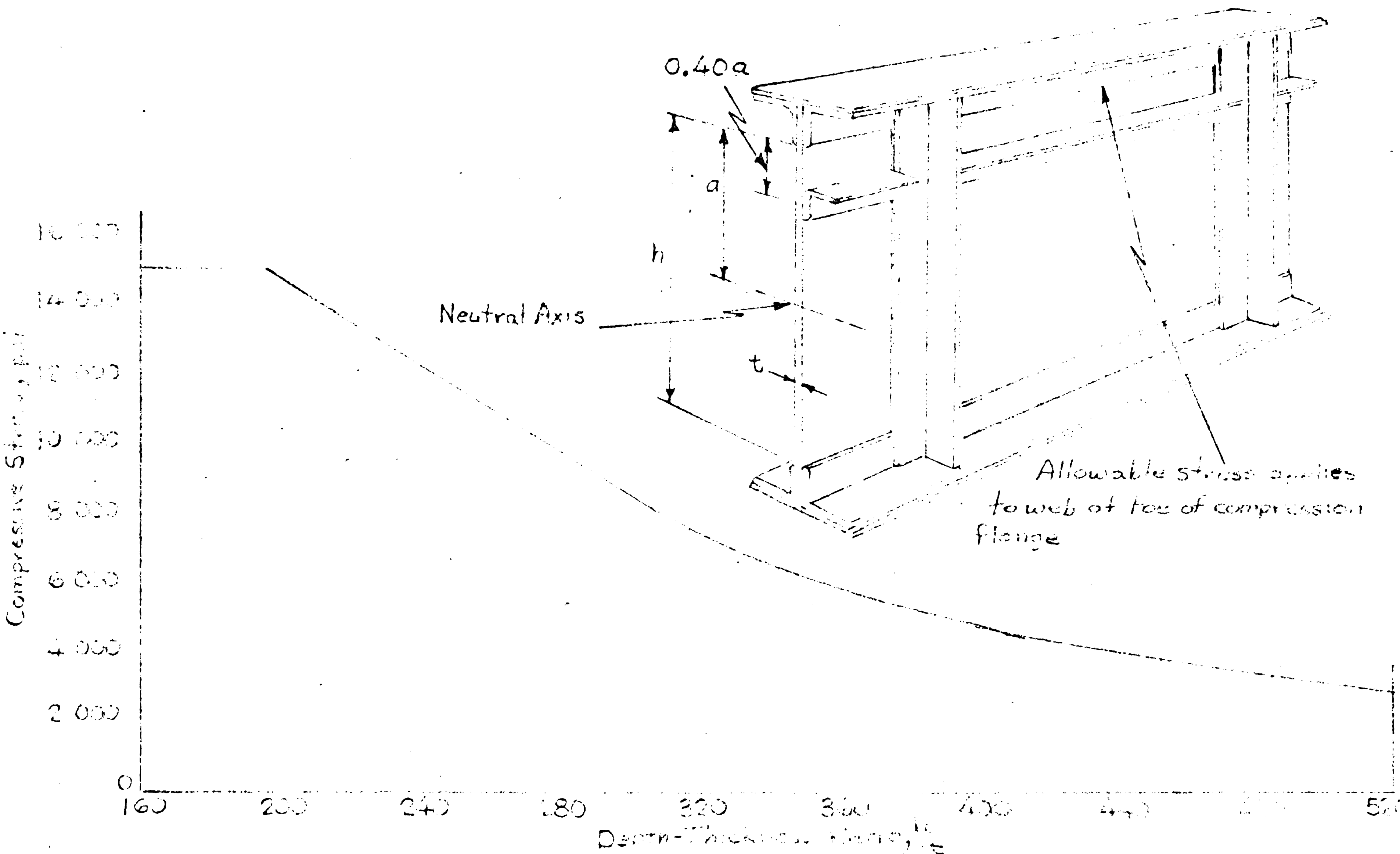
Allowable Compressive Stress on Flat Plates Built-in along Two Edges (Gross Section)
(Partial restraint assumed along both edges)

Fig 4



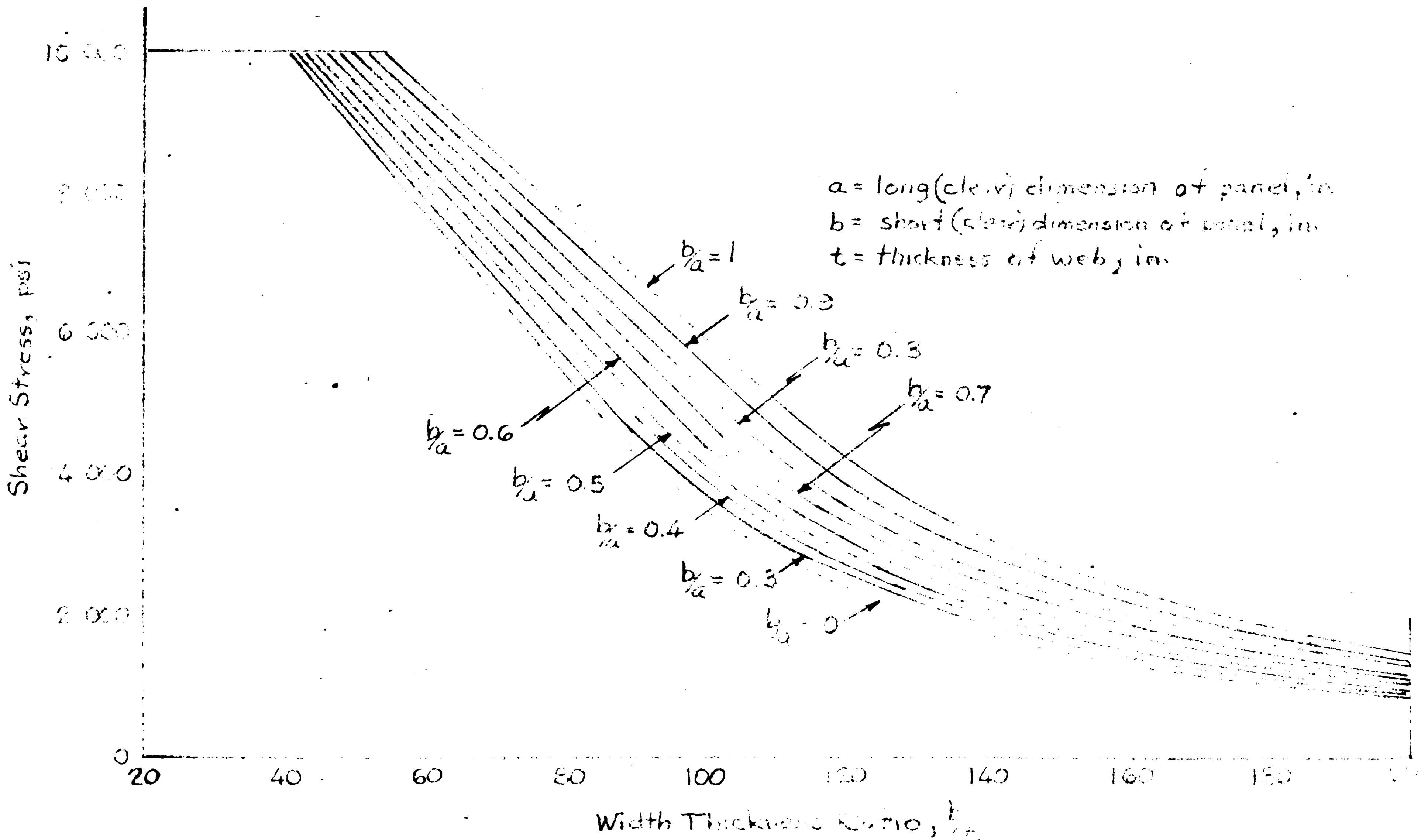
Allowable Compressive Stress [illegible] Webs of Beams & Girders (Girders [illegible]
No Horizontal Stiffener)
([illegible] restraint [illegible] along toe of [illegible])

Fig 5



Allowable Compressive Stress for Webs of Girders
with One Horizontal Stiffener (Gross Section)
(Partial restraint at [illegible])

Fig 5A



Allowable Shear Stress in Webs (Gross Section)

(Partial restraint assumed at edges of rectangular panels)

Fig. 6

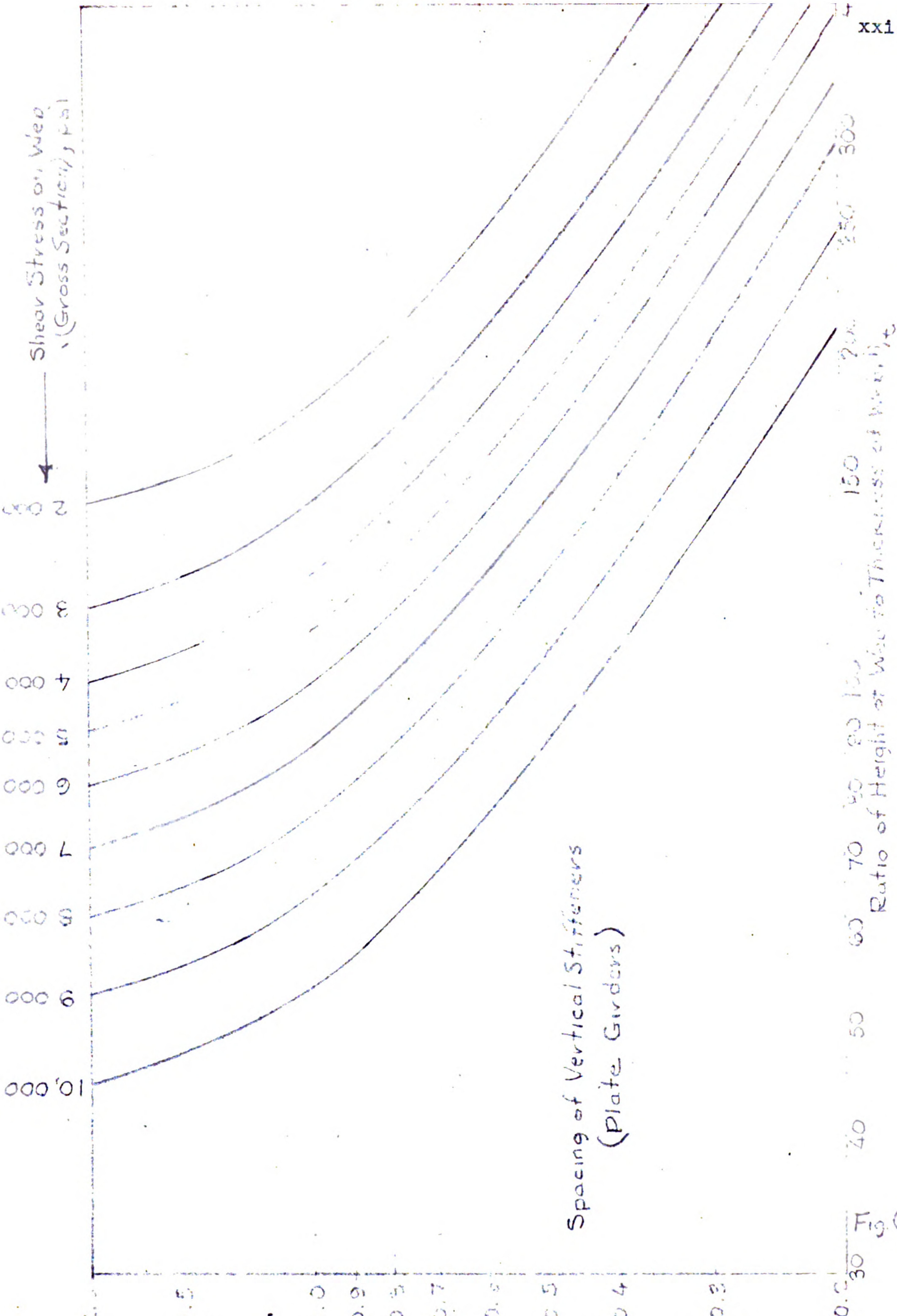

Shear Stress on Web (Gross Section), psi
2 000
3 000
4 000
5 000
6 000
7 000
8 000
9 000
10,000
Spacing of Vertical Stiffeners
(Plate Girders)
Ratio of Height of Web to Thickness of Web h/t
30
40
50
60
70
80
90
100
150
200
250
300
1.5
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2

Fig. 6.

FABRICATION

Quality and accuracy of workmanship shall be in accordance with standard specifications for steel structures. Details in which the fabrication of aluminum alloy 61S-T differs from that of structural steel are covered by the following specifications:

A. Laying Cut

1. Hole centers may be center punched and cut-off lines may be center punched or scribed. Center punching and scribing shall not be used where such marks would remain on fabricated material.

2. A temperature correction shall be applied where necessary in the layout of critical dimensions. The coefficient of expansion shall be taken as 0.000013 per degree Fahrenheit.

B. Cutting

1. Material 1/2 inch thick or less may be sheared, sawed or cut with a router. Material over 1/2 inch thick shall be sawed or routed.

2. Cut edges shall be true and smooth and free from excessive burrs or ragged breaks.

3. Edges of plates carrying calculated stresses shall be planed to a depth of 1/4 inch except in the case of sawed or routed edges of a quality equivalent to a planned edge.

4. Reentrant cuts shall be filleted by drilling prior to cutting.

5. Flame cutting of aluminum alloys is not permitted.

C. Heating

1. Structural material shall not be heated except:

   a. Material may be heated to a temperature not exceeding 400 degrees Fahrenheit for a period not exceeding 15 minutes to facilitate bending.

   b. Rivets shall be heated as specified in Section E-2.

D. Punching, Drilling and Reaming

1. Rivet or bolt holes in main members shall be subpunched or subdrilled 3/16 inch smaller than the nominal diameter of the fastener and reamed to finished size after assembly, except that if the metal thickness is greater than the diameter of the hole that hole shall be drilled.

2. Rivet or bolt holes in secondary material not carrying calculated stress may be punched or drilled to finished size before assembly.

3. The finished diameter of holes for cold driven rivets shall be not more than 4 percent greater than the nominal diameter of the rivet.

4. The finished diameter of holes for hot driven rivets shall be not more than 7 percent greater than the nominal diameter of the fastener.

5. The finished diameter of holes for unfinished bolts shall be not more than 1/16" larger than the nominal bolt diameter.

6. Holes for turned bolts shall be drilled or reamed to give a driving fit.

7. All holes shall be cylindrical and perpendicular to the principal surface. Holes shall not be drifted in such a manner as to distort the metal. Any chips lodged between contacting surfaces shall be removed before riveting.

E. Riveting

1. The driven head of aluminum alloy rivets preferably shall be of the flat or of the low cone type.

   a. Flat heads shall have a diameter not less than 1.4 times the nominal rivet diameter and a height not less than 0.4 times the nominal rivet diameter.

   b. Low cone heads shall have a diameter not less than 1.4 times the nominal rivet diameter and a height, to the apex of the cone, not less than 0.65 times the nominal rivet diameter. The included angle at the apex of the cone shall be approximately 127 degrees.

2. Rivets shall be driven hot or cold as called for on the plans.

   a. Hot driven rivets shall be heated in a hot air type furnace providing uniform temperatures throughout the rivet chamber and equipped with automatic temperature controls.

   b. For hot driven alloy 53S rivets the rivet temperature shall be held at 1030 to 1050 degrees Fahrenheit for not less than 15 minutes and not more than one hour before driving.

c. Hot rivets shall be transferred from the furnace to the work and driven with a minimum loss of time.

3. Rivets shall be driven with direct-acting riveters where practicable.

4. Rivets shall fill the holes completely. Rivet heads shall be concentric with the rivet holes and shall be in proper contact with the surface of the metal.

5. Defective rivets shall be removed by drilling.

F. Welding

1. Welding of aluminum alloys is not permitted except as specifically called for on the plans.

2. Where welding is employed, care shall be exercise d to remove all traces of welding flux.

G. Cleaning of Metal Surfaces

1. Surfaces of metal shall be cleaned immediately before painting by a method which will remove all dirt, oil, grease, and other foreign substances.

2. Either of the two following methods of cleaning may be used on exposed metal surfaces:

a. Sandblasting - Standard mild sandblasting methods may be used.

b. Chemical cleaning - Parts may be immersed in, or swabbed with a dilute water solution of phosporic acid and organic solvents such as Deoxidine No. 126. The solution temperature shall remain in contact with the metal not less than 5 minutes. Residual solution shall be removed with-clear water.

3. <u>For contacting surfaces only,</u> the metal may be cleaned in accordance with section G-2, or with a solvent such as mineral spirits or benzine.

4. Flame cleaning is not permitted.

H. <u>Painting</u>

1. Metal parts shall be painted unless the plans state that no painting is required.

2. Contacting metal surfaces shall be painted before assembly with one coat of zinc chromate primer in accordan ce with Navy Department Specification 52P18 or equivalent, or with one coat of Alumilastic (brushing consistency with zinc chromate added) or equivalent. Zinc chromate paint shall be allowed to dry before assembly of the parts.

3. In any case where aluminum work is to be fastened to steel members or other dissimilar metal parts, the aluminum shall be kept from direct contact with such parts by painting the aluminum surface as described in H-2 and by painting the dissimilar metal with a suitable primer paint.

4. Aluminum surfaced to be placed in contact with concrete or masonry construction shall, before installation, be given a heavy coat of an Alkali resistant bituminous paint. The quality of the bituminous paint used shall be equal to that called for in the army-navy aeronautical specification AN-P-31. The paint shall be applied as it is received from the manufacturer without the addition of any thinner.

5. All other surfaces shall be given one shop coat of zinc chromate primer in accordance with Navy Department Specification 52P18 or equivalent.

6. All surfaces, except those covered by sections h-2, H-3 and H-4 shall be given a second shop coat of paint consisting of two pounds of Alcoa Albron Standard Paste No. 205 per gallon of varnish to Federal Specification TTV81A, or equivalent. Sufficient Prussian Blue shall be added to permit detection of incomplete application of the subsequent paint coat.

7. After erection bare spots shall be touched up with zinc chromate primer followed by one coat of aluminum paint as specified in H-5 and H-6.

8. The completed structure shall be given one field coat of aluminum paint as specified in section H-6, exeept that Prussian Blue shall be ommitted from the field coat.

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